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Asphalt Materials and Mixtures

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Transportation Research Record 1115

Contents

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1115

Foreword

This Record should be of interest primarily to materials engineers and materials researchers, but it may also have marginal interest for design and maintenance engineers.

Coplantz et al. present the results of a study on the effectiveness of antistrip additives for materials used in the reconstruction of a highway in Nevada. The study covers laboratory and field mixes as well as pavement cores. Stroup-Gardner and Epps evaluated four variables that affect the success of lime as an antistripping agent: methods of addition, types of lime, different products, and air voids. These were related to moisture sensitivity and temperature susceptibility. Leung and Anderson evaluated the temperature susceptibility and low-temperature structural characteristics of asphalt cements from heavy crude sources in western Canada. Stock and Button compared the performance of asphalt cement from different sources by means of a series of full-scale trials and associated laboratory tests of penetration and viscosity of the asphalt cement, chromatography, and mix parameters such as void content and resilient modulus. Beazley et al. detail size exclusion chromatography and nuclear magnetic resonance techniques for predicting asphalt yield and viscosity as functions of type of crude oil and refining conditions. Noureldin and Wood present some of the findings from a laboratory characterization of hardened bituminous film around reclaimed aggregate particles and the influence of rejuvenator diffusion in the film on binder behavior. Such characterizations can be used in the prediction of long-term pavement performance.

Carpenter and VanDam discuss the results of a series of laboratory tests on polymer additives to establish test data that can serve as performance indicators for mixes. Five polymer blends, three standard asphalt grades, and crushed limestone were used. Higgins researched the value of chemical modifiers for improving the physical properties and durability of asphalt concrete using test data from laboratory and field specimens with and without modifiers. Hicks et al. report on an experimental Oregon road section that incorporates asphalt additives. The study covers mix design, construction, quality control, unit prices, and field performance. Kim et al. describe the evaluation of an oxidative aging procedure for asphalt mixtures using a pressure oxidation bomb. Parker and Gharaybeh used indirect tensile tests to study the stripping process and evaluate the stripping test for Alabama materials. Castedo presents the results of field and laboratory investigations to determine the role that materials variability, mix design factors, and other parameters play in the performance of recycled asphalt pavements on secondary roads. Bissada investigated the ability of foamed asphalt to stabilize marginal, local sand aggregates for use as base or subbase materials.

Bayomy and Khedr report on a full-scale experimental road in which sulfur was used as a partial replacement for asphalt. Beatty et al. surveyed 26 sulfur-extended asphalt paving projects constructed between 1975 and 1982 in 18 states to measure the severity of major visible pavement distress. Paulsen et al. conducted a study to determine the potential and technical feasibility of using waste roofing products as a partial substitute for asphalt cement or aggregate in asphalt paving mixtures. Salter and Rafati-Afshar describe the use of the indirect tensile test to investigate the effects of ethylene vinyl acetate, polypropylene fiber, rubber, and sulfur on the fatigue strength of bituminous mixtures.

Colwill and Daines report on the performance of previous macadam with polymer-modified binders to improve long-term durability and economic viability. Tam and Lynch review the developments that led to the adoption of a policy of using friction course mixes in Ontario as well as the design, construction, and results from experimental test sections. Roberts and Lytton developed a mix design procedure for asphalt-rubber binders for airport pavements using ground scrap tires. Miller relates the history and present-day applications of surface treatments and chip seals in New Brunswick.

Antistrip Additives: Background for a Field Performance Study

JOHN S. COPLANTZ, JON A. EPPS, AND LEDO QUILICI

Water sensitivity of asphalt concrete pavements is of great importance nationwide. Water-sensitive pavement may suffer damage that leads to reduced pavement life when subject to moisture. To alleviate this problem, various liquid antistripping additives have been developed. In this paper are presented the results of a study on the effectiveness of antistrip additives for materials used in the reconstruction of Nevada State Highway 207. Preconstruction mixtures containing various antistripping additives (liquids and solids), construction mixtures mixed in the field and compacted in the laboratory, and cores taken after construction were subjected to laboratory conditioning using vacuum saturation plus one cycle of freeze-thaw. Test results show that a slight reduction in water sensitivity was obtained in mixtures that contained the liquid antistrip additives in comparison with control mixtures without additives or mixtures containing portland cement as an antistripping material. Results of evaluation of mixtures during the preconstruction phase of the project indicate that mixtures that contain lime slurry exhibited significant reductions in water sensitivity. Test results of field cores show agreement with preconstruction mixtures in the prediction of water sensitivity. Test results of cores also indicate that no significant changes in mixture strength and water sensitivity have taken place during the first year of life. In addition, visual surveys have indicated that test sections that contain either the liquid antistripping additive or portland cement additive have performed well during the first year of pavement life.

Premature pavement distress in the form of raveling and cracking has occurred on several pavements in Nevada in the last several years *(1, 2)*. These types of distress are caused in part by water sensitivity (loss of bond between the asphalt cement and the aggregate or loss of strength in the presence of water, or both) of the paving mixtures.

Several techniques can be used to reduce the sensitivity of an asphalt concrete mixture to water or moisture. Liquid "antistrip" chemicals as an additive to asphalt cement and portland cement or lime as an additive to aggregate (dry or in slurry form) are commonly used throughout the United States as antistrip agents. Paving mixtures that have been designed to account for the effects of water sensitivity by the use of antistripping agents can be cost-effective for governmental agencies. Liquid chemical antistrip agents added to asphalts are preferred by several public agencies and contractors because of their cost advantage and ease of handling during construction. However, research has indicated that several liquid antistrip chemicals used in asphalt cements are not effective over a wide range of material types.

New chemical formulations that show promise for solving difficult stripping problems are being developed. One of these new products is being evaluated in Nevada. In this paper preconstruction, construction, and postconstruction portions of the research study are discussed. Laboratory comparisons of this relatively new product, a conventional liquid antistrip chemical in asphalt cement, portland cement, and lime are presented. A more detailed report of this project can be found elsewhere *(3)*.

DESCRIPTION OF PROJECT

A portion of Nevada State Highway 207 was reconstructed and realigned in 1984 and 1985. The project, locally known as Kingsbury Grade, is located in Douglas County and connects the Carson Valley with US-50 near Stateline, Nevada (Figure 1). The project is approximately 3.8 mi in length. The average daily traffic for 1985 was 14,670 over the length of the project. The projected traffic level for the year 2005 is 27,160 *(4)*. The test section within the project is located at elevations that range from 7,075 to 7,300 ft above sea level. Average annual precipitation for the area is 23 in. during a 70-day period. One hundred forty air freeze-thaw cycles occur annually. Maximum

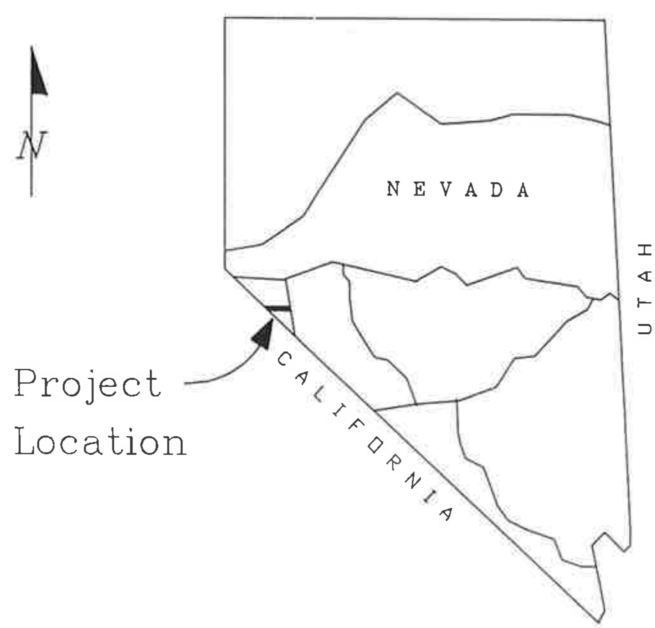

FIGURE 1 Location of project.

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air temperatures rarely exceed 90°F. Typical minimum temperatures are 0°F.

The typical reconstructed cross section of the roadway consists of a 3/4-in. open-graded bituminous wearing surface and 5 1/2 in. of dense-graded Type-2 asphalt concrete over a Type-2 Class-B aggregate base. The Type-2 dense-graded asphalt concrete used on the project contains one of two antistrip agents, portland cement, or a liquid antistrip additive. The open-graded surface layer contains portland cement as an antistrip material. Laboratory preconstruction tests to evaluate water sensitivity were performed on these and several other antistrip agents. Loose samples of field-prepared mixtures and core samples from the in-place pavement were also obtained and subjected to the laboratory testing program.

Because this project is located within a severe climatic area of the state, a test section to evaluate a new antistrip chemical was included within its bounds. The test section was placed during the second year of construction, and the change to include the test section was made after the project was under contract. Because the project was under contract and additional costs associated with test sections needed to be minimized, only one test section was placed although others would normally have been included to better define the field performance of antistrip materials and to provide additional life-cycle cost data on the various additives. Time constraints and equipment availability also severely limited the number of test sections.

MATERIALS

Asphalt Cement

The asphalt cement used for the project was obtained from a San Francisco Bay area refinery. The physical properties of the AR-4000 asphalt cement are given in Table 1.

Aggregate

The aggregate was obtained from a river deposit located near Gardnerville, Nevada, and a decomposed-granite pit located near the junction of US-50 and US-395. The physical properties and specifications of the aggregate are given in Table 2.

Portland Cement

A Type-IP portland cement was used on the project.

Lime

A hydrated high-calcium lime was used in the laboratory portion of the study.

Liquid Antistrip Chemicals

Two proprietary liquid antistrip chemicals were used in the study. Both materials are used as additives to asphalt cements. Liquid Additive 1, the first-generation additive, has been used throughout the United States for a number of years. Liquid Additive 2, a second-generation additive developed more recently, was also studied. Properties of the materials are given in Table 3.

TEST PROGRAM

Laboratory tests were performed on samples prepared before construction, during construction, immediately following construction, and 1 year after construction. Preconstruction sam-

TABLE 1 PROPERTIES OF ASPHALT CEMENT OBTAINED BEFORE CONSTRUCTION

	San Francisco Bay Area AR-4000		
Property	No Additive, Sampled 7/24/84	0.5% Liquid Additive 1, Sampled 11/5/84	0.5% Liquid Additive 2, Sampled 11/5/84
Original			
Viscosity at 140°F (poises)	2263	2280	2540
Penetration at 77°F (dmm)	47	37	36
Penetration at 39.2°F (dmm)		6	10
Viscosity at 275°F (cSt)	340	293	286
Softening point (°F)		125	126
Flash point, COC[a] (°F)	475+	475+	475+
Aged (after RTFOT[b], ASTM D 2872)			
Viscosity at 140°F (poises)	4617	4603	4614
Penetration at 77°F (dmm)	34	24	23
Penetration at 39.2°F (dmm)		3	9
Viscosity at 275°F (cSt)	469	413	406
Softening point (°F)		134	135

[a]Cleveland open cup.
[b]Rolling thin-film oven test.

TABLE 2 PROPERTIES OF AGGREGATE AS COMBINED FOR TYPE-2 GRADATION

Sieve Size	Percent Passing: Gradation	Percent Passing: Specification	Property	Result	Specification
1 in.	100	100	Specific gravity	2.58	
3/4 in.	98	90–100	Liquid limit	21	35 max
1/2 in.	84				
3/8 in.	74	63–85	Plasticity index	NP	6 max
No. 4	58	45–65			
No. 10	42		Fractured faces (%)	85	50 min
No. 16	34	20–40	Los Angeles, abrasion, 500 rev (%)	23.3	45 max
No. 40	18				
No. 50	13		Surface area (ft^2/lb)	26.8	
No. 100	8				
No. 200	5	3–9	Swell test (in.)	0.003	0.03 max

NOTE: Percentages as combined from pit are as follows:

Size	Percentage
3/4 in.	28.8
3/8 in.	17.6
Natural sand	43.6
Decomposed granite	10

ples were mixed and compacted in the laboratory. The variables considered in this portion of the study included

1. Type and amount of antistrip agent,
2. Asphalt content, and
3. Type and gradation of aggregate.

Table 4 gives the test matrix associated with this portion of the study. Figure 2 shows the test sequence used on the preconstruction samples. Conventional mixture design tests were also performed with various combinations of materials.

Loose samples of asphalt concrete were obtained from behind the paving machine during construction. Conventional quality control tests were performed by the Nevada Department of Transportation (NDOT). Additional tests were performed on laboratory-compacted samples of this loose mixture (Table 4 and Figure 2).

Core samples were obtained a few days after construction and also 1 year after construction. The core samples were obtained with a water-cooled coring unit, wrapped in plastic, and transported to the laboratory. At the laboratory, the cores were removed from the plastic and allowed to air dry for 48 hr before being subjected to the test program shown in Figure 2.

TEST METHODS

Conventional mixture design and quality control tests were performed by NDOT. These tests used standardized AASHTO *(5)* and NDOT *(6)* procedures. Resilient modulus, indirect tensile strength, and modified Lottman water sensitivity tests were performed by the Construction Materials Laboratory, University of Nevada-Reno. These test methods are briefly described.

The resilient modulus (Young's modulus for viscoelastic materials) was determined by ASTM D 4123 *(7)*. The test

TABLE 3 TYPICAL PHYSICAL PROPERTIES OF LIQUID ANTISTRIP AGENTS

Property	First-Generation Additive	Second-Generation Additive
Active ingredients (%)	100	100
Form	Liquid	Viscous liquid
Color	Dark brown	Dark
Type	Alkaline	Metallo amine complex
Pour point, ASTM D 97 (°F)	20–30	60
Viscosity, ASTM D 445-79 (cSt)		
At 77°F	750–2000	35 000[a]
At 100°F	250–860	–[b]
At 140°F	–[b]	1500[c]
Flash point, ASTM D 92-78, COC (°F)	275 min	375
Specific gravity, 77°F/60°F (U.S.P. method)	1.03	1.10
Weight at 77°F (lb/gal)	8.6	9.18

[a]Plus or minus 15 percent.
[b]Dash = not specified.
[c]Plus or minus 10 percent.

TABLE 4 VARIABLES CONSIDERED IN PROJECT

Amount of Decomposed Granite (%)	Asphalt Content (% by dry weight of aggregate)	Additive Type and Percentage						
		No Additive	1% Portland Cement Applied Dry[a]	1% Lime Applied Dry[a]	1% Lime Applied in Slurry[a]	2% lime Applied in Slurry[a]	0.5% Liquid Additive 1[b]	0.5% Liquid Additive 2[b]
0	6.0		PC	PC	PC	PC		
	6.25							
	6.5	PC					PC	PC
5	6.0							LC
	6.25							
	6.5							
10	6.0	PC	PC	PC	PC	PC	PC	PC
	6.25			LC, CC				LC, CC
	6.5							

NOTE: PC = preconstruction tests, LC = loose mix sampled during construction and completed in laboratory, and CC = cores obtained after construction.
[a]Percent by dry weight of aggregate.
[b]Percent by weight of asphalt.

procedure involves the application of a light repetitive load through a load cell along the vertical axis of the sample. Loads were applied for a duration of 0.1 sec at 3.0-sec intervals.

Resilient modulus values for all samples were obtained at 77°F. Control samples were tested in the dry condition, and samples subject to moisture conditioning were tested under saturated surface dry (SSD) conditions.

Indirect Tensile Strength

Indirect tension was determined by ASTM D 4123 *(7)*. The equipment required and the loading procedure are described in ASTM C 496 *(8)*. A deformation rate of 2.0 in./min was used until sample failure occurred. The calculation of tensile strength at failure is also described in ASTM C 496. It should be noted that tensile strength measurements for core samples taken in May 1986 were taken with a Marshall testing machine as described in ASTM D 1559 *(8)* because the previously used mechanical testing apparatus had suffered flood damage. Measurements of indirect tensile strength were obtained at 77°F in either the dry or the SSD condition.

Lottman Water Sensitivity

The procedure used for moisture conditioning is essentially that used by Lottman *(9)* with slight modification. Specimens were subjected to vacuum saturation at 26 in. Hg vacuum for 2 hr and then the resilient modulus test was performed under the SSD condition. The samples were again subjected to vacuum saturation for 10 min, tightly wrapped in thin plastic, and frozen at −20°F for 15 hr. The frozen specimens were unwrapped and submerged in 140°F water for 24 hr and then submerged in 77°F water for approximately 2 hr. Resilient modulus and indirect tension results were determined at SSD conditions, and these results were compared with the test results for unconditioned samples.

PRECONSTRUCTION TESTS

Mixture Design

A mixture design for the Type-2 dense-graded asphalt concrete was performed by NDOT using the Hveem method. The design

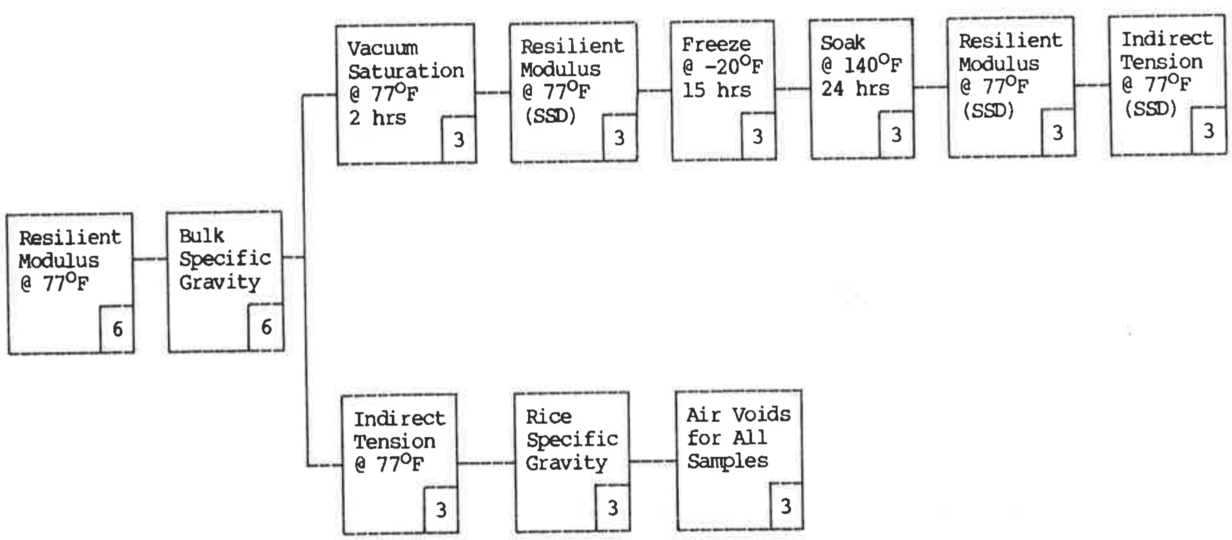

FIGURE 2 Test sequence for samples.

mixture was prepared from aggregates that contained 10.0 percent decomposed granite (DG) and no antistrip chemicals.

Mixtures with Antistrip Chemicals

Laboratory mixed and compacted samples were prepared at the University of Nevada and NDOT to determine the effectiveness of various types of antistrip agents. The type and quantities of antistrip agents used are listed next and are given in Table 4.

1. 1.0 percent portland cement (by dry weight of the aggregate) added dry to the aggregate,
2. 1.0 percent lime (by dry weight of the aggregate) added dry to the aggregate,
3. 1.0 percent lime (by dry weight of the aggregate) added as a slurry to the aggregate,
4. 2.0 percent lime (by dry weight of the aggregate) added as a slurry to the aggregate,
5. 0.5 percent (by weight of asphalt cement) Liquid Additive 1 (a first-generation liquid antistrip additive) added to the asphalt cement, and
6. 0.5 percent (by weight of asphalt cement) Liquid Additive 2 (a second-generation liquid antistrip additive) added to the asphalt cement.

These preconstruction mixes were prepared at 6.0 and 6.5 percent asphalt cement by dry weight of aggregate and at 0.0 and 10.0 percent decomposed granite (Table 4).

Test results are given in Tables 5 and 6 and shown in Figures 3–6. Properties of the mixtures before exposure to water are given in Table 5. A comparison of all antistrip agents investigated in this study is possible with mixtures containing 6.0 percent asphalt cement and 10.0 percent decomposed granite. Mixtures that contain either Liquid Additive 2 or slurried lime have the highest resilient modulus and tensile strength. This difference among types of antistrip agents is also evident for mixtures that contain 6.0 and 6.5 percent asphalt with no decomposed granite (Table 5, Figure 5). Note that the laboratory compaction effort was adjusted to provide air voids in the range of from 8 to 10 percent. This compactive effort was used to match the air voids obtained during laboratory measurements of initial field cores.

A comparison of properties of mixtures before and after they were subjected to the action of water is given in Table 6 and Shown in Figures 3–6. Figures 3 and 4 show comparisons of original resilient modulus, resilient modulus after soaking the sample with use of a vacuum, and resilient modulus after subjecting the sample to a freeze-thaw cycle (Lottman). Retained strength ratios as measured with resilient modulus and tensile strength after the Lottman tests are shown in Figures 5 and 6. A comparison of all antistrip agents investigated in this study is possible with mixtures containing 6.0 percent asphalt cement and 10.0 percent decomposed granite (Table 6, Figures 4 and 6). Retained resilient moduli after vacuum saturation only are at levels greater than 70 percent (Figure 4). After Lottman freeze-thaw conditioning, all mixtures except those that contained slurried lime were below the 70 percent retained strength level (Figure 6).

CONSTRUCTION TESTS

Quality Control Tests

Conventional quality control tests were performed by NDOT during the construction of the project. A complete review of the data is found elsewhere *(3)*. The data are summarized here.

Asphalt Cement Properties

Original and laboratory-aged viscosity and penetration data for the asphalt cement sampled during construction were obtained from samples with and without the liquid antistrip chemicals. The original and aged viscosities and penetrations were not found to be substantially affected by the liquid antistrip chemical.

TABLE 5 AVERAGE TEST RESULTS BEFORE MOISTURE CONDITIONING (preconstruction)

Sample Identification	Resilient Modulus (ksi)	Indirect Tension (psi)	Air Voids (%)
6.0% AC, no DG			
1.0% PC	408	115	10.9
1.0% lime (dry)	500	121	10.4
1.0% lime slurry	679	180	8.4
2.0% lime slurry	624	169	8.8
6.5% AC, no DG			
No additive	470	131	10.7
0.5% Liquid Additive 1	480	127	9.3
0.5% Liquid Additive 2	556	159	9.7
6.0% AC, 10% DG			
No additive	424	119	10.1
0.5% Liquid Additive 1	461	124	9.5
0.5% Liquid Additive 2	672	161	10.1
1.0% PC	456	129	9.3
1.0% lime (dry)	476	126	8.6
1.0% lime slurry	658	170	8.9
2.0% lime slurry	556	150	9.5

TABLE 6 AVERAGE TEST RESULTS FOR MOISTURE-CONDITIONED SAMPLES (preconstruction)

Sample Identification	Resilient Modulus: After Vacuum Saturation (ksi)	Resilient Modulus: Retained Strength (%)	Resilient Modulus: After 1 Cycle Lottman (ksi)	Resilient Modulus: Retained Strength (%)	Tensile Strength: After 1 Cycle Lottman (psi)	Tensile Strength: Retained Strength (%)
6.0% AC, No DG:						
1.0% PC	405	99	80	20	25	22
1.0% Lime (dry)	486	97	77	15	23	19
1.0% Lime slurry	637	94	437	64	151	84
2.0% Lime slurry	610	98	606	97	155	92
6.5% AC, No DG:						
No additive	405	86	82	17	24	18
0.5% Liquid Additive 1	398	83	132	28	38	30
0.5% Liquid Additive 2	692	124	293	53	60	38
6.0% AC, 10% DG:						
No additive	328	77	55	13	18	15
0.5% Liquid Additive 1	374	81	84	18	28	23
0.5% Liquid Additive 2	701	104	195	29	52	32
1.0% PC	487	107	68	15	22	17
1.0% Lime (dry)	557	117	54	11	16	13
1.0% Lime slurry	530	81	469	71	152	89
2.0% Lime slurry	641	115	558	100	161	107

Asphalt Content

Asphalt content was determined by both tank measurements and mixture extraction tests. Reported average asphalt cement contents were slightly lower than the target value.

Aggregate Gradation

Aggregate gradation was determined on residual aggregate from the extraction of plant mix used for asphalt content determination. Gradation data showed substantial compliance with Type-2 specifications.

Mixture Placement Conditions

The specified job mix formula range of temperature for dense-graded asphalt cements was 255°F to 325°F for the completed mixtures in the haul vehicles just before they left the plant *(10, 11)*. All reported values are within the specified range. Requirements for open-graded mixtures were 255°F to 290°F *(10, 11)*. Some values exceeded the desired range.

Special Laboratory Tests

Loose samples of the dense-graded mixtures were obtained during construction and compacted in the University of Nevada

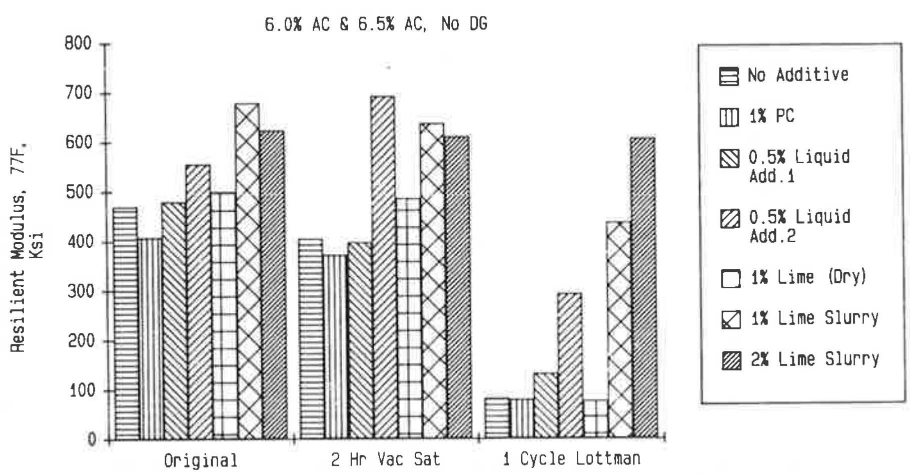

FIGURE 3 Resilient modulus through conditioning cycle, no DG (preconstruction).

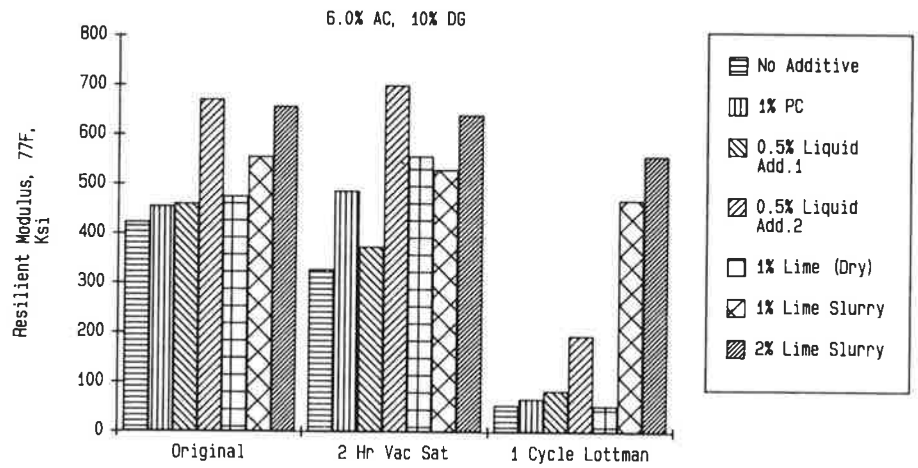

FIGURE 4 Resilient modulus through conditioning cycle, 10% DG (preconstruction).

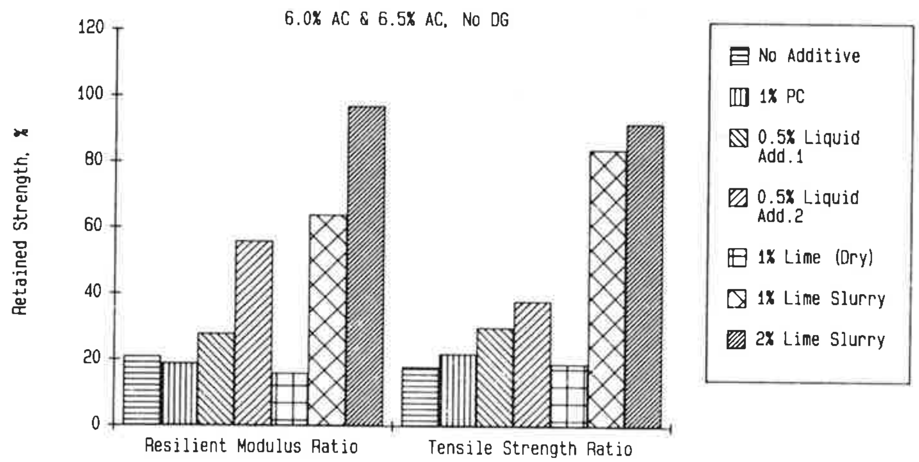

FIGURE 5 Resilient modulus and tensile strength ratios, no DG (preconstruction).

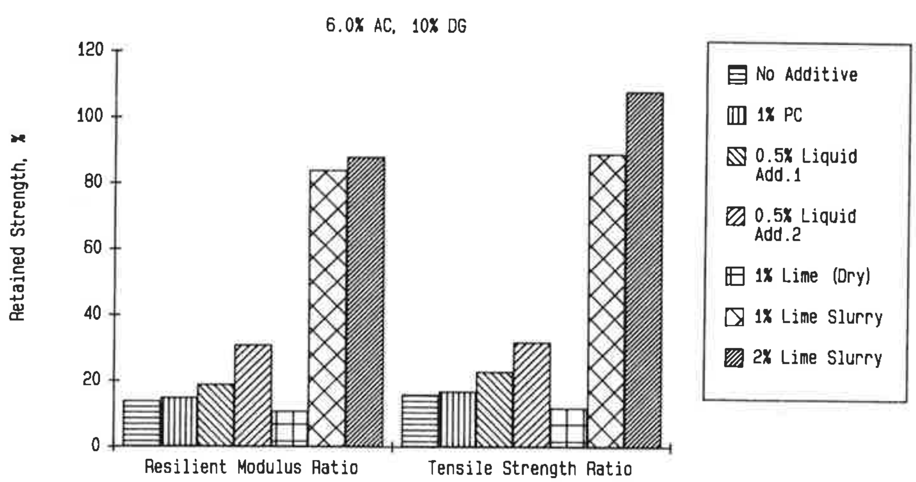

FIGURE 6 Resilient modulus and tensile strength ratios, 10% DG (preconstruction).

TABLE 7 AVERAGE TEST RESULTS BEFORE MOISTURE CONDITIONING (loose field mix and core samples)

Sample Identification	Resilient Modulus (ksi)	Indirect Tension (psi)	Air Voids (%)
Loose field mix samples			
1.0% portland cement	440	119	9.9
0.5% Liquid Additive 2, 5% DG	388	126	10.5
0.5% Liquid Additive 2, 10% DG[a]	345	111	11.2
0.5% Liquid Additive 2, 10% DG[b]	343	101	10.7
Cores taken July 1985			
1.0% portland cement	136	100	11.1
0.5% Liquid Additive 2, top lift	212	135	9.2
0.5% Liquid Additive 2, bottom lift	337	134	8.3
Cores taken May 1986			
1.0% portland cement	283	71	10.7
0.5% Liquid Additive 2, top lift	270	81	10.6
0.5% Liquid Additive 2, bottom lift	488	105	9.4

[a]Location 1.
[b]Location 2.

Construction Materials Laboratory. The samples were subjected to the test program shown in Figure 2. Test results are given in Tables 7 and 8 and shown in Figures 7 and 8. Portland cement and Liquid Additive 2 were used as antistrip agents in the field mixtures. The majority of the field mixtures contained 10.0 percent decomposed granite with a target asphalt content of 6.25 percent by dry weight of aggregate. A control section without antistrip agents was not placed on the project.

Slightly higher resilient modulus at 77°F and tensile strength were noted for the mixtures that contained portland cement (Table 7). The portland cement may be acting as a mineral filler and increasing the stiffness of the asphalt cement.

Retained resilient modulus and tensile strength after Lottman test are uniformly low for the various mixtures (Table 8, Figure 8). Note that the laboratory compaction effort was adjusted to produce samples with air voids in an 8 to 10 percent range to simulate air void measurements obtained from initial field cores (Table 7).

POSTCONSTRUCTION TESTS

Core samples of dense-graded mixtures were obtained a few days after construction (July 1985) and again 1 year after construction (May 1986). The cores were subjected to the test program shown in Figure 2. Test results are given in Tables 7 and 8 and shown in Figures 9 and 10.

Slightly higher resilient modulus and tensile strength values

TABLE 8 AVERAGE TEST RESULTS FOR MOISTURE-CONDITIONED SAMPLES (loose mix and core samples)

Sample Identification	Resilient Modulus: After Vacuum Saturation (ksi)	Resilient Modulus: Retained Strength (%)	Resilient Modulus: After 1 Cycle Lottman (ksi)	Resilient Modulus: Retained Strength (%)	Tensile Strength: After 1 Cycle Lottman (psi)	Tensile Strength: Retained Strength (%)
Loose Mix Field Samples:						
1.0% PC	185	42	52	12	18	15
0.5% Liquid Additive 2, 5% DG	210	54	63	16	21	17
0.5% Liquid Additive 2, 10% DG (1)	253	73	44	13	22	20
0.5% Liquid Additive 2, 10% DG (2)	132	38	48	14	25	25
Cores Taken July 1985:						
1.0% PC	127	94	31	23	13	13
0.5% Liquid Additive 2, top lift	231	109	80	38	32	24
0.5% Liquid Additive 2, bottom lift	315	93	112	33	45	34
Cores Taken May 1986:						
1.0% PC	227	80	42	15	16	23
0.5% Liquid Additive 2, top lift	270	100	65	24	28	35
0.5% Liquid Additive 2, bottom lift	536	110	146	30	46	44

(1) Location 1
(2) Location 2

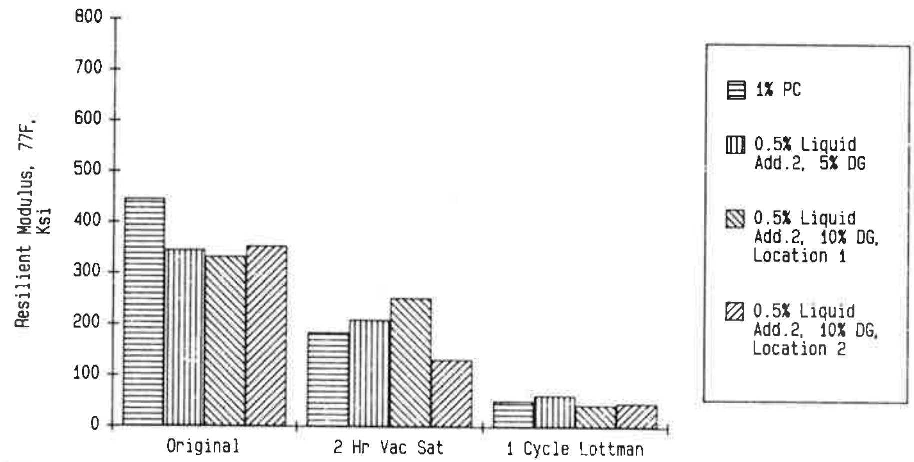

FIGURE 7 Resilient modulus through conditioning cycle (loose field mix).

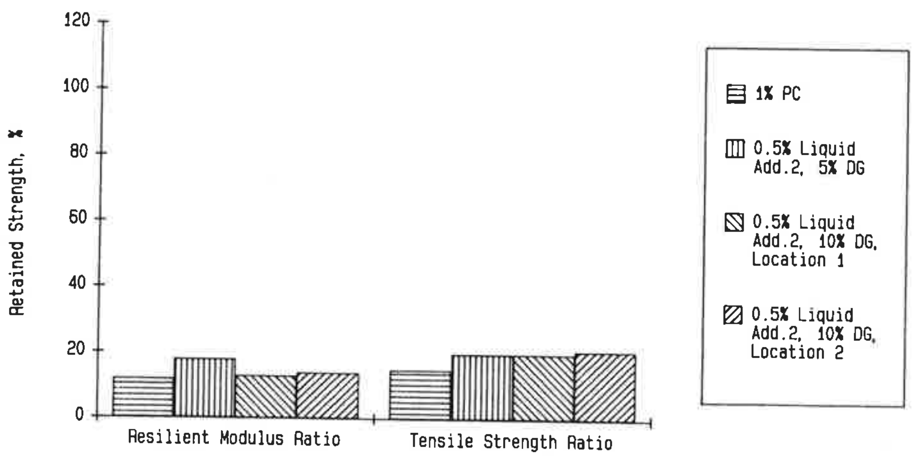

FIGURE 8 Resilient modulus and tensile strength ratios (loose field mix).

were obtained with the liquid antistrip additive (Table 6) for cores taken in July 1985. Part of this difference is associated with the higher air void content of the mixtures that contained portland cement.

Retained resilient modulus and tensile strength values after the Lottman test are low (Table 8, Figures 9 and 10). The mixtures that contained the liquid antistrip agent had slightly higher retained properties than did those mixtures that contained portland cement (Figure 10). Note that the field compaction procedure produced in-place air voids in the range of from 8 to 11 percent (Table 7).

Resilient modulus values for cores taken during May 1986 show an increase in stiffness over those taken in July 1985 (Table 7). The top lift, which contains portland cement, has

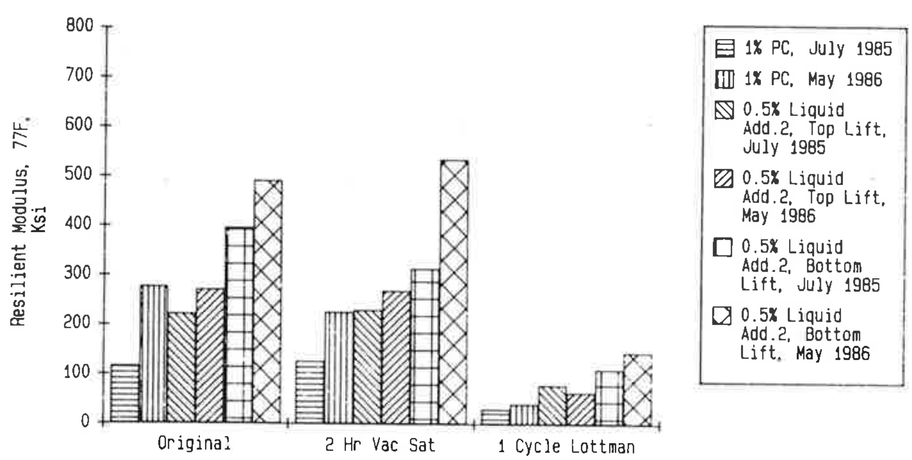

FIGURE 9 Resilient modulus through conditioning cycle (cores).

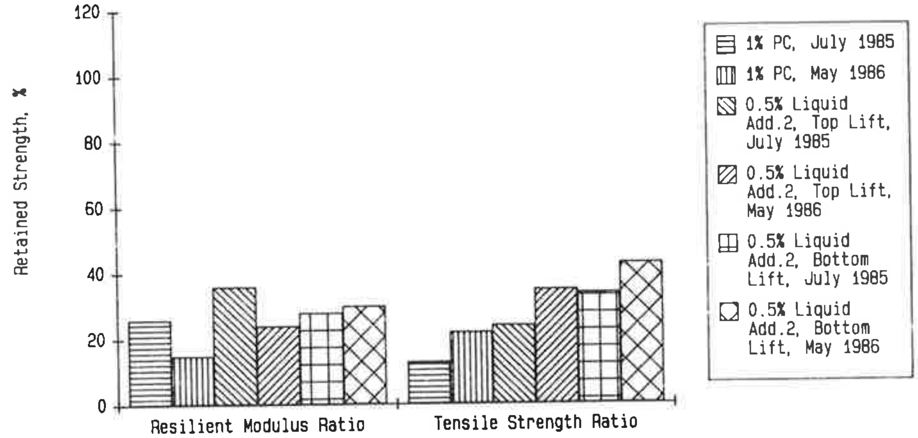

FIGURE 10 Resilient modulus and tensile strength ratios (cores).

stiffness values that have increased significantly relative to the stiffness increase for the top lift of Liquid Additive 2. Resilient modulus measurements on the top lifts of cores sampled during May 1986 show that stiffness values for cores that contain portland cement are quite similar to stiffness values obtained with Liquid Additive 2 (Table 7). Tensile strength values have decreased somewhat. Note that tensile strength values for cores sampled during May 1986 were obtained with a different testing apparatus than were the values reported for July 1985 cores. Resilient modulus values after moisture conditioning did not change significantly during this time (Table 8, Figure 9). Retained strength ratios show no definite trends between these two dates (Figure 10).

A visual survey of the construction project was made during November 1986. A roadway section representing the liquid antistripping agent and a section representing portland cement additive were selected for the survey. Both sections of the roadway appear to be in good condition. There were no occurrences of cracking, rutting, raveling, or bleeding evident in either section. The open-graded layer contains 1.0 percent portland cement as an antistripping agent and was placed over all portions of the project.

CONSTRUCTION PROBLEMS

During the placement of the mixture that contained the liquid antistrip additive, "tenderness" was noted during rolling. "Shoving" and "checking" were noted during steel breakdown rolling. Checking was more severe for downgrade rolling than for upgrade rolling. Only one pass of breakdown rolling was used because two passes proved detrimental to the mat. The temperature was reduced to 180°F for this pass to control the shoving associated with tenderness. Secondary rolling was performed with a pneumatic roller. Pneumatic rolling was performed at a temperature as low as 140°F. A steel-wheeled tandem roller was used for finish rolling.

The cause of the tenderness problem is not known at this time. Asphalt cement tests suggest no significant change in viscosity at compaction temperatures with or without the liquid antistrip additive in the binder. A review of aggregate gradations (although limited) suggests a slight change in gradation with and without the liquid antistrip additive.

This is the first U.S. project that contained the liquid antistrip to be reported as "tender" during construction. An investigation is under way in the manufacturer's laboratory to define the probable cause or causes.

CONCLUSIONS

Preconstruction Samples

Less sensitivity to water was obtained in the mixtures that contained the liquid antistrip chemicals than in control mixes without additives. This improved behavior did not reach the level of performance obtained by lime slurry (Figures 4 and 6).

No significant reduction in water sensitivity was noted with the use of dry portland cement and dry lime compared with control mixes without additive (Figures 4 and 6).

A significant reduction in water sensitivity was noted with the addition of lime slurry (Figures 4 and 6) to the mixtures. The mixtures that contained 2 percent lime slurry showed a slight improvement in resistance to water damage over those that contained 1.0 percent lime slurry.

Construction and Postconstruction Samples

A slight increase in resistance to water damage was obtained in the mixtures that contained the liquid antistrip additive compared with the mixture that contained dry portland cement (Figures 8 and 10). The desired level of improvement was not obtained.

No significant changes in moisture-conditioned mixture strength and water sensitivity occurred between July 1985 and May 1986 (Figures 9 and 10).

Visual surveys indicate that pavement sections with either the liquid antistrip agent or portland cement additive have performed well during the first year of pavement life.

ACKNOWLEDGMENT

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Four Variables That Affect the Performance of Lime in Asphalt-Aggregate Mixtures

MARY STROUP-GARDINER AND JON EPPS

Four variables that affect the success of lime as an antistripping agent were evaluated: (a) four methods of adding two types of lime to asphalt-aggregates, (b) four lime products, (c) two different aggregate sources, and (d) air voids. Effects of the four variables on moisture sensitivity were evaluated by determining the resilient modulus and the tensile strength of samples before and after one cycle of the Lottman accelerated conditioning procedure (a freeze-thaw cycle subjecting water-saturated samples to freezing at −20°F and thawing in a 140°F water bath). The effects of the variables on temperature susceptibility were evaluated by determining the resilient modulus values at four different test temperatures. The following conclusions can be drawn from this research project: (a) quicklime added to the asphalt or to the dry aggregate can be detrimental to the mixture; (b) dolomitic lime can improve mixture properties to the same degree as hydrated lime; (c) mixture properties can be enhanced by the addition of hydrated lime, regardless of the moisture susceptibility of the untreated mixes; (d) an increase in the volume of lime used can further improve the mixture properties; and (e) air voids significantly affect mixture properties, regardless of lime variations.

A significant number of premature pavement failures in the Southeast, intermountain West, and Southwest have been associated with water sensitivity of asphalt-aggregate mixtures. Significant strength losses in mixtures can result without noticeable debonding or "stripping" of the asphalt cement from the aggregate. Partial or complete stripping will lead to strength loss on the order of from 70 to 95 percent.

Stripping is caused by several factors:

1. Asphalt-aggregate interactions,
2. Surface coatings on the aggregate,
3. Smooth aggregate surface texture, and
4. Improper pavement design and construction control.

There are several methods for either eliminating or correcting the causes of stripping:

1. Modifying the physical-chemical properties of the mix,
2. Washing the aggregate before mixing,
3. Crushing smooth-surfaced aggregates,
4. Providing adequate drainage to prevent the accumulation of water in pavement layers,
5. Sealing surfaces to reduce permeability,
6. Controlling compaction of the pavement to reduce permeability, and
7. Replacing a stripping aggregate with a nonstripping aggregate if such a practice is economical.

Although there are many methods of improving mixture properties, the treatment of asphalt concrete with an additive appears to be the most acceptable. Additives are easy to use and have a minimal cost. Although the reasons for the success of additives are not fully understood, three types of additives are generally recognized as treatments for stripping mixtures: hydrated lime, liquid antistripping agents, and portland cement.

The use of these additives has produced varying results in construction projects. Hydrated lime appears to be the most effective antistripping additive. Liquid antistripping agents, usually amines, have had variable success and portland cement has had some general success.

There are several theories about why hydrated lime is effective. First, lime improves the bonding of calcium with silicates in aggregate. Second, there is a possible interaction with the acidic portions of the asphalts *(1)*. Third, if aggregates have clay coatings, there are ion exchange and pozzolanic reactions between the calcium in lime and the silica in clay.

The effect of lime on the moisture sensitivity of asphalt-aggregate mixtures is dependent on other variables:

1. Methods of introducing the lime into the mixture,
2. Types of lime products,
3. Changes in aggregate sources, and
4. Air voids present in the pavement.

This research program explores the effects of these variables.

RESEARCH PROGRAM

This research program was developed to individually evaluate the effect of four variables on the moisture sensitivity of asphalt-aggregate mixtures. The variables are methods of adding lime, various lime products, different aggregate sources, and effects of air voids.

Two types of lime were used in the investigation of methods of adding lime. Quicklime was introduced into the mixture by (a) adding it to the dry aggregate, (b) adding it to the asphalt, and (c) slurrying the quicklime before adding it to the aggregate. Hydrated lime was introduced into the mixture by (a) adding it to dry aggregate and (b) adding it to wet aggregate.

Four types of lime added dry to one source of aggregate were used in the investigation of the effects of various lime products on asphalt-aggregate mixture properties. Two of these limes

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were added dry to aggregate from a second source to establish the effects of different aggregate sources.

Two types of lime, added by various methods to aggregate from one source and compacted with air voids between 1 and 8 percent, were used to evaluate the effect of air voids on moisture sensitivity.

A control set of samples with no lime was used for comparison and to establish the effectiveness of the treatment methods.

MATERIALS AND SUPPLIES

Asphalt

Asphalt from only one source was used for this research program. This was an AR-4000. The physical properties of the asphalt are given in Table 1.

TABLE 1 PHYSICAL PROPERTIES OF AR-4000

Test	Original Asphalt	After Rolling Thin Film	After Extended Rolling Thin Film
Penetration			
39.2°F/100g/5sec	14	11	2
77°F/100g/5sec	54	34	–
Viscosity			
At 140°F (poise)	2184	3880	–
At 275°F (cSt)	268.4	344.8	8645.7
Ductility (cm)	100+	100+	0.5
Ring-and-ball softening point (°F)	123	127	185

NOTE: Dashes = data not available.

TABLE 2 PHYSICAL PROPERTIES OF AGGREGATES

	Bulk Specific Gravity	Bulk, SSD[a], Specific Gravity	Apparent Specific Gravity	Absorption Capacity (%)
Aggregate 1				
Coarse	2.565	2.631	2.746	2.60
Fine	2.492	2.567	2.722	3.40
Aggregate 2				
Coarse	2.616	2.644	2.693	1.12
Fine	2.555	2.601	2.678	1.80

[a]SSD = saturated surface dry.

Aggregates

Aggregates were obtained from Sparks, Nevada (Aggregate 1), and Phoenix, Arizona (Aggregate 2). These aggregates were chosen because of their history of stripping problems and the similarity of their rounded shape and surface texture.

Aggregate stockpiles were separated into 10 sieve sizes. All aggregate larger than the No. 30 sieve was washed to provide tighter control on the amount of fines in the mix. The sample gradation is discussed further in the section on Testing Program and Procedures.

The physical properties of the aggregates are given in Table 2. The major difference among the aggregates is their absorption capacity. The Nevada aggregate has an absorption capacity between 2.6 and 3.4 percent, and that of the Arizona aggregate is between 1.1 and 1.8 percent.

Lime Products

Lime is manufactured from either high-calcium or dolomitic limestone. High-grade commercial deposits usually contain not more than 3 percent total impurities. Heat, water, and carbon dioxide are used to transform limestone into three distinct forms.

Limestone (calcium carbonate) is calcined (burned) to produce quicklime. Quicklime and water react to produce hydrated lime. Carbon dioxide in the air recombines with hydrated lime in the presence of water and converts it to the carbonated state (limestone).

Dolomitic limestone is a combination of calcium and magnesium carbonate. When dolomitic lime is hydrated, only a small portion of the magnesium oxide is hydrated. Complete hydration of the magnesium oxide is accomplished by a continuous high-pressure system.

A quicklime (QL), two hydrated limes (HD1 and HD2), and a dolomitic lime (DL) were used for this research project. The available physical, gradation, and chemical properties are given in Tables 3–5. The hydrated limes were obtained from Industry, California (HD1), and Nelson, Nevada (HD2). The Arizona lime was laboratory hydrated before it was shipped.

The major differences among the lime products are (a) yield in cubic feet of putty, (b) setting rate, and (c) density. The quicklime has a yield of 85 ft^3 of putty compared with approximately 50 ft^3 for both California lime (HD1) and dolomitic lime. The California lime (HD1), quicklime, and dolomitic

TABLE 3 PHYSICAL PROPERTIES OF LIMES

Physical Property	Hydrated Lime 1	Hydrated Lime 2	Quicklime	Dolomitic Lime
Yield of putty (ft^3) per				
Ton	51	–	85	56
Cubic foot	1.1	–	2.56	1.20
50-lb bag	1.3	–	–	1.25
Setting rate to ½ volume[a] (min)	150	–	350	420
Loose density (lb/ft^3)	28	86	60	25
Specific gravity	2.23	2.63	3.15	2.22

NOTE: Dashes = data not available.
[a]ASTM C 110.

TABLE 4 SIEVE ANALYSIS OF TWO LIMES

	Cumulative Percentage Passing	
Sieve Size	Hydrated Lime 1	Dolomitic Lime
20 mm	100	100
30 mm	100	99.6
35 mm	100	99.6
48 mm	100	99
65 mm	100	98
100 mm	Trace	96
150 mm	99	90
200 mm	97	86
325 mm	88	79

limes have setting times to half volume (ASTM C 110) of 150, 350, and 450 min, respectively. The Arizona lime (HD2) has the highest loose density followed by the quicklime and the two hydrated limes.

The most significant differences between the two hydrated limes are their densities and specific gravities. The loose densities for the California lime (HD1) and the Arizona lime (HD2) are 28 and 86 and their specific gravities are 2.23 and 2.63, respectively.

TEST PROGRAM AND PROCEDURES

The test program included selection of an appropriate aggregate gradation, two mix designs to determine optimum asphalt content, and methods for adding lime as well as determination of compactive efforts necessary to produce a wide range of air voids. When the test program had been defined, all samples were tested as outlined in the test sequence shown in Figure 1.

Aggregate Gradation

The aggregate gradation attempted to meet three standard specifications: Nevada DOT Type 2, Caltrans 3/4-in. maximum size (operating range), and ASTM D 3515 dense mixture (1/2-in. maximum size) *(2–4)*. It should be noted that the 1/2-in. sieve size is slightly out of range on the Nevada DOT Type 2 and the Caltrans 3/4-in. maximum size. This was unavoidable because of the lack of overlap between the specifications at this sieve size.

The same gradation was used for aggregate from both sources. The sieve analysis of the aggregate gradations is given in Table 6. Because lime acts as a mineral filler as well as an admixture, a gradation was done for both aggregates with the HD1 lime added to dry aggregate (Table 6).

To control the fines, aggregate from both sources was sieved into individual sieve sizes. All aggregate above the No. 30 sieve was washed before sample preparation.

Mix Designs

Mix designs were completed for aggregate from both sources as outlined by the Asphalt Institute *(5)*. Samples were compacted according to ASTM D 1559 using 50 blows per side. The optimum asphalt content for the Nevada aggregate (Aggregate 1) was 6.5 percent by total weight of mix. The optimum asphalt content for the Arizona aggregate (Aggregate 2) was 7.0 percent by total weight of mix.

Preparation of Aggregate-Lime Treatments

Four methods of introducing the lime into the mixture were used although not all methods were used for every lime product. Each method of treatment used 1.5 percent lime by dry weight of aggregate. All aggregate was dried at 230°F for a minimum of 15 hr before treatment. Six samples were prepared for use in testing each variable.

The procedures for introducing the lime into the mixture, the lime products, and the aggregates used for each method were as follows.

1. Dry lime was added to cold aggregate, mixed well to coat the aggregate, then reheated before mixing. Quicklime, dolomitic lime, and both hydrated limes were added to both aggregates.
2. Lime was added to the asphalt before mixing with aggregate. The quicklime was the only lime added by this method, and only Aggregate 1 was used.
3. Lime was combined with water in a four-to-one ratio,

TABLE 5 CHEMICAL PROPERTIES OF LIME PRODUCTS

Property	Hydrated Lime 1 (%)	Hydrated Lime 2 (%)	Quicklime (%)	Dolomitic Lime (%)
Acid insoluble	1.5	1.0	2.0	0.5
Iron oxide	0.1	0.05	0.20	0.20
Aluminum oxide	0.5	0.2	0.7	1.0
Magnesium carbonate	–	0.5	–	–
Calcium carbonate	1.5	–	1.5	2.0
Calcium oxide	Nil	–	92.0	–
Magnesium oxide	1.0	–	2.0	1.0
Magnesium hydroxide	–	–	–	40.0
Calcium hydroxide	93.0	94.0	1.0	56.0
Moisture	0.5	–	Nil	0.5
ASTM available lime	91.5	92.0	91.0	–

NOTE: Dashes = data not available.

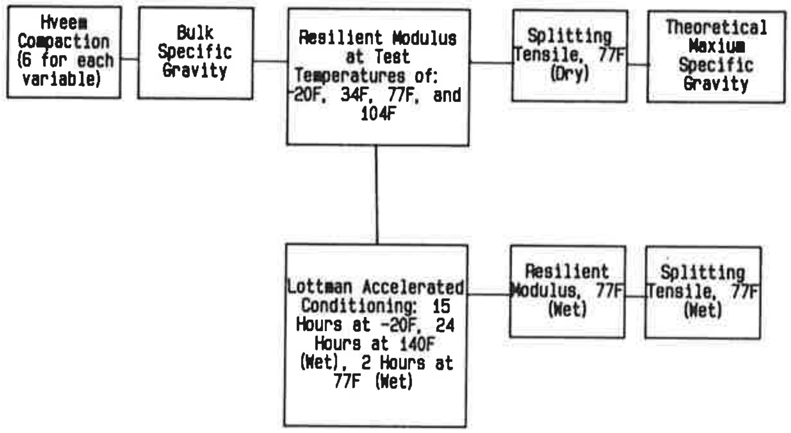

FIGURE 1 Test sequence flow chart.

then mixed with cold aggregate. The lime-aggregate mixture was then dried again at 230°F for a minimum of 15 hr before mixing. The quicklime was the only lime added by this method, and only Aggregate 1 was used.

4. Water (6 percent by dry weight of aggregate) was added to cooled oven dry aggregate before the lime was added. The lime-aggregate mixture was then dried again at 230°F for a minimum of 15 hr before mixing. The California hydrated lime (HD1) was the only lime added by this method, and only Aggregate 1 was used.

Several details were noted during preparation of lime treatments. During the mixing of dry lime with dry aggregate, it was noted that the California hydrated lime (HD1) coated the dry aggregate thoroughly whereas the quicklime failed to completely coat the coarse aggregate. Some foaming was noted during mixing the quicklime with asphalt; thickening of the asphalt and settlement of the lime to the bottom of the asphalt were also observed. An extensive exothermic reaction was noted when the quicklime and water were combined; to keep the reaction to a minimum, the quicklime was added in small quantities. A hard crust was noticed on the surface of the quicklime slurry–aggregate mixture after it was removed from the oven. This crust had to be broken before asphalt could be added.

TABLE 6 GRADATION OF AGGREGATES WITH AND WITHOUT HYDRATED LIME

	Percentage Passing			
	Aggregate 1		Aggregate 2	
Sieve Size	Without Lime	With Lime	Without Lime	With Lime
3/4 in.	100	100	100	100
1/2 in.	94.7	94.7	94.9	95.2
3/8 in.	72.4	72.0	72.7	72.9
No. 4	52.2	52.6	52.1	53.1
No. 8	36.2	37.4	36.5	37.6
No. 16	30.2	31.4	30.2	31.0
No. 30	19.8	22.1	16.4	16.7
No. 50	13.1	14.7	10.1	10.4
No. 100	9.0	10.1	6.7	7.0
No. 200	5.8	6.4	4.1	4.3

NOTE: Washed aggregate was batched with and without lime, and a sieve analysis was then performed.

Compactive Effort

Two Hveem compactive efforts were used to produce a wide range of air voids. The 100 percent compactive effort as described in ASTM D 1561 (150 strokes at 500 psi and a leveling load of 12,600 lb) produced between 1 and 3 percent air voids. The 95 percent compactive effort (ASTM D 1561 modified by using 30 strokes and 250 psi and a leveling load of 11,600 lb) produced between 4 and 8 percent air voids.

All samples used in the evaluation of the effects of methods of adding lime, lime products, and aggregate sources were prepared with the 95 percent compactive effort. Six samples were prepared with California hydrated lime (HD1) added dry, quicklime added dry, slurried quicklime, the control mix, and Aggregate 1 with the greater (100 percent) compactive effort.

Test Sequence

Six samples for evaluating each variable were prepared and tested according to the sequence shown in Figure 1. The testing procedures are outlined next.

After mixing, samples were heated at 140°F for 15 hr and then compacted according to ASTM D 1561 and modified ASTM D 1561. Bulk specific gravities were determined and the samples were then stored overnight at 77°F. Resilient modulus values were determined for sample temperatures of −20°F, 34°F, 77°F, and 104°F. Testing was performed according to ASTM D 4123; the load cycle was 0.1 sec applied load with a 3-sec pause between loads. The samples were then divided into two groups of three samples each.

Splitting tensile strength was determined for the first group of three samples. Theoretical maximum specific gravities were determined by ASTM D 2041; a correction was made for absorptive aggregates *(4)*.

The second group of three samples was subjected to one cycle of the Lottman accelerated conditioning procedure *(6)*. Resilient modulus values and splitting tensile strengths were determined for wet samples at a test temperature of 77°F.

TABLE 7 TEST RESULTS FOR METHODS OF ADDING QUICKLIME (Aggregate 1)

	Method of Adding Lime					
Test	QL to Dry Aggregate	QL to Asphalt	Slurried QL	HD1 to Dry Aggregate	HD1 to Wet Aggregate	Control
Resilient modulus (ksi) at						
−20°F	5,960	6,621	7,585	–	5,591	8,208
34°F	4,725	4,372	4,419	–	3,005	4,354
77°F	521	570	449	292	404	467
104°F	69	78	56	–	46	43
Resilient modulus (ksi) after one cycle of Lottman accelerated conditioning						
77°F, dry	521	570	449	292	404	467
77°F, wet	184	82	739	260	316	150
Ratio	35	14	165	89	78	32
Tensile Strength (psi) at						
77°F, dry	213	222	162	90	106	176
77°F, wet	70	43	192	70	117	131
Ratio	33	20	118	78	110	74
Air voids (%)	3	4	1	6	4	2

NOTE: Dashes = data not available.

TEST RESULTS

Test results are discussed in terms of the variables investigated.

Methods of Adding Lime

There was little difference in the temperature susceptibility of the five mixtures as evidenced by the negligible variations in stiffness at any given temperatures (Table 7).

The best retained resilient modulus values after one cycle of conditioning were achieved by the slurried quicklime with a resilient modulus ratio greater than 100 percent. The hydrated lime added to dry and to wet aggregate produced the next best results with resilient modulus ratios of 89 and 78 percent, respectively. Adding the quicklime to dry aggregate did not improve the mix and adding it to the asphalt actually decreased the resilient modulus ratio compared with the control. Test results are given in Table 7 and shown in Figure 2; the ratios are shown in Figure 3.

The best tensile strength ratio after one cycle of conditioning was achieved by the slurried quicklime, the hydrated lime added to wet aggregate, and hydrated lime added to dry aggregate. The quicklime added to dry aggregate and to the asphalt actually decreased the tensile strength ratios. The test results are given in Table 7 and shown in Figure 4; the ratios are shown in Figure 3.

Types of Lime

There was little difference in the temperature susceptibility of the five mixtures (Table 8).

The best retained resilient modulus values were obtained

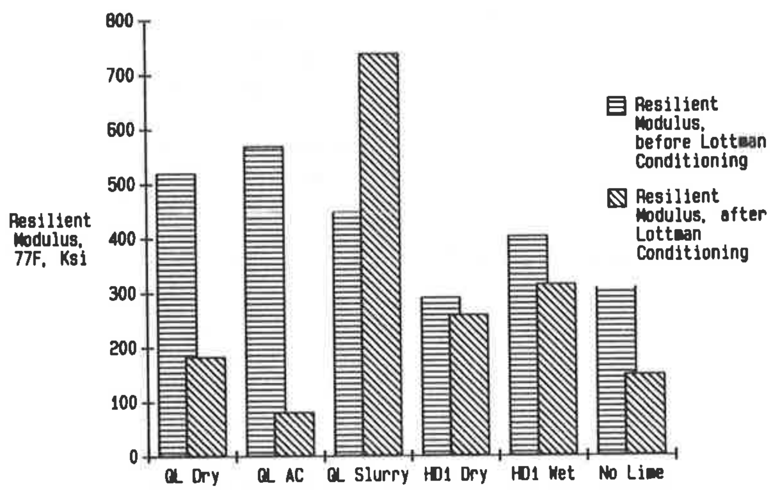

FIGURE 2 Resilient modulus for various methods of adding lime (Aggregate 1).

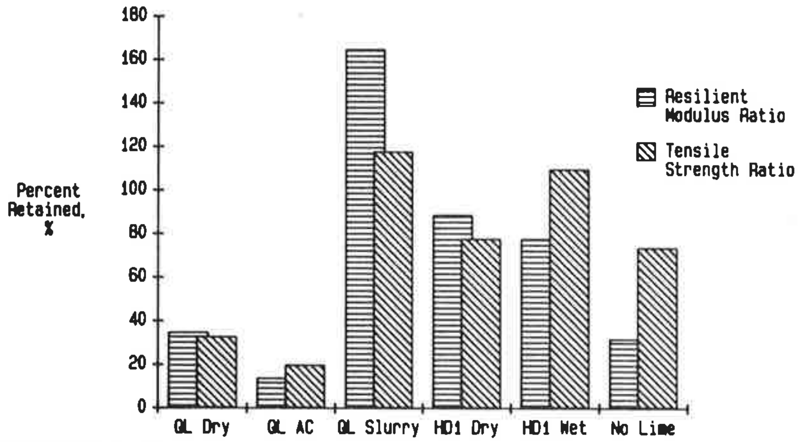

FIGURE 3 Percentage retained after Lottman cycling for various methods of adding lime (Aggregate 1).

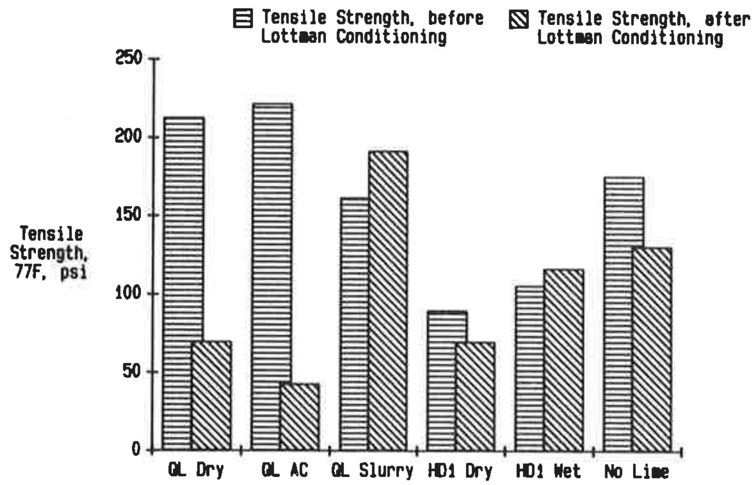

FIGURE 4 Tensile strengths for various methods of adding lime (Aggregate 1).

TABLE 8 TEST RESULTS FOR VARIOUS TYPES OF LIME (added to dry Aggregate 1)

Test	Types of Lime				
	Hydrated Lime 1	Hydrated Lime 2	Quicklime	Dolomitic Lime	Control
Resilient modulus (ksi) at					
−20°F	–	4,749	–	3,861	–
34°F	–	4,097	–	3,861	–
77°F	292	337	237	401	231
104°F	–	40	–	42	–
Resilient modulus (ksi) after one cycle of Lottman accelerated conditioning					
77°F, dry	292	337	237	401	231
77°F, wet	260	237	–	221	93
Ratio	89	71	–	55	41
Tensile strength (psi) at					
77°F, dry	90	115	65	99	73
77°F, wet	70	64	25	67	34
Ratio	78	56	39	67	47
Air voids (%)	6	7	8	5	8

NOTE: Dashes = data not available.

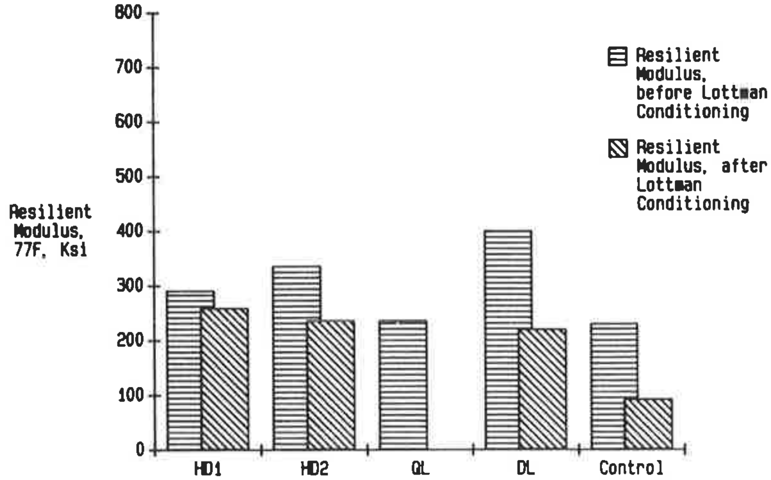

FIGURE 5 Resilient modulus for various types of lime (Aggregate 1).

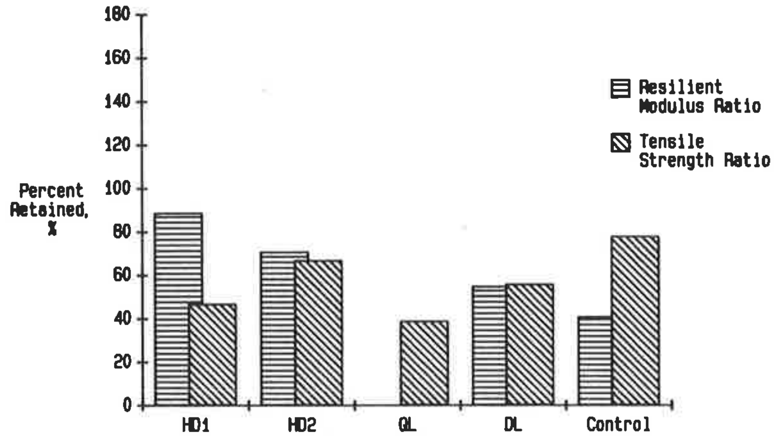

FIGURE 6 Percentage retained after Lottman cycling for various types of lime added dry to Aggregate 1.

with the hydrated limes. Although the dolomitic lime produced a resilient modulus value, before soaking, that was approximately equal to that of the hydrated limes, the original strength was higher and the resilient modulus ratio was therefore reduced. Test results are given in Table 8 and shown in Figure 5; the ratios are shown in Figure 6.

The HD1 produced slightly lower before-conditioning values than did the HD2 but produced higher strengths after conditioning. The difference in strengths might not be due to differences in lime products but to the volume of lime present in the mixtures. HD2 has a significantly higher specific gravity than does HD1; this difference in specific gravities results in a lower volume of HD2 when limes are added on a weight basis.

The best tensile strength ratios as shown in Figure 6 were

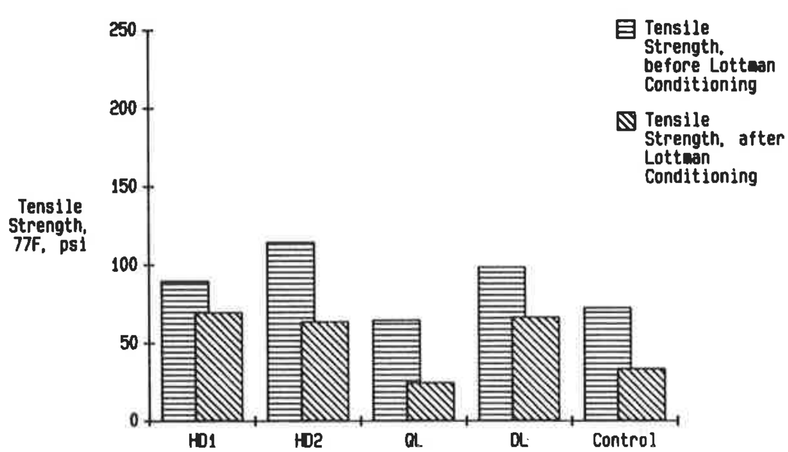

FIGURE 7 Tensile strengths for various types of lime added dry to Aggregate 1.

TABLE 9 TEST RESULTS FOR AGGREGATES FROM VARIOUS SOURCES (hydrated lime was added to dry aggregate)

Test	Aggregate 1			Aggregate 2		
	HD1	HD2	Control	HD1	HD2	Control
Resilient modulus (ksi) at						
−20°F	–	4,749	–	6,271	5,600	5,600
34°F	–	4,097	–	3,853	4,271	3,173
77°F	92	337	231	269	224	198
104°F	–	40	–	33	27	32
Resilient modulus (ksi) after one cycle of Lottman accelerated conditioning						
77°F, dry	292	337	231	269	224	198
77°F, wet	260	237	93	310	219	139
Ratio	89	71	41	118	99	70
Tensile strength (psi) at						
77°F, dry	90	115	15	100	86	90
77°F, wet	70	64	34	159	109	84
Ratio	78	56	47	160	126	93
Air voids (%)	6	7	8	5	4	7

NOTE: Dashes = data not available.

produced by HD1 and dolomitic lime. Although the ratios range from 78 to 56 percent for both of the hydrated limes and the dolomitic limes, the final tensile strengths for these three groups are approximately the same (Table 8 and Figure 7).

Aggregate from Different Sources

Changing aggregate sources or types of hydrated limes had little effect on the temperature susceptibility of the mixtures (Table 9).

The resilient modulus values for Aggregate 1 before conditioning were slightly higher than those for Aggregate 2 (Table 9 and Figure 8). The resilient modulus values for Aggregate 1 after conditioning were slightly lower than those for Aggregate 2. The HD1 produced slightly better resilient modulus values, both before and after conditioning, than did the HD2. This is, again, probably due to the difference in volumes of lime present in the mixtures. Both limes showed significant improvement over both sets of control samples.

Although Aggregate 2 was not as susceptible to water as Aggregate 1, as shown by the difference in the resilient modulus and tensile strength ratios (Figure 9), the presence of lime in either asphalt-aggregate mixture greatly improves the mix properties.

The tensile strength before conditioning was approximately the same for both aggregates with both hydrated limes (Table 9 and Figure 10). Aggregate 2 showed a significant gain in tensile strength after conditioning. The tensile strength ratios of both aggregates were improved by the addition of lime, although HD1 produced better results.

Air Voids

With one exception, there was a significant drop in resilient modulus values, both wet and dry, as the percentage of air voids increased (Table 10 and Figure 11). The exception was the mixture with hydrated lime 1; the wet resilient modulus was approximately the same regardless of the percentage of air

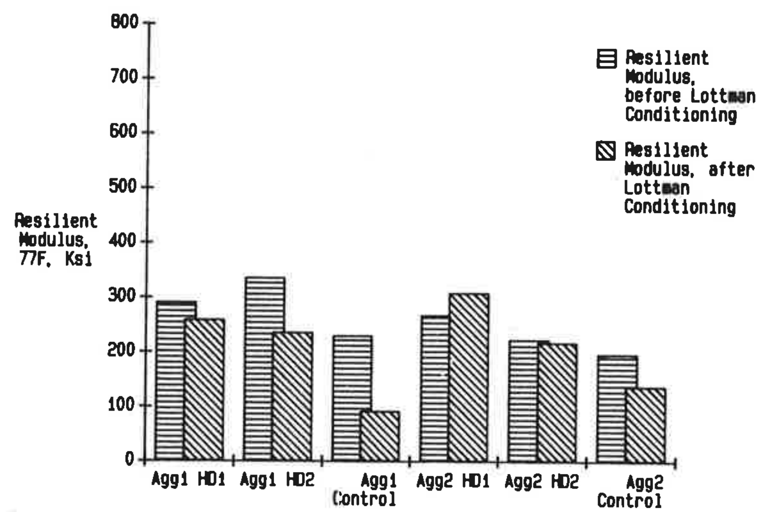

FIGURE 8 Resilient modulus for mixes with aggregate from different sources (lime added to dry aggregate).

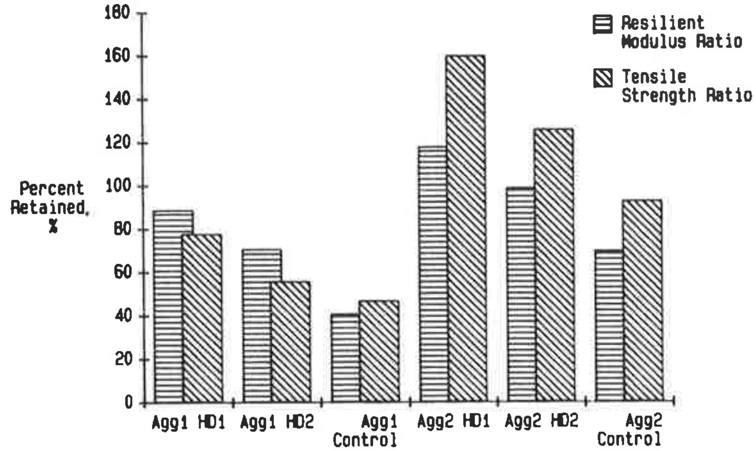

FIGURE 9 Percentage retained after Lottman conditioning for mixes with aggregate from different sources (lime added to dry aggregate).

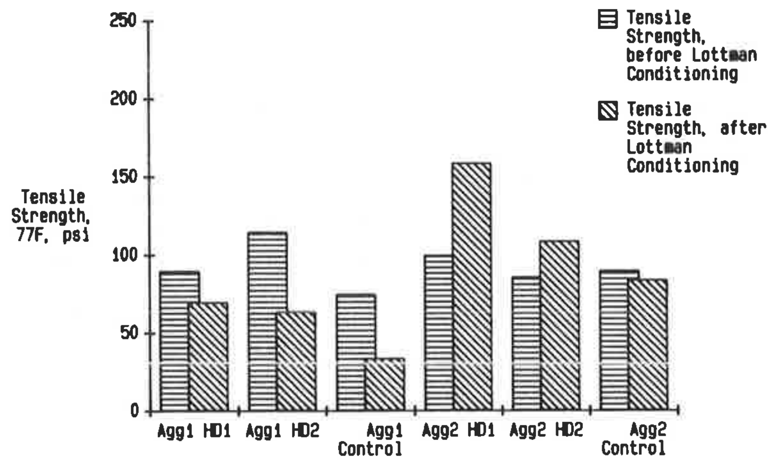

FIGURE 10 Tensile strengths for mixes with aggregates from different sources (lime added to dry aggregate).

TABLE 10 TEST RESULTS FOR VARIATIONS IN AIR VOIDS (Aggregate 1)

	95% Compactive Effort/100% Compactive Effort			
Test	Hydrated Lime 1 with Dry Aggregate	Quicklime with Dry Aggregate	Slurried Quicklime	Control
Resilient modulus (ksi) after one cycle of Lottman accelerated conditioning				
77°F, dry	292/473	238/521	321/449	231/467
77°F, wet	260/240	–/184	452/738	93/150
Ratio	89/51	–/35	142/165	41/32
Tensile strength (psi) at				
77°F, dry	90/180	65/213	127/162	73/176
77°F, wet	70/162	25/70	142/192	34/131
Ratio	78/92	39/33	118/118	47/74
Air voids (%)	6/1	8/4	3/1	8/2

NOTE: Dashes = data not available.

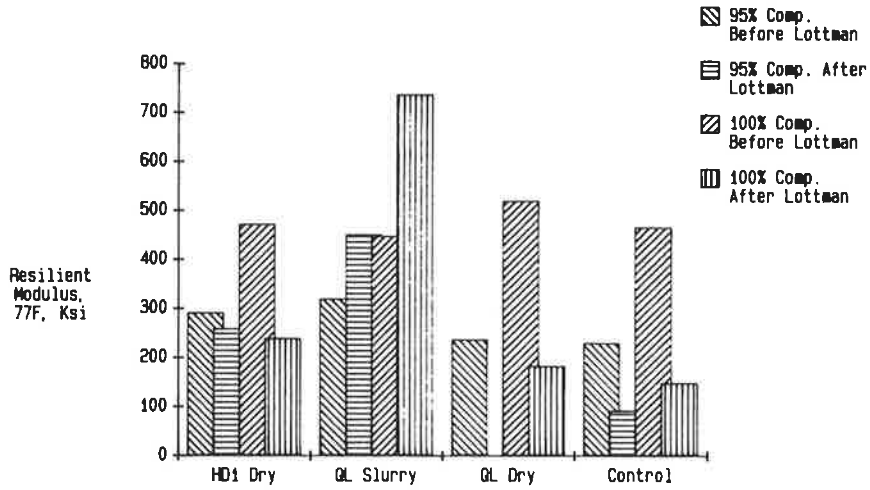

FIGURE 11 Comparison of resilient modulus values for samples prepared with different compactive efforts.

voids. All tensile strengths decreased with increasing air voids (Table 10 and Figure 12).

Although the change in air voids greatly affected the resilient modulus and tensile strength values, there were limited changes in percentage retained values (Figure 13).

CONCLUSION

Four variables that affect the success of lime as an antistripping agent were evaluated: (a) four methods of adding two types of lime to asphalt-aggregates, (b) four lime products, (c) aggregate from two different sources, and (d) air voids.

Effects of the variables on moisture sensitivity were evaluated by testing samples before and after one cycle of the Lottman accelerated conditioning procedure *(6)*. The effects of the variables on temperature susceptibility were evaluated by determining the resilient modulus values at four different test temperatures.

The following conclusions can be drawn from this research:

1. Neither methods of adding lime, types of lime, nor aggregate sources appear to have a significant effect on the temperature susceptibility of the mixtures.
2. Slurried quicklime improves resistance to water damage.
3. Both hydrated lime added to dry aggregate and hydrated lime added to wet aggregate improve resistance to water damage.
4. Adding quicklime to asphalt or adding it dry to aggregate is detrimental to the mixture.
5. Both of the hydrated limes and the dolomitic lime improve resistance to water damage.

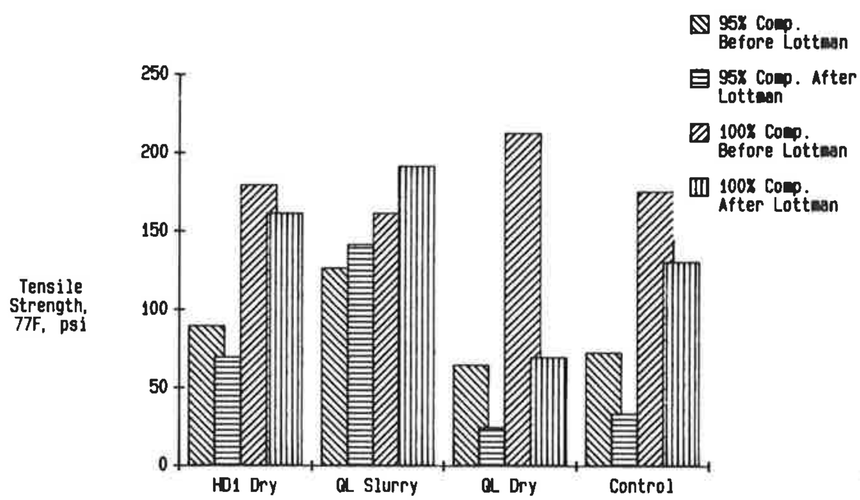

FIGURE 12 Comparison of tensile strengths of samples prepared with different compactive efforts.

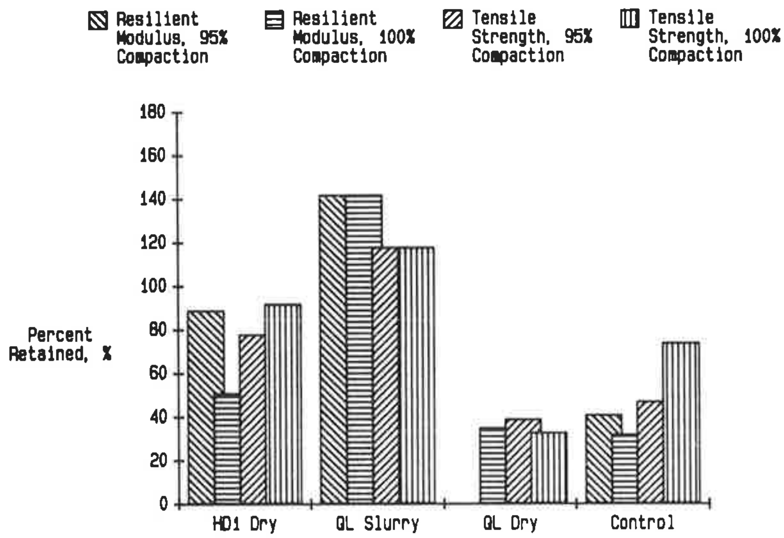

FIGURE 13 Percentage retained after Lottman conditioning for mixes with different air voids.

6. Either hydrated lime added to an asphalt-aggregate mixture, regardless of original moisture susceptibility, improves resistance to water damage.

7. Air voids greatly affect the strength of a mixture but have limited effects on percentage retained strengths.

ACKNOWLEDGMENT

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Evaluation of Asphalt Cements for Low-Temperature Performance

SUI C. LEUNG AND KENNETH O. ANDERSON

The primary objective of this research project was an evaluation of the temperature susceptibility and low-temperature fracture characteristics of asphalt cements from heavy crude sources in western Canada. Six samples of asphalt of grades 85/100 and 200/300 formulated from crude oils from Cold Lake, Lloydminster, and Redwater sources were tested to determine their physical properties. They were also used to prepare Marshall specimens for testing by the low-temperature indirect tensile test method at temperatures of 0°C, −10°C, −20°C, and −30°C. From the results of the laboratory tests, it is concluded that the Redwater asphalt is the most temperature susceptible of the three asphalts studied. It was also confirmed that temperature as well as grade and crude source of asphalts have quite marked effects on the tensile properties of asphalt concrete mixtures. The asphalt cements produced from heavy crude sources of the Cold Lake and Lloydminster areas have been found to perform better at low temperature than those produced from the lighter crude source of the Redwater area. The 200/300 asphalt is also expected to perform better than the 85/100 asphalt. These observations are based on failure strain and stiffness values obtained by means of the indirect tensile test at various temperatures.

Low-temperature cracking of asphalt pavements in cold regions continues to be a major concern for highway and airfield authorities. Improvements in temperature susceptibility and other characteristics of asphalt cements could have a significant impact on performance and costs of these pavements.

The temperature susceptibility of paving asphalt cements has been shown to correlate well with low-temperature behavior of asphalt concrete pavements, particularly with regard to transverse cracking. Asphalt cements produced from heavy crude sources in western Canada have been reported to perform better than those manufactured from lighter crude oils *(1, 2)*.

The primary objective of this research project was an evaluation of the low-temperature performance of asphalt cements produced from different locally available crude sources in western Canada.

Asphalt cements produced from a variety of heavy crude sources were evaluated using conventional physical tests to define rheological properties and temperature susceptibility parameters. Methods used to describe temperature susceptibility over various temperature ranges have been reviewed extensively in the report by Button et al. *(3)*. Four of the mathematical formulas for calculating temperature susceptibility have been used to define these parameters for the asphalt cements studied.

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A major problem with these methods is that physical tests are made at temperatures above the critical low-temperature range for expected pavement cracking. To provide information on the behavior of asphalt concrete mixes at temperatures from 0°C to −30°C, asphalt concrete specimens were prepared and tested to determine low-temperature tensile properties with the indirect tensile test used for many years at the University of Alberta *(4)*.

LABORATORY TESTS OF ASPHALTS

Testing Program

A laboratory testing program of selected asphalts was developed and conducted to fulfill several objectives. The first objective was to develop some test data on asphalts from identifiable crude sources. Such data would make possible comparison of the various common properties of other asphalt cements currently in use. The second objective was to evaluate the temperature susceptibility of asphalt cement produced from different crude oils. Last, attempts were made to evaluate the low-temperature tensile properties of the asphalt mixtures that contained asphalts from the different crude sources.

Two grades of asphalt cement samples from different crude sources, which represent asphalt cements of different temperature susceptibilities, were obtained from Esso Petroleum Canada and Husky Oil of Lloydminster.

Two types of laboratory tests were carried out. Conventional physical tests were carried out to define the rheological properties and temperature susceptibility parameters that were used in evaluating the low-temperature performance of the selected asphalt cements.

The indirect tensile test, as used at the University of Alberta and improved in this project, was used to test asphaltic concrete cylinders prepared from the different asphalt cements. The tensile properties obtained from the tests were also used in evaluating the asphalt cements for low-temperature performance.

Description of Asphalt Cement Samples

Two criteria were used for the selection of asphalt cements for laboratory testing. First, the samples were to represent a hard grade of asphalt cement (85/100) and a soft grade (200/300). Second, the selected samples within a given grade were to represent different temperature susceptibilities.

Asphalt cements manufactured from crude oils from different sources were chosen. The Cold Lake asphalt cement and

TABLE 1 PHYSICAL PROPERTIES OF ASPHALT SAMPLES

	Cold Lake Grade		Lloydminster Grade		Redwater Grade	
	85/100	200/300	85/100	200/300	85/100	200/300
Pen at 25°C (dmm)	95	263	94	254	93	242
Pen at 4°C (dmm)	9.0	19.3	6.8	26	5.7	10.3
Vis at 60°C (Pa/sec)[a]	158.2	43.4	189.0	44.8	52.9	19.5
Vis at 135°C (cSt)	340	187	391	202	169	104
Softening point (°C)	45.0	36.0	44.5	36.0	47.0	42.0
Ductility (cm)	+150	+150	+150	+150	+150	+150

[a]Pa/sec = 10 poise.

TABLE 2 TEMPERATURE SUSCEPTIBILITY PARAMETERS

	Cold Lake Grade		Lloydminster Grade		Redwater Grade	
	85/100	200/300	85/100	200/300	85/100	200/300
PI(dPen/dT)	−1.27	−1.89	−1.93	−1.06	−2.28	−2.96
PI(R&B)	−0.95	−0.61	−1.14	−0.81	−0.40	+1.87
PVN(25-60)	−0.36	−0.01	−0.19	−0.04	−1.56	−1.18
PVN(25-135)	−0.53	−0.25	−0.33	−0.16	−1.61	−1.46

the Redwater-Gulf blend asphalt cement were obtained from Esso Petroleum Canada. The Cold Lake asphalt cement, produced from heavy crude oils, was considered to have low-temperature susceptibility. The Redwater-Gulf blend asphalt cement was specially formulated from lighter crude oils to exhibit high-temperature susceptibility. This blend was chosen in order to have the greatest possible difference in temperature susceptibility among samples. For purposes of comparison, the asphalt cement produced from the heavy crude oils of the Lloydminster area was obtained from Husky Oil of Lloydminster. This asphalt cement was also considered to have low-temperature susceptibility. A total of six asphalt cement samples were chosen for laboratory testing.

Conventional Physical Tests

In this laboratory testing program, only the common physical tests of asphalt cements were carried out on the six samples. The primary emphasis was on the evaluation of the consistency properties of the materials, such as viscosity, penetration, and ductility, and the temperature susceptibility parameters of the materials, such as penetration index (PI) and pen-vis number (PVN).

Standard ASTM testing procedures were used for all of the physical tests. Penetration tests at 25°C and 4°C were made following ASTM D 5 procedures (loading the needle with 100 g for 5 sec). For viscosity tests at 135°C and 60°C, ASTM methods D 2170 and D 2171 were followed.

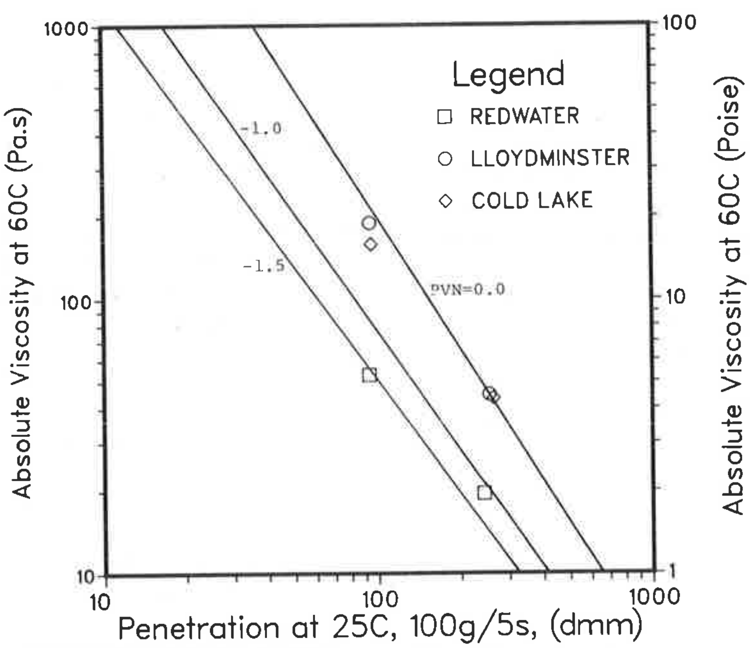

FIGURE 1 Relationship between absolute viscosity at 60°C and penetration at 25°C.

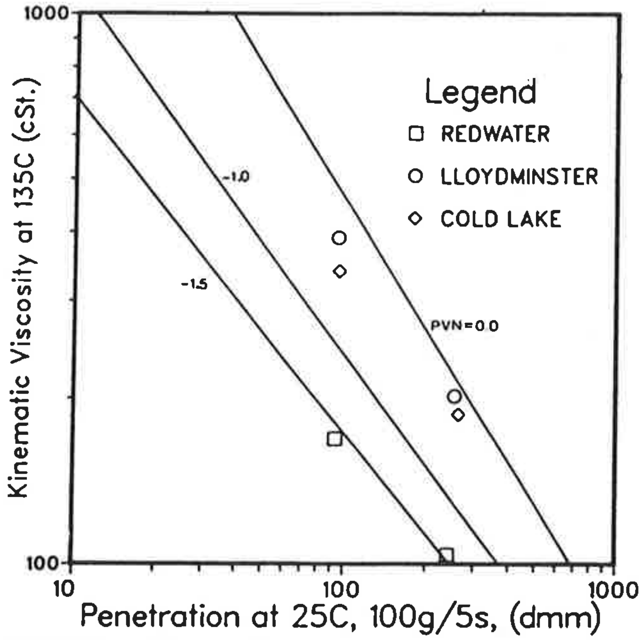

FIGURE 2 Relationship between kinematic viscosity at 135°C and penetration at 25°C.

DISCUSSION OF PHYSICAL TEST RESULTS

Physical Properties

Table 1 gives a summary of the results of the physical tests carried out in this phase of research. Reported values are averages of five individual tests, except for softening point and ductility.

From the table, it is noted that the penetration at 25°C of the asphalts from all sources is quite uniform. However, the penetration at 4°C differs substantially among sources of the same grade. The Redwater-Gulf blend asphalt has the lowest penetration value at 4°C of the three asphalts of each grade. This indicates that the Redwater-Gulf blend asphalt is harder than the Cold Lake and Lloydminster asphalts at low temperature even though they have similar penetration values at 25°C.

The viscosities (at both 135°C and 60°C) of the Redwater-Gulf blend asphalt are particularly low (for both grades) compared with the viscosities of the asphalts from Cold Lake and Lloydminster. The consistency measurements of the Cold Lake and Lloydminster asphalts are similar in most respects.

The Redwater-Gulf blend asphalt, particularly the 200/300 grade, has the highest ring-and-ball softening point of the three asphalts. The ductility of all of the asphalts meets the specification requirement of a minimum extension of 150 cm at 25°C.

Figures 1 and 2 show the absolute viscosity at 60°C and the kinematic viscosity at 135°C plotted against penetration at 25°C. The suppliers report slightly different values, but all are within acceptable multilaboratory precision.

Temperature Susceptibility Parameters

Table 2 gives the temperature susceptibility parameters, PVN(25-60), PVN(25-135), PI[ring and ball (R&B)] and PI(dPen/dT), using data from Table 1 and commonly used equations *(3)*.

The first two parameters are those introduced by McLeod *(5, 6)*, and the latter two are those developed by Pfeiffer and Van Doormal (*7*, p. 414) and Pfeiffer (*8*, p. 161). Numerous other investigators have used these methods to evaluate temperature susceptibility of asphalt cements, and for brevity further descriptions of the methods are not given in this paper.

The Redwater-Gulf blend asphalt is the most temperature susceptible in both grades according to the PVN and PI(dPen/dT) methods. On the contrary, the PI(R&B) method shows that this asphalt is the least temperature susceptible, which, obviously, is not correct. The Redwater-Gulf blend asphalt appears to be a waxy asphalt, which gives false R&B softening points that lead to erroneous values of PI(R&B).

The temperature susceptibilities of the Cold Lake and Lloydminster asphalts are quite similar, and the different parameters do not consistently distinguish the order of their temperature susceptibility. For the 85/100 asphalt samples, PVN-values indicate that the Cold Lake asphalt is slightly more temperature susceptible than the Lloydminster asphalt with a maximum numerical difference of PVN-values of 0.20. However, both the PI(dPen/dT) and the PI(R&B) values indicate the contrary. They indicate that the Lloydminster asphalt is more temperature susceptible as shown by the lower PI-values. The maximum numerical differences are 0.65 and 0.19, respectively.

For the 200/300 asphalt samples, PVN-values indicate that the temperature susceptibilities of both the Lloydminster and the Cold Lake asphalt are quite similar with a maximum numerical difference of only 0.09. However, the values of PI(dPen/dT) indicate that the Cold Lake asphalt is more temperature susceptible than is the Lloydminster asphalt whereas the values of PI(R&B) indicate the contrary. The PI(dPen/dT) of the Cold Lake sample is 0.83 more negative than the value of the Lloydminster sample whereas the PI(R&B) of the Lloydminster sample is 0.20 more.

The values of PVN(25-60) and PVN(25-135) are quite similar, and PVN(25-135) generally has a slightly lower value. The maximum numerical difference is 0.28.

The values of PI and PVN are not equal. The values of PI(dPen/dT) of all of the test samples are substantially lower than the corresponding values of PVN(25-60) and PVN(25-135). The maximum numerical difference is as much as 1.88. This observation is not surprising because Puzinauskas *(9)* has presented test results that show poor correlation between these parameters and others used to describe the temperature susceptibility of asphalts.

The difference may well be because the PI(dPen/dT) employs two penetration readings within the lower temperature range whereas the PVN methods use penetration and viscosity values within a higher temperature range. The lower values of PI(dPen/dT) indicate that the temperature susceptibility of these asphalts is greater at low temperature than at high temperature.

On the basis of these test data it may be observed that the values of PVN(25-135) are good indicators of temperature susceptibility in the high-temperature range; however, the PI using the slope of the log penetration versus temperature is a better indication of the temperature susceptibility of an asphalt over the lower temperature range.

INDIRECT TENSILE TEST

Summary of Method

The indirect tensile test method involves loading an asphalt concrete cylinder via loading strips across a diameter in a compression testing frame and within a controlled temperature chamber maintained at a constant low temperature. Output signals from a load cell and three linear variable differential transformers are recorded on floppy diskette by means of a datalog card installed on a microcomputer. Computer programs to analyze the test data developed for mainframe computers (*10*, p. 157) have been recently updated for microcomputer use (*11*). The raw data recorded in the diskette are processed using the Lotus 1-2-3 spreadsheet program, and the tensile failure stress, strain, stiffness, and stress-strain diagram are obtained.

TABLE 3 AGGREGATE GRADATION

Sieve Size (mm)	Approximate U.S. Standard	Percentage Passing
20	3/4 in.	100
12.5	1/2 in.	99.8
10.0	3/8 in.	95.4
5.0	No. 4	70.6
2.0	No. 10	50.6
0.800	No. 20	38.7
0.400	No. 40	28.9
0.160	No. 100	15.8
0.063	No. 230	11.2

Description of Asphalt Concrete Specimens

Two grades of asphalt from three different crude sources were used in preparing the laboratory asphalt concrete specimens. The rheological properties of these asphalts were described in the section on physical properties.

The laboratory specimens were prepared from locally available aggregates, TBG–Clover Bar 12.5-mm crushed gravel. The gradation of this aggregate is given in Table 3.

In accordance with standard Marshall design procedures (ASTM D 1559), an asphalt content of 6 percent by weight of aggregate was chosen as an approximate optimum content for the asphalt concrete mixtures. Twenty Marshall briquette specimens were fabricated with each of the six different asphalts. Each specimen was fabricated under the same conditions: 50 hammer blows at each end of the specimen and a compaction temperature of 130°C and 135°C, respectively, for the 200/300 and 85/100 asphalts.

The bulk specific gravity of each specimen was then determined by weighing each specimen in air and immersed in water. Groups of five specimens were arranged for testing at different temperatures according to their bulk specific gravities; each group had similar average densities.

Testing Conditions

All of the testing was carried out in accordance with the procedures described by Button et al. *(4)* and McLeod *(6)* at temperatures of 0°C, –10°C, –20°C, and –30°C.

The loading rate of the testing machine was set at a nominal rate of 1.5 mm/min and kept unchanged throughout.

A more comprehensive discussion of the test program and methods of testing is given in the thesis on which this paper is based (*12*, p. 164).

DISCUSSION OF INDIRECT TENSILE TEST RESULTS

Test Results

Table 4 gives a summary of the average stress, strain, and secant stiffness moduli of the test specimens at failure. Figures 3–5 show plots of average failure stress, failure strain, and failure stiffness modulus versus temperature for each of the six different asphalt concrete mixtures.

TABLE 4 AVERAGE FAILURE STRESS, STRAIN, AND STIFFNESS OF TEST SPECIMENS

Crude Source	Test Temp. (°C)	85/100 Failure Stress (kPa)	85/100 Failure Strain ($\times 10^{-4}$)	85/100 Failure Stiffness (mPa)	200/300 Failure Stress (kPa)	200/300 Failure Strain ($\times 10^{-4}$)	200/300 Failure Stiffness (mPa)
Redwater-Gulf Blend	0	1437	47	624	1049	87	236
	-10	2125	11	3895	2073	17	4706
	-20	2133	4	9639	2708	16	3282
	-30	2177	4	10019	2839	5	10242
Lloydminster	0	1258	102	236	660	140	91
	-10	2352	22	2468	1213	54	431
	-20	2554	7	7259	2815	40	1525
	-30	2406	6	7514	2770	4	12070
Cold Lake	0	1383	68	372	722	121	112
	-10	2179	23	1800	1640	28	1235
	-20	2616	6	8183	2424	21	2253
	-30	2710	6	9231	3013	6	9749

Conversions: 1 kPa = 0.145 psi 1 mPa = 145 psi

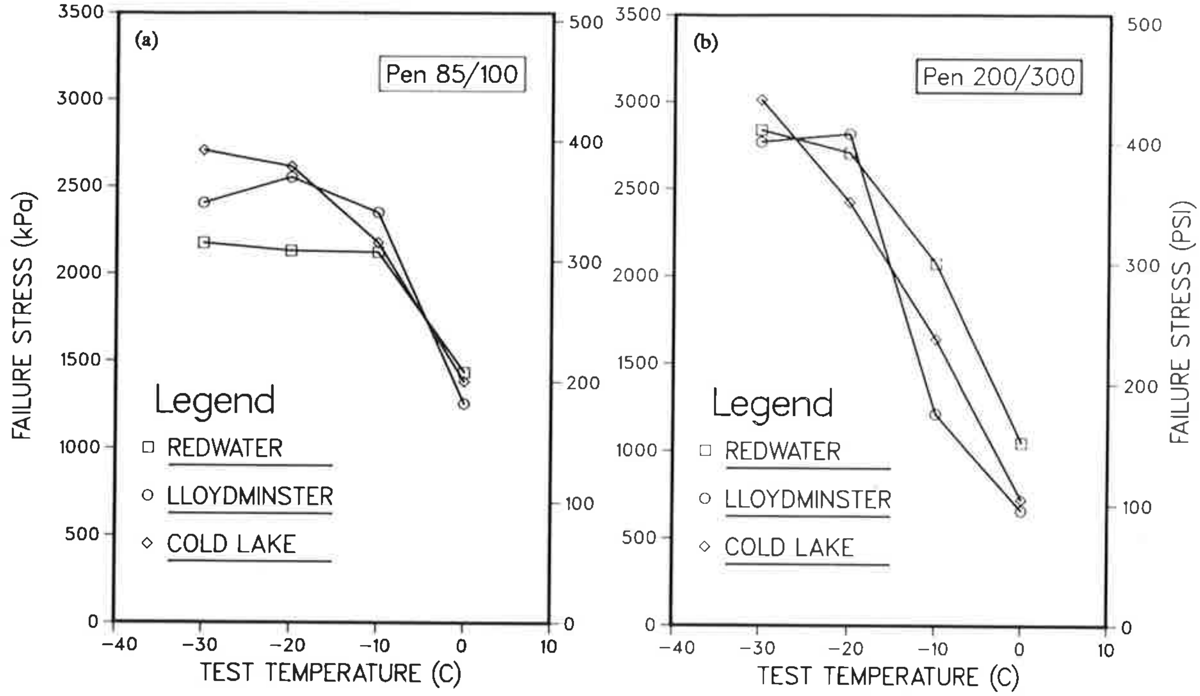

FIGURE 3 Failure stress–temperature relationships.

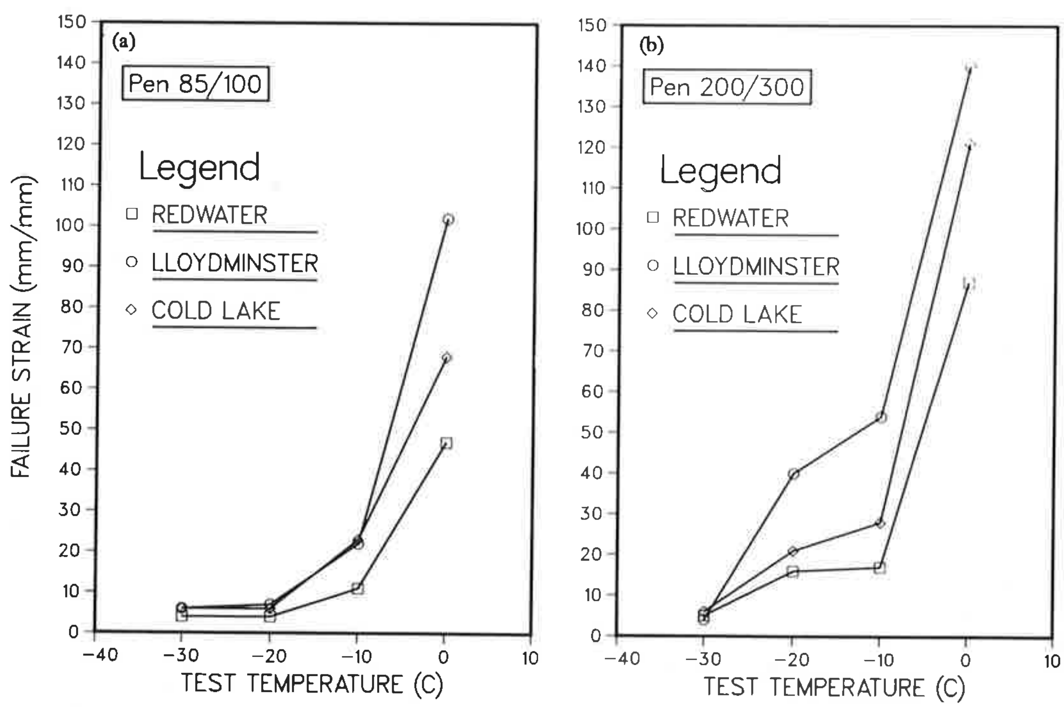

FIGURE 4 Failure strain–temperature relationships.

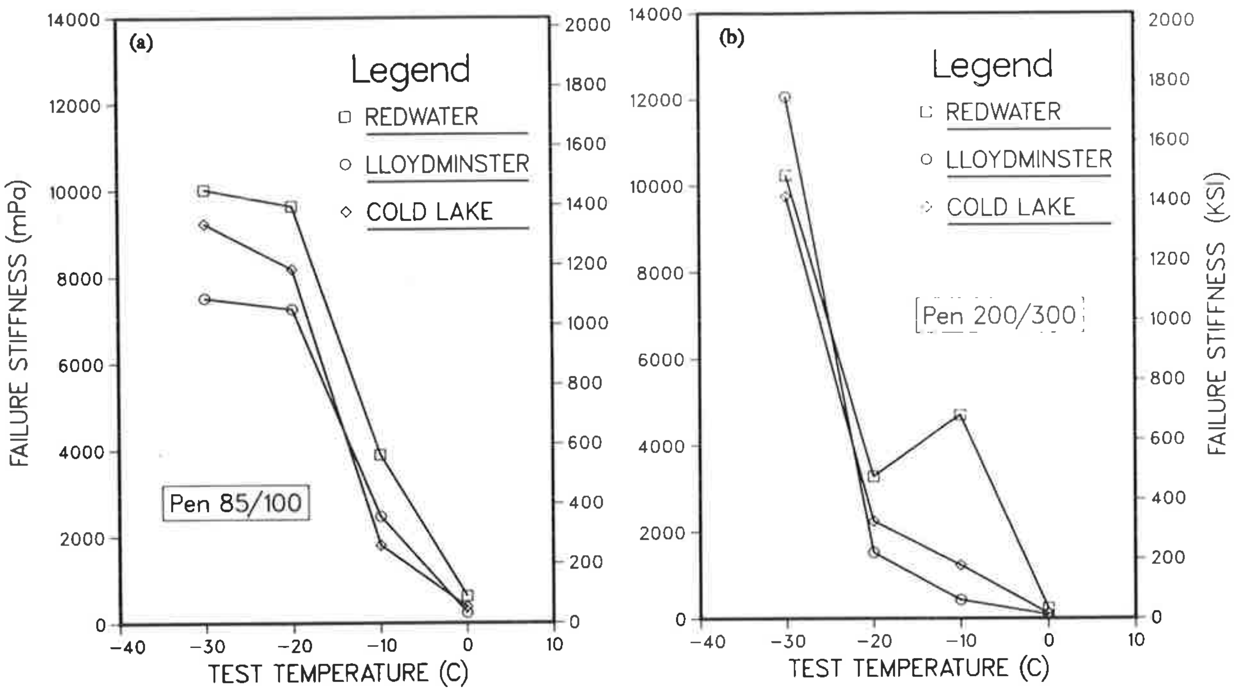

FIGURE 5 Failure stiffness–temperature relationships.

Failure Stress–Temperature Relationships

From the tabulated data and plots, it can be noted that test temperature has a quite significant effect on the failure stress of the asphalt concrete mixtures.

In general, failure stress increases as test temperature decreases. The trend is particularly apparent at moderately cold temperatures (i.e., 0°C to −10°C). At colder temperatures the rate of increase of failure stress with decreasing temperature appears to become smaller. From Figure 3, it can be noted that failure stress ceases to increase as rapidly at test temperatures below −10°C and −20°C for 85/100 and 200/300 asphalt mixtures, respectively.

Failure Strain–Temperature Relationships

The failure strain of the test specimens is also affected remarkably by test temperature. In general, failure strain decreases as test temperature decreases. The rate of decrease is large as the test temperature changes from 0°C to −10°C. The rate of change becomes smaller as the test temperature diminishes further. At very cold temperatures, for example below −20°C, the asphalt specimens show little strain at failure and the failure strain remains relatively constant.

It appears that there is some critical temperature below which failure strain remains relatively unchanged. This critical temperature appears to be a function of asphalt grade. For grade 85/100, this temperature is around −20°C, and for grade 200/300, it is approximately −30°C.

Failure Stiffness–Temperature Relationships

Figure 5 shows failure stiffness–temperature relationships for the two grades of asphalt cement. The stiffness value used has been considered the tensile stiffness modulus taking into account the biaxial state of stress in the cylinder under loading *(4, 5)*.

Failure stiffness generally increases as test temperature decreases. The 85/100 asphalt concrete mixture exhibits a rapid increase in stiffness when the test temperature drops from 0°C to −20°C. The increase in stiffness is only slight when the test temperature drops from −20°C to −30°C.

On the contrary, the 200/300 asphalt concrete mixtures show only slight increase in stiffness during a drop of test temperature from 0°C to −10°C, and there is a rapid increase in stiffness from −2°C to −30°C. There appears to be an anomaly with the Redwater material, which may be due in part to the lack of accuracy of the test method at these lower temperatures.

Effect of Crude Source

Figures 6–8 show the average stress-strain curves of the test specimens with asphalts from different crude source at different test temperatures.

For the 85/100 specimens, the average tensile failure stress is approximately the same irrespective of crude source. However, the average failure strain is markedly smaller for the Redwater asphalt concrete. The difference is greater at 0°C and −10°C and becomes negligible at −20°C and colder.

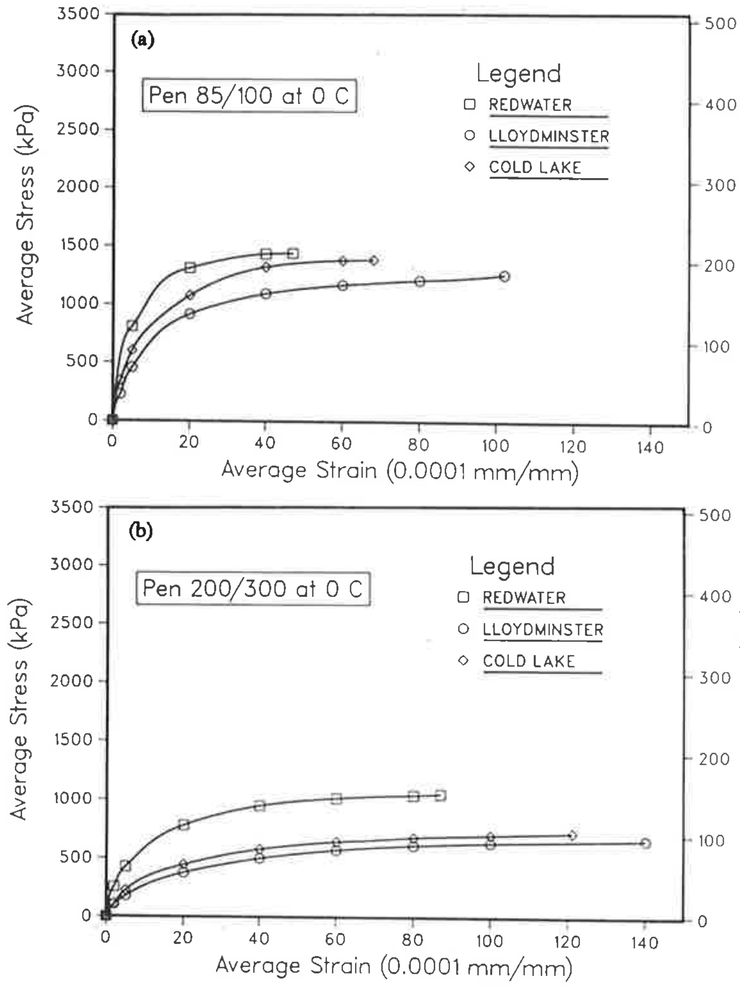

FIGURE 6 Average stress-strain curves at 0°C.

For the 200/300 specimens, the average tensile failure stress of the Redwater asphalt concrete is higher than that of the Cold Lake and the Lloydminster asphalt concretes at 0°C and –10°C. At temperatures of –20°C and colder, the difference becomes smaller. The average failure strain of the 200/300 Redwater specimens is similar to that of the 85/100 grade and is markedly smaller than that of the Cold Lake and Lloydminster mixtures. The difference is greater from 0°C to –20°C and becomes almost zero at –30°C.

The stiffness modulus of the Redwater asphalt concrete as shown in Figure 5 is slightly higher than those of the Cold Lake and Lloydminster mixtures at most test temperatures.

Effect of Asphalt Grade

Comparing Figures 3a and 3b, it can be noted that the failure stress of the softer grade asphalt concrete at 0°C and –10°C is smaller than that of the harder grade. Although this phenomenon is not unexpected, it is interesting to note that at –30°C the phenomenon is reversed and the failure stress of the 200/300 grade is higher than that of the 85/100 grade.

Again, comparing Figures 4a and 4b, it can be noted that the failure strain of the 200/300 grade is generally higher than that of the 85/100 grade except at a test temperature of –30°C. At –30°C the failure strains of both grades are quite similar.

Comparing Figures 5a and 5b, it can be noted that the failure stiffness modulus of the 85/100 grade is greater than that of the 200/300 grade at test temperatures about –20°C. At –30°C the 200/300 mixture becomes stiffer. This is in agreement with the results discussed previously.

Low-Temperature Performance of Asphalts

On the basis of the results of the laboratory tests to determine the tensile properties of asphalt cements and asphalt concrete mixtures, it is believed that the asphalt cements produced from heavy crude sources of the Cold Lake and Lloydminster areas perform better at low temperatures than do those produced from the lighter crude source of the Redwater area.

This observation is justified by the results of the indirect tensile test that show that the Cold Lake and Lloydminster asphalt concrete mixtures can sustain larger strain at failure. This is considered an important property for resistance to thermally induced cracking.

Furthermore, the lower tensile stiffness moduli of the Cold Lake and Lloydminster asphalt concrete mixtures imply that

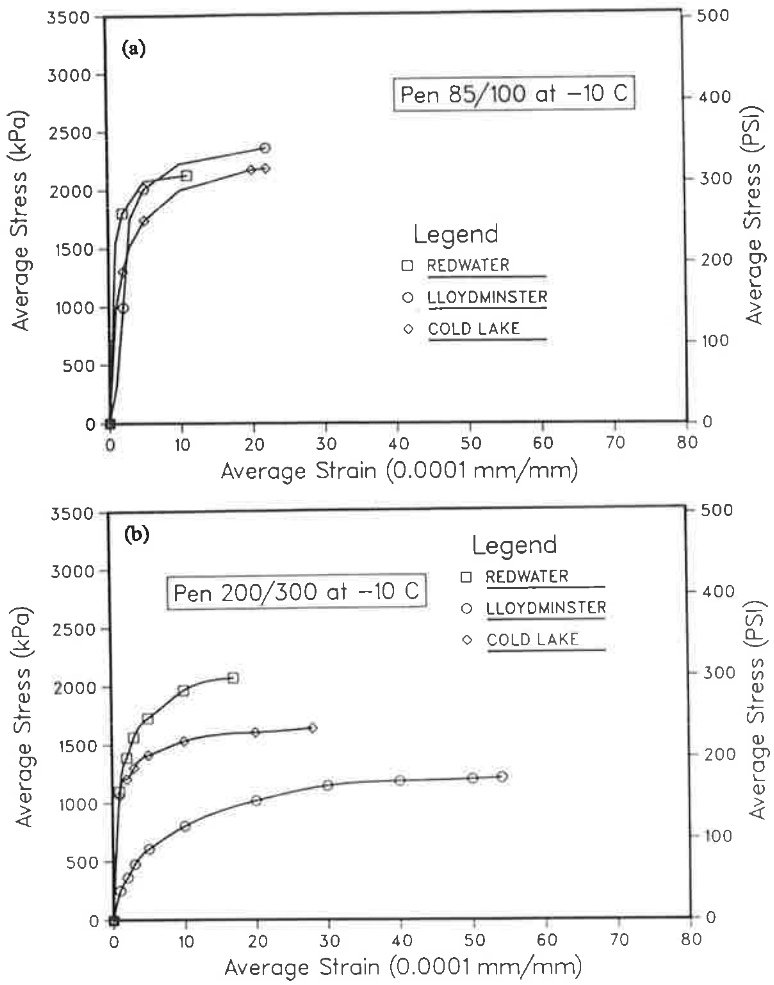

FIGURE 7 Average stress-strain curves at −10°C.

the induced tensile stresses due to temperature change in these mixtures will be smaller. This is advantageous in reducing the chance of thermal cracking because the tensile strength of the mixture will be less likely to be exceeded.

For similar reasons, the performance of the 200/300 asphalt at low temperatures is considered better than that of the 85/100 asphalt.

Data of this type have been used previously to compare observed cracking with that predicted by various stress analyses *(13)*. Such data have also been used with the distress prediction model COLD (COmputation of Low temperature Damage), developed by Finn et al. *(14)*, to estimate the cracking potential of a runway overlay *(11, 15,* p. 156). Further development and similar applications of these data are anticipated.

CONCLUSIONS

Asphalt cements of the same grade but produced from different crude oils possess different rheological properties. In this study, the properties of the Cold Lake and Lloydminster asphalts are found to be similar. The properties of the Redwater-Gulf asphalt are quite different from those of the Cold Lake and Lloydminster asphalts.

The specially formulated Redwater-Gulf blend asphalt is the most temperature susceptible of the three asphalts from the three different crude sources as shown by all of the temperature susceptibility parameters except the PI(R&B).

The PVN(25-135) is a good indicator of temperature susceptibility in the high-temperature range; however, PI using the slope of the log penetration versus temperature is a better indicator over the lower temperature range.

The indirect tensile test method employed in this study provides useful information for the evaluation of low-temperature tensile properties of asphalt cements and asphalt concrete mixtures.

Test temperature has a definite influence on tensile properties of asphalt concrete mixtures. Tensile failure stress increases with decreasing temperature. The rate of increase decreases as the temperature continues to drop.

Tensile failure strain decreases with decreasing temperature. It appears that there is some critical temperature below which failure strain remains unchanged with decreasing temperature. This critical temperature appears to be a function of asphalt grade.

Asphalt concretes made with the same grade but with asphalts from different crude sources have been shown to possess different indirect tensile properties.

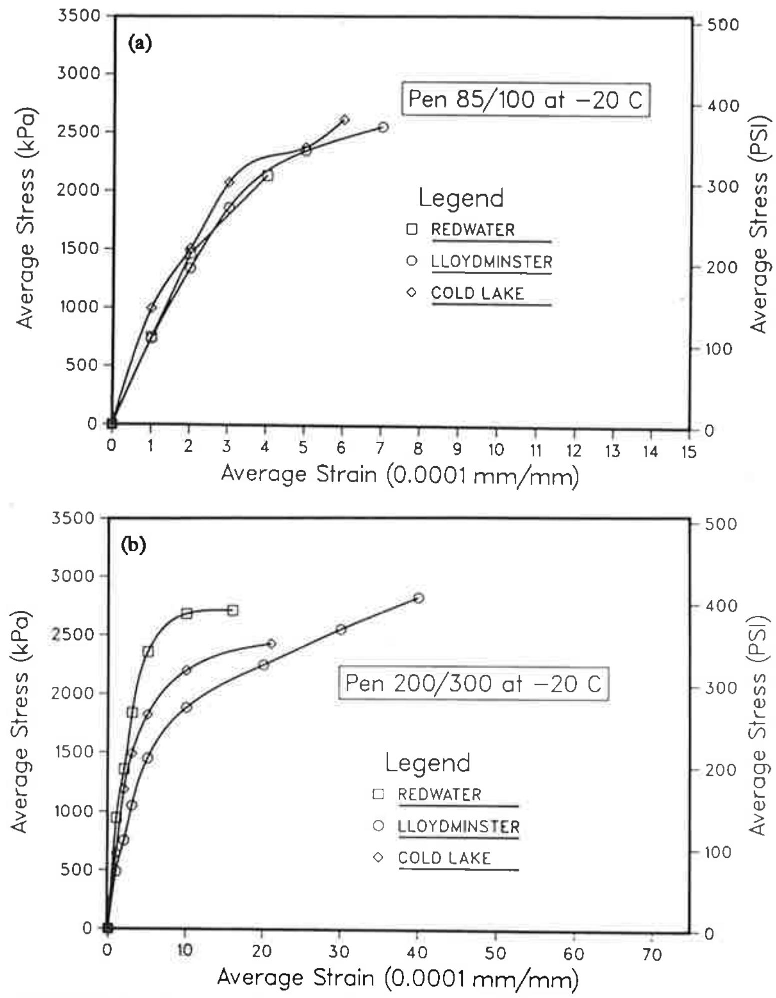

FIGURE 8 Average stress-strain curves at −20°C.

The tensile failure strain of the Redwater asphalt concrete is the smallest of the three different mixtures. The difference diminishes at very low temperatures.

Asphalt concretes made with different grades of asphalts also possess different tensile properties. At moderately cold temperatures, the harder grade asphalt concrete generally has higher failure stress and lower failure strain than does the softer grade. However, at very cold temperatures, the harder grade asphalt concrete has a slightly smaller failure stress, and the failure strains of both grades are similar.

On the basis of the results of the laboratory tests, it is believed that the asphalt cements produced from heavy crude sources of the Cold Lake and Lloydminster areas perform better at low temperature than do those produced from the lighter crude source of the Redwater area. The 200/300 asphalt is also expected to perform better than the 85/100 asphalt, provided both grades are produced from the same crude source.

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The opinions, findings, and conclusions expressed are those of the authors.

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Relationship Among Parameters Derived from High-Performance Liquid Chromatography, Physical Properties, and Pavement Performance in Texas

A. F. Stock and Joe Button

The performance of asphalt cement in pavement mixes has aroused considerable interest and debate in the past few years. As a result of this, the Texas State Department of Highways and Public Transportation has sponsored studies to compare the performance of asphalt cement from different sources through a series of full-scale trials and associated laboratory studies. The laboratory studies include penetration and viscosity tests on the asphalt cement, from which temperature susceptibility parameters have been derived; mix parameters such as void content and resilient modulus; and state-of-the-art chromatography. The purpose of this paper is to make the results obtained to date available to other researchers and to draw some tentative conclusions about the interrelationship among the various tests that have been performed.

Pioneering work in relating chromatograms of asphalt fractions to the performance of pavements was sponsored by the Texas Highway Department [now the State Department of Highways and Public Transportation (SDHPT)] as early as 1963. The results of this study were reported to the Association of Asphalt Paving Technologists at their 1970 annual meeting and subsequently published in the proceedings *(1)*. This early research showed that the technique can be useful in studying the behavior of asphalt binders in highway pavements, though the authors did note on occasion that "there is really no satisfactory explanation for the results."

In 1981 the Texas SDHPT sponsored a study of the performance in pavements of asphalt cements from several sources. Test pavements were constructed in three different locations selected to represent different climatic zones within Texas. This study included high-performance liquid chromatography (HPLC) analysis of the asphalt cements as well as a program of measurements of the properties of the asphalt cements and the mechanical properties of the mixtures.

The purpose of this paper is to provide a preliminary analysis of the results. This analysis will be directed toward an assessment of the value of the HPLC technique for evaluating asphalt cements in comparison with physical tests. In addition, the experience gained from these studies will be used to make recommendations on how to proceed with this type of investigation in order to maximize the value of the information that can be obtained.

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It is expected that this will be the first of a series of reports from the Texas Transportation Institute because the SDHPT has continued their sponsorship of this research to gain the maximum benefit of a long-term evaluation of their pavements.

TEST SECTIONS

The three test sites are located at Dickens, Dumas, and Lufkin, Texas, as shown in Figure 1. The study included AC-10 and AC-20 asphalts, which had widely varying chemical and physical properties, from five different Texas sources. Sources are labeled A through E, though neither all grades nor all sources of asphalt cement were used at all sites. Table 1 gives the sources of asphalt cement and the places from which samples were taken. The coding system used for presenting results in the following tables is introduced in Table 1.

FIGURE 1 Location of asphalt cement test sections.

TABLE 1 SUMMARY OF SOURCES OF ASPHALT CEMENT

Asphalt Cement Source and Grade	Code Number							
					Recovered Material			
	New Asphalt from				Laboratory Compacted from		Cores from	
	Refinery[a]	Dickens	Dumas	Lufkin	Dumas	Lufkin	Dumas	Lufkin
A AC-10	1	11	17	–	30	–	38	–
A AC-20	2	12	18	24	–	36	–	40
B AC-10	3	–	19	25	31	–	–	–
B AC-20	4	13	–	–	–	–	–	41
C AC-10	5	–	20	–	32	–	39	–
C AC-20	6	14	–	26	–	–	–	42
D AC-10	7	–	21	27	33	–	–	43
D AC-20	8	–	–	28	–	37	–	44
E AC-10	9	15	22	–	34	–	–	–
E AC-20	10	16	23	29	35	–	–	45

[a]These samples were obtained directly from the refineries several weeks before construction of the pavements in Dickens and Dumas, Texas. Test pavements in Lufkin were installed about 1 year after those in Dickens and Dumas.

The aggregate used on all sites was locally available crushed rock that met Texas SDHPT specifications. The Dickens aggregate was of siliceous mineralogy and nonabsorptive. At Dumas and Lufkin, the aggregates were absorptive; the Dumas rock was a limestone and the Lufkin rock a combination of limestone and sandstone. At a given location, the same equipment was used to mix, place, and compact the test pavements in the conventional manner. Further details of the construction processes can be found elsewhere *(2)*.

PHYSICAL TESTS

Asphalt Cement

The following tests were performed on the asphalt cements:

- Penetration at 39.2°F and 77°F and viscosity at 77°F, 140°F, and 275°F on new asphalt cement and on that recovered from cores and
- Percentage loss, penetration at 77°F, and viscosity at 140°F on neat asphalt after exposure in the thin-film oven.

Mixes

The following tests were conducted, when possible, on samples compacted in the laboratory and recovered from the field:

- Bulk specific gravity and resilient modulus at −13°F, 33°F, 77°F, and 104°F and
- Indirect tension at 77°F, Rice specific gravity.

RESULTS OF ASPHALT CEMENT TESTS

The study has generated a considerable volume of data on conventional physical properties and of HPLC chromatographic data on the asphalt cements. Chromatograms were normalized by a computer program developed by the chemical engineers working with HPLC at the Texas Transportation Institute. The method normalizes the data by adjusting the areas of the individual chromatograms based on the area of one that is selected arbitrarily as a standard. For the purposes of interpretation, each chromatogram has been divided into three sections. These are referred to as "early," "intermediate," and "late" elution times rather than large, medium, and small molecular size. This was done because there is some doubt *(3)* about the reliability of calibration of HPLC data with respect to molecular size.

Results of the chromatographic study are presented as percentages of total area under the portions of the curves described as early, intermediate, and late fractions and are given in Table 2. This table also includes two parameters derived from physical tests. The penetration index *(PI)* *(4)* is determined from Equations 1 and 2:

$$PI = \frac{20 - 500A}{1 - 50A} \qquad (1)$$

where

$$A = \frac{\text{Log } pen \text{ at } T_1 - \text{Log } pen \text{ at } T_2}{T_1 - T_2}, \qquad (2)$$

$pen\ T_1$ = penetration at temperature T_1, and
$pen\ T_2$ = penetration at temperature T_2.

Viscosity-temperature susceptibility *(VTS)* *(5)* is determined from Equation 3 using viscosities at 140°F and 275°F:

$$VTS = \frac{\log \log (n_2) - \log \log (n_1)}{\text{Log } T_1 - \text{Log } T_2} \qquad (3)$$

where

n_1 = viscosity at T_1 (poises),
n_2 = viscosity at T_2 (poises), and
T = temperature (degrees Kelvin).

Tables 3 and 4 give the results of the physical tests performed on the asphalt cements. Table 5 gives the results of mechanical tests on selected paving mixtures. Table 6 gives the results from the thin-film oven tests.

PAVEMENT PERFORMANCE

Dickens

The test pavements appeared to be satisfactory after construction. After 1 year in service, no major distress was evident, but the surface appeared dry and coarse. As a result of this, a fog seal was applied after 3 years. There were no visually discernible differences in performance between the asphalts from the different sources or of different grades.

Dumas

The pavement appeared to be satisfactory after construction. Within 2 months, the sections with Asphalts B and C had begun to ravel severely; in some cases, raveling reached the base. At the end of 1 year of service, the section with Asphalt C was replaced and the section with Asphalt B was partly replaced. No further deterioration was evident after 3 years.

Lufkin

The pavement appeared to be satisfactory after construction and showed no distress on inspection after 1 and 2 years of service.

TABLE 2 CHROMATOGRAPHIC AND TEMPERATURE SUSCEPTIBILITY DATA

Asphalt Source (code no.)	Percentage			*PI*	*VTS*
	Early	Intermediate	Late		
1	71.3	24.6	4.1	1.01	3.52
2	71.6	24.2	4.2	1.54	3.19
3	49.4	44.1	6.5	1.5	3.42
4	48.5	43.8	7.7	5.53	3.44
5	42.7	46.5	10.8	1.26	3.60
6	43.7	46.6	9.7	3.58	3.72
7	50.8	40.6	8.6	3.56	3.38
8	51.8	39.9	8.3	3.79	3.44
9	45.0	51.0	4.0	1.53	3.66
10	43.0	46.8	10.2	3.14	3.70
11	45.4	44.5	10.1	0.31	3.20
12	70.8	25.0	4.1	−0.53	3.08
13	54.3	39.6	6.1	4.39	3.47
14	44.7	47.4	7.9	0.43	3.70
15	41.1	49.7	9.2	1.44	3.70
16	42.1	49.4	8.5	0.59	3.68
17	71.7	24.2	4.1	0.22	3.07
18	73.5	23.6	2.9	2.85	3.16
19	51.6	41.2	7.2	3.22	3.28
20	41.0	50.1	8.9	1.32	3.58
21	49.9	44.8	5.3	3.07	3.41
22	41.0	49.8	9.2	1.81	3.64
23	42.1	48.8	9.1	0.93	3.76
24	65.5	29.7	4.8	1.88	3.26
25	57.4	36.6	6.1	–	3.27
26	43.3	49.0	7.7	−1.2	3.65
27	46.7	44.3	9.0	3.31	3.51
28	49.0	42.7	8.3	3.18	3.49
29	44.9	47.7	7.9	0.65	
30	74.7	24.1	1.2	2.68	3.23
31	51.8	41.5	6.7	8.17	3.46
32	48.4	44.5	7.1	0.26	3.69
33	53.0	39.6	7.4	5.28	3.51
34	45.3	45.8	8.9	2.44	3.71
35	45.5	45.2	9.3	3.21	3.90
36	35.7	50.8	13.5	3.13	3.27
37	53.3	39.2	7.5	−0.83	3.63
38	72.9	24.7	2.4	2.6	3.46
39	45.3	47.4	7.3	4.71	3.51
40	68.9	26.6	4.5	1.44	3.29
41	57.3	38.5	4.2	−0.48	
42	47.6	46.2	6.2	1.64	3.66
43	51.9	42.8	5.3	2.41	3.49
44	54.2	40.3	5.5	1.92	3.48
45	46.4	45.0	8.7	0.68	

DISCUSSION OF RESULTS

Differentiation Between Grades of Asphalt

Table 7 gives the ratio of the three components of the chromatograms for grades AC-10 and AC-20 from the refineries. For Asphalts A through D the ratio for the early and intermediate fractions is quite close to unity; the range is plus or minus 2 parts in 100. A larger variation is evident in the ratio of the late fractions for these asphalts; the range is from minus 16 to plus 11 parts in 100. However, the late fraction makes only a small contribution to the total area and occurs in the part of the curve that tails off and is difficult to define. To assess the precision of this variability, the results of two additional replicate tests on samples of Asphalt Cement A were analyzed. Comparison of the results of the three tests shows that, on the basis of the mean value, the variation in the early and intermediate fractions is plus and minus 1 part in 100. In the late fraction, it is plus 5 minus 9 parts in 100. In addition, the difference between grades for Asphalt Cements A through D is of the same order of

TABLE 3 PHYSICAL TEST RESULTS ON THE ASPHALT CEMENTS

		Rheological Properties									
		Directly from Refinery					As Delivered to Construction Site				
		Viscosity			Penetration		Viscosity			Penetration	
Asphalt Refinery & Grade	Location of Test Pavement	77	140	275	39.2	77	77	140	275	39.2	77
A AC-10	**	0.66	973	2.76	20	106					
	Dickens	-	-	-	-	-	1.35	1220	4.51	15	95
	Dumas	-	-	-	-	-	0.56	958	4.65	16	104
A AC-20	**	3.55	2240	6.42							
	Dickens	-	-	-	-	-	4.00	2180	7.15	8	65
	Dumas	-	-	-	-	-	1.90	2160	6.39	16	61
	Lufkin	-	-	-	-	-	1.80	1730	5.05	16	70
B AC-10	**	0.22	773	2.76	35	166					
	Dumas	-	-	-	-	-	0.36	961	3.63	39	133
	Lufkin	-	-	-	-	-	0.76	932	3.63	25	95
B AC-20	**	1.55	3010	5.33	26	64					
	Dickens	-	-	-	-	-	1.20	2520	4.64	27	77
C AC-10	**	0.66	1268	2.85	16	80					
	Dumas	-	-	-	-	-	0.83	1388	3.06	15	74
C AC-20	**	1.70	2180	3.22	18	58					
	Dickens	-	-	-	-	-	2.75	2580	3.55	7	43
	Lufkin	-	-	-	-	-	1.55	1810	3.19	6	64
D AC-10	**	0.50	930	3.18	35	113					
	Dumas	-	-	-	-	-	0.53	1030	3.21	30	105
	Lufkin	-	-	-	-	-	0.42	1040	2.88	33	111
D AC-20	**	1.11	1810	4.09	26	81					
	Dickens	-	-	-	-	-	2.50	2150	4.53	22	69
	Lufkin	-	-	-	-	-	0.96	1910	3.96	23	79
E AC-10	**	0.88	955	2.34	17	80					
	Dickens	-	-	-	-	-	1.15	1260	2.55	15	72
	Dumas	-	-	-	-	-	0.97	1038	2.48	16	71
E AC-20	**	2.25	1910	3.10	13	45					
	Dickens	-	-	-	-	-	1.90	1520	2.87	9	53
	Dumas	-	-	-	-	-	1.60	2350	3.17	10	54

* Viscosity at 77°F given in poise x 10^6

Viscosity at 140 and 275°F given in poise

** These data are representative of asphalts obtained directly from refineries, ie, not from a specific construction site.

magnitude or less than the difference between replicate samples of original AC-10 asphalt from Source A and so could be attributed to experimental error.

The elution time ratios between grades show larger differences for Asphalt E than for Asphalts A through D. The only fraction that is significant in relation to the replicate tests on Asphalt A is the late fraction.

It should be noted that the penetration and viscosity data (Tables 3 and 4) do not suggest that asphalts from Refinery E are significantly different from the others, but the tests on the residue from the thin-film oven (Table 6) show substantially larger loss, decrease in penetration, and increase in viscosity after testing than do tests on cement from other sources.

Differentiation Among Samples from the Refinery, New Material on Site, and Recovered Material

To date, the test program has provided chromatograms of asphalt cement from the refinery, new material delivered to site,

TABLE 4 PHYSICAL TEST RESULTS ON ASPHALT CEMENTS RECOVERED FROM LABORATORY MIXES AND CORES

		Recovered from Lab Mixes					Recovered from Cores				
		Viscosity			Penetration		Viscosity			Penetration	
Asphalt Refinery & Grade	Location of Test Pavement	77	140	275	39.2	77	77	140	275	39.2	77
A	**										
AC-10	Dickens	12.0	2000	5.93	21	62	17.0	12400	7.29	15	37
	Dumas	1.6	1723	5.24	20	75	2.0	2600	4.81	15	57
A	**										
AC-20	Dickens	18.4	9560	11.0	14	20	12.0	12300	11.8	5	25
	Dumas	3.4	2980	7.13	12	51	5.8	4470	8.41	10	41
	Lufkin	4.5	3780	7.45	15	52	3.55	3470	6.92	10	48
B	**										
AC-10	Dumas	0.56	1360	3.49	57	107	0.7	1450	3.59	30	90
	Lufkin	3.45	3600	5.46	22	57	3.70	3890	6.26	7	56
B	**										
AC-20	Dickens	7.0	11250	8.30	20	38	21.0	9790	5.67	3	17
C	**										
AC-10	Dumas	2.9	3000	3.86	7	45	2.06	2480	4.41	15	62
C	**										
AC-20	Dickens	16.0	8670	6.06	8	21	8.00	5520	8.08	8	32
	Lufkin	3.3	2939	4.40	9	48	3.80	2750	3.87	10	46
D	**										
AC-10	Dumas	1.5	1990	3.98	26	66	1.28	1930	4.04	26	71
	Lufkin	1.28	1870	3.90	19	73	1.20	2420	4.44	16	63
D	**										
AC-20	Dickens	1.40	11400	8.74	21	32	21.0	8670	5.79	2	20
	Lufkin	4.10	5940	5.77	5	45	3.80	2975	4.98	12	52
E	**										
AC-10	Dickens	1.30	4322	4.16	8	28	18.5	23100	9.70	10	21
	Dumas	2.00	1940	3.08	12	47	-	-	-	-	-
E	**						30.0	15500	7.45	0	18
AC-20	Dickens	14.0	4750	4.49	9	29	-	-	-	-	-
	Dumas	5.0	2374	3.42	12	41	14.5	8790	5.06	4	23

* Viscosity at 77°F given in poise x 10^6

Viscosity at 140 and 275°F given in poise

** These data are representative of asphalts obtained directly from refineries, ie, not from a specific construction site.

TABLE 5 RESULTS OF MECHANICAL TESTS ON ASPHALT PAVING MIXTURES

Source Code	Asphalt Content, %	Air Voids, %	Resilient Modulus, psi x 106			
			-13°F	33°F	77°F	104°F
30	6.2	5.5	1.81	1.36	0.318	0.072
31	5.3	8.5	2.02	1.66	0.285	0.080
32	5.7	10.7	2.21	1.35	0.416	0.122
33	5.7	7.5	1.79	1.26	0.318	0.099
34	5.8	6.1	1.87	1.68	0.519	0.141
35	5.6	8.6	1.93	1.39	0.561	0.148
36	5.0	7.4	1.52	1.16	0.422	0.074
38	6.0	6.2	1.60	0.958	0.151	0.033
39	5.4	15.4	1.32	0.918	0.195	0.039
40	5.6	2.6	2.50	1.28	0.254	0.072
42	5.2	3.0	2.58	1.43	0.204	0.045
43	5.7	4.2	2.41	1.64	0.499	0.141
44	4.9	2.2	2.24	1.26	0.165	0.040
45	5.4	3.2	2.60	1.84	0.509	0.082

TABLE 6 THIN-FILM OVEN TEST RESULTS ON AC-10 ASPHALTS FROM REFINERIES

	Asphalt Source				
	A	B	C	D	E
Test 1					
Loss (%)	0	0.04	0.03	0.13	0.37
Viscosity at 140°F (poises)	1280	1210	2539	2381	2436
Penetration at 77°F (dmm)	69	121	50	68	42
Test 2					
Loss (%)	0	0.04	0.05	0.08	0.15
Viscosity at 140°F (poises)	4681	5008	4017	3236	4285
Penetration at 77°F (dmm)	41	53	32	56	32

TABLE 7 COMPARISON OF CHROMATOGRAPHIC DATA ON AC-10 AND AC-20 ASPHALTS FROM EACH REFINERY SOURCE

	Ratio Between Fractions (AC-10/AC-20) of Asphalts from Source				
Fraction	A	B	C	D	E
Early	1.00	1.02	0.98	0.98	1.10
Intermediate	1.02	1.01	1.00	1.02	1.11
Late	0.98	0.84	1.11	1.04	0.31

TABLE 8 COMPARISON OF CHROMATOGRAPHIC DATA ON ASPHALT CEMENTS AT VARIOUS AGES

Source, Grade, and Site	New, On Site			Recovered from Laboratory Molded			Recovered from Field Cores		
	E	I	L	E	I	L	E	I	L
A AC-10, Dumas	1.01	0.98	1.00	1.05	0.98	0.02	1.02	1.00	0.59
A AC-20, Dumas	0.91	1.23	1.14	–	–	–	0.96	1.10	1.07
B AC-20, Lufkin	1.18	0.84	0.79	–	–	–	1.18	0.88	0.55
C AC-10, Dumas	0.96	1.08	0.82	1.13	0.96	0.66	1.06	1.11	0.68
C AC-20, Lufkin	0.99	1.05	0.79	–	–	–	1.09	0.99	0.64
D AC-10, Lufkin	0.92	1.09	1.05	–	–	–	1.02	1.05	0.62
D AC-20, Lufkin	0.95	1.07	1.00	1.03	0.98	0.66	1.05	1.01	0.66
E AC-20, Lufkin	1.08	1.02	0.60	–	–	–	1.13	0.96	0.66

NOTE: E = early, I = intermediate, L = late.

$$\text{Ratio} = \frac{\text{Area under given section of chromatogram for given asphalt obtained either during construction or recovered from mix}}{\text{Area under given section of chromatogram for given asphalt obtained directly from refinery at an earlier date}}$$

material recovered from samples compacted in the laboratory, and that recovered from cores. These data are given in Table 8 in the form of ratios of the areas under the early, intermediate, and late portions of the chromatograms. Virgin asphalts obtained from construction sites and asphalts recovered from laboratory-compacted samples and field cores are compared with asphalts obtained directly from the refineries. Unfortunately the majority of the data relates to the pavements at Lufkin where the field cores were removed after only 1 week of service.

As would be expected, the physical property data show an increase in viscosity and a decrease in penetration from the new material to the recovered material.

Data from the chromatograms show differences between the asphalt obtained from the refinery, new asphalt on site, laboratory-compacted asphalt, and field cores. In general, this difference is relatively small: the maximum change in the early fraction from new at refinery to recovered from field cores is 6 percent; for the intermediate fraction it is 5 percent (both for Asphalt A, AC-20 at Lufkin); and for the late fraction it is 3.1 percent (Asphalt D, AC-10 at Lufkin).

The changes show most clearly when the ratios of the fractions are examined as a function of the components of the refinery samples (Table 8). The maximum change from refinery to field is plus 18 parts and minus 4 parts in 100 for the early fraction. In both cases, this difference is matched or exceeded

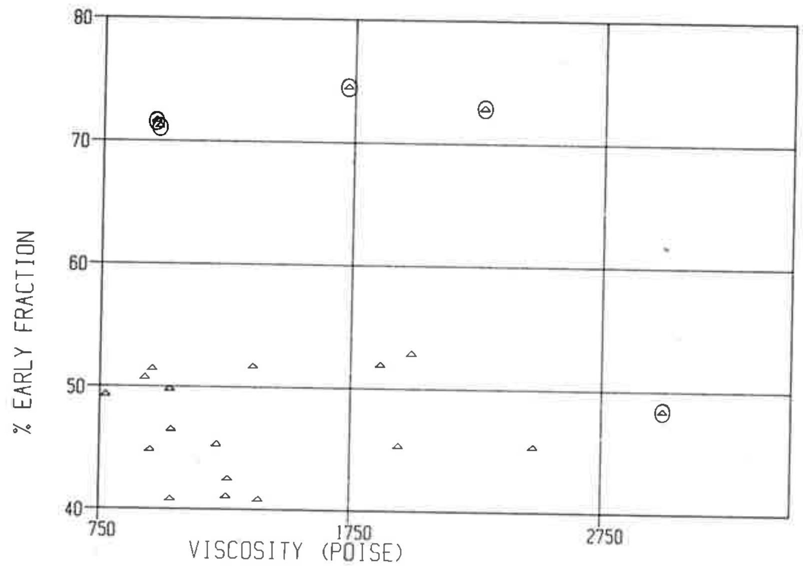

FIGURE 2 Early fraction as a function of viscosity.

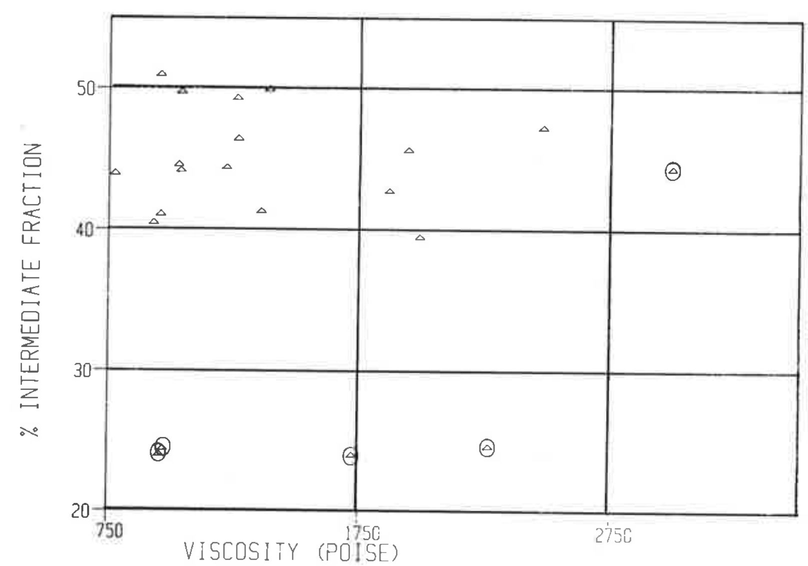

FIGURE 3 Intermediate fraction as a function of viscosity (AC-10 asphalts).

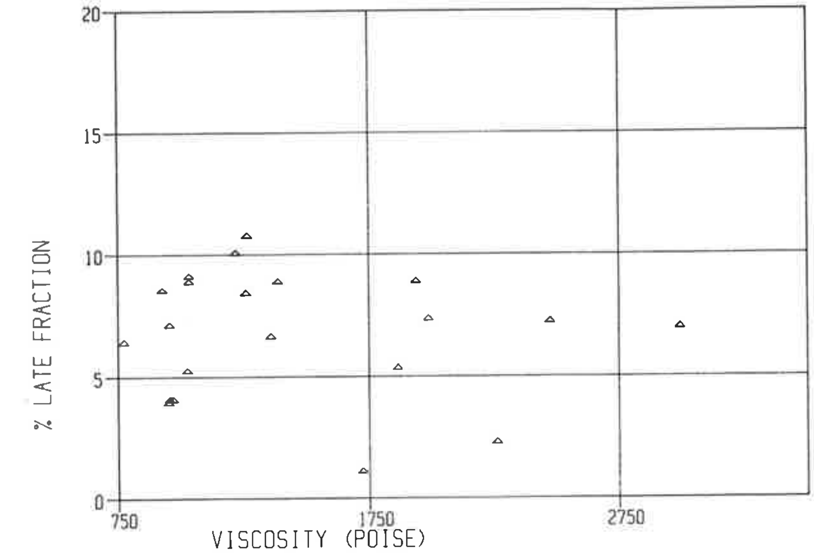

FIGURE 4 Late fraction as a function of viscosity.

by the difference between the new asphalt as delivered to site and that directly from the refinery. Ratios for the intermediate fraction range from plus 23 to minus 16 parts in 100. Once again, the greatest differences are between the virgin asphalt from the refinery and that delivered to the site.

The changes in the asphalts, as shown by ratios of areas for the late fractions, are much greater than for the other two fractions. In all cases but one (Asphalt A, AC-20, Lufkin), the late fraction has decreased from the refinery sample to the field sample. Also, in all cases but one, the changes from refinery to field for the late fractions are greater than the differences between the two new asphalts of different grades. In interpreting these data, it must be remembered that the actual changes in area are quite small; the greatest difference is a reduction from 7.7 percent of total to 4.5 percent of total, which yields a ratio of 0.58.

Relationship Between HPLC and Physical Properties

Figures 2–4 show the relationship among the early, intermediate, and late fractions determined from the chromatographs and viscosity measured at 140°F for the AC-10 asphalts. The data relating the chromatographic parameters to penetration have also been plotted, but, because they show a pattern similar to that of the viscosity data, they are not reproduced here. The three figures show that there is considerable scatter in the results. The data suggest that the quantity of material in each of the three sections of the chromatograph is independent of viscosity. There is, however, one interesting feature in Figures 2 and 3. In both of these figures, there are four points, which have been circled, detached from the main body of the data. These points are all for asphalt from Source A. It may be conjectured from this that there may be a family of lines that represents approximately constant quantity of either the early or the int rmediate fraction across the range of viscosity and that these lines may be unique to a particular source of asphalt cement. However, it should be noted that one point from the data on Asphalt A, also circled, lies within the body of the data, which represent all other sources, so the conjecture may be inappropriate. It is noteable that there is no separation of the points related to Asphalt A in Figure 4.

Figures 5–7 show the relationship between the parameters derived from the chromatograph and the *PI* for the AC-10 asphalts. These plots show that transforming penetration data to a temperature susceptibility parameter does not improve the relationship between physical tests and chromatographic data. Indeed, the general picture presented by the data is quite similar to that shown in Figures 2–4. Graphs of *VTS* are not reproduced in the interest of brevity and because they show the same pattern shown in Figures 5–7. Thus it would appear that there is no interrelationship between the standard rheological properties and the chromatographic plots for the AC-10 asphalts.

A similar analysis has been performed for the AC-20 asphalts. Figures 8–11 show a selection of these data. These figures show trends that closely resemble those observed for the AC-10 asphalts. If anything, the separation of the data into groups is more obvious than for the AC-10 asphalts, but again there are insufficient data to confirm the hypothesis that this suggests.

Relationship Between Parameters Derived from HPLC and Resilient Modulus

Figures 12–14 are plots of the area fractions from the chromatographs as a function of resilient modulus at 77°F. As was the case for penetration and viscosity, the chromatographic parameters and resilient modulus are not closely related. This is not surprising in view of the dependence of resilient modulus on the rheological properties of the asphalt cement.

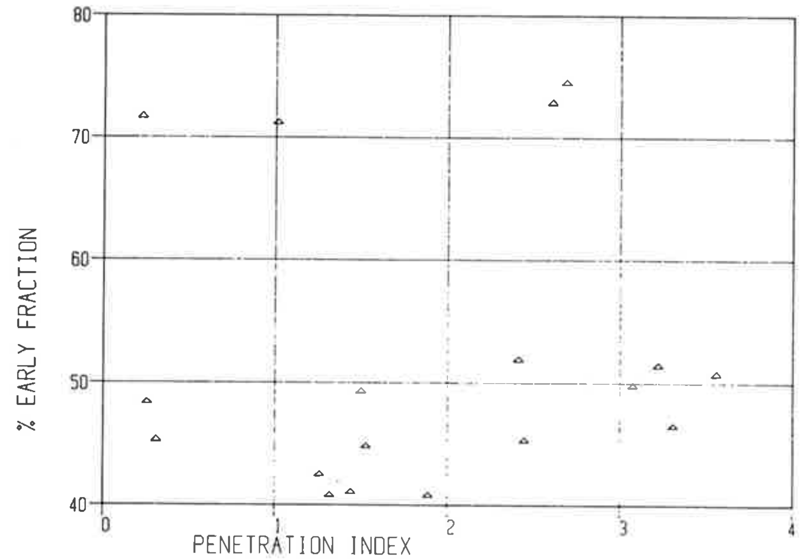

FIGURE 5 **Early fraction as a function of *PI*.**

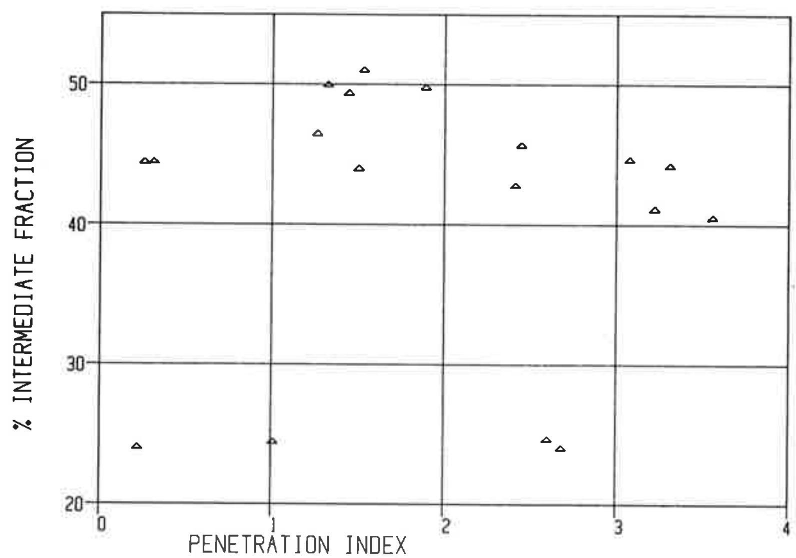

FIGURE 6 **Intermediate fraction as a function of *PI* (AC-10 asphalts).**

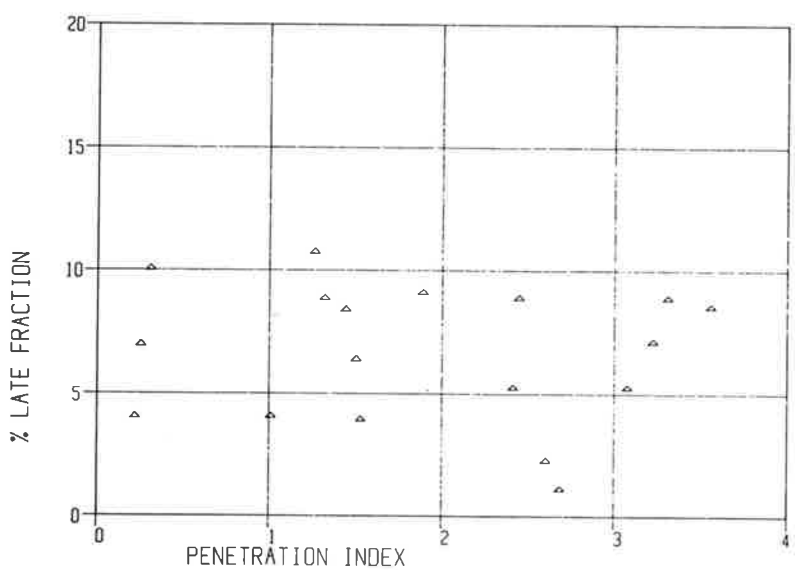

FIGURE 7 **Late fraction as a function of *PI*.**

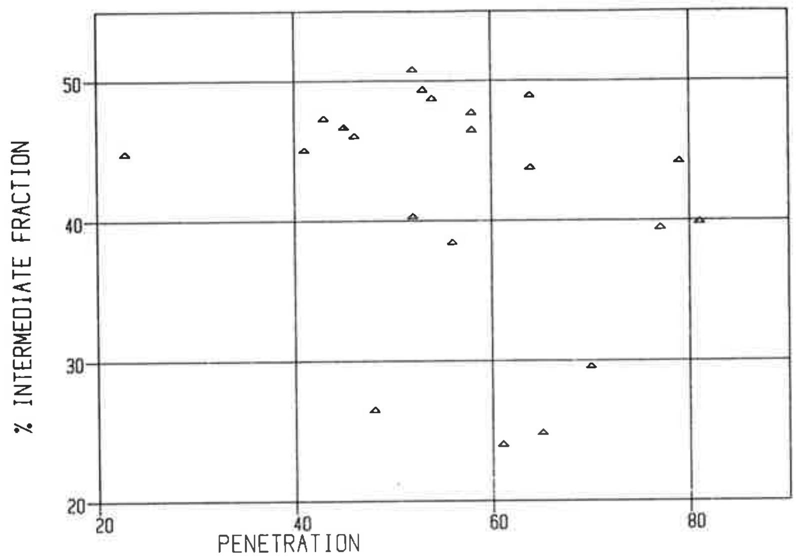

FIGURE 8 Intermediate fraction as a function of penetration.

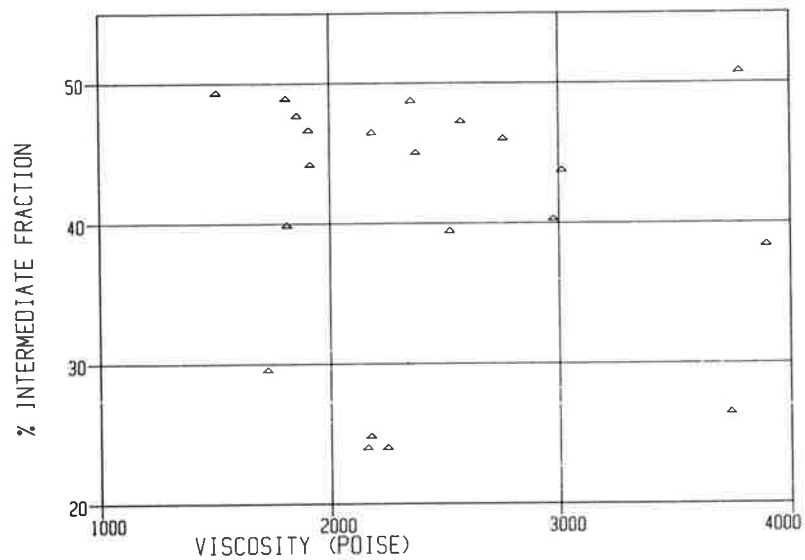

FIGURE 9 Intermediate fraction as a function of viscosity (AC-20 asphalts).

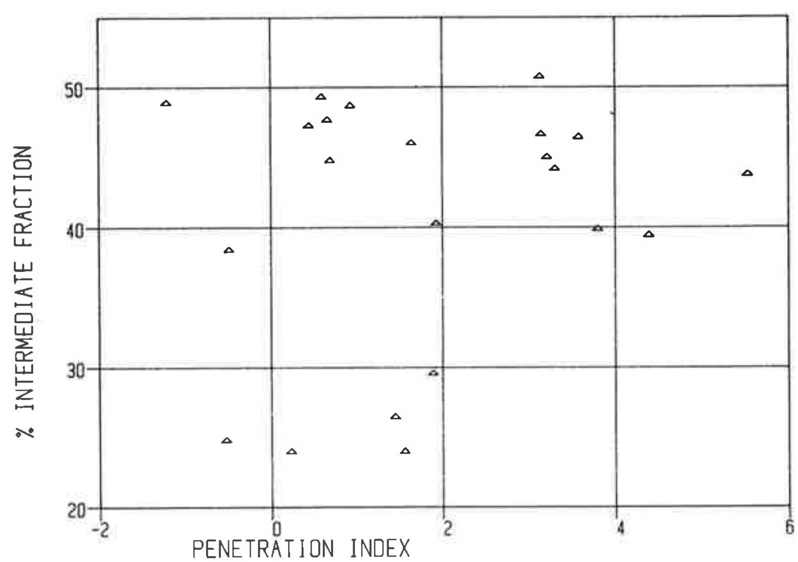

FIGURE 10 Intermediate fraction as a function of *PI* (AC-20 asphalts).

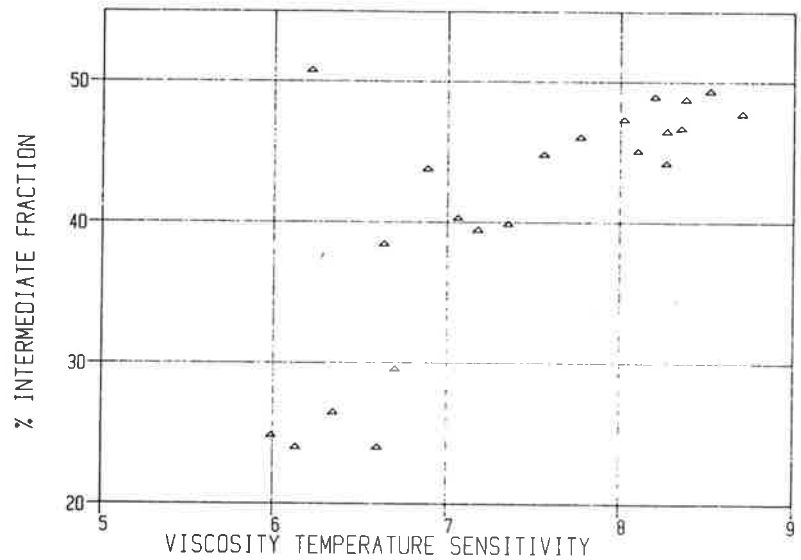

FIGURE 11 Intermediate fraction as a function of *VTS*.

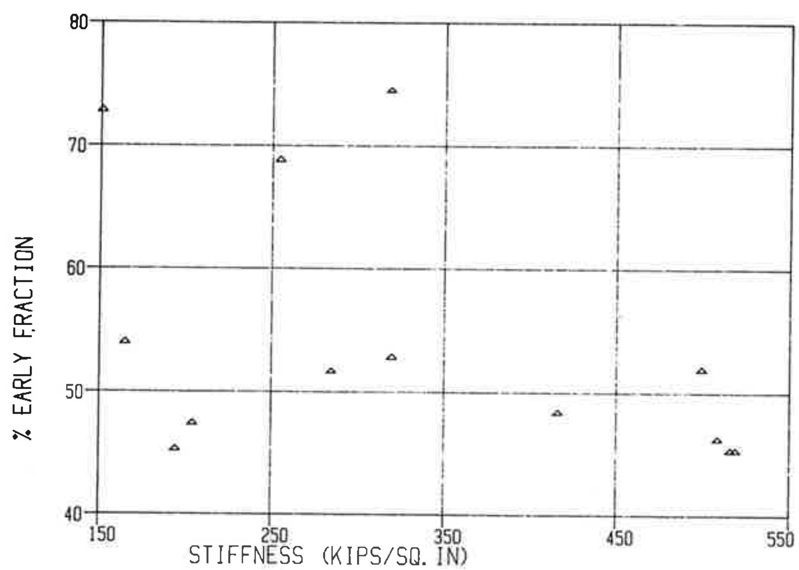

FIGURE 12 Early fraction as a function of stiffness.

Relationship Between Parameters Derived from HPLC and Pavement Performance

There are, to date, relatively few field data to use in an attempt to correlate the results of the chromatographic analysis with pavement performance at Dickens, Dumas, and Lufkin, Texas. The raveling observed in selected sections (containing Asphalts B and C) at the Dumas test site was most likely promoted by poor compaction of the mix. It should be pointed out, however, that all test sections were compacted to approximately equivalent air void contents and only two of them exhibited significant raveling. The raveling was reportedly due to extended periods of exposure to snow, ice, and moisture. One test section (Asphalt C) exhibited extremely severe raveling. If all other construction factors are reasonably constant, it may be possible to attribute the raveling to the character of the asphalts. The test sections at Dickens and Lufkin, which contain asphalts from the same refineries, showed no visible signs of distress after as much as 4 years. These inconsistencies in field performance and the limited quantity of data make it difficult to derive and support conclusions about asphalt quality. However, it is appropriate to comment on the potential of the chromatographic technique for predicting the performance of asphalt mixes. First, it must be remembered that the technique is applied to the asphalt cement alone; it does not provide information on the characteristics of the asphalt mix, and so it can only be of use in considering pavement distress that is related solely to the properties of the asphalt cement.

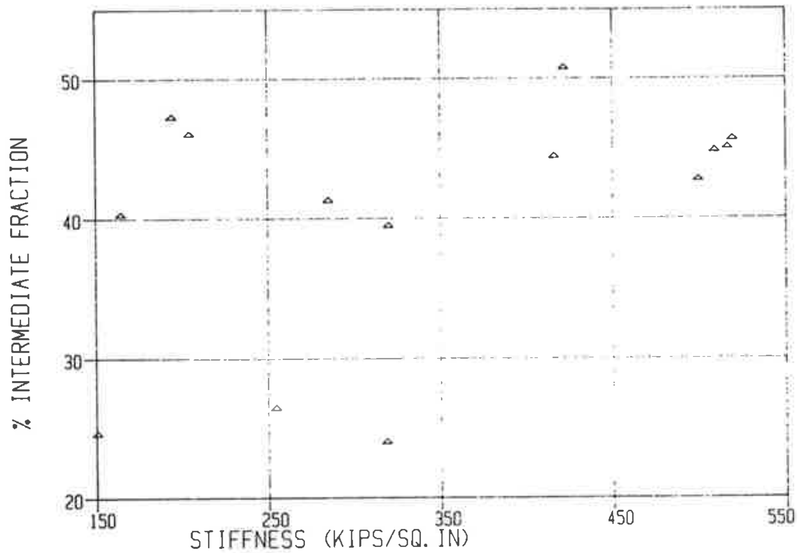

FIGURE 13 Intermediate fraction as a function of stiffness.

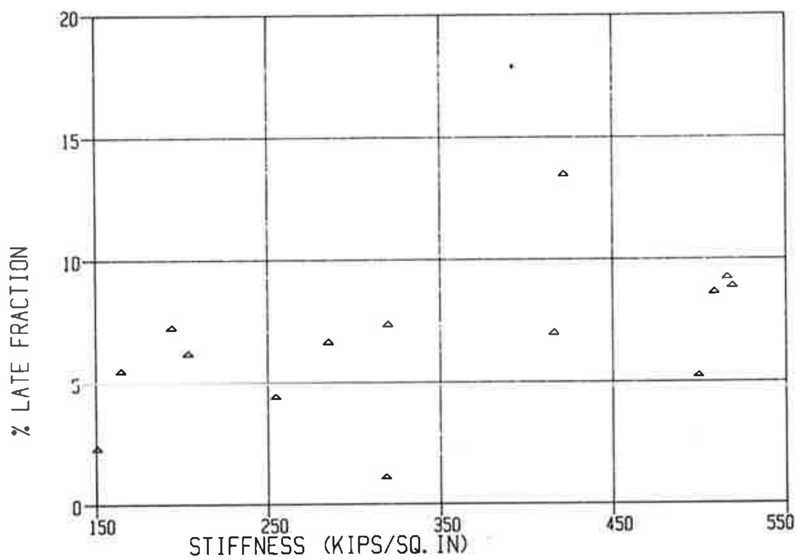

FIGURE 14 Late fraction as a function of stiffness.

If an asphalt cement does not possess the required adhesive characteristics, the aggregate particles with which it has been mixed will be dislodged by the passage of traffic, particularly in the presence of moisture, and the surface will disintegrate. However, if an asphalt layer is not properly compacted, there is a high probability that the surface will disintegrate, but, in this case, disintegration is not due to any intrinsic characteristic of the asphalt cement.

The situation becomes much more complex when cracking is considered. Laboratory testing has shown that fatigue-type cracking is related to the characteristics of the asphalt in the mix. These studies also show that fatigue-type cracking is related to the volume of asphalt cement in the mix. That is, low binder content or poor compaction can accelerate cracking in a mix with a perfectly satisfactory binder. Even if the mix is well compacted and contains adequate asphalt cement, if it is placed over a weak structure and is therefore subjected to large tensile strains it will crack. Hence there are many potential causes of fatigue cracking that are not related to intrinsic characteristics of the asphalt cement.

This discussion could be extended to other forms of cracking as well as other modes of distress; however, it is hoped that sufficient comments have been made to emphasize the necessity of extremely careful monitoring and investigation of pavement performance before any useful conclusions can be drawn with respect to the relationship between chromatographic data and pavement performance.

CONCLUSIONS

The technique of dividing the complete chromatograph obtained from an asphalt sample into three sections as described permits the following conclusions to be drawn:

1. There is no relationship among the early, intermediate, or late partial areas that can be used to predict penetration, viscosity, penetration index, viscosity, temperature sensitivity, and resilient modulus.

2. The technique does not differentiate between different grades of asphalt obtained from the same refinery.

ACKNOWLEDGMENT

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Size Exclusion Chromatography and Nuclear Magnetic Resonance Techniques for Predicting Asphalt Yields and Viscosities from Crude Oil Analyses

P. M. BEAZLEY, L. E. HAWSEY, AND M. A. PLUMMER

Size exclusion chromatography (SEC) and nuclear magnetic resonance (NMR) techniques for predicting asphalt yield and viscosity as functions of crude oil type and manufacturing conditions are detailed in this paper. SEC conditions were varied to yield crude oil component distributions to the asphalt phase that agreed with the thermodynamic requirements for distillation processes. Silanized-silica column packing and ultraviolet detection yielded better results than polystyrene packing and refractive index detection. Relatively standard NMR conditions were used to determine the ratio of aromatic protons to paraffinic protons. This ratio was used as a measure of the relative polarity of crude oil and asphalt components.

Asphalt producers have historically relied on fixed domestic crude sources to yield consistent quality. Recently, some producers have been required to use variable crude slates; this makes asphalt quality control more difficult. In response to this situation, mathematical models were developed to predict asphalt yield and viscosity as functions of crude oil type and manufacturing conditions *(1)*. Inputs to these models are analytical results that require only 2 to 4 hr to obtain. The purpose of this paper is to detail the size exclusion chromatography (SEC) and nuclear magnetic resonance (NMR) techniques developed for these models.

To predict asphalt yield, molecular weight or size distribution and polarity of the crude oil constituents need to be known. The larger the concentration of the higher molecular weight asphalt components, the greater the asphalt yield over other constituents. For a given molecular weight, more of the aromatic compounds than paraffinic compounds will preferentially fractionate to the asphalt product. The viscosity of an asphalt depends on its molecular size distribution and on its aromatic/paraffinic composition. A more aromatic asphalt with higher polarity or an asphalt with larger molecular sizes will exhibit higher viscosities.

To obtain data needed for yield and viscosity predictions, SEC was selected for size distribution and proton NMR was chosen for component polarity. To achieve the most accuracy and flexibility in modeling, the size distribution and polarity parameters had to be measured as independently as possible.

A typical analytical approach for obtaining molecular weight or size distribution data on a crude or asphalt is to first fractionate the material. Then SEC and molecular weight data are obtained on the fractions. From these results, a SEC retention time–versus–molecular weight calibration curve is established for the entire material *(2, 3)*. This approach is quite time consuming. Also, it is difficult to determine if all molecular association and adsorption forces have been eliminated.

The SEC approach used in this work was to analyze, in its entirety, a given crude oil and all of the various asphalts distilled from the crude oil. Then, the percentage of component distribution to the asphalt phase was calculated and plotted versus SEC retention time. These experimental distribution–versus–retention time data were compared with those expected from thermodynamic principles for distillation processes.The SEC procedures were varied until the actual component distribution–versus–retention time data agreed with the thermodynamic requirement of a monotonically decreasing function (Figure 1). In this manner, SEC conditions were developed that assured a true component molecular size distribution essentially free of association or adsorption problems.

The typical NMR approach for obtaining aromatic/paraffinic (polar/nonpolar) component results is analysis by both carbon-13 and proton NMR *(4–6)*. Then various carbon/hydrogen ratios are calculated to obtain a measure of the aromatic/paraffinic composition. This approach is quite time consuming. Only proton NMR was used in this study because it was found that the polarity interactions that control asphalt yield and viscosity could be modeled with the aromatic/paraffinic proton ratio.

EXPERIMENTAL PROCEDURE

The following SEC and NMR techniques were used to obtain molecular size and aromatic/paraffinic breakdowns on 5 crude oils and 11 asphalts distilled from these crudes. Using the results, asphalt yields and viscosities were correlated to distillation conditions at average accuracy levels of 0.6 percent *(1)*.

Marathon Oil Company, 7400 South Broadway, Littleton, Colo. 80122.

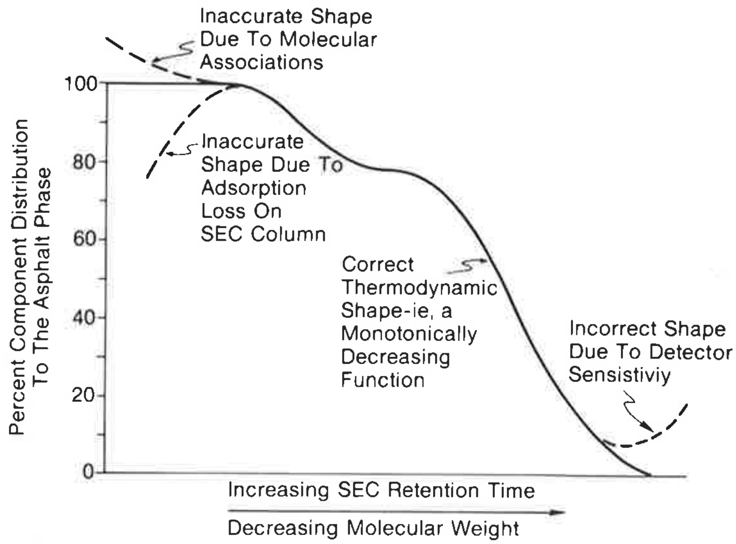

FIGURE 1 Thermodynamic requirement for component distributions to asphalt phase versus SEC retention time.

SEC

The liquid chromatograph consisted of a Spectra-Physics Model SP8700 pump, a Spectra-Physics Model SP8400 UV/VIS variable wavelength detector, a Spectra-Physics Model 748C column oven, a Bascom-Turner Model 4120 data acquisition system, a Valco Model HH-60 injection valve, and a Phase Separation LTD flow meter. The chromatograph was controlled with an in-house–designed microprocessor system based on a Motorola 6800 CPU and a Pro-Log STD BUS. A silanized 60-Å silica-based (6.2-mm inside diameter × 25 cm) DuPont Zorbax® PSM 60S SEC column was used with a Brownlee (4.6-mm inside diameter × 3 cm) RP-18 guard column.

The column was eluted with helium-purged tetrahydrofuran (THF, Burdick and Jackson) at a flow rate of 1 ± 0.01 mL/min. Best results were obtained with the column heated to 90°C and the detector set at a wavelength of 220 nm. The detector was not thermostated. During each 10-min run, the data system acquired 1,000 absorbance measurements. Solutions (25 μL) of 0.05 percent by weight of crude or asphalt in THF were injected. Both valve and column were thermostated in the column oven. Samples were prepared with helium-purged THF and stored in nitrogen in septum vials.

NMR

A Varian EM-390 90-MHz spectrometer was used in this study. Asphalt and crude oil samples were dissolved in carbon tetrachloride (spectrophotometric grade) to obtain low-viscosity solutions. Tetramethylsilan was used as the reference standard. Other NMR operating parameters are as follows:

RF power: 0.30 mG
Filter time constant: 0.05 sec
Sweep time
  Spectrum: 5 min
  Integrals: 1 min
Sweep width: 10 ppm
End of sweep: 0 ppm
Sample spin rate: 40–60 rps
Sample temperature: 34°C

To obtain relative polarity values, two regions of the NMR spectrum were integrated. Paraffinic protons were assigned to the 0 to 5.33 ppm region and aromatic protons to the 5.33 to 9.00 ppm region.

DISCUSSION OF RESULTS

As previously mentioned, the models need independent measurements of polarity and molecular size distribution. Development of NMR techniques for determining the polarity input was relatively straightforward. However, considerable difficulty, which has also been noted by others (7, 8), was encountered with the molecular size distribution analysis. Gel permeation chromatography or, to use the modern term, size exclusion chromatography (SEC) was the technique of choice. The first attempts used Waters μ-Styragel® columns, THF solvent, and refractive index (RI) detection. The data obtained from this system could not be used. The apparent molecular size distribution of various distillation fractions did not agree with the distributions predicted from thermodynamic principles. The moderately high polarity of the Waters polystyrene column packing gave separations based on polarity and absorption in addition to molecular size. Also, the RI detector emphasized the polarity separation mechanism. For example, the chromatograph of a typical crude oil obtained under these conditions clearly shows the effects of absorption and RI detector re-

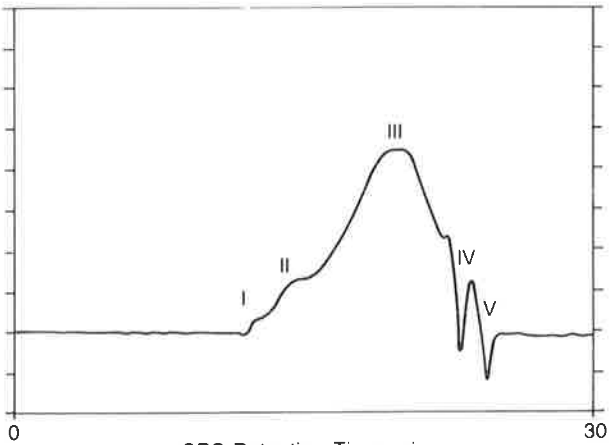

FIGURE 2 Chromatogram of crude oil. Obtained with a set of two Waters 500-Å µ-Styragel® columns at 50°C eluted with THF at 1 mL/min; Waters RI detector at X8 sensitivity; and 100-µL sample of 2 percent by weight.

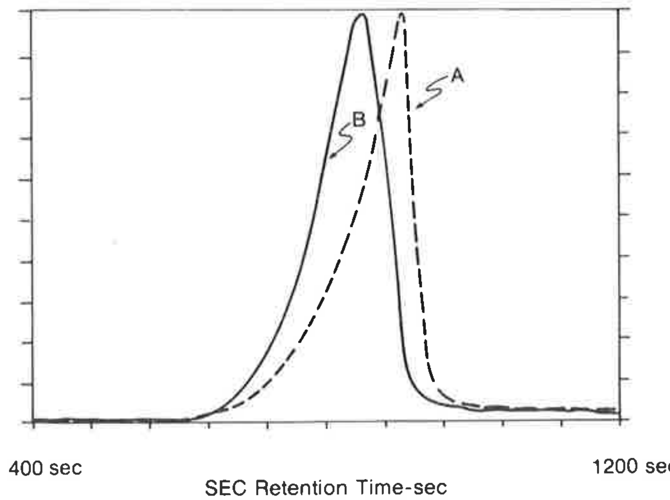

FIGURE 4 Chromatogram of (A) whole crude oil and (B) asphalt distilled from this crude. Obtained with three Zorbax® PSM 60S columns at 50°C eluted with THF at 1 mL/min; UV detection at 254 nm; and 10-µL samples of 0.2 percent by weight in THF.

sponse (Figure 2). In particular, it is believed that the separated Fractions I and II are not collections of material with similar molecular size but rather represent material with similar polarity. The large negative peaks, IV and V, rendered the chromatogram useless for this analysis because meaningful component distributions to the asphalt phase could not be calculated. In an effort to improve this situation, an ultraviolet (UV) detector was substituted for the RI detector. Using the new detector, the chromatograms of the same crude and of an asphalt derived from it were significantly improved by the elimination of the negative peaks (Figure 3). Meaningful component distributions could then be calculated from this plot, but the result would not accurately correspond to molecular size because multiple peaks were still observed.

Then silanized-silica Zorbax® PSM 60S columns were used. The Zorbax columns gave smooth chromatograms with no separated peaks or shoulders (Figure 4). These chromatograms appeared to be reasonable for molecular size distributions of complex materials like crude oil and asphalts.

It was suspected that both the styrene- and the silica-based columns could absorb polar materials from crude oils or asphalts. Hence, chromatographic responses from both types of columns were compared. The same amount of sample was run on each column under the same chromatographic conditions. The chromatographic results show that recovery from the silanized-silica column was much greater than recovery from the polystyrene column (Table 1).

TABLE 1 RECOVERY DATA

	Total Area	
Column	Crude	Asphalt
Zorbax PSM 60S	1798	4556
µ-Styragel 10^3Å	920	3223

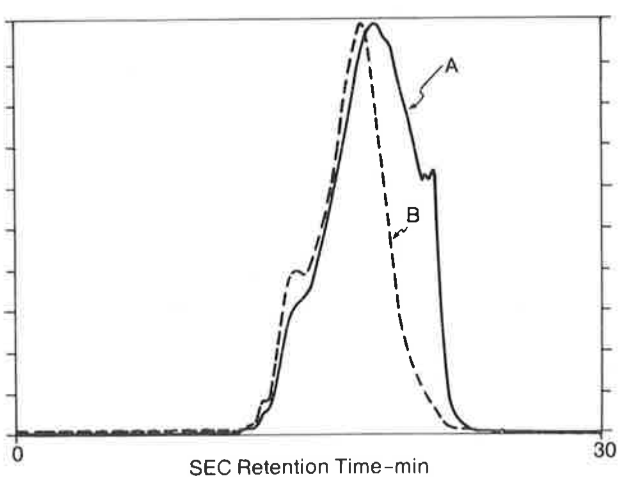

FIGURE 3 Chromatogram of (A) whole crude oil and (B) asphalt distilled from this crude. Obtained with two 500-Å µ-Styragel® columns at 50°C eluted with THF at 2 mL/min; UV detection at 254 nm; and 100-µL samples of 2 percent by weight in THF.

To test the resolution of the silanized-silica column, two asphalts distilled in a standard refinery process from a Wyoming crude were analyzed. The normalized chromatograms of these asphalts compared with that of the crude clearly reflect the expected different component distributions (Figure 5). Also, a crude and its overhead and residuum fractions obtained by the ASTM D 1160 distillation procedure were run (Figure 6). To see if the chromatographic resolution corresponded to the distillation, the normalized residuum curve, multiplied by its weight fraction from the whole crude, was subtracted from the normalized whole crude oil curve. The normalized difference curve was found to be equivalent to the actual overhead curve (Figure 7). Because these curves were essentially identical, it was thought that the SEC separation with the silanized-silica column corresponded to the distillation.

This initial work was performed using a detector setting of 254 nm, a column temperature of 50°C, and a sample concentration of 0.2 percent by weight in THF. These conditions

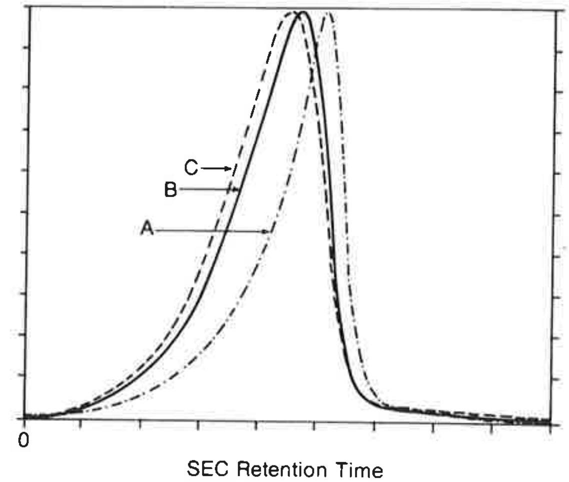

FIGURE 5 Chromatogram of (A) a Wyoming crude oil, (B) 200-250 pen, and (C) 85-100 pen asphalts distilled from this crude. Obtained with a Zorbax® PSM 60S column at 50°C eluted with THF at 1 mL/min; UV detection at 254 nm; and 10-μL samples of 0.2 percent by weight.

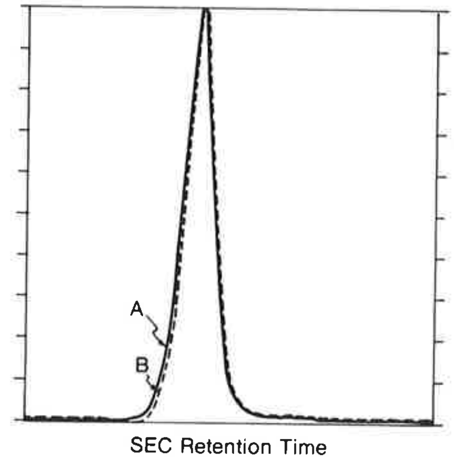

FIGURE 7 (A) Chromatogram of overhead from ASTM D 1160 distillation of crude. Obtained on a Zorbax® PSM 60S column at 50°C eluted with THF at 1 mL/min; UV detection at 254 nm; and 10-μL sample of 0.2 percent by weight in THF. (B) Calculated chromatogram from whole crude chromatogram curve minus that of the residuum fraction.

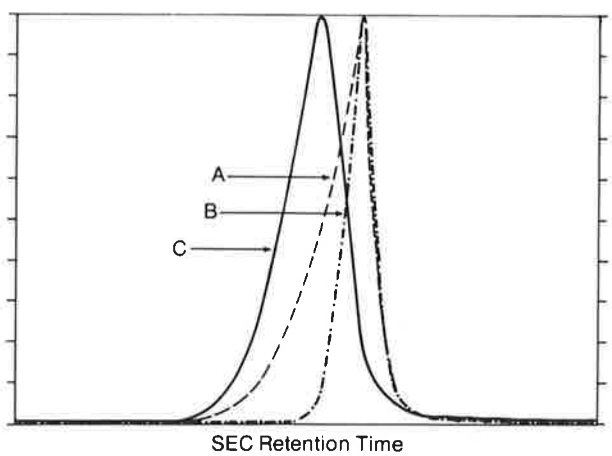

FIGURE 6 Chromatogram of (A) crude oil, (B) overhead fraction, and (C) residuum from ASTM D 1160 distillation. Obtained with a Zorbax® PSM 60S column at 50°C eluted with THF at 1 mL/min; UV detection at 254 nm; and 10-μL samples of 0.2 percent by weight.

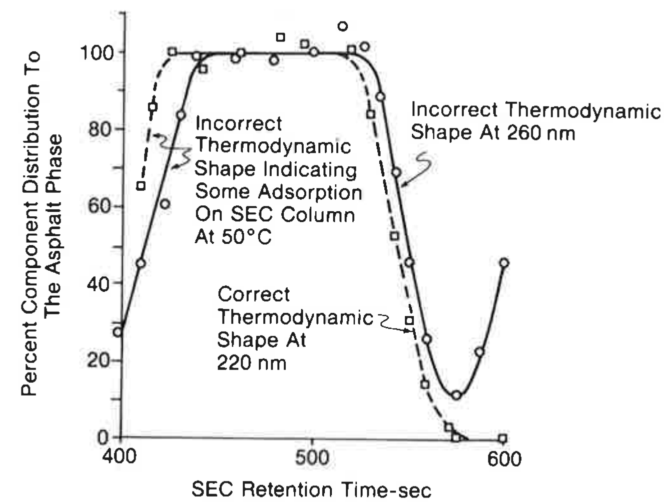

FIGURE 8 Distribution of components from a Wyoming crude to a 85-100 pen asphalt versus UV detector wavelength. Obtained on a Zorbax® PSM 60S column at 50°C eluted with THF at 1 mL/min and 25-μL sample of 0.05 percent by weight.

were accurate enough for selection of column and detector types. However, in terms of the thermodynamic model, the best SEC results were obtained at 220-nm wavelength, 90°C column temperature, and 0.05 percent by weight sample concentration in THF. In the thermodynamic analysis component distributions to the asphalt phase were determined as a function of SEC retention time or molecular size. A typical analysis showed that decreasing the UV wavelength from 260 to 220 nm yielded the correct component distributions at high retention times (Figure 8). At the higher wavelength, small molecular size components were overemphasized. Another study showed that increasing column temperature from 50°C to 90°C yielded the correct thermodynamic shape at low retention times (Figure 9). At 90°C, the adsorption of large molecular size components on the SEC column was essentially eliminated. Using the thermodynamic analysis, UV wavelengths of 210 to 280 nm, column temperatures of 25°C to 130°C, and sample concentrations in THF of 0.002 to 2.0 percent by weight were evaluated. In these evaluations, the asphalts used were manufactured by a standard refinery distillation process.

CONCLUSIONS

SEC and NMR conditions have been developed for use in predicting asphalt yield and viscosity as functions of crude oil type and manufacturing conditions. The SEC conditions were optimized by comparing experimental crude oil component

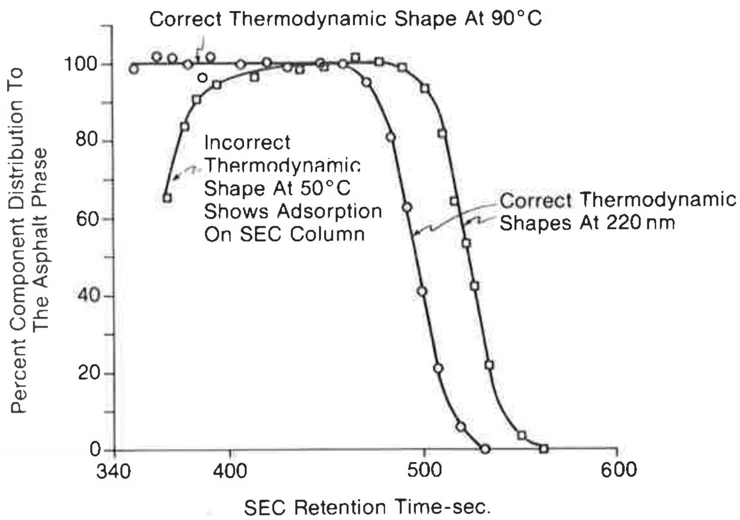

FIGURE 9 Distribution of components from a heavy Arabian crude to an AC-20 asphalt versus column temperature. Obtained on a Zorbax® PSM 60S column eluted with THF at 1 mL/min; UV detection at 220 nm; and 25-µL samples of 0.05 percent by weight.

distributions to the asphalt phase with the thermodynamic requirement for distillation processes. Aromatic/paraffinic proton ratios from the NMR results were developed to measure the relative polarity of the crude oil and asphalt components.

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Rejuvenator Diffusion in Binder Film for Hot-Mix Recycled Asphalt Pavement

AHMED SAMY NOURELDIN AND LEONARD E. WOOD

In hot-mix recycling of bituminous mixes a rejuvenator is commonly used to restore the aged asphalt cement to a condition that resembles that of virgin asphalt cement. The type and amount of rejuvenator to be used are generally determined from a characterization of the recovered weathered binder. However, the extent to which the salvaged bitumen will be softened by the recycling agent during the hot-mix operation, and hence the characteristics of the rejuvenated binder and the resulting performance of the recycled pavement, has not been widely reported. An investigation was undertaken to determine the extent to which certain types of rejuvenators diffuse into the hardened asphalt film coating the aggregate and affect its properties during a specified period of time. A partial extraction technique that had the effect of dividing the asphalt film into microlayers was used. The binder recovered from each microlayer was characterized by means of consistency tests. This technique was used to evaluate the consistency distribution of the binder film around the aggregate in (a) the extracted mix containing recycled asphalt pavement (RAP) only; (b) the extracted mix containing RAP and a rejuvenator; and (c) the extracted mix containing RAP, virgin aggregate, and a rejuvenator.

Recycling operations have grown rapidly in recent years. This growth has resulted from increased awareness of the potential for cost savings and material conservation. More important, the effort put forth by equipment manufacturers has also increased. In recent years there have been rapid advances in pulverizers, millers, and hot-mix plants that facilitate recycling operations. With the increase in recycling operations has come an increased awareness that the recycled material must be properly characterized in order to ensure a quality pavement. If the recycled pavements show excessive deterioration, the cost and energy savings realized during construction may be lost through excessive maintenance. Initial indications are that a quality pavement is being constructed. However, these pavements have not been in service long enough to permit a definite judgment of their long-term performance.

The aged binder present in a recycled asphalt pavement (RAP) has physical properties that make it undesirable for reuse without modification. Materials have been developed to restore these old binders to a condition suitable for reuse. This concept is not new and has been the subject of a number of extensive studies during the last several years *(1–3)*.

The common procedure for analyzing a recycled mixture involves (a) extraction of the asphalt cement, measurement of its properties, and selection of a proper amount and type of rejuvenator to restore the classification properties to a preselected level and (b) examination of the gradation of the salvaged aggregate fraction to determine the amount and gradation of new aggregate that may be required *(4–6)*.

The comparison of laboratory performance predictions for recycled materials and new construction requires that the binder content and consistency be the same. These two values represent the parameters that are influenced by the recycling operation. The consistency of the recycled asphalt cement should be similar to that of the virgin asphalt samples that represent new construction *(7, 8)*.

It has been generally recognized that the effectiveness of a recycling agent is related to its uniform dispersion throughout the pavement binder. This is an important issue for recycling because changes in properties with time have been attributed to inadequate mixing of the reclaimed bitumen and rejuvenating agent during processing *(9)*.

An investigation was undertaken to determine the extent to which certain types of rejuvenators diffuse into the hardened asphalt film coating the aggregate and affect its properties during a specified period of time. A partial extraction technique that had the effect of dividing the asphalt film into microlayers was used. The binder recovered from each microlayer was characterized by means of consistency tests.

This technique was used to evaluate the consistency distribution of the binder film around the aggregate in (a) the extracted mix containing RAP only; (b) the extracted mix containing RAP and a rejuvenator; and (c) the extracted mix containing RAP, virgin aggregate, and a rejuvenator.

CONCEPTUAL DISCUSSION OF DIFFUSION PROCESS

Common practices call for totally extracting and recovering the weathered asphalt and thoroughly mixing it with various percentages of the rejuvenator in order to determine the amount of rejuvenator required. The Asphalt Institute *(10)* has recommended an easy way to determine an initial value for this amount, if the viscosities at 140°F for the weathered asphalt and the rejuvenator and the target classification (consistency) of the resulting binder are known, by using a group of curves. Hence, it was assumed that (a) the rejuvenator is thoroughly mixed with the weathered asphalt and (b) the weathered asphalt film around the aggregate has a uniform consistency throughout its layer. Unfortunately this may not be the case in a typical hot-mix recycling project.

Carpenter and Wolosick *(9)* outlined the way in which a rejuvenator diffuses into the weathered asphalt film of a cold-mix recycled bituminous material (given that no virgin aggregate is used) as follows:

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1. The rejuvenator forms a very low-viscosity layer that surrounds the aggregate that is coated with aged asphalt cement (Time Step 0).
2. The rejuvenator begins to penetrate into the asphalt cement layer, thereby decreasing the amount of raw rejuvenator that coats the particle and softening the old asphalt cement (Time Step 1).
3. No raw rejuvenator remains and penetration continues; the viscosity of the inner layer is lowered and gradually the viscosity of the outer layer is increased (Time Step 2).
4. Equilibrium is approached over the majority of the film of asphalt except right at the asphalt-aggregate interface, which may remain at a higher viscosity level (Time Steps 3 and 4).

The study also indicated that the time span between these four phases may be critical and hence the structural parameters may or may not be sufficiently developed to provide resistance to wheel loads during the initial portion of the life of the recycled pavement.

Careful selection and testing of recycling agents must be conducted to shorten this time span and cause structural parameters to develop more rapidly.

SAMPLING PLAN AND MATERIALS

Salvaged Asphalt Pavement Samples

A stockpile of representative salvaged bituminous pavement was obtained for laboratory evaluation. The material used was milled from US-52 (south of Indianapolis, Indiana) and randomly selected under the supervision of the Indiana Department of Highways for the purpose of this study. Sampling of the stockpile was also done at random to obtain statistically representative bituminous materials for this study.

Virgin Aggregate Samples

Crushed limestone and local sand were selected to represent the coarse and fine aggregate material for virgin aggregate samples. The selection was based on materials that are generally used in Indiana to produce hot-mix bituminous pavements.

Recycling Agents

Three types of recycling agents were selected for use in combination with the age-hardened salvaged bituminous binder. The selection was based on previous use of these agents in other recycling techniques and the knowledge of their physical and chemical properties *(3, 11)*. The following recycling agents were used: an AC-2.5 obtained from Amoco Oil Company, an AE-150 (Indiana-designated, high-float, medium-setting type of asphalt emulsion), and Mobilsol-30 (Type-101 oil, ASTM designation in an emulsified form) supplied by McConnaughay, Inc.

TEST RESULTS AND ANALYSIS

Salvaged Material

Samples of the RAP were randomly chosen, reduced in size, and characterized. Asphalt extraction and recovery were conducted using ASTM D 2172-67 Method A and the Abson method (ASTM D 1856), respectively. The salvaged binder was characterized by penetration, softening point, and viscosity tests. The amount of asphalt present was determined, and the salvaged aggregate obtained from extraction was characterized by sieve analysis.

Tables 1 and 2 give the characteristics of the extracted hard asphalt and the gradation of salvaged aggregate, respectively. The values given represent an average of 10 samples. The Indiana specifications for No. 12 surface are also included in Table 2 for purposes of comparison and for use in future determination of the feasibility of using the salvaged aggregate in a high-quality hot surface mix. It should be noted that the amount of hardening that occurred in the old binder was relatively low compared with that in previous recycling projects. In addition, the sieve analysis of the salvaged aggregate indicated a gradation that is within the specification for No. 12 surface (Indiana DOT specifications). However, a lower percentage of material passing the 3/8-in. sieve was noted.

TABLE 1 CHARACTERISTICS OF EXTRACTED HARD ASPHALT

Test	Value
Penetration at 77°F, 100 g, 5 sec	28
Viscosity at 140°F (poises)	20 888
Kinematic viscosity at 275°F (cSt)	726
Softening point (°F)	137
Asphalt content (percentage of total weight)	6

TABLE 2 GRADATION OF SALVAGED AGGREGATE

	Sieve Size							
	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Percentage passing	93	74	62	44	28	15	7.5	5
Indiana specification for No. 12 surface (%)	96–100	70–80	36–66	19–50	10–38	5–26	2–17	0–8

Recycling Agents (rejuvenators)

Three types of recycling agents were used to restore the old binder to the AC-20 classification range. Selection of the three types was based on their previous usage in recycling techniques other than hot-mix recycling. In addition, the AC-20 classification range was a target because AC-20 is widely used in producing high-quality hot-mix paving mixtures in Indiana.

The three types used were AC-2.5 ASTM designation; AE-150 Indiana designation; and Mobilsol-30, which is a commercial type. Table 3 gives the characteristics of Mobilsol-30.

TABLE 3 CHARACTERISTICS OF MOBILSOL-30

Characteristic	Measurement
Asphaltenes (%)	0
Polar compounds (%)	8
Aromatics (%)	79
Saturates (%)	13
Solids in emulsified form (%)	66.7
Flash point (°F)	505
Kinematic viscosity at 140°F (cSt)	164

NOTE: Constituents were obtained using clay-gel analysis (ASTM D 2007-75). All characteristics except solids in emulsified form are those of residue.

Determination of the Amount of Rejuvenator

Asphalt Institute curves *(10)* were used to determine an initial value for the percentage of rejuvenator (AC-2.5 and AE-150) to be added to the old binder to restore the properties to the AC-20 range of classification. The curves suggest the percentage of rejuvenator on the basis of its viscosity at 140°F, the old binder viscosity at 140°F, and the required viscosity for the new rejuvenated binder at 140°F. The initial value for the percentage of Mobilsol-30 was chosen on the basis of previous recycling projects *(3, 11)*.

A series of extraction and recovery tests was conducted to justify these initial values. Table 4 gives the characteristics of the salvaged asphalt, the rejuvenators, and the three rejuvenated binders.

TABLE 4 CHARACTERISTICS OF SALVAGED ASPHALT, REJUVENATORS, AND REJUVENATED BINDERS

Binder	Penetration	Viscosity at 140°F (poises)
Old asphalt	28	20 888
AC-2.5	200	292
AE-150 residue	200	270
40% old asphalt plus 60% AC-2.5	62	2 112
45% old asphalt plus 55% AE-150 residue	68	1 994
85% old asphalt, 15% Mobilsol-30 residue	69	1 974
AC-20 specification	60+	1 600–2 400

NOTE: Characteristics of Mobilsol-30 are given in Table 3.

Concept of Stage Extraction

A stage extraction technique was used to determine the extent to which the salvaged bitumen would be softened by the recycling agent during the laboratory-simulated hot-mixing operation. The method used (explained later in detail) divides the asphalt binder film coating the aggregate into four successively extracted microlayers. Each layer is then characterized separately to determine how much it is affected by the rejuvenator (in other words, to what extent does the rejuvenator diffuse into the old asphalt binder film and affect its properties). The same technique was used to investigate the consistency distribution of the binder film around the aggregate in (a) the extracted mix containing RAP only; (b) the extracted mix containing RAP and a rejuvenator; and (c) the extracted mix containing RAP, virgin aggregate, and a rejuvenator.

Method

The RAP sample was heated in an oven at 240°F for 30 min. The rejuvenators (AC-2.5, AE-150, and the Mobilsol-30) were heated in an oven at 180°F. The RAP, virgin aggregate, and one of the rejuvenators were mechanically hot mixed for 2 min to ensure proper mixing. The loose samples were stored in an oven for 15 hr at 140°F and directly extracted at different stages using Method A (ASTM D 2172). To fully extract the binder from a 1200-g sample of RAP, 1400 mL of trichloroethylene (TCE) solvent were used. Seven samples were used to obtain the proper amount of recovered asphalt from each microlayer for characterization. The solvent was applied to the mix in increments of 200, 200, 300, and 700 mL, respectively, to have the extracted asphalt film in four components. A 5-min soaking period was required between the successive increments. Asphalt binders were then recovered separately from each of the four fractions by using the Abson method (ASTM D 1856) and characterized by means of penetration and viscosity tests. For those mixes that called for the addition of virgin aggregate, the aggregate was heated at 240°F for 30 min before it was mechanically mixed with the RAP and the rejuvenator.

Results of Fractional or Stage Extraction Process

RAP Only

The RAP, without the addition of either virgin aggregate or recycling agent, was stage extracted for purposes of comparison. Stage extraction gave rise to some interesting results. Table 5 gives the penetration and viscosity (140°F) values of the reclaimed stage-extracted old binder. The original asphalt used was AC-20, and it can be observed that the outer microlayer of the asphalt was severely hardened by direct exposure to weathering actions. However, the second microlayer was less hardened and the third one was almost unchanged (compared with original AC-20 characteristics). On the other hand, the last microlayer at the binder-aggregate interface was slightly hardened, probably because of the tendency of limestone (commonly used in Indiana) to absorb light fractions of the binder. Figure 1 is a schematic diagram of these four

TABLE 5 TEST RESULTS ON RECLAIMED STAGE-EXTRACTED RAP

TCE Increment (mL)	Binder (% by weight)	Penetration	Viscosity at 140°F (poises)
200	55.5	24	24 000
200	26.5	33	15 000
300	11.2	65	2 500
700	6.8	57	3 300

NOTE: The results are averages of three replications, each conducted on seven samples of 1200 g; percentage of asphalt cement is 6 percent by weight of mix; and original asphalt was AC-20.

microlayers and the penetration and viscosity distribution along the old asphalt film.

Rejuvenator Effect, No Virgin Aggregate

It was decided in this portion of the study not to add any virgin aggregate in order to clarify the effect of the rejuvenator on the older binder during the laboratory-simulated hot-mix operation. Table 6 gives the penetration and viscosity (140°F) values of reclaimed stage-extracted treated binder. The results suggest that the three rejuvenators used (AC-2.5, AE-150, and Mobilsol-30) restored the two outer layers of the old binder to the AC-20 range of specification to almost the same extent and that the other two inner layers were almost unaffected. However, these two layers were not significantly hardened as previously indicated by the results of stage extracting the RAP only. Figures 2–4 are schematic diagrams of the four layers and the penetration and viscosity distributions along the treated asphalt film.

Effect of Rejuvenator in Combination with Virgin Aggregate

Because a hot-mix recycling operation generally requires the use of virgin aggregate, it was imperative to include the effect of rejuvenators on old binder in the presence of virgin aggregate. The amount and gradation of aggregate added were selected to keep the treated binder content at 6 percent by weight of mix (same as binder content in RAP) and the total aggregate fraction gradation within the No. 12 surface range of specification, which is commonly used in Indiana for producing hot-mix bituminous pavement. These two requirements were met by using 60, 55, and 15 percent of virgin aggregate by total aggregate weight for the mixes treated with AC-2.5, AE-150, and Mobilsol-30, respectively. The gradation used was the specification midpoint of No. 12 surface given in Table 2. The heated rejuvenator (AC-2.5, AE-150, or Mobilsol-30) was added during the mixing of the heated virgin aggregate-RAP combination except for the Mobilsol-30 that was mixed with the RAP directly before the addition of hot virgin aggregates. Table 7 gives the penetration and viscosity (140°F) values of the reclaimed stage-extracted binder. The results suggest that both rejuvenators (AC-2.5 and Mobilsol-30) were attracted to the old asphalt binder, softened it, and then covered the virgin aggregate. However, this was not the case for the AE-150; its

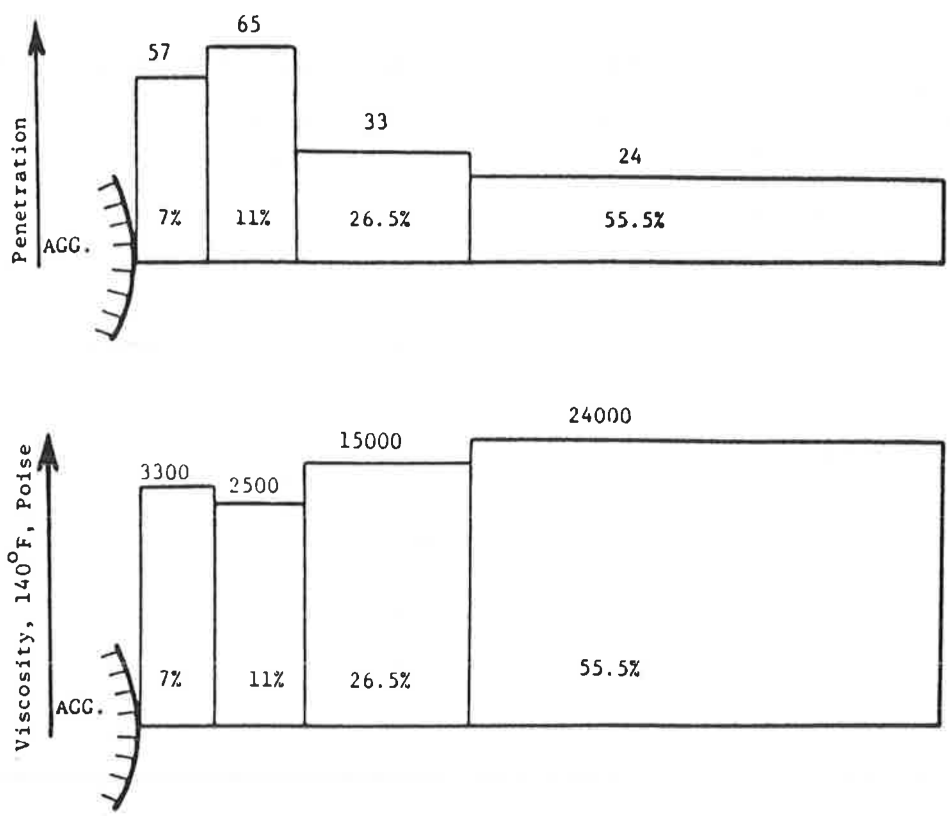

FIGURE 1 Consistency distribution throughout the binder film, RAP only.

TABLE 6 TEST RESULTS ON RECLAIMED, STAGE-EXTRACTED, TREATED BINDER—NO VIRGIN AGGREGATE

Binder	TCE Increment (mL)	Binder (% by weight)	Penetration	Viscosity at 140°F (poises)
60% AC-2.5,	200	67.5	67	1674
40% old asphalt	200	21.5	68	1880
	300	7	59	2394
	700	4	50	3000
55% AE-150 residue,	200	69	75	1683
45% old asphalt	200	16.5	70	2010
	300	8.5	62	2290
	700	6	49	3020
15% Mobilsol-30 residue,	200	71	75	1864
85% old asphalt	200	18	69	1980
	300	6	63	2040
	700	4	48	3152

NOTE: It was not possible to keep the percentage of treated binder at 6 percent (original percentage in RAP) because no virgin aggregate was added. Treated binder contents by weight of mix were 13.75, 12.5, and 7 percent for the AC-2.5, AE-150, and Mobilsol-30, respectively.

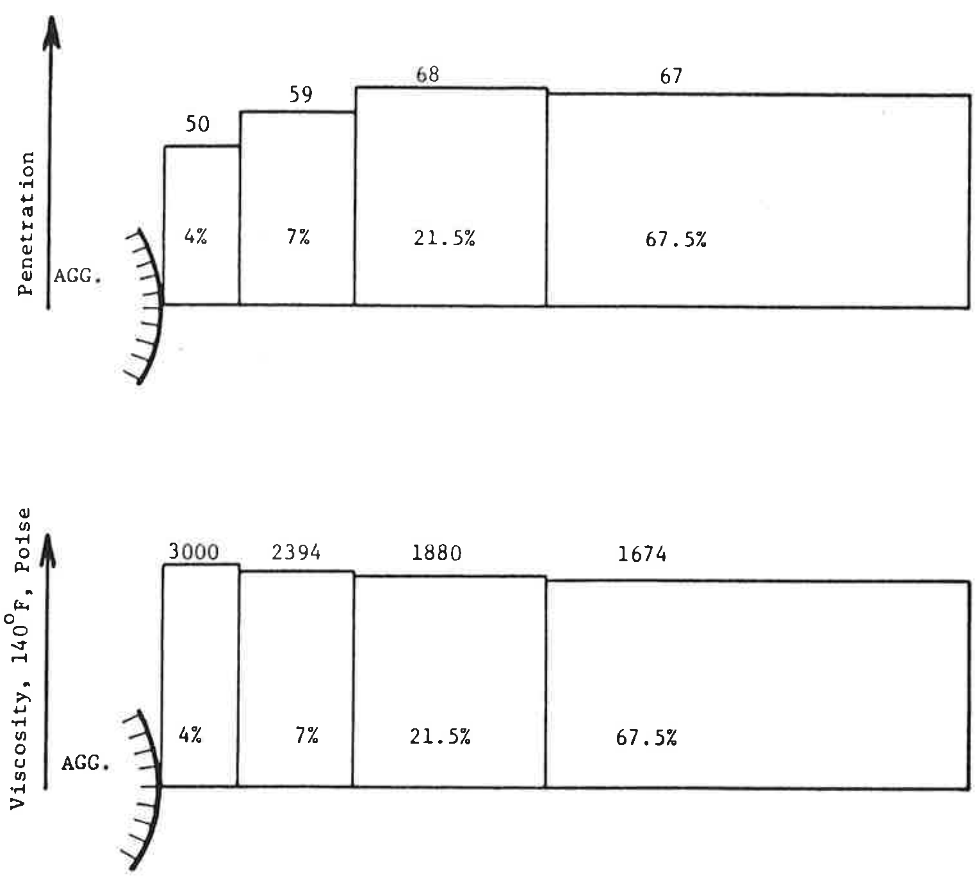

FIGURE 2 Consistency distribution throughout the binder film (RAP plus AC-2.5), no virgin aggregate.

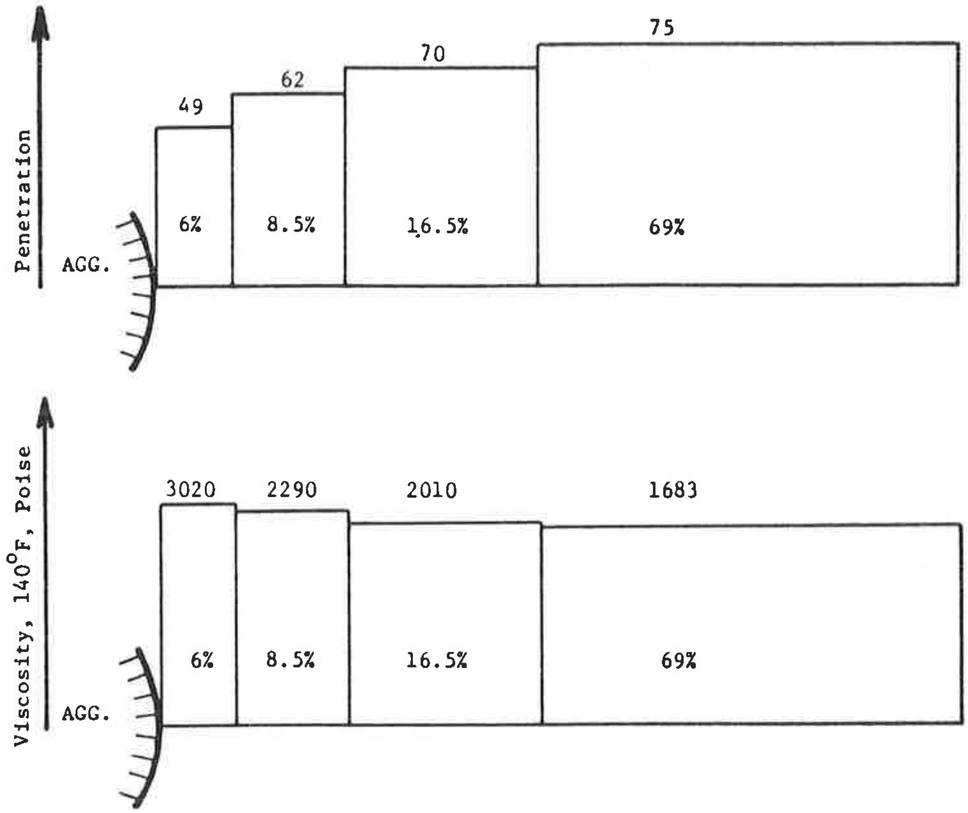

FIGURE 3 Consistency distribution throughout the binder film (RAP plus AE-150), no virgin aggregate.

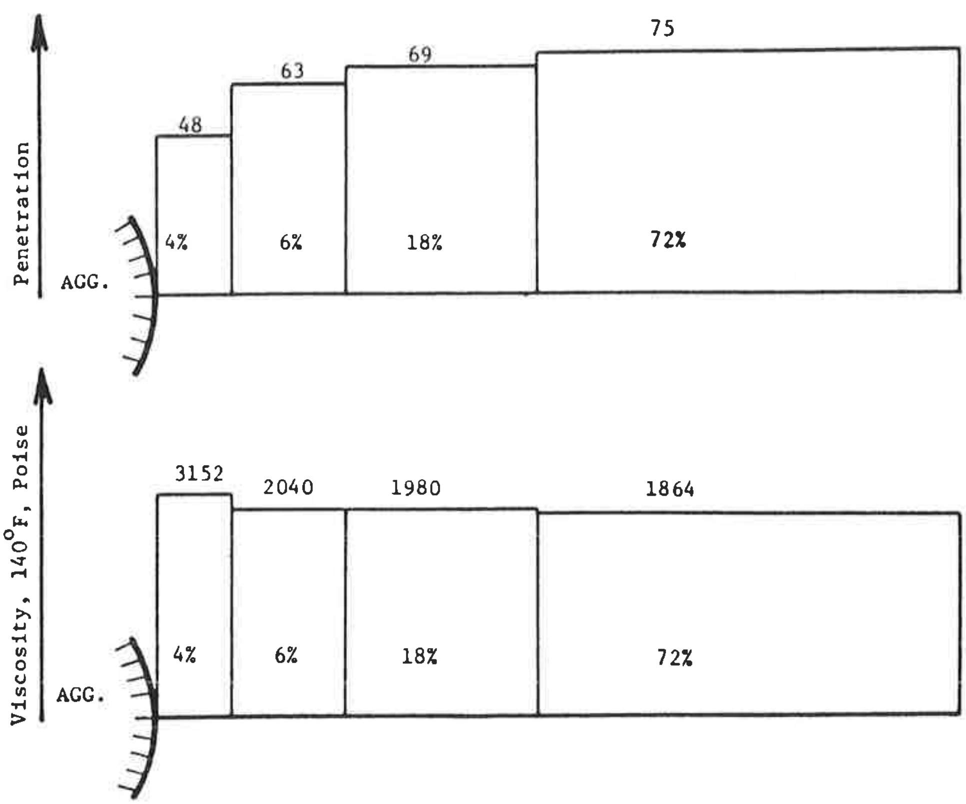

FIGURE 4 Consistency distribution throughout the binder film (RAP plus Mobilsol-30), no virgin aggregate.

TABLE 7 TEST RESULTS ON RECLAIMED, STAGE-EXTRACTED, TREATED BINDER—VIRGIN AGGREGATE USED

Binder	TCE Increment (mL)	Binder (% by weight)	Penetration	Viscosity at 140°F (poises)
60% AC-2.5,	200	72	60	2100
40% old asphalt	200	19	51	2892
	200	19	51	2892
	300	5.5	52	2470
	700	3.5	130	809
55% AE-150 residue,	200	71	70	1972
45% old asphalt	200	19	67	1734
	300	6	60	2424
	700	4	50	3616
15% Mobilsol-30 residue,	200	74	73	2049
85% old asphalt	200	17.5	80	1664
	300	5.5	90	1260
	700	3.5	100	1240

NOTE: 6 percent binder was used in all mixes.

results indicated a consistency gradient that was almost identical to the one for no virgin aggregate.

In general, the consistency of the four microlayers of the treated binder (representing the whole film of asphalt coating the aggregate) characterized by penetration and viscosity (at 140°F) results was similar to that of AC-20, which indicates that the rejuvenators (AC-2.5, AE-150, and Mobilsol-30) diffused well through the hard asphalt film and restored its properties to the AC-20 specification range. Figures 5–7 are schematic diagrams of the four layers and the penetration and viscosity distributions along the treated asphalt film.

Development of Microlayers and Theoretical Implications

It has been observed that the penetration and viscosity (at 140°F) values for the four microlayers of asphalt film extracted and reclaimed from all samples used in this study are logarithmically additive. In other words, if $\mathrm{Log}_{10}A$, $\mathrm{Log}_{10}B$, $\mathrm{Log}_{10}C$, and $\mathrm{Log}_{10}D$ represent the logarithmic values for the penetration or the viscosity (140°F) of the four microlayers and $\mathrm{Log}_{10}T$ represents that value for the whole asphalt film, it was observed that

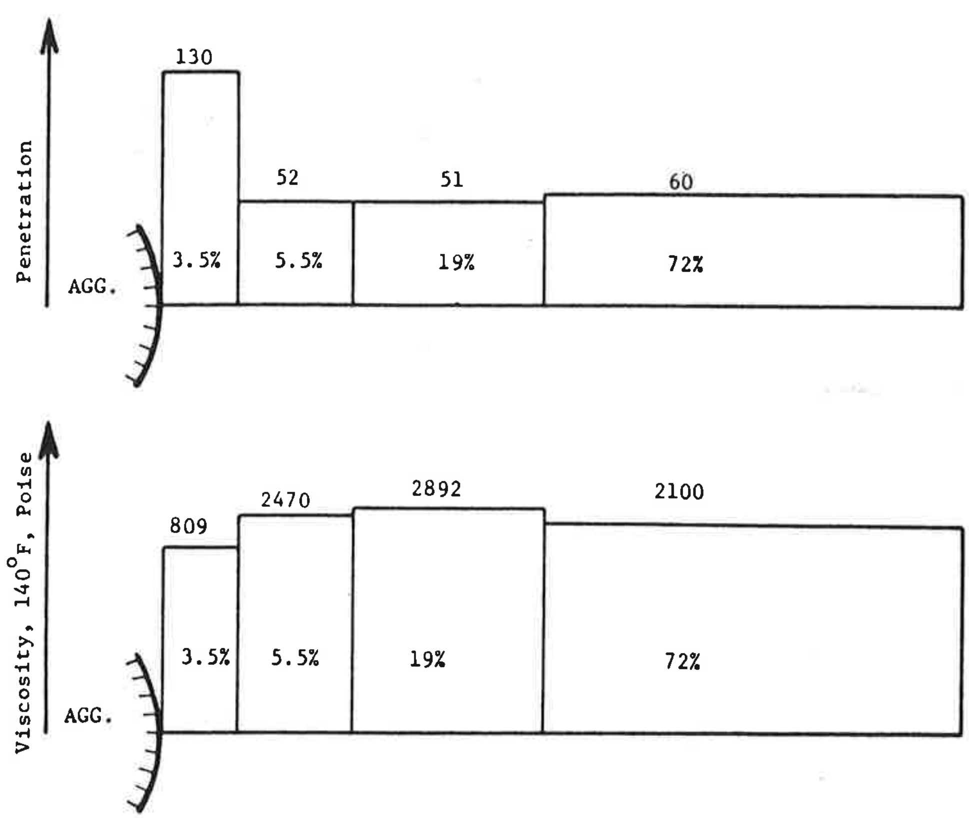

FIGURE 5 Consistency distribution throughout the binder film (RAP plus AC-2.5), with virgin aggregate.

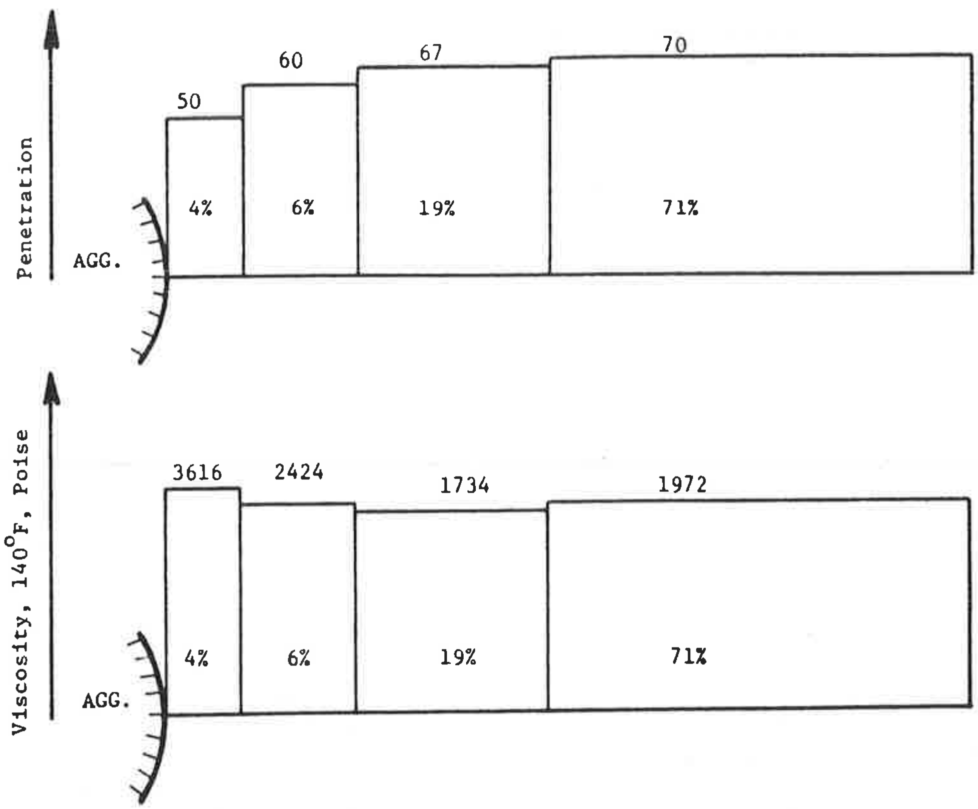

FIGURE 6 **Consistency distribution throughout the binder film (RAP plus AE-150), with virgin aggregate.**

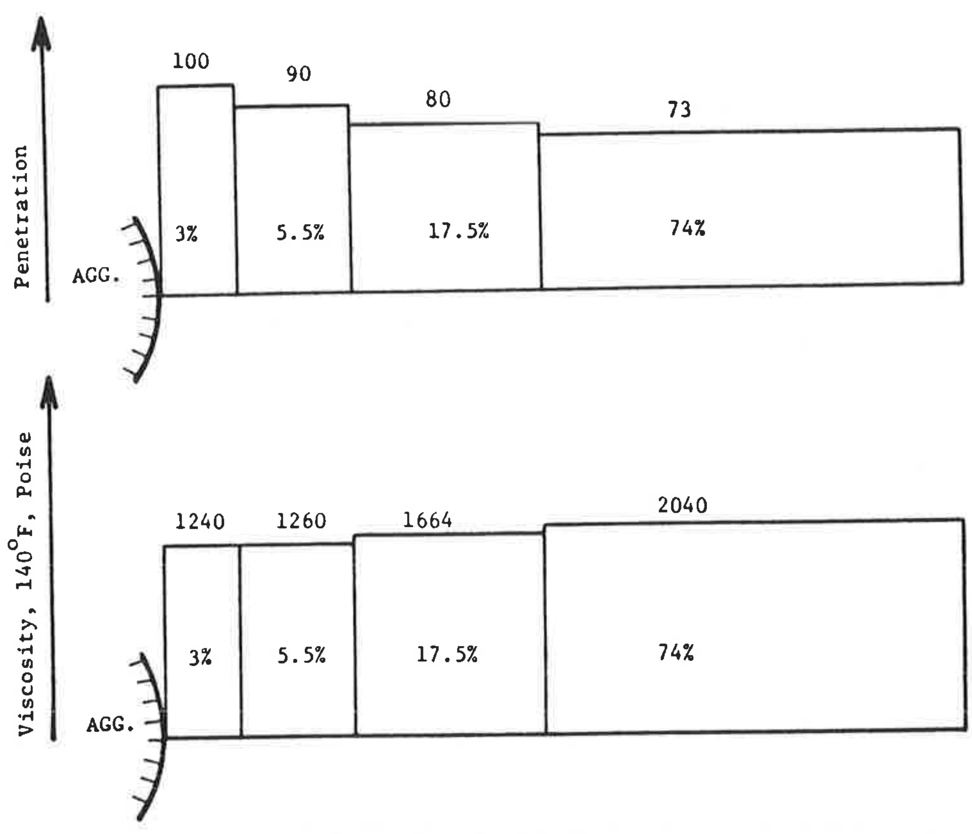

FIGURE 7 **Consistency distribution throughout the binder film (RAP plus Mobilsol-30), with virgin aggregate.**

$$\text{Log}_{10}T = P1\ \text{Log}_{10}A + P2\ \text{Log}_{10}B + P3\ \text{Log}_{10}C + P4\ \text{Log}_{10}D$$

where $P1$, $P2$, $P3$, and $P4$ are the percentages by weight of the four microlayers. Taking the RAP rejuvenated by Mobilsol-30 as an example, values for $P1$, $P2$, $P3$, and $P4$ are 0.72, 0.18, 0.06, and 0.04 (Table 6) and viscosity values A, B, C, and D are 1864, 1980, 2040, and 3152 poises (Table 6). If these values are substituted in the equation, T is 1935, which is close to its test value of 1974 (Table 4).

Because the proof of this relationship would entail a research effort that is beyond the magnitude of this study, it was necessary to include it only as an observation. However, this relationship can be used to develop the results for the four microlayers obtained in this study into 10 microlayers or more. Figures 8 and 9 show the relationship between the percentage of binder extracted and the penetration or the viscosity (at 140°F) of the extracted old binder (RAP) and the RAP treated with AC-2.5, AE-150, and Mobilsol-30 when virgin aggregate is used. It would be possible to predict the penetration or viscosity value of the last 5 percent microlayer (at the binder-aggregate interface) by obtaining the value of viscosity or penetration at 95 percent binder extracted (A) and at 100 percent binder extracted (T) and substituting these values in the previous expression. Taking the untreated RAP as an example, the A penetration value is 27 at 95 percent binder extracted (Figure 8), and the T penetration value at 100 percent binder extracted is 28 (Figure 8). $P1$ and $P2$ are 0.95 and 0.05. Substituting these values in the equation yields a B penetration value of 56, which is close to the test value of 57 given in Table 5.

SUMMARY OF RESULTS

The salvaged material used in this study was obtained from US-52 in Indiana. The recycling agents applied to the salvaged material were AC-2.5, AE-150, and a commercial type (Mobilsol-30). Stage extraction was conducted using Method A (ASTM D-2172). Analysis and evaluation of the test data revealed some new aspects of hot-mix recycled bituminous pavement. However, it is imperative to indicate that the significant results obtained may be limited to the materials used and the test conditions applied in this study.

The main findings can be summarized as follows:

1. Stage extraction of hard asphalt film for the RAP indicated a nonuniform consistency distribution. The outer microlayer of the binder film was severely hardened by direct exposure to weathering actions. However, the second microlayer was less hardened and the third layer appeared to retain its original consistency. The slight hardening of the inner microlayer (at the asphalt-aggregate interface) may be due to the

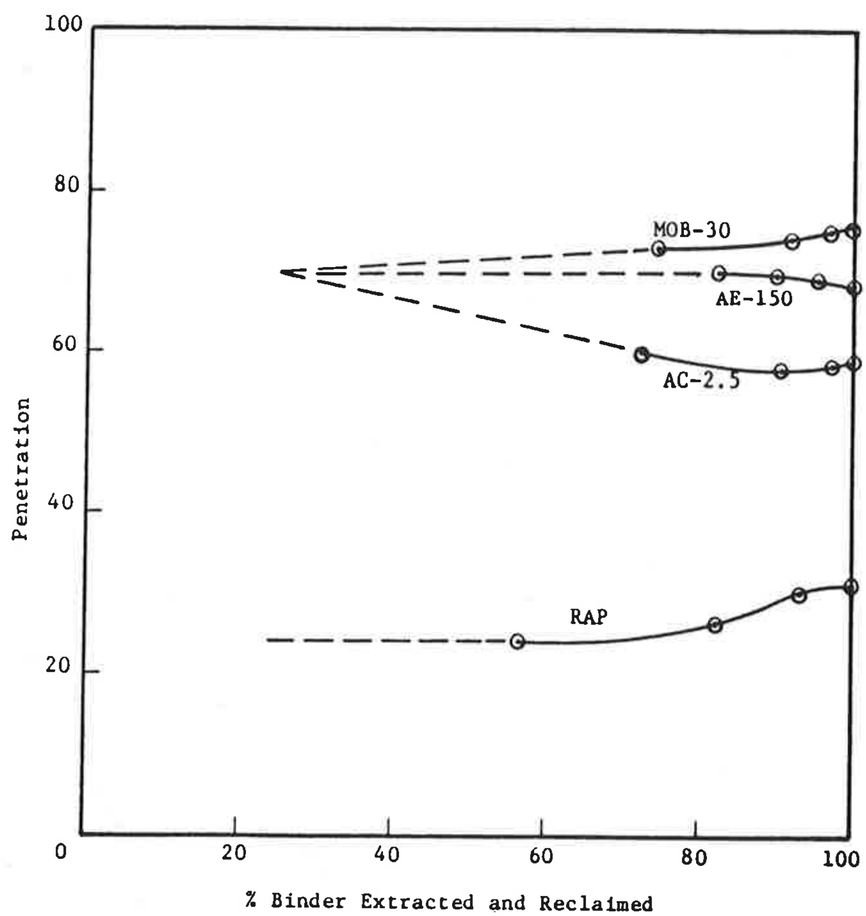

FIGURE 8 Relationship between percentage of extracted and reclaimed binder and penetration (with virgin aggregate).

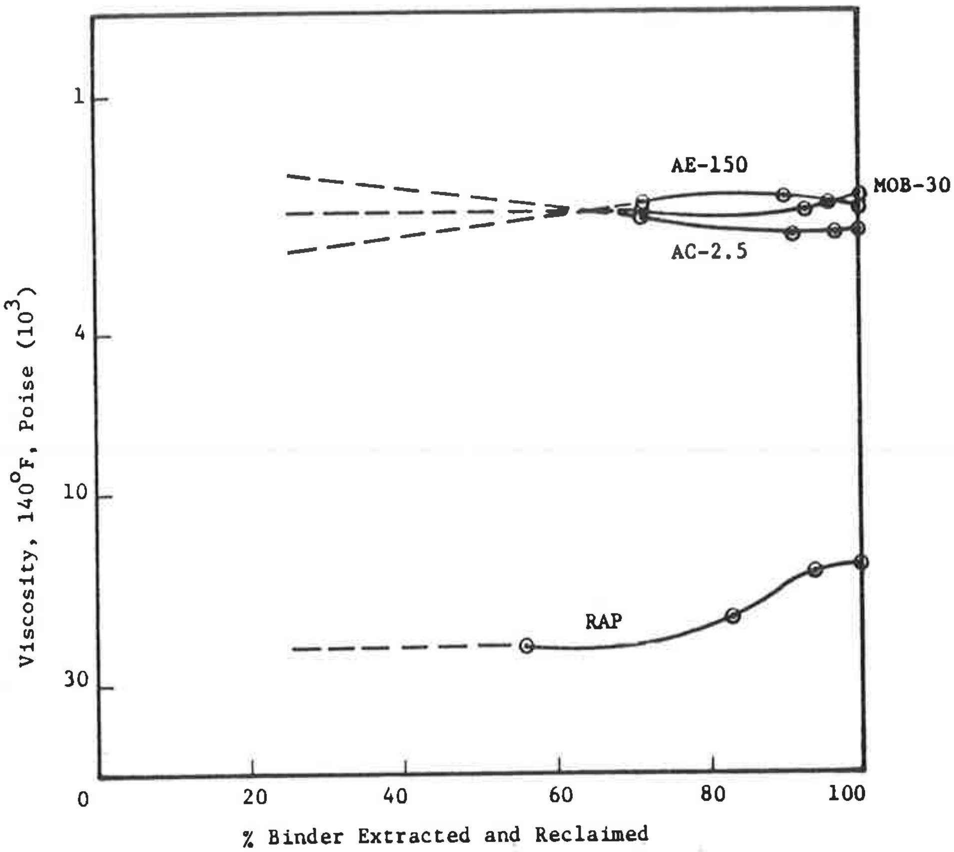

FIGURE 9 Relationship between percentage of extracted and reclaimed binder and viscosity (with virgin aggregate).

tendency of limestone (commonly used in Indiana) to absorb light fractions.

2. Stage extraction of the binder rejuvenated by AC-2.5, AE-150, or Mobilsol-30 without the addition of virgin aggregate indicated that the rejuvenators are most effective at softening the hardened binder on the outer two microlayers of the asphalt film.

3. Stage extraction of rejuvenated binders in the presence of virgin aggregate indicated variable trends in the consistency distribution of the asphalt film on the aggregate. The attraction of the low-viscosity rapidly softened binder to the virgin aggregate may have been the cause of these inconsistent trends.

4. In general, all three rejuvenators were successful in restoring the old hardened asphalt film to the AC-20 specification range.

5. The three recycling agents used displayed good efficiency in diffusing through the hard asphalt film as indicated by stage extraction test results after 15 hr.

ACKNOWLEDGMENTS

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Laboratory Performance Comparisons of Polymer-Modified and Unmodified Asphalt Concrete Mixtures

S. H. CARPENTER AND TOM VANDAM

The use of additives to improve the performance of asphalt cement and asphalt concrete mixtures has increased in recent years. Polymeric additives have been proposed as a potential source of specific improvements to asphalt cements. The major use has been in surface dressings; work in asphalt concrete has only recently been conducted. Presented in this paper are the results of a comprehensive series of laboratory tests on a series of polymer additives to establish test data that can serve as performance indicators for the mixes. Five polymer blends were manufactured from a base AC-5 asphalt cement. Three standard grades of asphalt cement, the base AC-5, an AC-10, and an AC-20, were used for control. A crushed limestone was used to prepare dense-graded mixtures that met the Illinois Interstate overlay mixture criteria. Testing included the diametral resilient modulus test at three temperature levels, indirect tensile testing at temperatures ranging from 72°F to −20°F, permanent deformation testing at 72°F and 100°F, and Lottman moisture susceptibility testing. The testing indicated that the polymer additives reduced stiffness at low temperatures yet maintained adequate stiffness at elevated temperatures. Low-temperature performance was greatly improved over that of untreated asphalt cements of all grades. The permanent deformation characteristics were greatly improved at elevated temperatures. No moisture sensitivity was noted in any of the samples. In general, the polymeric additives improved the base asphalt characteristics to those of the next stiffer grade at normal temperatures; made them better than the base asphalt at low temperatures; and made them better than an AC-20, two grades stiffer than the base asphalt, at elevated temperatures. Long-term fatigue characteristics will require further testing before the influence of the modifiers can be evaluated.

A major reason for using polymer-modified asphalt cements is to increase the level of field performance of the asphalt concrete pavement. To be successful in the marketplace, this increased performance should offset any increased expenditures associated with incorporating the polymer in the asphalt cement. The areas of performance critical to the long-term performance of flexible pavements are

- Stiffness and stiffness-temperature relationships,
- Fatigue resistance,
- Permanent deformation resistance,
- Low-temperature cracking resistance, and
- Strength characteristics.

Although all of these properties can be presented independently, it is obvious that they are all interrelated. It is difficult to present data for one property without discussing overall performance. As an example, the tensile strength of a mixture is useful in categorizing fatigue performance at 72°F, and at extremely low temperatures it, along with stiffness, also indicates resistance to low-temperature cracking.

Presented in this paper are the results of laboratory testing to characterize performance differences among five different polymer blends and three unmodified asphalts. The base asphalt is an AC-5 that was used in all polymer blends. The remaining unmodified asphalts are an AC-10 and an AC-20. The polymers used were various amounts and types of Kraton®, a proprietary polymer from Shell Development Company, Houston, Texas.

PREPARATION OF SAMPLES

The asphalts prepared for use in this study were made from an AC-5 modified with the following polymers:

1. 3 percent Kraton® D-1101,
2. 6 percent Kraton® D-1101,
3. 3 percent Kraton® D-1650,
4. Experimental polymer, and
5. 2.85 percent Kraton® D-1116 with 1.14 percent Kraton® D-1107.

The three grades of asphalt cement used for control purposes were

6. AC-5 (base asphalt cement used in all polymer blends),
7. AC-10, and
8. AC-20.

The properties of the asphalt cements formed with these polymer combinations are given in Table 1. Marshall mix design was performed to determine optimum properties for 4 percent air voids. The optimum values used in this study *(1)* are given in Table 2. The aggregate was a crushed limestone blended to the dense gradation required by the Illinois Department of Transportation for new Interstate overlay mixes as shown in Figure 1. This mix design was used for both the 50- and the 75-blow Marshall test. The 50-blow Marshall test was used for the main laboratory testing program because this is the standard mix design procedure used in Illinois.

Marshall samples (4 in. diameter and 2.5 in. high) were compacted at both 75 and 50 blows at optimum asphalt content and at asphalt levels 0.5 percent above and below optimum.

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TABLE 1 PROPERTIES OF ASPHALT CEMENTS EVALUATED IN THIS STUDY

	Modified Exxon AC-5 Base Asphalt with Treatment					Unmodified		
Property	1[a]	2[b]	3[c]	4[d]	5[e]	Exxon AC-5 with Treatment 6	Shell AC-10 with Treatment 7	Shell AC-20 with Treatment 8
Penetration at 25°C (dmm)	100	78	104	103	97	128	84	59
Viscosity at 80°C (poises)	560	–	200	282	628	78	131	207
Viscosity at 135°C (cPs)	570	1675	525	660	905	250	365	480
Ductility, 4°C, 5 cm/min (cm)	98	91	23	34	104	31	3	0
Softening point, ring and ball (°F)	121	193	118	120	130	112	118	123
Penetration index	+0.5	+6.8	+0.2	+0.5	+1.0	−0.9	−0.6	−0.7
Pen-vis no.	+0.2	+1.0	+0.2	+0.5	+1.0	−0.9	−0.6	−0.5
Toughness (in./lb)	85	171	85	177	144	17	42	73
Tenacity (in./lb)	67	141	20	147	126	10	11	12

[a]3 percent Kraton® D-1101.
[b]6 percent Kraton® D-1101.
[c]3 percent Kraton® G-1650.
[d]Experimental polymer.
[e]2.86 percent Kraton® D-116 with 1.14 percent Kraton® D-1107, 4 percent polymer content.

TABLE 2 MIX DESIGN PROPERTIES OF SAMPLES TESTED

Treatment	Asphalt Content (%)	G_{mm}[a]	G_{mb}[b]	Air Voids (%)
1	5.75	2.457	2.384	2.97
2	5.75	2.457	2.388	2.77
3	6.5	2.431	2.407	1.02
4	6.2	2.442	2.399	2.09
5	6.25	2.440	2.399	1.68
6	6.0	2.448	2.379	2.80
7	6.25	2.450	2.392	2.37
8	6.5	2.432	2.400	1.32

[a]G_{mm} = maximum theoretical density or specific gravity of an asphalt mixture.
[b]G_{mb} = density or bulk specific gravity of an asphalt mixture.

This was done to determine any relative influence of asphalt content and compaction on performance in the indirect tensile strength determinations.

The creep compliance and permanent deformation testing required cylindrical samples 4 in. in diameter and 7 in. tall. The California kneading compactor was used to compact these cylinders. This type of compaction has been shown by other investigators *(2)* to produce a sample that better approximates a field-compacted material for use in determining permanent deformation characteristics. The kneading compactor was calibrated with an AC-10 asphalt cement to establish operating characteristics that would produce compacted samples with the same density as produced by 50-blow Marshall compaction. On average, the densities were higher with the kneading com-

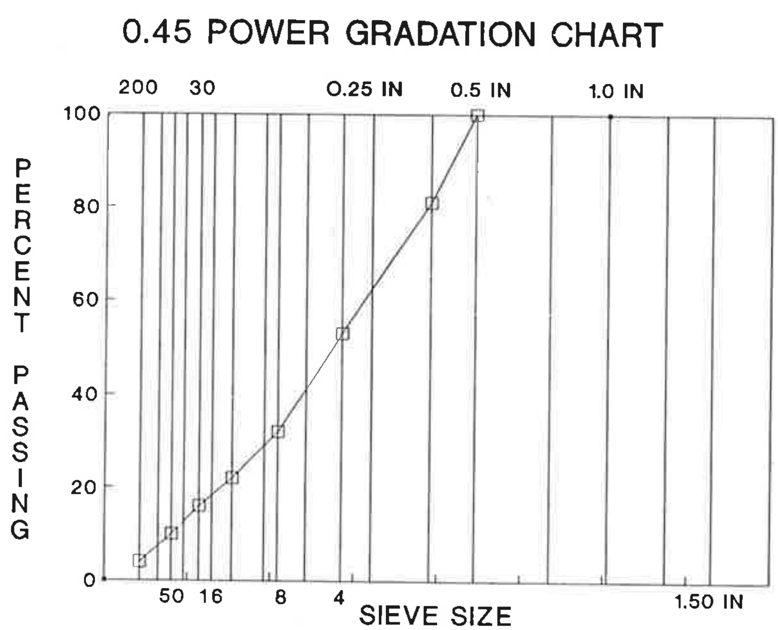

FIGURE 1 Gradation of Illinois dense-graded Interstate concrete overlay mixture.

pactor, and the air voids were lower than anticipated (Table 2). All asphalt contents were chosen to provide 4 percent air voids. This increase in density illustrates an interesting point about polymer-modified asphalt cements: at large strain levels, the resistance of the modified cement to deformation is significantly less than that of a normal asphalt cement of the same grade consistency compacted at the same temperatures *(3)*. In comparison with the Marshall hammer method of compaction, the kneading compactor is a large-strain device.

The cylindrical samples were trimmed using a concrete saw to provide parallel, polished ends for creep and permanent deformation testing. This produced a sample with a minimum length of 6 in. Samples not used for creep and permanent deformation testing were prepared for use as thermal contraction bars and samples for low-temperature indirect tensile strength determinations by cutting them into samples 4 in. in diameter with heights of approximately 2 in. The use of these samples for indirect tensile testing was significant because it showed that the influence of the kneading compactor extends to strains measured in the indirect tensile test as well as in the permanent deformation test.

TESTS

Stiffness

Stiffness determinations were performed using the diametral resilient modulus device. A 0.1-sec pulse load is applied along the vertical diameter of the sample, and the horizontal deformation is recorded.

The test is performed on two diameters of the sample, 90 degrees apart, and averaged. Testing was performed at three temperatures, 40°F, 72°F, and 100°F, to cover the values normally found in typical pavement structures in the United States that would be used as yearly design temperatures. These temperatures cover the range for high-temperature stability. Low-temperature cracking requires testing at lower temperatures and does not involve the diametral resilient modulus device.

Tensile Strength

The tensile strength of compacted asphalt concrete specimens is typically determined by the indirect tensile test or the Brazilian split test. This test is conceptually similar to the diametral resilient modulus test, except that the load is applied at a constant rate and is increased until failure occurs. The load and deformation applied to the vertical diameter are recorded along with the deformations along the horizontal axis. Indirect tensile strength, stiffness, tensile strain, compressive strain, Poisson's ratio, and vertical deformation at failure can be calculated from formulas given elsewhere *(4)*. The coefficients for the equations will vary with sample diameter. All samples tested in this study were standard Marshall size with a diameter of 4 in.

Testing was performed at 72°F with a rate of deformation of 2.0 in./min. Testing under these conditions evaluates the quality of the mixture and can provide an indication of the fatigue resistance of mixtures compared with that of normal asphalt cements. There is some question about whether polymer-modified asphalt cements tested in indirect tension provide accurate fatigue data as will be seen later. Subsequent testing was conducted at 40°F, 20°F, 0°F, and −20°F at a deformation rate of 0.05 in./min to provide an indication of the low-temperature performance of the mixtures.

Permanent Deformation

The permanent deformation or rutting resistance of a mixture must be evaluated using cylindrical samples. Loads are applied to the vertical axis of the cylinder, and the total deformation under the load is recorded by a noncontacting sensor. The test equipment used can perform a standard creep compliance test and a continual repeated load test. The procedure used in this study to develop permanent deformation characteristics was the FHWA incremental-static procedure, which defines the VESYS *ALPHA* and *GNU* parameters *(5)*.

The VESYS incremental-static test sequence consists of a series of load applications and measurement of the deformation remaining after a specified period of rest with no load applied. Permanent deformation versus loading time on logarithmic scales furnishes an indication of the relative resistance to permanent deformation of the mixture. Previous studies have clearly shown that the time of loading in a creep test is directly related to the accumulation of permanent deformation under repeated loadings *(6)*. The calculations for *ALPHA* and *GNU* are shown in Figure 2.

Thermal Coefficient of Expansion

Determination of the coefficient of expansion or contraction is an integral part of the analysis of the thermal behavior of mixes. The cylindrical samples formed with kneading compaction for use in the creep and permanent deformation study were used to determine the thermal coefficient of contraction. Samples not used in creep testing were cut into quarters. Set points were epoxied into the ends of two of the quartered bars. The length of the bars was monitored with a 0.0001-in. dial gauge. The asphalt concrete bars were placed in an environmental chamber for 24 hr at each temperature, and the length of the bar was recorded after the 24-hr temperature cycle. Temperature levels investigated included 72°F, 40°F, 20°F, 0°F, and −20°F. Readings were taken during both the cooling and the heating cycle for comparison.

TEST RESULTS

Diametral Resilient Modulus

The plots of stiffness as a function of temperature are shown in Figure 3. The relationship of stiffness and temperature for each type of asphalt is not unexpected when the temperature susceptibility parameters of the asphalt cements are considered. In particular, the pen-vis number (PVN) and the penetration index (PI) values indicate that the polymer-modified asphalt cements (Table 1) are less temperature susceptible than are the unmodified cements. Although the use of PI or PVN for modified asphalts is of questionable accuracy, these values show that the

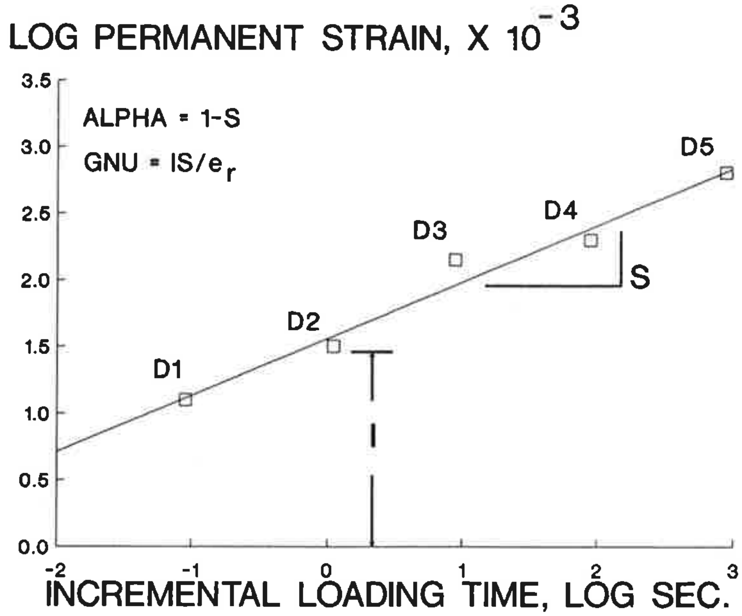

FIGURE 2 Schematic results of VESYS incremental-static analysis.

polymer-modified asphalt cements have the potential to provide an asphalt concrete that is stiffer at high temperatures and less stiff at low temperatures than are mixes made with asphalt cements that are a stiffer grade than the base asphalt used to formulate the polymer-modified asphalt cement. This has the potential to provide an asphalt concrete capable of resisting rutting at high temperatures and thermal cracking at low temperatures.

The influence of the polymers on the stiffness of the mixes is readily apparent, and the following comparisons between them and the three control asphalt cements can be made. The polymer-modified mixes are stiffer than the base AC-5 until the temperature drops below approximately 20°F. Below this temperature the polymer mixes are softer than the AC-5. The modified mixes show stiffnesses intermediate between those of the AC-5 and the AC-20 at elevated temperatures. Treatment 2 produced stiffness values similar to those of the AC-20 at 72°F. Extrapolating the data below 40°F indicates that the blends will remain more flexible than the AC-10 and AC-20, and below 20°F the polymer blends are more flexible than the base asphalt.

These mixtures were all compacted at the optimum asphalt content determined for each particular asphalt blend from the Marshall procedure. In general, a stiffer asphalt cement will

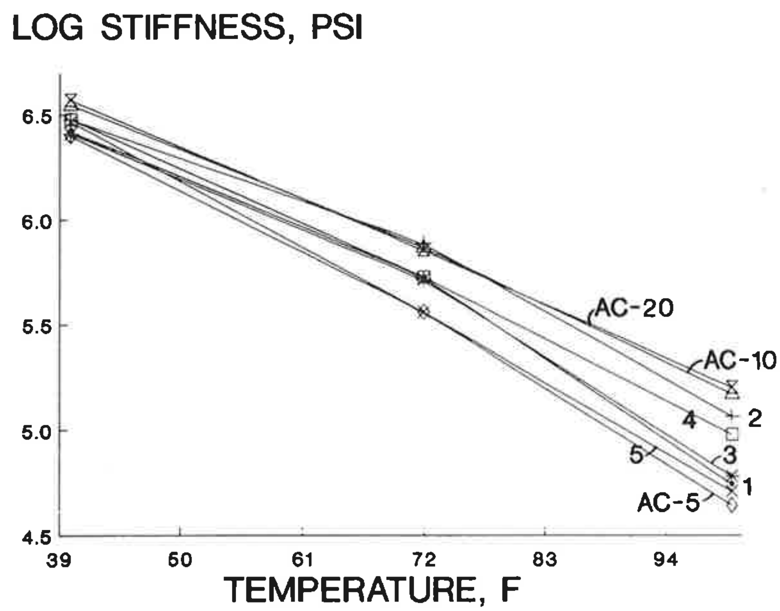

FIGURE 3 Stiffness versus temperature for treated and untreated samples.

TABLE 3 TENSILE STRENGTH RESULTS

Asphalt	Blows	Tensile Strength (psi)	Strain
1	50	105	0.00547
	75	132	0.00670
2	50	139	0.00654
	75	123	0.00812
3	50	110	0.00806
	75	136	0.00535
4	50	135	0.00609
	75	155	0.00569
5	50	124	0.00871
	75	128	0.00622
6	50	95	0.00791
	75	95	0.00846
7	50	164	0.00663
	75	152	0.00546
8	50	228	0.00561
	75	214	0.00507

require from 0.25 to 0.50 percent more asphalt cement in a mixture compacted at optimum *(1)*. A higher asphalt content typically produces lower stiffness, which may not be totally offset by a stiffer asphalt cement. The influence of the polymers is apparent in the overlapping stiffness curves (Figure 3). The mixtures are stiffer at normal or slightly above normal temperatures and significantly softer at low temperatures compared with the AC-5. Treatment 2 produced a very stiff mix at normal temperatures but was significantly softer at the extremes compared with the AC-10 and the AC-20.

Because of the limited number of data points, the stiffness values of each mixture at asphalt contents above and below optimum are not shown. In general, increasing asphalt content produced a decrease in stiffness. Sometimes, however, this trend was not easily discerned, and a maximum stiffness was indicated at the optimum asphalt content. These trends are typical of asphalt concrete samples with a dense gradation. All stiffness measurements were made on Marshall-compacted samples.

TABLE 4 STIFFNESS AND TENSILE STRENGTH DATA FOR MARSHALL COMPACTED SAMPLES

Treatment	Compaction Level					
	50			75		
	AC %	S_{it}	Stiffness	AC %	Sit	Stiffness
	5.75	110.6	554600	5.75	130	470200
1	5.75	105.2	517800	5.75	132.9	647000
	5.25	128.5	821250	5.25	148.8	700850
2	5.75	134.8	785950	5.75	121	546950
	5.75	143.2	759950	5.75	124.7	662150
	6.25	136.1	693400	6.25	138	651550
	6.00	124	560000	5.25	130.3	752100
3	6.00	98.1	484400	5.75	137.9	754050
	6.50	124	511350	5.75	133.3	801500
	6.50	119.6	592700	6.50	124	757750
	5.75	147.6	702300	5.25	163.6	870100
4	5.75	140.5	539800	5.25	146.7	875150
	6.25	129.6	527300			
	5.75	120.2	484400	5.00	145	754350
5	6.25	132.3	535200	5.50	121.3	500900
	6.25	115.5	458200	5.50	134.4	607000
	6.75	118.9	510850	6.00	133.6	555350
	5.5	92.2	267150	5.50	88.4	467600
6	6.00	93.2	338700	6.00	101.8	369000
	6.00	96.9	390150	6.00	88.9	359350
	6.50	84.6	270100	6.50	94.8	281550
	5.75	162.2	729850	5.00	155.8	773100
7	6.25	170.8	743100	5.50	151.9	732100
	6.25	156.4	738900	5.50	151.9	603300
	6.75	143.1	723300	6.00	166.6	817550
	6.50	202.8	743000	4.75	221.8	1083350
8	6.50	226.2	692050	5.25	230.4	873700
	7.00	185	699150	5.25	226.7	1288500
				5.75	225.9	977050

Indirect Tensile Strength

Indirect tensile strengths were determined at 72°F with a loading rate of 2.0 in./min. Additional testing at 40°F, 20°F, 0°F, and −20°F was done with a loading rate of 0.05 in./min. The 72°F test can be used to indicate fatigue resistance of a mixture prepared with unmodified asphalt cements. The lower temperatures are necessary for characterization of the low-temperature performance of the mixes.

Results at 72°F

The 72°F tensile test results are given in Table 3 for the eight mixes. Tests at these levels were conducted only on Marshall-compacted samples at both compaction levels and several asphalt contents. The results of the stiffness measurements at 72°F, given earlier, are mirrored by the tensile strengths. The data in Table 4 can be used to establish a relationship between indirect tensile strength and dynamic modulus, which has been proposed by others. The data have been plotted in Figure 4 to indicate the relationship. The relationship for this particular gradation, using the different asphalt cements, asphalt contents, and compaction efforts at 72°F, is

$$MR = 35{,}632 + 4{,}446(S_{it}) \qquad (1)$$

where S_{it} is the indirect tensile strength (psi) and MR is the diametral resilient modulus (psi). The R^2 coefficient for this relationship was 0.85. This equation included only Marshall samples, compacted at 50 and 75 blows, below, at, and above optimum asphalt content in 0.5 percent increments.

At 72°F the polymer modifiers clearly increased the tensile strength of the mixes above the levels provided by the base AC-5 asphalt cement but not above that provided by an AC-10 asphalt cement. The tensile strain at failure for the polymer treatments was generally greater than that for the AC-10 and AC-20 mixtures. Increased strain at failure indicates that more strain energy is required to fail the specimen of that mixture, which is sometimes interpreted as a "tougher" mixture. The improvement at 72°F, however, is minimal in terms of increased performance.

Results at Low Temperatures

The tensile strengths and strains from the low-temperature tests are given in Table 5. The tests at the lower temperatures were conducted only on the samples compacted by the kneading compactor.

Tensile strength as a function of temperature is shown in Figure 5 for the untreated asphalts and polymer-modified blends. These curves are typical of dense-graded asphalt concrete mixtures *(4)*. The AC-20 mixture developed its peak strength at approximately 0°F whereas the AC-10 and the AC-5 developed peak strength at approximately −15°F. Typically, the softer grade of asphalt cement provides better resistance to low-temperature cracking by lowering the temperature at which failure occurs as well as increasing the tensile strength at failure.

The polymer-modified mixes demonstrate distinct differences from the untreated asphalt mixes. At approximately 10°F all mixes possess the strength of the AC-20 mixture. Below 10°F, even at very low temperatures, all polymer mixes maintain a higher strength than does the AC-20 mixture. It is thought that the peak strengths of several of the polymer blends may not have been reached because of temperature equipment control limitations that precluded going below −20°F. Improvement in the strength-temperature relationship has a significant impact on resistance to low-temperature thermal cracking in the field.

Tensile strain as a function of temperature is shown in

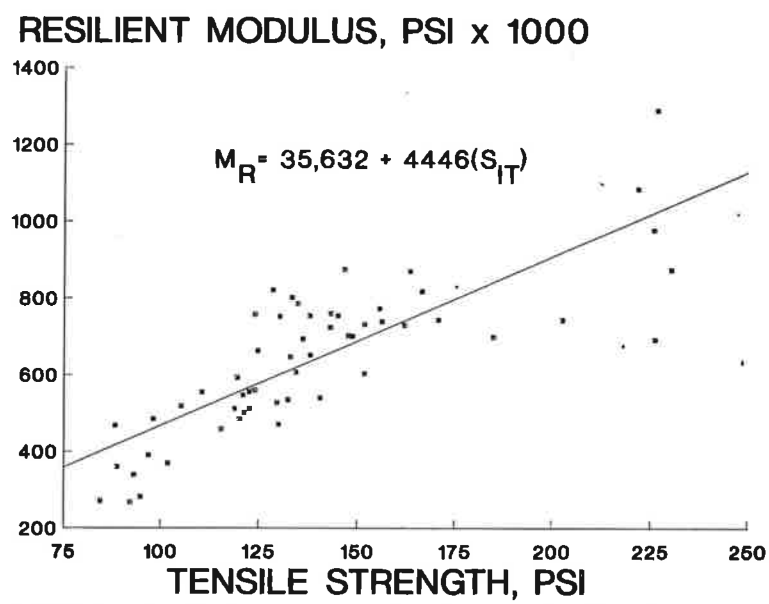

FIGURE 4 Resilient modulus as a function of indirect tensile strength at 72°F.

TABLE 5 TENSILE TEST RESULTS AT VARIOUS TEMPERATURES

Asphalt No.	40 °F Strength, psi	40 °F Strain in/in	10 °F Strength, psi	10 °F Strain in/in	-10 °F Strength, psi	-10 °F Strain in/in	-20 °F Strength, psi	-20 °F Strain in/in
1	-	-	398	0.0048	512	0.0030	476	0.0024
2	162	0.0091	417	0.0051	499	0.0036	479	0.0039
3	155	0.0073	460	0.0011	459	0.0026	492	0.0032
4	170	0.0068	450	0.0023	530	0.0014	559	0.0031
5	159	0.0066	436	0.0031	515	0.0030	510	0.0022
6	139	0.0070	299	0.0018	497	0.0013	432	0.0008
7	199	0.0070	299	0.0018	388	0.0009	450	0.0016
8	-	-	453	0.0012	400	0.0009	372	0.0019

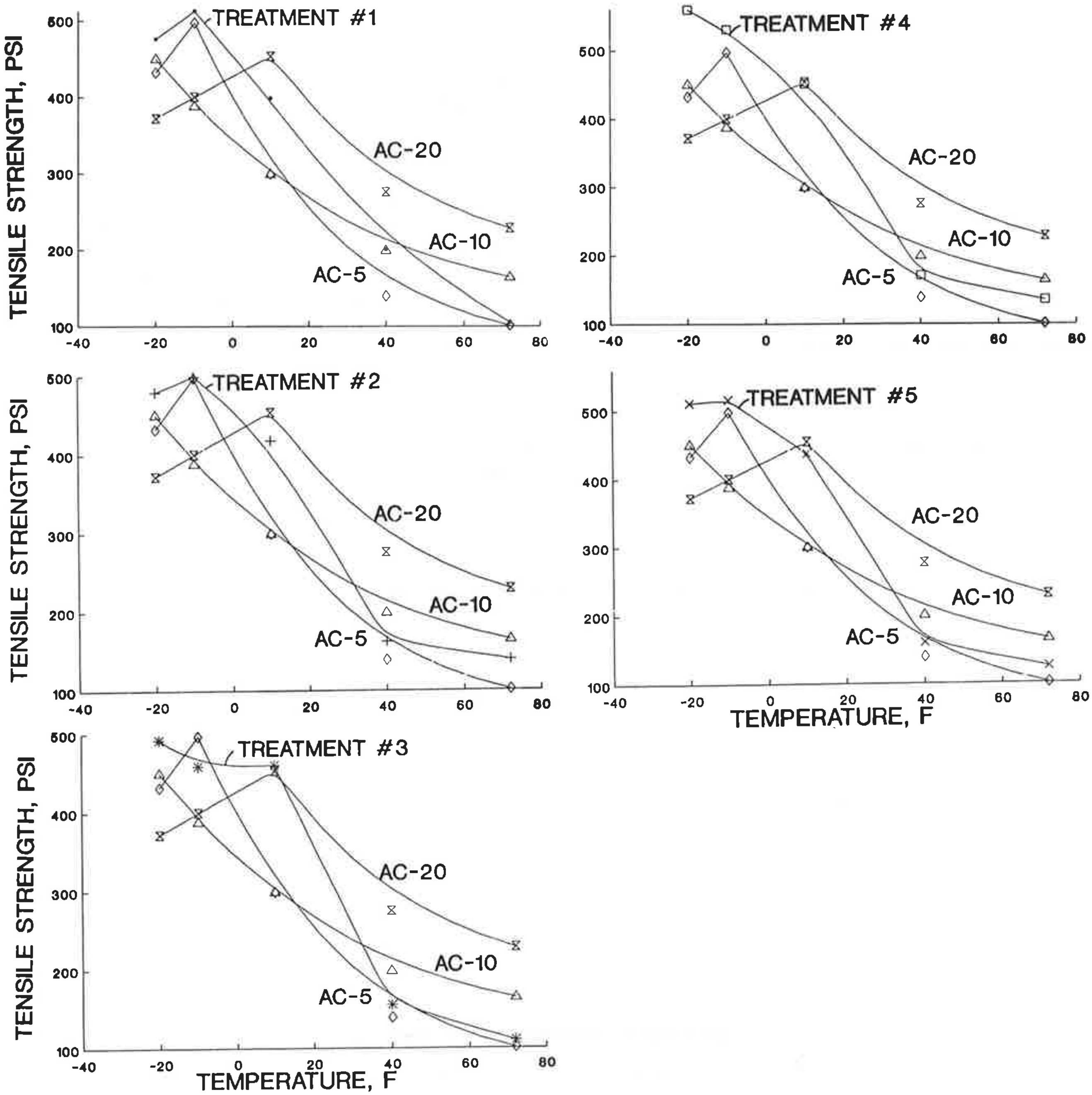

FIGURE 5 Indirect tensile strength as a function of temperature.

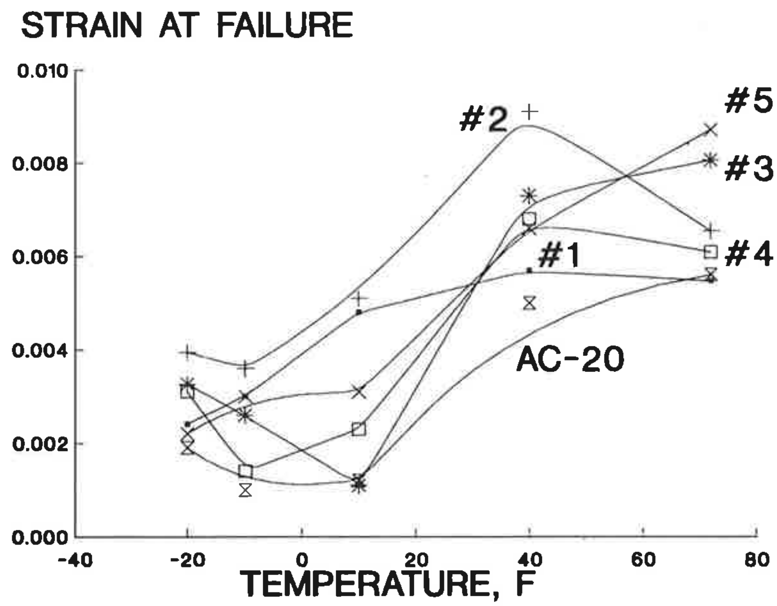

FIGURE 6 Indirect tensile strain as a function of temperature (AC-20).

Figures 6–8 for the untreated and polymer-modified blends. Each figure shows the untreated asphalt cement results superimposed on the results from the treated mixes. The strain at failure in the normal asphalt mixes rapidly decreases to a minimum value, even at moderate temperature levels, and remains at this level to extremely low temperatures. The polymer-modified blends, however, show a significantly different relationship. As shown in these figures, the tensile strain at failure for the polymer-modified blends remains much higher than those of the normal asphalt cements. Most significantly, at −20°F the failure strain in the polymer mixes is two to three times greater than those of the normal asphalt cements. This corresponds to the lower resilient modulus stiffness values indicated for these mixes at low temperatures. From these data, it would appear that Mixture 2 provides the best strain at failure and that Mixtures 1 and 3 perform quite well. It is evident that the polymers significantly modify the low-temperature performance of the mixes. This modification is not possible with normal asphalts even if they are of very different grades.

Fatigue Characteristics

Maupin *(7)* has developed statistically valid relationships between the indirect tensile strength and the fatigue characteristics of a dense-graded asphalt concrete mixture compacted by

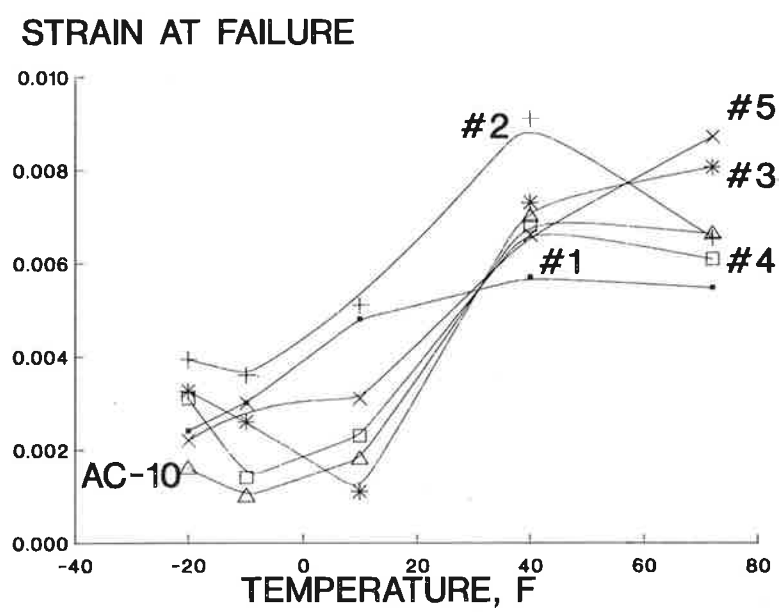

FIGURE 7 Indirect tensile strain as a function of temperature (AC-10).

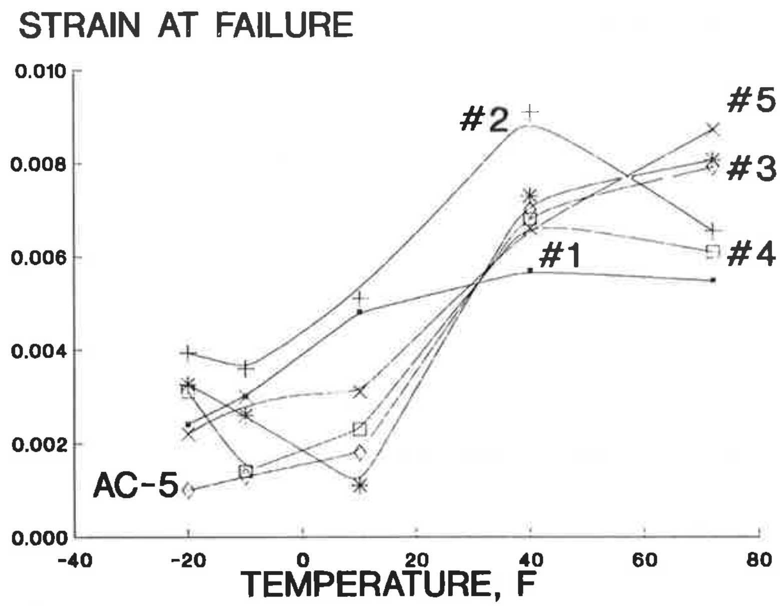

FIGURE 8 Indirect tensile strain as a function of temperature (AC-5).

the Marshall device and tested at 72°F at a rate of 2.0 in./min. These relationships are for the constant strain and constant stress mode of fatigue testing. The relationship for constant strain is

$$N_f = K_2(1/e)^n \tag{2}$$

where

N_f = number of loadings required to reduce the dynamic stiffness modulus by one-third,
e = radial tensile strain in the asphalt concrete layer,
K_2 = $10^{(7.92 - 0.0122\, S_{it})}$,
n = $0.0374\, S_{it} - 0.744$, and
S_{it} = indirect tensile strength (psi).

The constant strain representation of fatigue data is most widely used for normal fatigue testing because of the ease of testing.

The indirect tensile strength data presented in the previous section are all typical of high-quality dense-graded mixes similar to those tested in the development of Maupin's relationships. However, the Illinois gradation and the use of crushed stone and nonstandard asphalts produce mixes with significantly higher tensile strengths than those tested by Maupin. The mixes used in this study may provide a fatigue life that is significantly different from that of a more normal mix that contains natural gravels or sands instead of the crushed limestone. The relative effects of the addition of the polymer asphalts should remain similar to those reported here. The actual use of this relationship for predicting fatigue life for polymer-modified mixes cannot be recommended at present because of the use of different asphalts and the problems inherent in using regression relationships that place limitations on the data used to develop the equation.

As was noted earlier, a problem with polymer-modified asphalt cements is the influence on strain-related performance, particularly strains at large magnitudes, which appears to radically alter the performance of the asphalt cement. Thus it could be expected that actual fatigue testing would show different results than were seen in the indirect tensile strength tests that have not been validated for modified binders. It is expected that the polymeric properties would provide increased fatigue resistance, but this parameter requires further testing in an appropriate fatigue testing configuration.

Permanent Deformation

The *ALPHA* and *GNU* parameters calculated from the static-incremental creep test are given in Table 6. Static-incremental data furnish input parameters for the equation

$$F_{(N)} = GNU^{*}(N)^{-ALPHA} \tag{3}$$

where

N = cumulative load applications;

TABLE 6 *GNU* AND *ALPHA* PERMANENT DEFORMATION PARAMETERS AT TWO TEMPERATURE LEVELS

	72°F		100°F	
Asphalt	*GNU*	*ALPHA*	*GNU*	*ALPHA*
1	0.1770	0.5562	0.1114	0.5521
2	0.1516	0.3731	0.1483	0.5759
3	0.2352	0.5298	0.1136	0.4838
4	0.2713	0.4282	0.1948	0.6622
5	0.5303	0.5626	0.0533	0.5311
6	0.2100	0.4746	0.1193	0.5197
7	0.2984	0.5243	0.1271	0.5855
8	0.2625	0.4775	0.1358	0.4557

GNU = IS/e_r;
I = intercept at Log $t = 0$;
S = slope, as shown in Figure 5;
e_r = resilient strain in the sample under the stress used in the test;
ALPHA = 1.0 − *S*; and
$F_{(N)}$ = percentage of the resilient strain occurring in the asphalt layer under load *N* that will become permanent.

It can be seen from Equation 3 that as *GNU* increases permanent deformation increases and that as *ALPHA* decreases permanent deformation increases. However, this increase is only in the percentage of the total deformation under a load that becomes permanent and thus calculation of the actual magnitude of permanent deformation is dependent on the stiffness of the mixture. Any one mixture may have *ALPHA* and *GNU* parameters that allow a higher percentage of the strain to become permanent, but, if the mixture has a very high stiffness, the total strain occurring under one wheel load will be small, and the accumulation of permanent deformation will develop at a much slower rate than in another sample with different (better) parameters but a low stiffness.

To show the true relative amount of permanent deformation that develops in each of the mixtures tested, a computer program was prepared to use the *ALPHA* and *GNU* parameters and perform the accumulation of the $F_{(N)}$ percentage in a 15-year period with Interstate levels of traffic. A 3-in. asphalt concrete layer and an 80-psi vertical stress were used. The diametral resilient modulus of each mixture as determined from the 50-blow Marshall sample was used to calculate the strain occurring in a 3-in. layer under the 80-psi stress. Equal traffic levels were applied to all samples. The accumulation of rutting for the untreated asphalt samples is shown in Figure 9 for the 72°F test sequence. Rutting is given in a relative unit for comparison only because the calculated rut depths are for comparison of mixture quality only and are not indicative of actual rut depths that would develop in a complete pavement structure. It is interesting that the AC-10 showed less rutting potential than the AC-20. This may be because the asphalt content of the AC-10 is less than that of the AC-20 from the mix design (6.25 versus 6.5) and because the AC-20 was compacted in the kneading compactor, which produced a denser mix with low air voids of 1.3 percent that increase the rutting potential. The dramatic influence of the soft AC-5 binder is clear even though these samples were constructed with an asphalt content of 6.0 percent.

The rutting curves for the polymer-modified asphalt cements are shown in Figure 10 for the 72°F test sequence. Asphalt 4 showed a much higher potential for permanent deformation than even the AC-5, and all of the other polymer treatments, particularly Treatments 1 and 3, showed less rutting potential than did the AC-5. The low amount of rutting for Treatment 3 is significant because the samples for Treatment 3 were compacted to air voids in the range of 1 percent. This low amount of air voids should produce a mixture that is highly susceptible to rutting, as was shown with the AC-20 mixture. The low amount of rutting potential indicates that the polymer treatment improves the integrity of the mixture.

The rutting curves for the asphalt concretes at 100°F are given in Figures 11 and 12 for the untreated and treated samples, respectively. This temperature level is important because this is where the asphalt cement plays a greater role in resisting rutting than does gradation variation. The untreated asphalt samples show a dramatic increase in the potential for rutting, as would be expected. In particular, the AC-5 shows an almost complete loss of stability at this temperature. The polymer-treated asphalts also show an increase in rutting potential, but the increase is not nearly as dramatic as it is for the untreated samples. This is particularly true for Asphalt 4 that did not change its potential for rutting at all, which demonstrates a very stable temperature influence. Treatments 1 and 3 showed the largest increase in rutting potential, but they still performed better than the AC-10 and nearly as well as the AC-20 at similar asphalt contents. Treatment 5 also showed no change, and Treatment 2 actually showed a decrease in rutting.

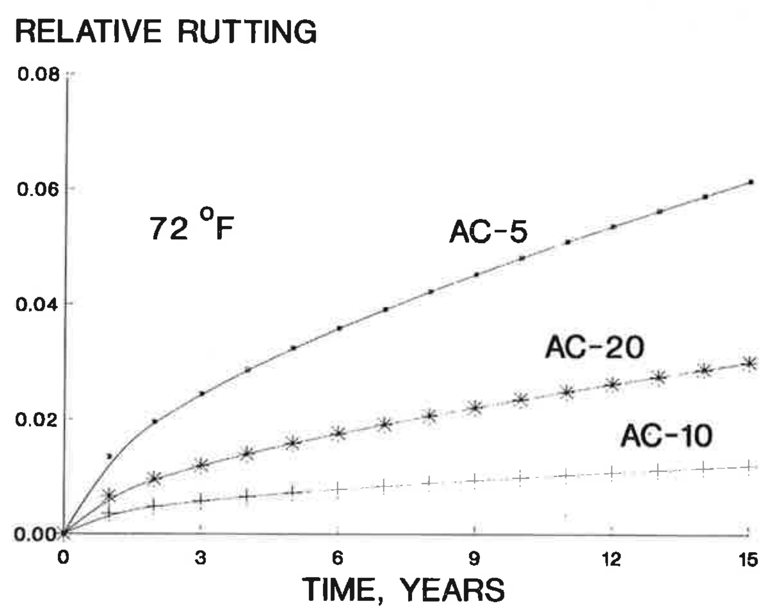

FIGURE 9 Development of rutting in untreated samples at 72°F.

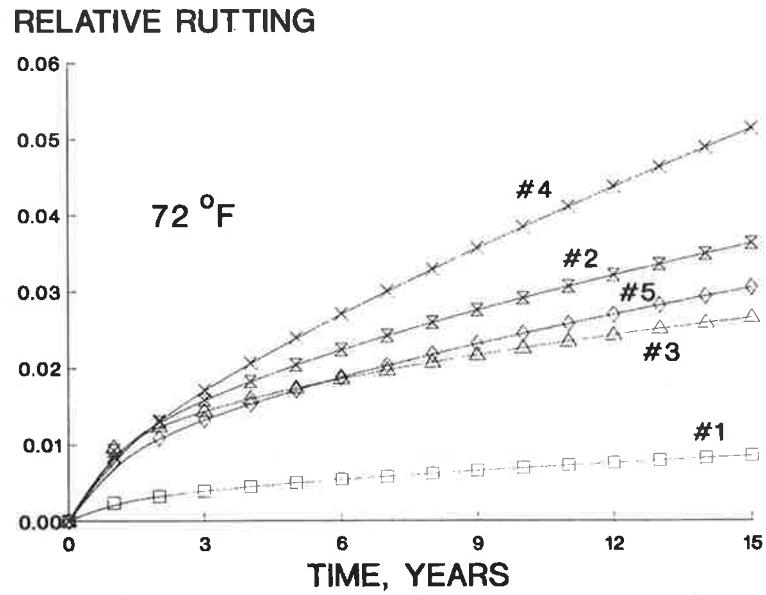

FIGURE 10 Development of rutting in treated samples at 72°F.

Coefficient of Thermal Contraction

The coefficients of thermal contraction are given in Table 7 for each of the temperature ranges examined. These coefficients are typical of any dense-graded mixture and do not appear to be affected by the asphalt grade used or the type of polymer treatment. The polymer-treated mixtures did show a difference from the untreated asphalt cements in that the coefficients did not show the same linear relationship with temperature. The polymer-treated mixes exhibited a nonlinear relationship in the 40°F temperature range. Although this does not cause any significant difference in performance, it may be indicative of the polymer's influence.

Moisture Sensitivity

Cores cut from the cylindrical samples (kneading compaction) were prepared and run through the Lottman vacuum saturation freeze-thaw procedure to induce stripping *(8)*. This procedure relies on the indirect tensile strength before and after conditioning to indicate the potential for stripping to develop in the mixture in service. A 50 percent loss in tensile strength in laboratory-prepared samples indicates severe moisture sensitivity of the mixture; such sensitivity has been correlated to stripping in field samples. The strength values of the mixtures evaluated in this program are given in Table 8. The combination of virgin asphalt and limestone aggregate used in this study

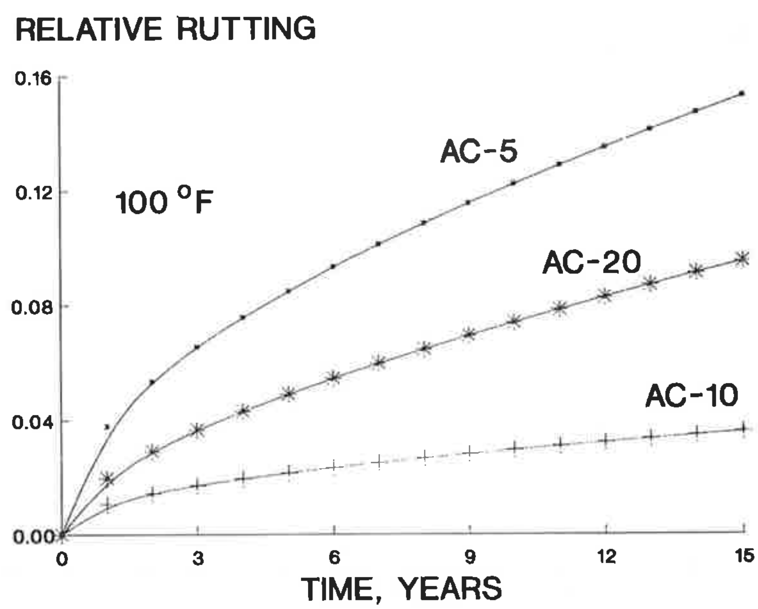

FIGURE 11 Development of rutting in untreated samples at 100°F.

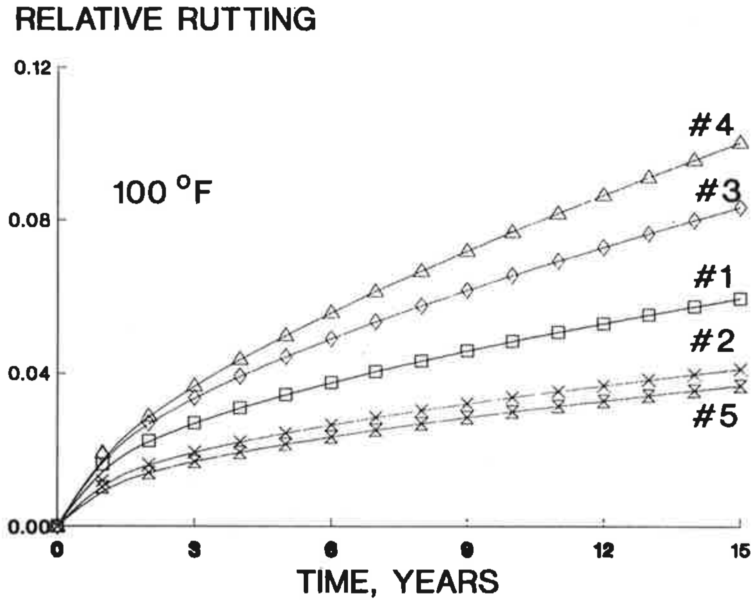

FIGURE 12 Development of rutting in treated samples at 100°F.

was apparently not moisture sensitive. Therefore any potential of the modified blends for improving resistance to stripping cannot be investigated with these mixtures. It is just as important to note that the modified asphalt cements did not increase the potential for moisture sensitivity in these mixes. Further research on moisture susceptible mixtures should be performed to evaluate any improvements in resistance to stripping that may be achieved through the use of polymer-modified binders.

CONCLUSIONS

It must be emphasized that, even for the same mixture, merely selecting one grade of asphalt cement over another does not guarantee better performance. Different grades of asphalt cement are selected to provide resistance to specific types of distress that are related to the environment of a specific area. Low-stiffness AC-5 cements are selected in northern climates to enhance resistance to thermal cracking; stiffer AC-20 cements are selected in warmer climates where rutting is the major problem. The function of the polymer treatment is to improve performance-related material properties of an asphalt cement. Thus relative comparisons of the test data are made against a soft and a stiff asphalt cement to demonstrate how the properties of an asphalt cement can be modified to change its performance at extremes of temperature.

The study presented here was designed to illustrate differences in performance of several different polymer blends, not to recommend any one of the blends as better than another. The data presented here represent the scope of information a state materials engineer should expect to see on any polymer additives blended with his specific asphalt cements. Less information would not allow for an adequate investigation of material differences. Further testing for aging and fatigue characteristics may also be advisable and recommended.

The laboratory test data developed in this study clearly illustrate that polymer modification of an asphalt cement produces a mixture that is quite different from the normal asphalt cement. Beginning with a base AC-5 asphalt, performance can be enhanced to a level expected of an AC-10, and in some

TABLE 7 THERMAL COEFFICIENTS OF CONTRACTION ($\times 10^{-5}$/°F) IN TEMPERATURE RANGES

	Temperature Range (°F)			
Asphalt	72 to 40	40 to 20	20 to 0	0 to −20
1	1.06	1.10	1.22	1.10
2	0.99	1.24	1.22	1.14
3	1.04	1.24	1.40	1.16
4	0.92	1.08	1.33	1.25
5	0.96	1.29	1.49	1.23
6	–	1.51	1.33	1.10
7	1.37	1.32	1.41	1.21
8	1.42	1.66	1.33	1.22

TABLE 8 CHANGE IN INDIRECT TENSILE STRENGTH AFTER LOTTMAN PROCEDURE

	Tensile Strength (psi)		
Asphalt	Dry	After Conditioning	Tensile Strength Ratio
1	105	91	0.87
2	139	116	0.83
3	110	107	0.97
4	135	105	0.77
5	124	110	0.89
6	95	82	0.86
7	164	126	0.77
8	228	175	0.77

instances to that found in an AC-20 depending on which distress mechanism is being evaluated. At temperatures around 72°F, the performance of the polymer-modified asphalts is better than that of the untreated asphalt cements with similar consistencies (penetration at 77°F). The performance of these asphalts is most similar to that of an AC-10. At low temperatures, the performance of the modified asphalts approaches and surpasses that of an AC-5, and, at higher temperatures, the performance of the modified asphalt can meet or exceed that of an AC-20 in the permanent deformation test.

The different polymer blends and amounts used in the mix design all provide a different level of performance modification. Some blends were more beneficial than others, and some blends provided improvement in one area and not in another. In certain instances, the performance of the original asphalt was better.

The following conclusion can be drawn from the laboratory study of the polymer blends investigated here.

1. The dynamic resilient modulus–temperature relationship of the base asphalt can be enhanced to provide a stiffer asphalt at elevated temperatures yet maintain a stiffness below that of the base asphalt at low temperatures. This alteration becomes most significant when evaluated in the static mode of testing.
2. The fatigue resistance of a mixture is improved slightly with a polymer-modified asphalt cement, particularly at high temperatures where the elevated stiffness values produce lower radial tensile strains. Further laboratory testing is required to validate this trend thoroughly.
3. The potential for low-temperature thermal cracking is significantly reduced by use of a polymer-modified asphalt cement. Significantly higher tensile strengths are provided while a lower stiffness, which provides increased resistance to thermal cracking at temperatures in the range of from –20°F to 0°F, is maintained. The tensile strengths at extremely low temperatures are above those of an untreated asphalt concrete sample and they maintain this relationship to significantly lower temperatures than were possible in these tests.
4. A significant improvement in low-temperature performance properties is seen in the dramatic increase in the tensile strain at failure at low temperatures. This is the most important factor in polymer modification for low-temperature performance of asphalt concrete mixtures. With increased strains at failure, the polymer-modified mixes are not as brittle as unmodified mixes at low temperatures and provide improved resistance to thermal cracking.
5. Polymer modification provides a significant improvement in rutting resistance in comparison with the mixture prepared with the base asphalt. The performance of the modified blends is similar to, but not significantly better than, that of an AC-10 or an AC-20 at 72°F. The improvement provided by the polymer modification is seen when rutting at 100°F is compared. At these elevated temperatures, the improvement provided by the polymer treatment over the untreated AC-5 is quite dramatic. Several of the polymer blends actually showed no increase in rutting at the elevated temperature. One even had a lower potential for rutting at the elevated temperature; this may be due to mix design variability rather than polymer effect. At the elevated temperature, the performance of the polymer-modified blends was substantially equal to that of the untreated AC-10 and AC-20 mixtures.

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Evaluation of Asphalt Additives: Lava Butte Road–Fremont Highway Junction

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In August 1985 an experimental road section that incorporated different asphalt additives was constructed near Bend, Oregon. Ten sections were constructed using mix designs furnished by the additive supplier or by the Oregon Department of Transportation (ODOT). In this paper are described the experimental study, including the mix design process, and the construction process, including quality control data and unit prices. Preliminary data on mix properties and field performance are also presented. Significant findings include (a) mix design techniques used by the additive suppliers are, in some cases, not well defined or documented; (b) mix design results obtained by ODOT after construction differ slightly from those recommended by the additive suppliers; (c) there were no major problems during construction of the different mixes; (d) there are significant differences in the preliminary mix properties of additive types; and (e) the performance of all of the test sections after 1 year is good.

A considerable number of Oregon highways are in need of a stable and durable overlay to regain an acceptable serviceability rating. Principal reasons for this include surfacing deficiencies such as fatigue cracking, raveling, deformation, and thermal distress. The primary overlay treatments to date have been thick (2- to 6-in.) dense-graded hot mixes on high-volume highways and open-graded cold mixes on lower volume highways. In recent years, thick hot asphalt concrete overlays have been effective in delaying reflective cracking but have experienced premature longitudinal cracking and stripping. Emulsion cold mixes, surface seals using cationic or high-float emulsions, and hot mixes with lime-treated aggregates have not appeared to exhibit these early performance problems *(1)*.

Today there are numerous additives being sold that are reported to improve the performance of asphalt concrete overlays by eliminating or reducing deformation, surface raveling (stripping is a major problem), and reflective or thermal cracking *(2)*. Because these additives usually add significantly to project costs, it is important to determine their effectiveness under field conditions and to evaluate the cost-effectiveness of their use.

In the summer of 1985, the Oregon Department of Transportation (ODOT) initiated a field study to investigate the use of various asphalt additives. The purpose of the study was twofold:

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1. To evaluate the effectiveness of 10 hot-mix overlay test sections, incorporating various additives to extend the life of asphalt concrete pavements, and
2. To determine the cost-effectiveness of each compared with a conventional asphalt concrete mix.

Products evaluated included

1. PlusRide® 12—coarse-ground rubber in a mix with modified aggregate gradation and asphalt containing Pave Bond (antistripping agent),
2. Arm-R-Shield—asphalt concrete containing fine-ground rubber in asphalt in a mix with conventional aggregate gradation,
3. Fiber Pave®—polypropylene fiber in a mix with asphalt containing Pave Bond and a conventional aggregate gradation,
4. BoniFibers®—polyester fiber in a mix with asphalt containing Pave Bond and a conventional aggregate gradation,
5. Pave Bond®—asphalt containing an antistripping agent in a mix with a conventional aggregate gradation,
6. Pave Bond® and lime—lime-treated aggregate and asphalt containing an antistripping agent in a mix with a conventional aggregate gradation,
7. Lime—lime-treated aggregate in a mix with a conventional aggregate gradation,
8. No additive—a conventional asphalt concrete mix,
9. CA(P)-1—polymer contained in asphalt in a mix with a conventional aggregate gradation, and
10. CA(P)-1 with lime—polymer contained in asphalt with lime-treated aggregate in a conventional mix.

In this paper are presented mix design, construction process, initial mix property, and performance data for the test sections. The project will continue to be monitored during the next 3 years to identify performance differences among the various materials.

DESCRIPTION OF PROJECT

The experimental project is located on US-97 (Oregon Highway 4) approximately 20 mi south of Bend (Figure 1). The weather in the area is considered very severe. Temperatures range from −10°F in the winter to 100°F in the summer, with daily temperature ranges of about 40°F. There are snow and ice from November through February *(3)*.

The test sections were part of an overlay project scheduled for a 20-mi section of roadway that was structurally inadequate and suffering considerable distress. An asphalt concrete overlay was selected to correct the deficiencies. Instead of using

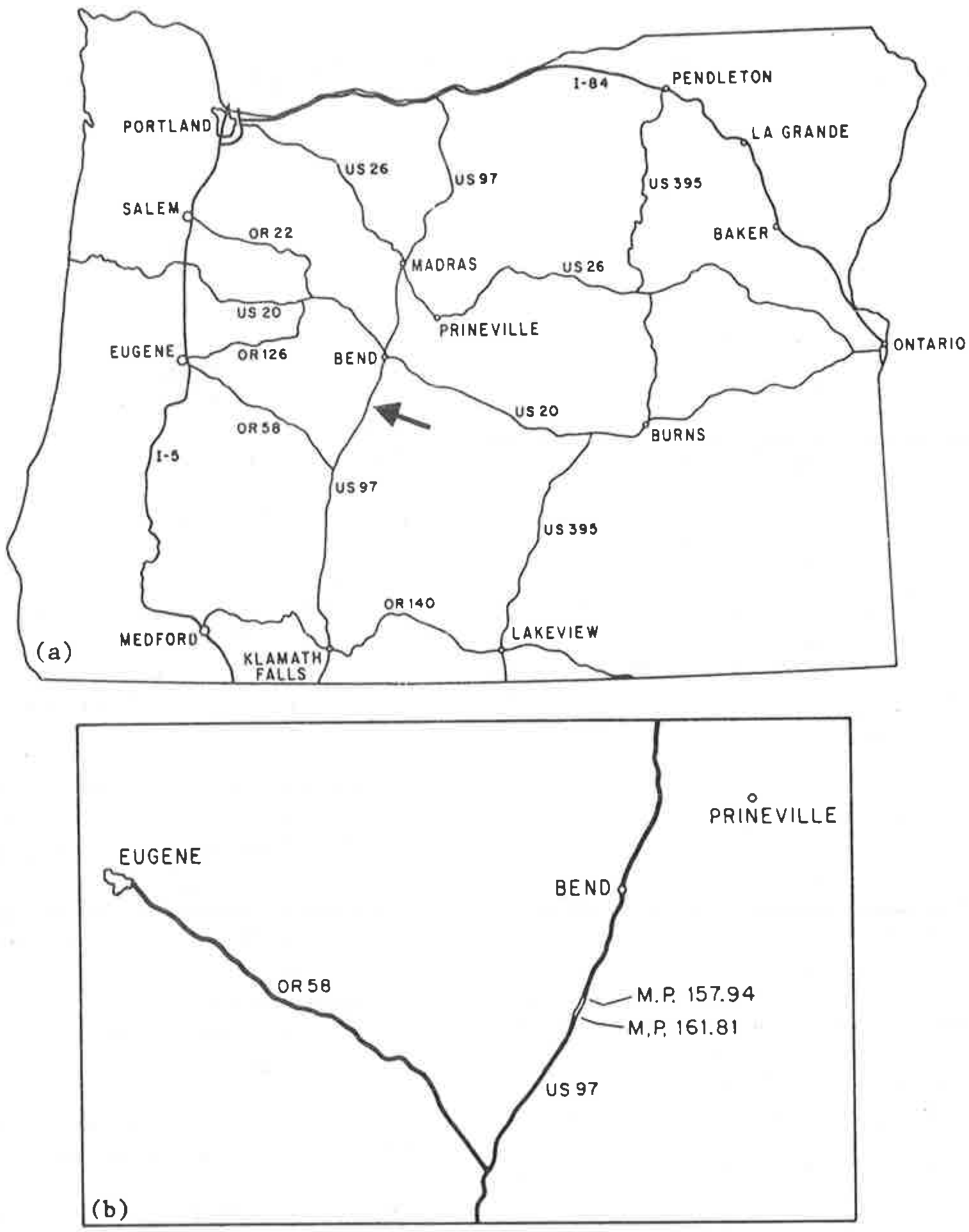

FIGURE 1 Location of asphalt additives test road: (a) general and (b) close up.

conventional asphalt concrete throughout the project, 10 sections with experimental features were placed for evaluation. Each test section was a minimum of 0.5 mi in length and included a 12-ft-wide travel lane. A 0.5-mi section of dense-graded hot mix with no additive served as the control. A layout of the test sites as they were actually constructed is shown in Figure 2. The 0.5-mi sections were selected for the following reasons:

1. The handling and placement characteristics of each material are different and adjustments would be necessary during construction and
2. It is advantageous to measure performance over long sections to minimize statistical errors.

Condition of Pavement

Before construction of the overlays, an extensive survey was made to evaluate the type and extent of distress along the existing pavement. Within each designated test section, a 250-ft site that represented conditions of the entire section was selected. For each 250-ft inspection site, a record of distress types, including a map of all cracks, was made *(4)*. Figure 2 also shows the general location of the inspection sites. In general, there was considerable alligator and thermal cracking as well as patching (Figure 3). The overall condition rating for the project was poor.

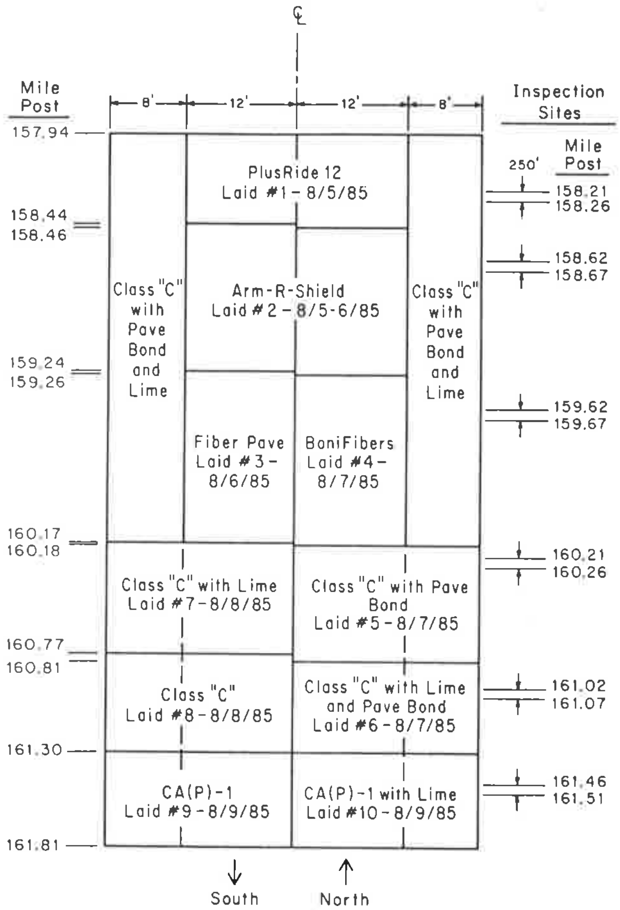

FIGURE 2 Layout of test sections.

Pavement Deflections

Pavement surface deflections to evaluate the structural adequacy of the existing roadway were taken before the overlay was placed. The ODOT Dynaflect was used to measure surface deflections. Deflection measurements were taken every 50 ft within each test section for a distance of about 500 ft. This 500-ft section was selected to overlap the 250-ft inspection site for evaluating pavement condition. In general, there was considerable variation among the sections in terms of structural adequacy. The deflection data were used to determine the overlay requirements, following a modification of the California overlay design procedure *(5–7)*. The recommended section is as folows:

1. Top lift (13/4 in.)—experimental feature,
2. Bottom lift ($1^1/_2$ in.)—Oregon Class-C mix, and
3. Leveling course (as needed)—Oregon Class-C mix.

MIX DESIGNS

Job Mix Designs

For each of the experimental sections, the additive supplier or the ODOT Materials Section recommended the job mix asphalt content and gradation. All mixes, except PlusRide, were designed using the C-mix aggregate gradation given in Tables 1 and 2. In some cases, both coarse and fine aggregate were treated by pug mill mixing dry lime and water. The 5-day minimum period for mellowing aggregates in a stockpile was extended to from 60 to 90 days to fit the contractor's operations.

TABLE 1 GRADATION OF MIX (% passing)

Gradation	C-Mix	PlusRide
$^3/_4$ in.	100	–
$^5/_8$ in.	–	100
$^1/_2$ in.	99	89
$^3/_8$ in.	89	76
$^1/_4$ in.	66	38
No. 10	32	31
No. 30	–	19
No. 40	14	17
No. 200	5.8	8.9

The asphalt cement used was an AC-20 from Chevron's Willbridge Refinery in Portland, Oregon. Properties of the AC-20 and its specification requirements are given in Table 3. This material was used in all experimental features except where the polymer-modified asphalt was used. Table 4 gives a summary of the properties of Chevron's CA(P)-1.

FIGURE 3 Typical pavement condition before overlay: (*top*) milepost 158.21 looking south and (*bottom*) milepost 158.26 looking north.

TABLE 2 AGGREGATE PROPERTIES AND SPECIFICATIONS

	Actual		Specification	
Property	Coarse	Fine	Coarse	Fine
Specific gravity (AASHTO T-85				
Bulk	2.57	2.63	–	–
Saturated surface dry	2.64	2.69	–	–
Los Angeles abrasion (AASHTO T-96) (%)	28.4	–	30 max	–
Sand equivalent (AASHTO T-176)	–	–	–	–
Percentage crushed faces (OSHD T-213)	90	–	60 min	60 min
Sulfate soundness (OSHD T-206) (%)	0.6	2.7	12 max	12 max
Degradation (OSHD T-208)				
Passing No. 20 sieve (%)	21.9	10.8	30 max	30 max
Sediment height (in.)	0.3	0.3	3.0 max	4.0 max
Friable particles (AASHTO T-112) (%)	0.2	0.4	1.0 max	1.5 max

NOTE: OSHD = Oregon State Highway Division.

TABLE 3 PROPERTIES OF AC-20 ASPHALT CEMENT

Property	Actual	Specification
Viscosity at 140°F (poise)	2040	2000 ± 400
Viscosity at 275°F (cSt)	352	230 min
Penetration at 77°F (dmm)	58	50 min
Flash point, COC[a] (°F)		
(AASHTO T-73)	600	450 min
Solubility in trichloroethylene (%)	99.86	99 min
Tests on residue		
Viscosity at 140°F (poise)	6122	8000 max
Ductility at 77°F (cm)	–	75 min

[a]COC = Cleveland open cup.

The mix procedures and criteria for each experimental feature are given in Table 5. As noted, for some of the additives the mix design procedures and criteria are not well defined. The resulting asphalt contents are given in Table 6.

ODOT Mix Designs

After the project was constructed, ODOT performed detailed mix designs using their current mix design procedures *(8)*. This was done to verify the potential use of the current ODOT design method with modified asphalts. Mix design criteria used to evaluate the various experimental features (except PlusRide) are summarized as follows:

TABLE 4 PRELIMINARY PRODUCT SPECIFICATION, CHEVRON POLYMER ASPHALT CA(P)-1

Property	ASTM Test Method	CA(P)-1 Specification	CA(P)-1 Properties
Original test Properties			
Penetration at 77°F (dmm)	D 5	85 min	113
Viscosity at 140°F (poise)	D 2171	1600–2400	2092
Viscosity at 275°F (cSt)	D 2170	325 min	676
Flash point, COC (°F)	D 92	450 min	500
Ductility at 77°F (cm)	D 113	100 min	150+
Ductility at 39.2°F (cm) (5 cm/min pull rate)	D 113	25 min	32
Toughness (in.-lb)	–[a]	75 min	124
Tenacity (in.-lb)	–[a]	50 min	101
Properties After Rolling Thin-Film Oven Test			
Viscosity at 140°F (poise)	D 2872	10 000 max	4980
Ductility at 77°F (cm)	D 113	100 min	150+
Ductility at 39.2°F (cm) (5 cm/min pull rate)	D 113	8 min	13
Toughness (in.-lb)	–[a]	100 min	325
Tenacity (in.-lb)	–[a]	75 min	346

[a]Benson method of toughness and tenacity: 20 in./min pull rate, 7/8-in.-diameter tension head.

TABLE 5 MIX DESIGN PROCEDURES AND CRITERIA USED, ADDITIVE SUPPLIERS AND ODOT

Feature	Method	Compactive Effort	Additive	Design Criteria	Comments
PlusRide	Marshall	50 blows/side	3% rubber granules by weight of total mix	3% air voids	Mix is rich in asphalt and filler, has high coarse aggregate content, and is gap graded
Arm-R-Shield	Marshall	75 blows/side	20% rubber by weight of asphalt binder	Stability (lb): 1,500 min Flow: 8–18 in. × 10^{-3} Voids: 3–5%	Asphalt-rubber is reacted at elevated temperature before use Mix is rich in asphalt
Fiber Pave (polypropylene)	None given	–	0.3% fiber by weight of total mix	Asphalt content increased 0.3% over the standard mix	
BoniFibers (polyester)	None given	–	0.25% fiber by weight of total mix	Asphalt content increased 0.3% over the standard mix	
Chevron CA(P)-1	Hveem	150 blows/500 psi	5.0% of asphalt binder	Stability: 30 min Appearance: shiny	Polymer with and without lime-treated aggregate
All other mixes	Hveem (ODOT)	150 blows/500 psi	0.5% Pave Bond by weight of asphalt binder or 1.0% lime slurry by weight of aggregate, or both	Stability: 30 min Voids: 4–5% IRS: 75% min Modulus ratio: 70% min	For details of mix design procedure see Sullivan et al. *(8)*

- Asphalt film thickness — Sufficient to thick
- Air voids (%) — 3.0 to 5.0
- Stability, first compaction — 30 min
- Stability, second compaction — 30 min
- Index of retained strength, IRS (%) — 75 min
- Modulus ratio (%) — 70 min

For PlusRide, the asphalt content was selected at a void content of 3 percent. A summary of these mix designs, with appropriate comments, is given in Table 7. As indicated, there are only

TABLE 6 MIX DESIGN RESULTS (from additive suppliers)

Additive	Recommended Asphalt Content (%) With Lime	Without Lime
PlusRide	–	8.0
Arm-R-Shield	–	8.0
Fiber Pave	–	6.7
BoniFibers	–	6.7
CA(P)-1	6.5	6.5

NOTE: Percentage by weight of total mix.

TABLE 7 SUMMARY OF MIX DESIGNS PERFORMED BY ODOT FOLLOWING CONSTRUCTION

Material	Additive (%)	Basis for A/C Recommendation	Properties of Mix at Design Asphalt Content: Recommended Asphalt Content (% of total mix)	Hveem Stability	IRS (%)	Voids (%)	Diametral Modulus (psi)	Modulus Ratio (freeze-thaw)
PlusRide	3.00	3% voids	7.5	4	53	2.9	183,900	0.65
Arm-R-Shield	20.00[a]	Std[b] ODOT criteria	8.2	31	47	4.8	86,500	0.53
Fiber Pave[c]	0.30	Std ODOT criteria	7.0	36	100+	5.9	182,600	0.58
BoniFibers[c]	0.25	Std ODOT criteria	7.0	38	90	5.7	273,000	0.74
Treated with Pave Bond[c]	0.50[a]	Std ODOT criteria	5.9	39	99+	4.9	327,000	0.75
Treated with Pave Bond	0.50[a]	Std ODOT criteria	6.5	32	100+	4.7	350,000	1.03
Control	–	Std ODOT criteria	6.5	37	79	5.0	280,000	0.44
Control[c]	–	Std ODOT criteria	6.0	39	93	4.9	337,000	0.92
Chevron CA(P)-1	5.00[a]	Std ODOT criteria	6.5	39	73	4.9	160,000	0.68
Chevron[c] CA(P)-1	5.00[a]	Std ODOT criteria	6.9	39	91	4.9	110,000	0.79

[a]Percentage liquid binder.
[b]Std = standard.
[c]Aggregate is precoated with 1% lime slurry.

slight differences in asphalt contents recommended by the additive supplier and ODOT.

CONSTRUCTION PROCESS

Construction Procedures

Standard construction equipment was used for all sections. A conventional batch plant (Cedar Rapids Model 6000) was used to prepare the mixtures. Bottom dump trucks were used to transport the material to the job site. The mix was laid with a Cedar Rapids paver (Model 520). Compaction was accomplished using a 10-ton vibratory roller for breakdown, a 5-ton pneumatic roller as intermediate, and a 10-ton steel roller for finish compaction. The target density was 92 percent of maximum gravity (AASHTO T-209) for all sections. For the PlusRide and fiber sections, the additive was added as a dry mix cycle before the mixing operation. For the Arm-R-Shield, Pave Bond, and Chevron polymer-modified mixes, the additive was added to the asphalt before mixing. The standard tack coat was 0.03 gal/yd^2 of Chevron CSS-1. Specific construction details for each product are summarized next.

PlusRide®

In this product, the granulated rubber produced from shredded tires replaces a portion of the 1/4-in. to No. 10–sized aggregate. The PlusRide material was added to the batch plant via the top hopper. One hundred eighty pounds of rubber were added to every 3-ton batch of mix produced (3.0 percent). Asphalt content was set at 8 percent of the total weight of mix. Mix temperature was 340°F to 355°F, and laydown temperature was approximately 320°F in the windrow ahead of the paver. Compaction temperature was about 270°F to 280°F.

Construction proceeded rapidly without any major problems. The supplier recommended that rubber-tired rollers not be used and that a soap solution be used with the other rollers to prevent "pickup." Three roller passes, one static (breakdown) followed by a vibration pass and a static finish pass at 140°F, achieved the desired percentage of compaction. No pneumatic rolling was allowed because of pickup. Two additional passes increased the density, but the next pass resulted in cracking and lowered the density. Traffic traveled over the mat after it had been finish-rolled and the temperature had dropped to approximately 140°F. Initially, the mixture appeared to flush under traffic, but by the next day evidence of flushing could not be found. The mixture also was quite sticky and tacky on the surface.

Arm-R-Shield

The second section constructed incorporated a rubber-modified asphalt from Arizona Refining Company. Recycled rubber was melted with the AC-20 asphalt at 400°F in a mobile mixing truck supplied by Arizona Refining. After the rubber and asphalt had been blended, the asphalt-rubber mixture was transferred to a distributor truck for storage. Introduction of this additive into the mix presented some unique problems. Because the plant storage tanks already contained unmodified liquid asphalt, the asphalt modified with Arm-R-Shield had to be pumped from the distributor truck into the pug mill. The asphalt content for mix containing modified asphalt was set at 8 percent by weight of total mix. Because the binder contains 20 percent rubber, the actual components were 6.4 percent liquid asphalt and 1.6 percent rubber additive. Each 3-ton batch was mixed for from 3 to 15 sec before the modified asphalt was added and for 35 additional seconds after it was added. The production of the mix was extremely slow: 3-ton batches took from 2 to 7 min to mix. The plant operator thought that the slow production resulted from the material being very viscous and extremely hard to pump. Under normal production, this problem probably would not have occurred because larger plant storage tanks and hoses would have been used.

Mix temperatures of from 340°F to 350°F at the plant discharge, laydown temperatures between 315°F and 350°F in the windrow ahead of the paver, and mat temperatures after laydown of from 285°F to 295°F were recorded during the 2 days of operation. Blue smoke and steam appeared during laydown.

The roller operator attempted to compact directly behind the paver; however, the mix lacked stability and started to be picked up by the roller wheels. Because of these problems and the hot laydown temperature, the breakdown compaction equipment started rolling 600 to 800 ft behind the paver. The normal rolling pattern both days was two vibratory passes and one static pass with another vibratory roller pass for finish. During the first day, the mix moved under the rollers and wrinkled badly even though it was laid down without excessive cracking under the finish roller. On the second day, mix placed did not exhibit the tendency to "crawl." Neither the factory representative nor others at the job site could determine a reason for the difference in mix behavior. The only significant change in conditions was a 40°F to 50°F reduction in the surface temperature. It was difficult to achieve the desired 92 percent compaction as measured by the nuclear gauge. Even after 11 passes with both static and vibratory rollers, the mixture never achieved the desired compaction of 92 percent. However, compaction of 93.1 percent was obtained for a core taken August 8. Readings with the nuclear density gauge taken August 8 at the same location indicated compaction of 91.9 percent.

The pavement material was much "stickier" than PlusRide and remained in this condition until traffic had been on it for some time. This presented no problem. Extraction of asphalt from the mix (using a vacuum extractor) was difficult; washing took approximately 2 hr.

Fiber Pave®

Polypropylene fiber was used in the next test section constructed. The manufacturer of the Fiber Pave 3010 is Hercules, Inc. The fiber material was added to the pug mill. A crane hoisted the crates of material to the top of the batch plant and two workers fed one 18-lb bag of material for each 3-ton batch (0.3 percent). Each batch took approximately 30 sec; the workers were signaled from the control shack when to add the fibers. There were some initial clogging problems, but these were resolved by dumping the material down another hopper chute.

The specification for use of the material stated that the mix temperature could not be above 290°F. The addition of fiber and a 0.3 percent increase in the liquid asphalt content were the only deviations from normal Class-C mix components. The technical representative from Hercules was on site to oversee production. He noted that the mixture should have a stringy texture. If the mix temperature is too hot, the fibers melt and do not produce the desired consistency. However, the stringy texture made the mix difficult to rake.

Mix temperatures recorded were 285°F at the plant discharge, 265°F to 280°F in the windrow ahead of the paver, and 239°F to 248°F behind the paver. The existing surface temperatures ahead of the paver were 120°F to 125°F. The original rolling pattern called for two vibratory rollers, but this pattern was later modified to include a pneumatic roller for breakdown. The pneumatic rollers were added because of difficulty in meeting compaction criteria (92 percent of maximum density).

BoniFibers®

BoniFibers is the trade name of the polyester fiber used on this section. The method used to add Fiber Pave to the mix was also used for BoniFibers: two workers fed 15 lb of material into each 3-ton batch (0.25 percent). Asphalt content was increased 0.3 percent from that of the standard mix to 6.7 percent in this mix. The rest of the material and plant settings were not changed from the standard Class-C requirements. Mix temperatures of 305°F to 310°F at the plant discharge, laydown temperatures of 275°F to 305°F in the windrow ahead of the paver, mat temperatures directly behind the paver of 256°F to 285°F, and surface temperatures ahead of the paver of 60°F to 70°F were recorded. After the first pass with the breakdown roller this material appeared very brown, similar to a conventional mix with insufficient binder content. After 2 days of traffic it turned black. It should be noted that the fibers were not dry mixed before addition of the asphalt. This resulted in poor fiber dispersion in the mix and formation of fiber balls throughout the section. It is uncertain what effect this will have on the performance of this section.

The desired 92 percent compaction was difficult to attain. This could have been the result of one roller having mechanical difficulty, which delayed the paving operation. To provide adequate equipment, one static steel roller and two pneumatic rollers were added. The use of additional rollers resulted in overcompaction and a subsequent reduction in the density values.

Pave Bond® (with and without lime)

Two sections were specifically constructed to evaluate asphalt treated with 0.5 percent Pave Bond: one with lime and one without. Mix temperature at the plant was recorded as 300°F ± 10°F. Laydown temperatures of 295°F to 305°F in the windrow ahead of the paver, mat temperatures of 280°F to 290°F, and a surface temperature of 80°F were recorded. Four rollers were used to obtain compaction: two passes of a three-wheeled roller for breakdown, three passes with a vibratory roller (one static and two vibratory), two to four passes with a rubber-tired roller for intermediate rolling, and one or two static passes with the vibratory roller for finish rolling. Densities were reasonably close to the desired 92 percent compaction. No problems were encountered during construction of either of the two mix sections. Both mixes laid the same as a standard Class-C asphalt concrete mix.

Control (with and without lime)

Control sections consisted of a standard Class-C asphalt concrete mix. Mixing and compaction techniques were similar to those used for sections containing Pave Bond. Two control sections were constructed: one with lime treatment and one without. No special problems were encountered during construction of either section.

Chevron Modified Asphalt [CA(P)-1]

Two sections were constructed using Chevron polymer-modified asphalt: one section with lime-treated aggregate and one without. Mix temperatures at the plant discharge were originally set at 340°F. Laydown temperatures were 315°F to 320°F in the windrow ahead of the paver, and a mat temperature behind the paver of 285°F and an air temperature of 66°F were recorded for the mixture without lime-treated aggregate. Mix temperatures of 340°F at the plant discharge, laydown temperatures of 315°F to 335°F in the windrow ahead of the paver, mat temperatures behind the paver of 290°F to 305°F, and surface temperatures ahead of the paver of 110°F were recorded for the mixture with lime-treated aggregates. The Chevron representative considered the elevated temperature essential to obtain good bond of binder to stone, so all trucks were covered with tarps. The increased temperature was the only deviation from normal Class-C mix settings and components.

Breakdown rolling was accomplished with two passes of a three-wheeled steel roller, intermediate rolling with two or three passes with the pneumatic roller and three passes with the vibratory roller (one static and two vibratory), and finish rolling with one static pass of the vibratory roller. The mix without lime was deformed under the rollers, but the mix with lime was quite stable. This may have been caused by the CSS-1 tack coat. The section without lime-treated aggregate received 0.05 gal/yd^2 whereas the section with lime-treated aggregate and all other test sections received 0.03 gal/yd^2. When construction was completed, both sections looked satisfactory and compaction exceeded the desired 92 percent.

Quality Control Data

Tables 8 and 9 give summaries of the construction test results from daily plant reports. These results indicate that

1. The asphalt content and mix gradation generally fell within the mix design tolerances for all mixes. The exception

TABLE 8 SUMMARY OF CONSTRUCTION TEST RESULTS FROM DAILY REPORTS—SPECIAL ADDITIVES

	PlusRide		Arm-R-Shield		Fiber Pave Polypropylene Fiber		BoniFibers Polyester Fiber		CA(P)-1 Without Lime		CA(P)-1 With Lime	
	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance
Gradation (% passing)												
3/4 in.	–	–	100	100	100	100	100	100	100	100	100	100
5/8 in.	100	94–100	–	–	–	–	–	–	–	–	–	–
1/2 in.	–	–	98	95–100	97	95–100	97	95–100	97	95–100	97	95–100
3/8 in.	78–79	70–82	–	–	–	–	–	–	–	–	–	–
1/4 in.	47–48	32–44	60	60–72	63	60–72	66	60–72	65	60–72	65	60–72
No. 10	36–37	27–35	31	28–36	31	28–36	32	28–36	30	28–36	32	28–36
No. 30	21	15–23	–	–	–	–	–	–	–	–	–	–
No. 40	–	–	12	8–26	14	8–26	14	8–26	14	8–26	14	8–26
No. 200	7.4–7.9	6.9–10.9	5.0	3.8–7.8	5.5	3.8–7.8	5.5	3.8–7.8	6.5	3.8–7.8	6.2	3.8–7.8
Asphalt content (%)	8.5–9.0	7.6–8.4	7.6[a]	7.5–8.5[a]	6.7	6.3–7.3	6.5	6.2–7.2	6.4[a]	5.9–6.9[a]	6.6[a]	5.9–6.9[a]
Additives (%)	3.0 rubber in mix 0.5 Pave Bond 0.0 lime	2.85–3.15	20 rubber in asphalt 0.0 Pave Bond 0.0 lime		0.3 fiber 0.5 Pave Bond 0.0 lime		0.25 fiber 0.50 Pave Bond 0.00 lime		5.0 polymer based on asphalt weight 0.0 Pave Bond 0.0 lime		5.0 polymer based on asphalt weight 0.0 Pave Bond 1.0 lime	
Mix temperature (°F)	325–355	325–360	340–350	340 min	285	290 max	305	325 max	325–340	340	340	340
Laydown temperature (°F)	317–321	300 min	317–350	285–325	265–280	245–290	275–305	280±	315–320	–	315–335	–
Density (%)	92.5–96.7	92 min	85.9–92.9	92 min	86.0–90.9	92 min	86.2–92.7	92 min	93.1–95.4	92 min	92.8–94.5	92 min

[a]Includes asphalt additive.

TABLE 9 SUMMARY OF CONSTRUCTION TEST RESULTS FROM DAILY PLANT REPORTS—CONVENTIONAL ADDITIVES

	Control With Lime		Control Without Lime		Pave Bond Without Lime		Pave Bond With Lime	
	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance	Mix Test Value	Mix Design Tolerance
Gradation (% passing)								
3/4 in.	100	100	100	100	100	100	100	100
5/8 in.	–	–	–	–	–	–	–	–
1/2 in.	98	95–100	98	95–100	97	95–100	98	95–100
3/8 in.	–	–	–	–	–	–	–	–
1/4 in.	69	60–72	64	60–72	66	60–72	64	60–72
No. 10	32	28–36	32	28–36	33	28–36	31	28–36
No. 30	–	–	–	–	–	–	–	–
No. 40	14	8–26	13	8–26	14	8–26	14	8–26
No. 200	5.8	3.8–7.8	5.5	3.8–7.8	5.9	3.8–7.8	5.4	3.8–7.8
Asphalt content (%)	6.4	5.9–6.9	6.3	5.9–6.9	6.2[a]	5.9–6.9[a]	6.2[a]	5.9–6.9[a]
Additives (%)	0.0 Pave Bond 1.0 lime based on aggregate weight		0.0 Pave Bond 0.0 lime		0.5 Pave Bond 0.0 lime		0.5 Pave Bond 1.0 lime	
Mix temperature (°F)	305	325 max	305	325 max	305	325 max	305	325 max
Laydown temperature (°F)	290	280±	290	280±	285	280±	285	280±
Density (%)	93.1–94.7	92 min	89.4–94.0	92 min	89.1–91.7	92 min	91.4–92.4	92 min

[a]Includes asphalt additive.

TABLE 10 UNIT PRICES

Material	Asphalt Concrete Mixture[a] ($/ton)	Liquid Asphalt[b] (% binder/ton and $/ton mix)	Lime Credit ($/ton mix)	Total Cost ($/ton)
PlusRide	30.00	8.0 15.72		45.72
Arm-R-Shield	20.00	8.0 60.00		80.00
Fiber Pave	30.00	6.8 13.36	−1.45	41.91
BoniFibers	30.00	6.7 13.17	−1.45	41.72
C-mix with Pave Bond without lime	11.00	6.4 12.58	−1.45	22.13
C-mix with Pave Bond with lime	11.00	6.4 12.58		23.58
Control without Pave Bond with lime	11.00	6.4 12.10		23.10
Control without Pave Bond without lime	11.00	6.4 12.10	−1.45	21.65
C-mix with CA(P)-1	11.00	6.4 17.86	−1.45	27.41
C-mix with CA(P)-1 with lime	11.00	6.4 17.86		28.86

[a]Excludes liquid asphalt and additives in liquid asphalt.
[b]AC-20 = $189.00/ton, AC-20 with 0.5% Pave Bond = $196.50/ton, AC-20 with CA(P)-1 = $279.00/ton, and Arm-R-Shield = $750.00/ton.

was the asphalt content for PlusRide. All tests for the PlusRide mix showed an asphalt content higher than the design tolerance.

2. The mix and laydown temperatures generally conformed to specifications.

3. Many of the results of nuclear density tests failed to meet the specified 92 percent minimum value based on AASHTO T-209.

In general, the quality control tests indicated no major problems in the mix with the exception of low densities (high voids).

Unit Cost Evaluation

Unit costs given in Table 10 are predominantly contractor bid prices with a few negotiated costs included. Even though small quantities are involved and the contractor had no experience with most of the materials, these bid prices are considered a reasonable approximation of actual installation costs.

A separate bid item was included for each class of asphaltic concrete mix to cover the contractor's cost of aggregate, mixing, handling, and placing. The cost of fibers and crumb rubber added directly to the mixture was also included in the mixture bid item. Liquid asphalt, including additives added to the liquid asphalt before mixing, was bid separately. Total cost of the mix in place was dependent on the quantity of liquid asphalt incorporated in the mix. The designed percentage of asphalt for each type of mix is also included in Table 10 and used to calculate the cost per ton of mix.

Lime treatment of aggregate was specified for all mixtures except PlusRide and Arm-R-Shield. Therefore, when lime treatment was not used in a mixture, a $1.45/ton credit was subtracted from the mix unit price.

CA(P)-1 polymer-modified asphalt was furnished by Chevron USA at the price of AC-20 liquid asphalt. Chevron reported that the polymer-modified asphalt is being sold at from $80 to $100 per ton premium. For the sake of a fair evaluation, an average of $90 was assumed for this evaluation.

MIX PROPERTIES

Mix Properties—September 1985

Two 4-in. and three 6-in. cores were taken from each experimental section shortly after construction. The 4-in. cores were tested for density, voids, modulus, and stability. The 6-in. cores were tested for gradation, asphalt content, and asphalt properties. In addition, mix sampled during construction (box samples) was compacted and tested for Hveem stability, modulus fatigue, and index of retained strength.

The results of the tests on the 4-in. cores are given in Table 11. The following significant items are noted:

1. Modulus values (at 77°F) range from 93,000 psi for Arm-R-Shield to 590,000 psi for the C-mix with Pave Bond and lime-treated aggregate;

2. In-place voids range from 3.7 percent for PlusRide to 8.1 percent for BoniFibers; this is in conflict with the results of the construction quality control tests reported in Tables 8 and 9; and

3. Hveem stability values (in place) are in the normal range, except for PlusRide.

Table 12 gives a summary of the results of gradation and asphalt property tests on box samples of mix (obtained during construction) and on 6-in. cores obtained shortly after construction. The results presented in this table indicate that

TABLE 11 SUMMARY OF TEST RESULTS, 4-IN. CORES (September 1985)

Property	PlusRide	Arm-R-Shield	Fiber Pave	BoniFibers	Pave Bond Without Lime	Pave Bond With Lime	Control With Lime	Control Without Lime	CA(P)-1 Without Lime	CA(P)-1 With Lime
Unconditioned modulus at 77°F (1,000 psi)	264	93	111	137	275	590	209	256	352	366
Gravity										
In place (voids)	2.21 (3.7)	2.27 (6.9)	2.28 (6.5)	2.26 (8.1)	2.34 (5.3)	2.31 (6.6)	2.31 (6.9)	2.30 (7.1)	2.36 (4.9)	2.31 (6.9)
Recompacted (voids)	2.23 (2.8)	2.38 (2.4)	2.42 (0.8)	2.41 (2.0)	2.44 (1.2)	2.45 (0.9)	2.43 (2.1)	2.41 (2.6)	2.46 (0.9)	2.45 (1.2)
Relative compaction (%)	96.3	93.1	93.5	91.9	94.7	93.4	93.1	92.9	95.1	93.2
Maximum theoretical gravity	2.295	2.438	2.439	2.458	2.470	2.473	2.481	2.475	2.481	2.480
Hveem stability at 140°F										
In place	2	29	12	14	29	16	24	18	25	22
Recompacted	1	18	21	29	19	32	3	34	22	24

TABLE 12 SUMMARY OF MIX AND ASPHALT PROPERTY TEST RESULTS, 6-IN. CORES AND BOX SAMPLES (September 1985)

	PlusRide		Arm-R-Shield	Fiber Pave	BoniFibers	Pave Bond Without Lime	Pave Bond With Lime	Control With Lime	Control Without Lime	CA(P)-1 Without Lime	CA(P)-1 With Lime
	1	2									
Gradation[a] (% passing)											
3/4 in.	100	100	100	100	100	100	100	100	100	100	100
1/2 in.	97	95	98	97	98	97	99	98	98	100	99
3/8 in.	77	72	86	84	85	88	88	83	84	88	85
1/4 in.	47	43	62	64	66	68	67	66	66	66	63
No. 4	42	38	51	54	54	56	56	55	54	54	52
No. 10	35	33	30	32	31	32	32	32	32	32	32
No. 40	18	16	13	14	14	14	14	14	13	14	14
No. 20	7.0	6.2	5.8	6.1	5.8	6.2	6.4	6.7	6.0	5.9	5.5
Asphalt content (%)	8.5	7.8	6.8	7.0	6.9	6.2	6.3	6.3	6.3	7.2	6.4
Asphalt properties[a]											
Viscosity at 140°F (poises)	3319	4479	3302	8064	9130	8120	7637	5650	6560	8013	10 100
Kinematic viscosity at 275°F (cSt)	445	514	849	597	591	624	572	534	568	1040	1137
Penetration (dmm)	40	42	75	27	22	23	22	25	21	40	36
Asphalt properties[b]											
Viscosity at 140°F (poises)	2070	–	2733	5739	7231	7669	2574	6542	8435	9962	12 478
Kinematic viscosity at 275°F (cSt)	374	–	929	539	599	599	635	560	609	1093	1369
Penetration (dmm)	55	–	78	32	29	28	25	26	28	37	34
Mix properties											
Stability	8	5	37	44	39	44	40	39	41	41	37
IRS (%)	64	82	93	94	92	95	100	94	97	84	91
Modulus ratio (freeze-thaw)	0.73	0.63	0.70	0.76	0.84	0.92	0.85	0.90	0.78	0.73	1.00

[a]Box samples.
[b]6-in. cores.

1. The asphalt content and mix gradation were more or less in compliance with the job mix formula.
2. The viscosities of the recovered asphalt from the box samples generally were higher than those measured on the core samples. This is because these loose materials were tested up to 1 or 2 months after sampling. The highest viscosity at 140°F was measured on the polymer-modified asphalt in both cases.
3. The viscosities at 140°F of the rubber-modified asphalts were lower than those of the other mixes.
4. The penetration values at 77°F for the rubber- and polymer-modified asphalts were higher than for the other materials.
5. The Hveem stability values of laboratory-compacted box samples were all greater than 30 except for the PlusRide mix.
6. The index of retained strength (IRS) of all mixes was greater than 75 percent minimum except for PlusRide.
7. The modulus ratios (after freeze-thaw conditioning) were all greater than the 0.70 minimum except for PlusRide.

Mix Properties—March and June 1986

Diametral modulus and fatigue tests were performed on cores taken from all sections in March 1986 *(9)*. The tests were run in accordance with ASTM D 4123. In March both modulus and fatigue tests were run at 200 microstrain and 73°F. In June modulus and fatigue tests were conducted at both 200 and 100 microstrain and at 73°F. Table 13 gives a summary of the results of the tests on field cores.

Discussion of Test Results

The results of the testing are discussed in the following subsections.

Hveem Stability

All materials except PlusRide have in-place stability values within the expected range for cores. PlusRide has a stability value of 2. This is also true for laboratory-compacted samples. Despite the low value for PlusRide, there is no evidence of rutting. This would indicate that the use of stability criteria for evaluating the PlusRide mix may be inappropriate.

Modulus Values

At present, modulus values are not considered directly in the mix design or selection of additives. The tests on cores taken in 1985 and 1986 indicate that most of the materials are increasing in stiffness (Figure 4). The exceptions are the polymer-modified asphalt mixes and one of the conventional mixes.

Modulus Ratio

A minimum modulus ratio of 0.7 is required in mix designs to provide adequate resistance to pavement damage from freeze-thaw effects. The mix design modulus ratios (Table 7) indicate satisfactory freeze-thaw resistance for all mixes with lime-treated aggregate, Pave Bond, and BoniFibers.

Fatigue Results

The fatigue results on the cores clearly indicate that the polymer-modified mixes and the PlusRide mix are more resistant to cracking. The significance of the testing is that all of the other mixes have fatigue properties comparable to those of conventional mixtures. The design of a durable flexible pavement

TABLE 13 SUMMARY OF MODULUS AND FATIGUE TEST DATA (field cores)

Mix Type	Avg Density (pcf)	Avg Modulus[a] (1,000 psi)	Load Applications to Failure
March 1986			
PlusRide	137.6	272	15,942
Arm-R-Shield	141.7	194	4,171
Fiber Pave	144.2	400	6,708
BoniFibers	142.2	387	4,487
Pave Bond without lime	144.0	475	5,347
Pave Bond with lime	145.0	506	6,052
Control	144.9	457	7,094
Control with lime	147.4	511	4,986
CA(P)-1 without lime	148.4	284	21,187
CA(P)-1 with lime	144.4	298	37,375
June 1986			
PlusRide	136.2	341	88,500[b]
Arm-R-Shield	139.4	196	19,876[b]
CA(P)-1 without lime	147.7	304	19,208 122,043[b]
CA(P)-1 with lime	143.8	287	200,598[b] 31,432

[a]Tests run at 73°F and 200 microstrain, except as noted.
[b]Tests run at 73°F and 100 microstrain.

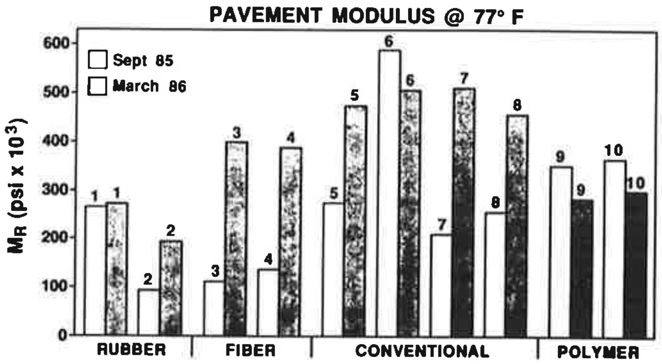

FIGURE 4 Variation in asphalt concrete stiffness with time.

FIGURE 5 Typical pavement condition of PlusRide after overlay (milepost 158.21 looking south).

FIGURE 6 Typical pavement condition of Arm-R-Shield after overlay (milepost 158.62 looking south).

FIGURE 7 Typical pavement condition of Class-C mix with polymer-modified asphalt and lime-treated aggregate (milepost 161.46 looking south).

requires high-level fatigue properties along with adequate resistance to freeze-thaw effects.

Future Test Schedule

Additional cores are scheduled to be taken in September 1986, 1987, and 1988. These results are expected to provide a better indication of changes in mix properties and features over time.

PERFORMANCE EVALUATION

During September and October 1985 the test road was evaluated for

- Pavement condition,
- Surface deflection,
- Skid resistance, and
- Ride.

The pavement surface within the limits of the project and within the limits of the test site is in good condition. Within the limits of the test sites, only one small crack and no significant rutting were observed. Figures 5–7 show the typical condition of the pavement.

Tables 14 and 15 give summaries of the results of deflection measurements and skid and ride tests. The results generally indicate that

1. The average deflection of the before condition (May 1985) varied considerably among sections;
2. The average deflections of the after condition (September 1985) are fairly uniform with most values ranging between 0.015 and 0.021 in.;
3. The reduction in deflection generally ranged between 50 and 70 percent;
4. The skid numbers for all sections are considered good and were about the same; the exception was the PlusRide section that exhibited the lowest value; and
5. The ride numbers for all sections were about the same and generally on the same order as those for conventional state projects.

In general, the results would indicate little variation in structural adequacy, skid, or ride among the various sections.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

On the basis of the results of this design and construction project, the following conclusions appear to be warranted:

1. The test site appears to be an excellent choice for evaluating the effects of asphalt additives on resistance to cracking and stripping because of previous performance problems in the vicinity.
2. Mix design techniques used by some additive suppliers are not well defined or documented.
3. Mix design results generated by the ODOT agree reasonably well with those recommended by the additive suppliers.
4. There were no major problems with the construction of the different mixes.

5. There are significant differences in the mix properties of the different materials.

6. The performance of all of the test sections after 1 year of service is good. There is no cracking, rutting, or extensive raveling of the pavement sections. However, failures have been experienced in this general area within 2 to 4 years after construction.

TABLE 14 SUMMARY OF EQUIVALENT BENKLEMAN BEAM DEFLECTIONS (10^{-3}) BEFORE AND AFTER OVERLAY

Material	Lane	Before 5/85	After 9/85	Percentage Reduction
PlusRide	SB	36.6	16.2	56
	NB	52.4	18.2	65
Arm-R-Shield	SB	59.3	17.2	71
	NB	53.9	15.7	71
Fiber Pave	SB	39.2	15.9	59
BoniFiber	NB	56.1	15.7	72
Class C (with Pave Bond)	NB	71.2	20.6	71
Class C (with lime and Pave Bond)	NB	48.9	20.6	58
Control (with lime)	SB	75.8	31.1	59
Control (without lime)	SB	48.0	18.1	62
CA(P)-1 (without lime)	SB	49.8	25.4	49
CA(P)-1 (with lime)	NB	32.3	23.1	28

NOTE: Equivalent Benkleman beam corrected to 70°F. SB = southbound and NB = northbound.

TABLE 15 SUMMARY OF SKID TESTS AND RIDE TESTS (October 1985)

Direction	Product	Avg Skid No.	Avg Mays Meter Ride Tests (in./0.1 mi/mi)
SB	PlusRide	44.3	34.3
NB	PlusRide	46.9	34.8
SB	Arm-R-Shield	48.4	33.7
NB	Arm-R-Shield	51.1	39.1
SB	Fiber Pave	52.1	34.9
NB	BoniFibers	52.9	26.4
NB	Class C (with Pave Bond)	55.6	35.0
NB	Class C (with lime and Pave Bond)	57.3	23.6
SB	Control (with lime)	53.3	32.4
SB	Control (without lime)	56.7	30.0
SB	CA(P)-1 (without lime)	52.9	28.4
NB	CA(P)-1 (with lime)	56.9	35.5

NOTE: SB = southbound; NB = northbound.

Recommendations

The following recommendations are warranted as a result of the findings:

1. Continue to monitor each of the sections for changes in performance. This should be done twice a year (fall and spring).

2. Continue to core the project to detect changes in mix properties. This should be done at least once a year, beginning fall 1986.

This monitoring program is expected to identify clearly how the mixes perform under severe traffic and environmental conditions.

ACKNOWLEDGMENT

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Chemical Modifiers To Improve the Strength and Durability of Asphalt Concrete

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The purpose of this paper is to discuss the value of chemical modifiers as a means of improving the physical properties and durability of asphalt concrete. A review of the factors that adversely affect the integrity and durability of asphalt concrete pavements is included. The ability of several different types of chemical modifiers to improve the properties and durability of asphalt concretes is illustrated by test data on laboratory and field specimens with and without chemical modifiers. Liquid chemical modifiers appear to provide a practical means of improving the strength, age-hardening resistance, water resistance, and overall integrity of asphalt concrete pavements.

The United States has the most extensive road system in the world. A vast majority of these roads have been constructed or renewed, or both, with asphalt concrete. Highway departments at all governmental levels are concerned about the quality and integrity of these pavements. Some of the factors that adversely affect the quality and durability of pavements are

- Poor-quality aggregates,
- Poor aggregate gradations,
- Poor-quality asphalts,
- Unstable substrates,
- Unusual and damaging weather,
- Traffic volume and heavily loaded vehicles,
- High temperatures and oxygen,
- Daily temperature cycling,
- Very low temperatures, and
- Water and ice.

In this paper are discussed efforts to improve the strength and durability of asphalt concrete by incorporating chemical additives in hydrocarbon asphalt cements. Chemical additives have been used for many years to improve other hydrocarbon products such as lubricants, fuels, and hydraulic fluids.

ASPHALT ADDITIVES AND MODIFIERS

A variety of materials has been proposed over the years as modifiers to improve the performance of asphalt concrete. These include fillers and fibers to reinforce the mixture; sulfur as a partial or complete replacement for asphalt; and polymers, elastomers, antistripping agents, and metal complexes to improve the properties of the asphalt cement and ultimately the properties of the asphalt concrete.

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The chemical modifiers discussed in this paper are liquid products that readily dissolve in asphalt cements. They do not significantly change the physical properties of the asphalt cements and do not require changes in current road-building procedures. These chemical modifiers are heat stable and remain effective after substantial storage periods in molten asphalt at temperatures in the 250°F-to-350°F range.

Three categories of chemical modifiers will be discussed. Their performance characteristics will be illustrated by test data obtained before and after various aging cycles. Evaluations of (a) asphalt cements, (b) laboratory-prepared asphalt concrete specimens, and (c) cores cut from test roads with and without chemical modifications are presented.

The three types of chemical modifiers are designed to improve the strength of asphalt concrete, improve the resistance of asphalt cements to age hardening, or improve the water resistance of asphalt concrete. The modifiers may be used to obtain a specific improvement, or they may be used in combination to address several problems.

MANGANESE MODIFICATION TO IMPROVE STRENGTH

Low concentrations of asphalt-soluble manganese compounds improve the strength and stability of asphalt concrete as measured by indirect tensile, Marshall stability, and unconfined compression tests *(1–3)*. The manganese is inactive until the modified asphalt cement becomes a thin coating on aggregate surfaces in the hot-mix plant. Manganese then accelerates oxidation during hot mixing, during transport to the job site, during placement, and for a period of time after placement *(4)*. The rate at which the consistency of the asphalt cement and the strength of the asphalt concrete increase is partly dependent on the ambient temperatures that prevail at a given site. The end result is the production of a high-consistency asphalt cement in the asphalt concrete. It is possible to obtain much higher stabilities with manganese-modified AC-5, AC-10, or AC-20 grade asphalts without the mixing and placement difficulties that would occur if a very high-viscosity asphalt cement (i.e., an AC-70 or AC-80 grade) were used.

The 140°F stabilities in the figures and tables that follow were obtained using the Marshall apparatus. The word Marshall has been omitted because the cylindrical test specimens were not prepared strictly in accordance with ASTM D 1559. The specimens were prepared with an automatic triple mechanical compactor rather than with a hand-operated compaction hammer. In addition, the number of blows to compact, in most cases, was not 50 per side but the number required to obtain specimens with approximately 7 percent air voids. Specimens compacted to lower densities are important for determining whether adequate stabilities will be obtained immediately after construction and for evaluating the resistance of the pavement to the degrading effects of air and water over extended periods at various temperatures after placement. The method of specimen preparation in a given comparative study is consistent so that effects of chemical modifiers on a given control asphalt cement can be determined without extraneous variables. Three replicates were used for each data point, and the numbers reported are the average of the three.

The effect of temperature on both a control and a manganese-modified asphalt concrete system is shown in Figures 1 and 2. Note that the time required for the modified, softer asphalt to provide increased stability becomes shorter as the curing temperature increases. Oxidation of asphalt is a chemical reaction. As a general rule the rate of chemical reactions doubles for every 10°C (18°F) increase in temperature. Catalysts increase the rate of reaction at any temperature. Manganese is an oxidation catalyst. The curing temperatures are continuous in these laboratory curing cycles. The times to obtain comparable increases in stability in actual service will be longer because peak temperatures on the road are experienced for only a few hours a day and vary with depth. The manganese modifier ultimately becomes inactive as it complexes with oxidized sites in the asphalt cement and the reaction stops. This is indicated by the leveling of the curve in Figure 2. The most effective way to illustrate the effect of manganese modification is to present engineering data on cores taken from highways.

A highway was constructed in West Virginia in 1982. This highway carries a large volume of heavily loaded coal trucks and thus high stability was required. The data given in Table 1 indicate that the modified wearing course had 54 percent higher stability than the control after 4 years and had reached a maximum after 2 years. In the base course the benefit of manganese modification was quite apparent after 2 years: its stability was 82 percent higher than that of the control. The data again indicate that the stability increase induced by the manganese appears to have leveled off after 2 years. Deflection tests run by West Virginia highway personnel after 2 years indicated a 25 percent higher deflection for the control pavement.

A section of Route 97 was placed in the same area of West Virginia in 1984–1985 near Beckley. Data are given in Table 2 that compare the properties of preconstruction laboratory specimens with cores cut from the subsequently placed highway. In this case a 33 percent higher stability was produced in the modified wearing course after 2 years, but the modified base course does not appear to have reached maximum stability. The laboratory results give an indication of the relative stabilities of control and modified systems. The only factor that causes change in the laboratory curing cycle is heat; changes in actual pavements are the result not only of heat but also of water and traffic. The laboratory specimens were all compacted to approximately 7 percent air voids to simulate initial pavement densities and to determine whether the initial stability of the modified asphalt concrete would be adequate to carry initial traffic loads. This proved to be true after placement of this road.

The low-temperature indirect tensile data on road cores from the Beckley, West Virginia, highway also indicate that the

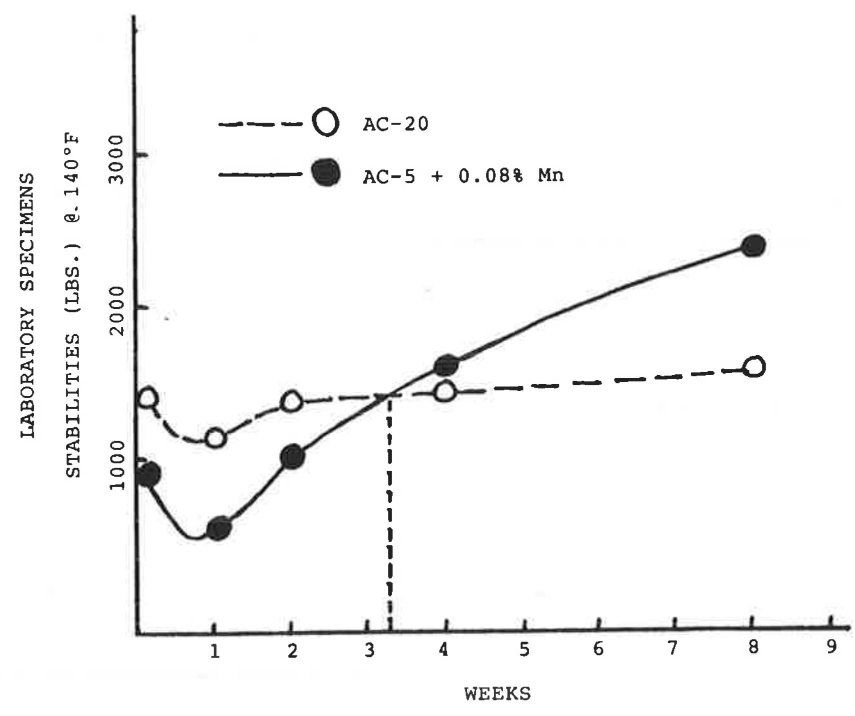

FIGURE 1 Curing rate study, manganese modification, 77°F.

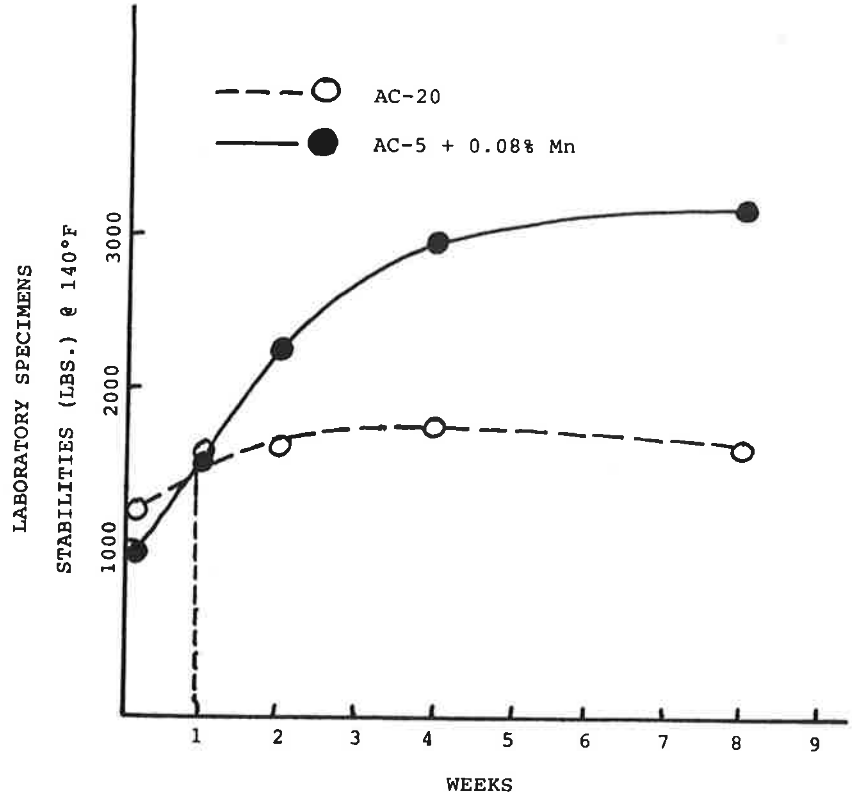

FIGURE 2 Curing rate study, manganese modification, 104°F.

TABLE 1 ROAD CORE PROPERTIES OF ROUTE 97 NEAR PINEVILLE, WEST VIRGINIA (1982 placement)

	1984		1986	
Core	Stability (lb)	Flow (0.01 in.)	Stability (lb)	Flow (0.01 in.)
Wearing course				
Control	2,582	8	2,166	13
Modified	3,738	10	3,325	15
Base course				
Control	1,236	15	1,906	20
Modified	2,247	27	2,432	27

NOTE: There are two 3-in. base courses and one 1-in. wearing course. Control asphalt is AC-20 grade; modified asphalt is AC-10 with 0.125 percent Mn. Cores are 4 in. in diameter. Measurements are at 140°F.

modified asphalt concretes have not reached maximum strength after 2 years. This is indicated by data given in Table 3. The rate of load application for all indirect tensile tests reported in this paper was 0.05 in./min and moduli were computed on the basis of a Poisson's ratio of 0.33.

In general, manganese modification is not recommended for thin overlays. However, in some cases the need for improved stability to prevent excessive rutting and shoving overrides the possibility of cracking caused by having a stiffer surface layer over a lower-strength and more flexible base. The data in Tables 4 and 5 for a modified 1.5-in. overlay put in place in 1985 in Houston, Texas, indicate the potential benefits of low-level manganese treatment to prevent surface rutting and shoving, which are serious problems in that locality. The modified section is currently outperforming the control section.

TABLE 2 PROPERTIES OF LABORATORY SPECIMENS AND ROAD CORES

	Initial Laboratory		Cured Laboratory		2-Year-Old Road Cores	
Specimen	Stability (lb)	Flow (0.01 in.)	Stability (lb)	Flow (0.01 in.)	Stability (lb)	Flow (0.01 in.)
Wearing course						
Control	1,009	15	1,294	7	1,111	15
Modified	680	13	2,003	19	1,478	12
Base course						
Control	1,842	14	1,553	20	1,882	20
Modified	1,186	18	3,270	28	1,617	21

NOTE: Pavement on Route 97 near Beckley, West Virginia, placed 1984–1985. Base courses are 3 in.; wearing courses are 1 in. Control asphalt is AC-20; modified asphalt is AC-10 with 0.125 percent Mn. Laboratory cure = 9 days at 140°F.

TABLE 3 INDIRECT TENSILE PROPERTIES OF 2-YEAR-OLD ROAD CORES FROM BECKLEY, WEST VIRGINIA

Core	Property Strength[a] (psi)	Strain (in./in. × 10^{-3})	Modulus (psi × 10^{-3})
Wearing course			
Control	121	2.5	110
Modified	103	2.4	119
Base course			
Control	183	3.4	134
Modified	92	3.8	50

[a]At 39.2°F.

Table 6 gives the engineering properties of cores taken in late 1985 and early 1986 from a highway constructed near Carlsbad, New Mexico, in March 1984. The data indicate ideal high-temperature stability, low-temperature flexibility, and water resistance properties for the modified section in comparison with the control. Nevertheless, cracking was observed at the edges of the modified pavement and in some places in the pavement itself after 21 to 24 months. This unexpected cracking may be due to the relatively large day-night temperature changes that occur in this geographic area. Manganese modification can increase asphalt coefficients of expansion and contraction and, in this case, the modified section has not responded to temperature changes as well as the control section.

As a result of studies of many manganese-modified highways, it has been concluded that the major benefits of manganese modification are (a) to improve the strength of full-depth asphalt concrete so that pavements will support heavy loads and (b) to provide an effective means of improving stabilities of local asphalt concrete highways where only poor-quality aggregates are available at a reasonable cost.

For the last 4 years, the performance of manganese-modified highways has been closely monitored and treatment levels have been decreased in accord with field experience. The early experimental manganese-modified roads contained relatively high levels of manganese, and as a result excessive stiffness, which led to cracking, was induced. In general, manganese modification for overlays is no longer recommended because stiffer surface courses do not have the flexibility to accommodate lower stability substrates as the pavement deflects under traffic. Many of the early manganese-modified highways were overlays, and they failed for this reason.

The degree of improvement obtained with manganese modification has steadily increased as the effects of this chemistry have become better understood (Table 7). Since late 1982 much lower levels of manganese have been recommended and, as the data in Tables 4–6 indicate, increased stabilities have been obtained.

ANTI-AGE-HARDENING AGENTS

Many laboratory studies and documented field experiences have shown that asphalt concrete is subject to cracking and raveling after service exposure. This deterioration has been related to increased hardness and decreased ductility of asphalt cement as it ages in service *(5)*. It is generally recognized that asphalts differ greatly in their resistance to age hardening because they are derived from different crude oils and in some cases produced by different refining processes *(6)*. There is a variety of theories about the mechanism of age hardening of asphalts, but, because of the chemical complexity of these organic materials and their variability from source to source, it has been difficult to identify a completely acceptable mechanism. It is known that oxidation changes the physical and chemical properties of asphalts and that asphalts recovered from deteriorated roads are oxygen rich. Oxidation of asphalt

TABLE 4 COMPARISON OF STABILITY AND FLOW OF LABORATORY SPECIMENS AND FIELD CORES AT 140°F

	Laboratory				Road Cores at			
	Initial		Cured[a]		1 Month		7 Months	
Asphalt	Stability	Flow	Stability	Flow	Stability	Flow	Stability	Flow
AC-20 control	215	7	279	7	515	8	706	9
Modified AC-20 with 0.06% Mn	180	6	1,290	10	570	10	1,018	10

NOTE: 1.5-in. overlay, city of Houston, placed in 1985. Stability is in pounds and flow in 0.01 in.
[a]Laboratory cure = 9 days at 140°F.

TABLE 5 COMPARISON OF INDIRECT TENSILE STRENGTH, STRAIN, AND ELASTIC MODULUS OF LABORATORY SPECIMENS AND CORE SAMPLES AT 39.2°F

				Road Cores at					
	Laboratory-Cured[a] Specimen			1 Month			7 Months		
Asphalt	Strength	Strain	Modulus	Strength	Strain	Modulus	Strength	Strain	Modulus
AC-20 control	171	4.2	82	226	5.9	81	224	4.4	108
Modified AC-20 with 0.06% Mn	171	2.0	169	164	3.1	110	212	1.6	278

NOTE: 1.5-in. overlay, city of Houston, placed in 1985. Strength is given in pounds per square inch, strain in in./in. × 10^{-3}, and modulus in psi × 10^{-3}.
[a]Laboratory cure = 9 days at 140°F.

TABLE 6 PROPERTIES OF FIELD CORES FROM US-62/US-180, CARLSBAD, NEW MEXICO

	Field Cores at			
	21 Months		24 Months	
Property	Control[a]	Modified[b]	Control[a]	Modified[b]
Stability at 140°F (lb)	1,528	1,981	1,761	1,915
Flow at 140°F (0.01 in.)	18	12	20	14
Strength at 39.2°F[c] (psi)			171	174
Strain at 39.2°F (in./in. × 10^{-3})			2.2	2.6
Modulus at 39.2°F (psi × 10^{-3})			167	133
Voids (%)			9	4
77°F Lottman cycle strength (psi)				
Dry			156	126
Wet			119	114
Wet/dry			0.76	0.91
Voids (%)			5.4	3.6

NOTE: 5-in. construction over a paved base. Aggregate treated with 1.5 percent hydrated lime. Placed March 1984.
[a]Control = 80–100 pen.
[b]Modified = 120–150 pen with 0.06 percent Mn.
[c]Indirect tensile strength.

TABLE 7 FIELD PERFORMANCE RATINGS: MANGANESE-MODIFIED PROJECTS VERSUS CONTROL PAVEMENTS

Year	Average Rating
1980–1982	−1.74
1982	+2.00
1983	+0.16
1984	+0.70
1985	+1.70

NOTE: +3 = modified pavement superior to control, 0 = modified pavement equal to control, and −3 = modified pavement much poorer than control. Intermediate number ratings were assigned, if necessary, on the basis of inspection of the projects.

cements is believed to be, directly or indirectly, a major cause of failure of asphalt concrete pavements.

Thin-film oven tests of asphalt cements are widely used to measure changes in properties when asphalts are exposed, as thin films, to air at elevated temperatures. The ASTM D 1754 procedure exposes a static film of asphalt to heat and air, and the ASTM D 2872 procedure exposes a moving or rolling film. The California Department of Transportation has developed a correlation between an extended rolling thin-film test and long-term aging of asphalt cement in California deserts *(7)*.

The rolling thin-film test procedure has been used to evaluate the effectiveness of a new series of liquid anti-age-hardening agents in a variety of asphalts. These materials vary in chemical functionality and molecular weight. In general they minimize the property changes of asphalts during exposure, as thin films, to air at elevated temperatures. The data given in Table 8 illustrate some of the effects of these anti-age-hardening modifiers in rolling thin-film oven tests. Four tubes, each with 35 g of asphalt, must be used for a given sample in order to have enough aged asphalt to run viscosity, penetration, and ductility tests. Use of an aged asphalt sample in more than one test may give misleading results.

The data in Table 8 illustrate several interesting points regarding age hardening of asphalts without and with chemical modifiers. The proprietary anti-age-hardening asphalt (AAHA) modifiers are identified as M-1 through M-7.

1. Note the property changes within a given asphalt type. Compare Samples 1, 2, and 3; 10 and 11; and 17. Asphalts B and C are much more susceptible to age hardening than is Asphalt A.
2. Anti-age-hardening agents, M-1 through M-7, minimize age hardening of asphalt cements. They prevent consistency increase as indicated by the 140°F viscosities and 77°F penetrations after aging. They also minimize the loss of ductility of asphalt cements.
3. The variability of asphalt cements and their aging tendencies require anti-age-hardening agents of more than one type of chemical functionality. The most effective anti-age-hardening modifier for a given asphalt and the optimum concentration of the modifier must be determined in each case in order to provide the most effective treatment for a given problem asphalt.

The temperature at which extended rolling thin-film oven tests are run is critical. The data given in Table 9 indicate the significantly increased age hardening that occurs when the temperature of the air contacting the rolling thin film of asphalt is increased from 215°F to 235°F. This increase in temperature is sufficient to at least double the rate of the reactions that cause age hardening. The 140°F viscosity values after 7 days at 235°F are as much as seven times higher than those obtained at 215°F. The requirements for the Caltrans durable asphalt specification are based on a temperature of 235°F. To meet this specification a combination of a reasonably stable asphalt and an effective anti-age-hardening additive is required.

The testing of chemically modified asphalt films by exposure to air in thin films at elevated temperatures provides a method of identifying potential anti-age-hardening agents. Their true value, however, can only be assessed in actual asphalt mixtures

TABLE 8 SCREENING OF ANTI-AGE-HARDENING AGENTS IN AN EXTENDED ROLLING THIN-FILM OVEN TEST (properties after 7 days at 215°F)

Sample	Asphalt	Grade	AAHA Modifier	140°F Vis (poises)	77°F Pen (0.01 mm)	77°F Ductility (cm)
1	A	AR1000	None	7 725	20	150+
2	A	AR2000	None	16 660	11	139
3	A	AR4000	None	20 335	10	13
4	A	AR4000	1% M-1	17 300	14	150+
5	A	AR4000	2% M-1	14 045	16	150+
6	A	AR4000	4% M-1	10 170	17	150+
7	A	AR4000	2% M-5	8 698	16	150+
8	A	AR4000	2% M-6	9 864	20	150+
9	A	AR4000	2% M-7	9 076	16	150+
10	B	AC-10	None	56 639	10	6
11	B	AC-20	None	99 500	8	6
12	B	AC-20	2% M-1	85 840	9	6
13	B	AC-20	3% M-1	13 406	17	26
14	B	AC-20	4% M-1	11 905	17	24
15	B	AC-20	6% M-1	11 466	19	37
16	B	AC-20	2% M-5	16 255	17	14
17	C	AR4000	None	87 020	19	9
18	C	AR4000	2% M-1	69 770	17	13
19	C	AR4000	4% M-1	14 963	35	92
20	C	AR4000	2% M-2	21 236	33	52
21	C	AR4000	3% M-2	20 706	32	150+
22	C	AR4000	3% M-2	23 206	27	150+

TABLE 9 CRITICAL TEMPERATURE EFFECTS IN EXTENDED ROLLING THIN-FILM OVEN TESTS

Test Temperature (°F)	Asphalt	AAHA Modifier	140°F Vis (poises)	77°F Pen (0.01 mm)	77°F Ductility (cm)
215	A	None	20 335	10	13
235	A	None	34 123	10	14
215	A	2% M-6	9 864	20	150+
235	A	3% M-6	31 423	12	35
215	A	2% M-5	8 698	16	150+
235	A	2% M-5	28 014	11	35
215	C	None	87 020	19	9
235	C	None	139 092	19	8
215	C	3% M-2	23 206	27	150+
235	C	3% M-2	159 217	19	7
235	Caltrans durable asphalt specification		75 000 max	25 min	30 min

NOTE: Properties are those of grade AR-4000 asphalts after 7 days.

because the aggregate substrate can influence the durability of thin films of asphalt cements. Inorganic aggregate surfaces can adsorb or absorb components of asphalt cement, much like the substrate absorbs specific chemical structures in a chromatographic column. The chemistry of an aggregate surface can influence chemical changes in asphalt cements in the same manner that inorganic catalysts influence cracking and reforming of hydrocarbon crude oil fractions.

The possibility of aggregates influencing the durability of asphalt cement and possibly interfering with the functioning of anti-age-hardening agents led to evaluations of these chemicals in asphalt concrete mixtures. The testing procedures are the same as those described earlier in the paper for evaluating the physical properties of manganese-modified laboratory specimens and road cores. The following data, however, illustrate the abilities of anti-age-hardening agents to minimize changes in the properties of asphalt concrete specimens during heat-aging cycles.

New, properly designed and constructed asphalt concrete pavements usually have the strength and flexibility to support traffic. Most highways would remain intact if it were not for the degradation of the asphalt cement by the effects of heat, air, and water. The data given in Tables 10 and 11 indicate that these new anti-age-hardening agents, at relatively low levels, will function in asphalt concretes and minimize changes in properties during simulated aging. The choice of the most

TABLE 10 SCREENING OF ANTI-AGE-HARDENING AGENTS IN LABORATORY SPECIMENS: STABILITY AT 140°F

Sample	Asphalt	AAHA Modifier	Initial Stability (lb)	Initial Flow (0.01 in.)	Aged Stability (lb)	Aged Flow (0.01 in.)	Percent Change
California Granite Aggregate and Mix							
1	A (AR-4000)	None	1,896	12	2,196	14	+16
2	A (AR-4000)	2% M-1	2,129	12	2,146	13	+1
3	C (AR-4000)	None	1,214	12	1,431	16	+18
4	C (AR-4000)	2% M-1	1,162	15	1,246	18	+7
Ohio Limestone Aggregate and Mix							
5	A (AR-4000)	None	559	8	790	8	+41
6	A (AR-4000)	4% M-1	407	8	474	7	+17
7	D (AC-20)	None	687	8	1,244	8	+81
8	D (AC-20)	4% M-1	514	7	711	9	+38
9	C (AR-4000)	None	468	8	1,092	10	+133
10	C (AR-4000)	2% M-2	709	7	1,125	10	+59

effective anti-age-hardening agent and its optimum concentration requires tests on each combination of asphalt and aggregate.

A large area was resurfaced in Wickliffe, Ohio, in the fall of 1985 to determine the effectiveness of anti-age-hardening agent M-1. Cores were taken shortly after placement and again 9 months later in July 1986. The engineering data on these field cores are summarized in Table 12. The high-temperature stability data and the low-temperature strain values obtained on these cores clearly indicate that chemical modifier M-1 minimizes hardening of asphalt concrete under service conditions.

ANTI-AGE-HARDENING AGENTS TO MEET ASPHALT SPECIFICATIONS

The new anti-age-hardening agents can also be used in asphalt cements that, as produced, do not meet certain governmental specifications. In particular, ductility requirements after thin-film oven aging are in some cases difficult for an asphalt supplier to meet. Table 13 gives examples of how these chemicals can provide solutions for problems of this type. Note that, after aging, each anti-age-hardening modifier, at the 0.5 percent level, has enabled the asphalt to maintain higher penetrations, lower 140°F viscosities, and higher 60°F ductilities. In this particular evaluation, both M-2 and M-4 produced an AC-15 grade asphalt that met aged specification requirements of the responsible department of transportation.

CHEMICAL MODIFIERS TO IMPROVE WATER RESISTANCE

It is meaningless to discuss the durability of asphalt concrete without considering the damaging effects of water *(8)*. Many aggregate surfaces are hydrophilic in nature and therefore will attract water that can, under certain conditions, saturate asphalt pavements. Water may fill voids and weaken asphalt-aggregate bonds. After the water has run off or evaporated, the asphalt-aggregate bond may not recover. Water, to some extent, is a solvent for most materials, and as it percolates through an asphalt concrete network it extracts components of both the aggregates and the asphalt. The amount of asphalt components extracted could be substantial because some oxidized asphalt components are water soluble. Water as a liquid can degrade the asphalt-aggregate mixture, and as water freezes and expands it can severely stress the pavement structure.

TABLE 11 SCREENING OF ANTI-AGE-HARDENING AGENTS IN LABORATORY SPECIMENS: INDIRECT TENSILE STRENGTH AT 39.2°F

Sample	Asphalt	AAHA Modifier	Indirect Tensile Strength (psi)	Strain (in./in. × 10^{-3})	Modulus (psi × 10^{-3})
California Granite Aggregate and Mix					
1	A (AR-4000)	None	360	4.4	165.4
2	A (AR-4000)	2% M-1	242	4.2	117.1
Ohio Limestone Aggregate and Mix					
3	A (AR-4000)	None	253	2.2	228.8
4	A (AR-4000)	4% M-1	191	3.9	98.0
5	D (AC-20)	None	166	6.6	50.3
6	D (AC-20)	4% M-1	122	7.4	33.2

NOTE: Measurements made after curing for 9 days at 140°F.

TABLE 12 SCREENING OF ANTI-AGE-HARDENING AGENT IN 1.5-in. OVERLAY

	Initial Core		Core at 9 Months		Percentage Change	
Property	Control	Control + 2% M-1	Control	Control + 2% M-1	Control	Control + 2% M-1
Stability at 140°F (lb)	456	405	872	469	+91	+16
Flow at 140°F (0.01 in.)	9	9	10	9		
Strength at 39.2°F[a] (psi)	111	85	82	110		
Strain at 39.2°F (in./in. × 10^{-3})	6.7	7.0	2.2	4.2		
Modulus at 39.2°F (psi × 10^{-3})	34	27	87	56		

NOTE: Placed October 1985 in Wickliffe, Ohio. Ohio aggregate and AC-20 Asphalt D.
[a]Indirect tensile strength.

TABLE 13 ANTI-AGE-HARDENING AGENTS FOR ASPHALT SPECIFICATIONS

	Sample			
Property	1	2	3	4
AAHA modifier	None	0.5% M-1	0.5% M-2	0.5% M-4
Weight loss (%)	0.54	0.59	0.50	0.50
77°F pen				
Initial	48	56	63	64
Aged	31	35	37	38
140°F Vis (poises)				
Initial	1442	1327	1572	1672
Aged	3969	2899	2830	3479
275°F Vis (cSt)				
Initial	322	310	332	318
Aged	430	433	459	440
60°F ductility (cm)				
Initial	150+	150+	150+	150+
Aged	14.6	14.9	20.8	32.3

NOTE: Thin-film oven test (ASTM D 1754-83), 5 hr at 325°F; Test Asphalt E = AC-15.

Antistrip additives have been used for many years to displace water from wet aggregate surfaces in order to enable asphalt cutbacks to bond to aggregate in the field. Heat-stable antistrip compounds have been developed that can be used in asphalt cements to improve the asphalt-aggregate bond in hot mixes and help maintain the bond during periods when pavements are saturated with water. Most antistrip additives are water soluble yet produce stable solutions or dispersions at low concentrations in asphalt.

Hydrated lime is widely used as a treatment for aggregates before they are coated with asphalts in hot-mix plants. This treatment has been found to be effective in improving the water resistance of asphalt pavements, and it appears to be less sensitive to aggregate and asphalt variations than are asphalt dispersible antistrip compounds. The effectiveness of the latter varies widely, although there are commercially available materials the effectiveness of which approaches that of hydrated lime. Hydrated lime is water soluble and not soluble in asphalt, and therefore, to be effective, it must be uniformly distributed over the surface of the aggregate before the aggregate is coated with asphalt cement.

The detrimental effect of water on the strength of an asphalt concrete is shown in Figures 3 and 4. The laboratory specimens were in one case immersed in water at room temperature the sixth day of each week and in the other case held continuously in an oven at the indicated temperatures. Note in Figure 3 that, at the 77°F cure temperature, periodic water immersion has decreased stability by as much as 45 percent whereas, as indicated in Figure 4, the decrease in stability at the 104°F cure temperature is, at one point, 55 percent.

A liquid modifier, which is much more soluble in asphalt and less soluble in water than are conventional antistrip compounds, has been developed to improve water resistance of asphalt concretes. It is identified as Additive L in the tables and figures that illustrate its performance characteristics.

The Lottman test is gaining increasing acceptance as a means of evaluating the water-resistance properties of asphalt concrete specimens. This test may be too severe for evaluating materials for temperate climates because of its freeze cycle, but it provides a realistic method of comparing the water resistance of asphalt concretes without and with various chemical modifiers.

Table 14 gives an overview of the performance characteristics of Additive L vis-à-vis aggregates, asphalts, and several commercial antistrip compounds.

Figure 5 shows that Additive L provides as much water resistance after seven cycles of Lottman test exposures as does the untreated asphalt concrete after one Lottman cycle.

Figure 6 shows that Additive L does not provide the long-term resistance to water damage that may be obtained by treatment of aggregate with hydrated lime. The relatively small additional improvement is, however, costly in terms of both

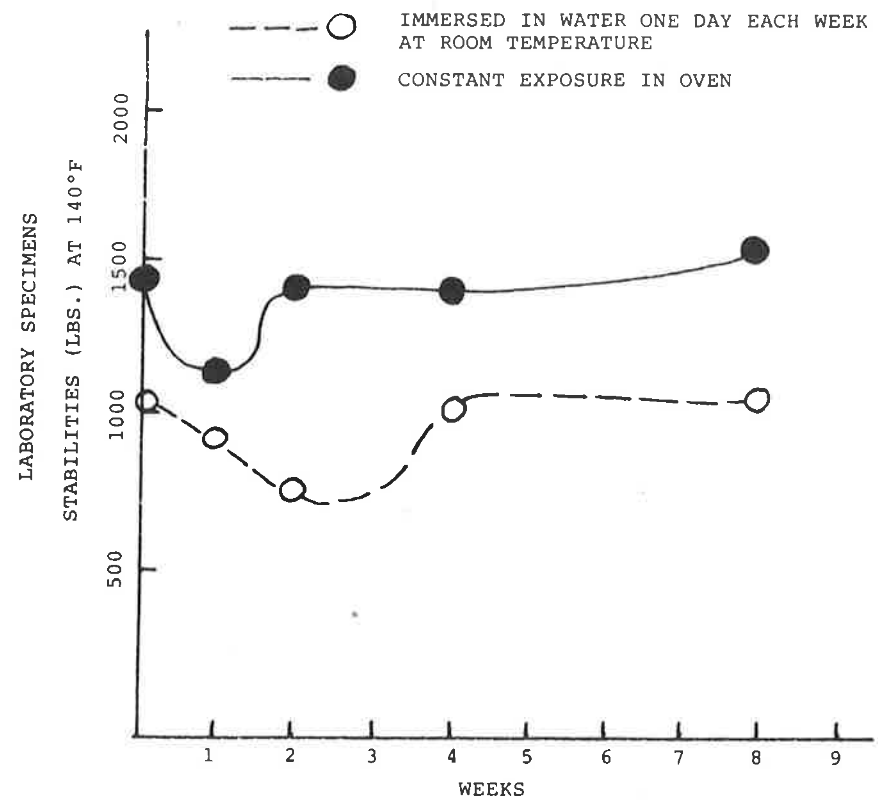

FIGURE 3 Curing rate study with water, 77°F.

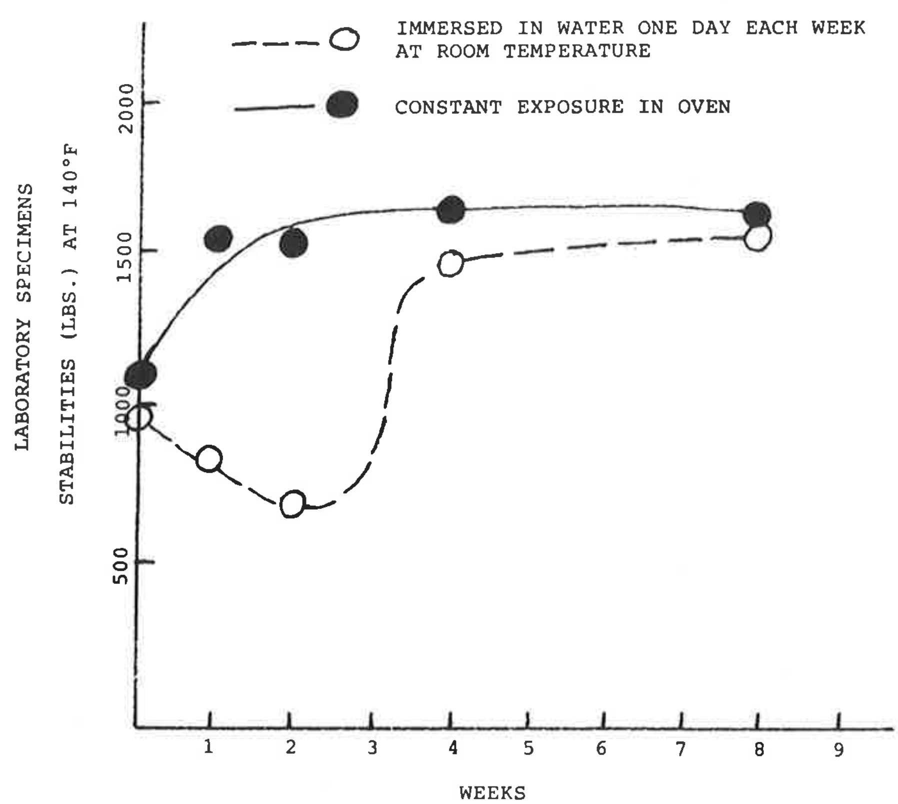

FIGURE 4 Curing rate study with water, 104°F.

TABLE 14 EFFECT OF AGGREGATES, ASPHALTS, AND MODIFIERS ON WATER RESISTANCE

Aggregate	Asphalt	0.5% Additive	Strength (psi) Dry	Strength (psi) Wet	Wet/Dry Ratio
Texas pea gravel	I	None	169	70	41
		Commercial A	152	81	54
		Commercial B	167	103	62
		Commercial C	175	127	72
		Additive L	181	132	73
	II	None	129	70	54
		Commercial A	141	110	79
		Commercial B	127	102	80
		Commercial C	122	103	85
		Additive L	117	115	98
Georgia granite	III	None	85	28	33
		Commercial A	87	80	93
		Commercial B	94	82	87
		Commercial C	99	85	86
		Additive L	104	89	86
	IV	None	121	64	53
		Commercial A	145	105	73
		Commercial B	154	134	87
		Commercial C	140	114	82
		Additive L	146	132	91

NOTE: Lottman method; AC-20 asphalt content = 5.2 percent by weight; air voids = 6.5 to 8.5 percent by volume.

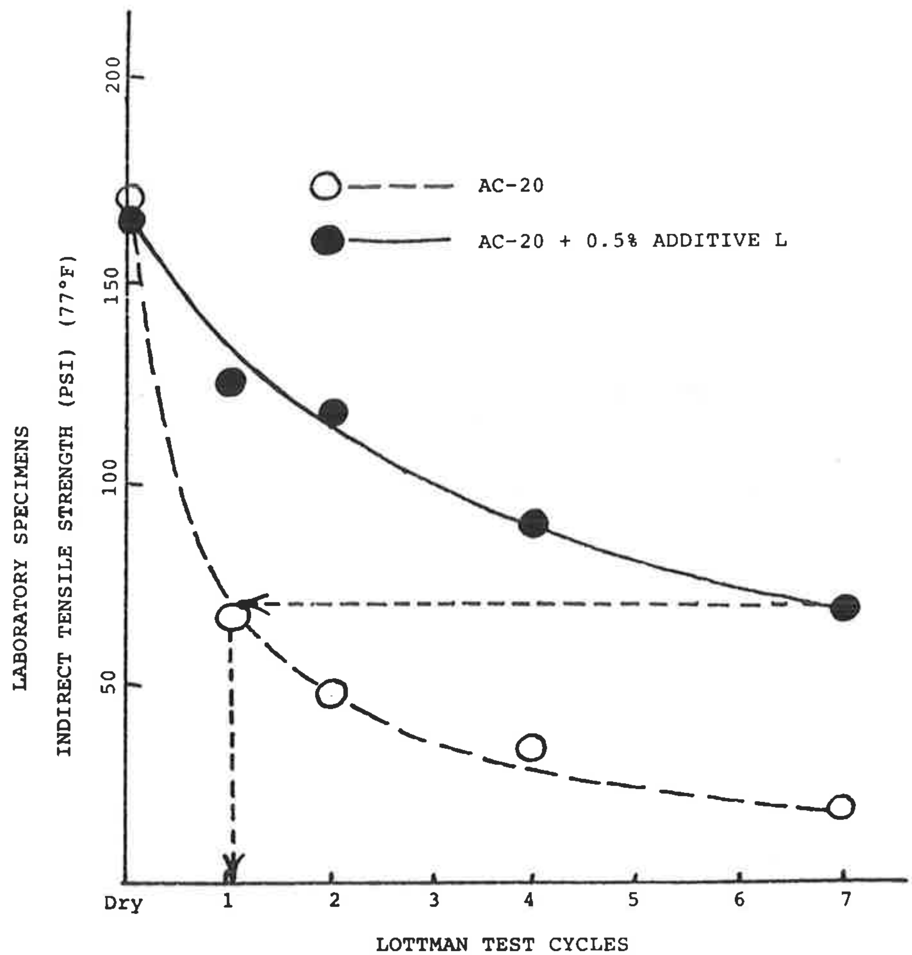

FIGURE 5 Multiple-cycle Lottman testing with and without Additive L.

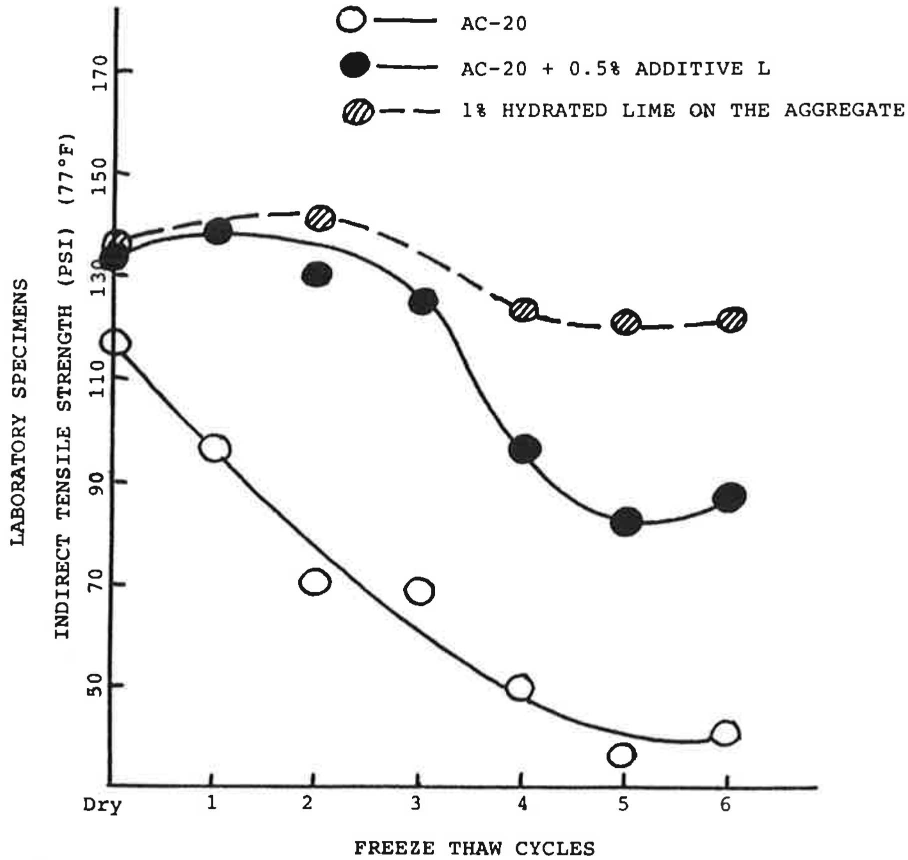

FIGURE 6 Multiple-cycle Lottman testing with and without Additive L and hydrated lime.

materials and plant complexity. Because the multiple-cycle Lottman test is severe, the additional improvement provided by hydrated lime may not be indicative of field performance.

Additive L in combination with other types of functional modifiers may well provide synergistic and beneficial results to enhance the durability of asphalt concrete. An example of potential synergism to provide improvement in both strength and water resistance is given in Table 15. Sample 4 provides an initial strength improvement of 28 percent over Sample 1 and excellent water resistance with only 0.045 percent Mn whereas Samples 2 and 3 provide only 14 and 16 percent strength improvement, respectively, with higher levels of manganese, and little improvement in water resistance. The combination has not only provided significantly improved water resistance, it has also enhanced the strength improvement of the manganese modifier.

Another interesting example of the efficacy of multifunctional modifiers is given in Table 16 by data obtained after several Lottman cycles. Although these data are not conclusive, a multifunctional treatment with a low level of manganese (0.024 percent) has provided a 66 percent increase in cured stability over the control and, as the multicycle Lottman data indicate, a remarkably stable system as far as water resistance is concerned. The wet/dry indirect tensile strength ratios obtained with the multifunctional additive system in these repeated Lottman cycles are similar to those that can be obtained with hydrated lime.

TABLE 15 MULTIFUNCTIONAL MODIFIER: STRENGTH AND WATER RESISTANCE

Sample	Asphalt Modifier	Indirect Tensile Strength at 77°F (psi)		Wet/Dry Ratio	Relative Binder Cost ($/ton)
		Dry	Wet		
1	None	129	55	0.43	150
2	0.06% Mn	147	62	0.43	188
3	0.08% Mn	150	66	0.44	201
4	Multifunctional modifier (0.045% Mn)	165	120	0.73	197

NOTE: Lottman method; aggregate is Texas pea gravel; asphalt is AC-20; asphalt content is 4.6 percent.

TABLE 16 MULTIFUNCTIONAL MODIFIER: STRENGTH, WATER RESISTANCE, AND AGE-HARDENING RESISTANCE

	Without Modifier		With 0.024% Mn Modifier	
Property	Initial	Cured	Initial	Cured
Stability at 140°F (lb)	577	1,102	592	1,833
Lottman test wet/dry ratio after cycles				
1	110		95	
2	44		101	
3	55		88	
4	40		89	

CONCLUSIONS

The data presented in this paper, which are representative of a large amount of data on chemical modification of asphalt cements, lead to several conclusions.

1. Certain liquid, asphalt-soluble chemical modifiers do not require changes in procedures currently used to build asphalt concrete highways.

2. Manganese modifiers may improve the strength and reduce the deflection of full-depth asphalt concrete so that heavy loads may be supported over extended periods.

3. Manganese modifiers make it possible to use poor-quality aggregate to produce asphalt concrete pavements with acceptable stabilities.

4. Certain anti-age-hardening chemicals minimize the decrease in penetration, the increase in viscosity, and the loss of ductility in asphalts exposed to air in thin films at elevated temperature.

5. Anti-age-hardening chemicals may extend the useful life of asphalt concrete pavements.

6. A new asphalt-soluble additive that provides improved water resistance in asphalt concretes has been developed. Its effectiveness approaches that of the hydrated lime treatment of aggregates.

7. Preliminary results indicate that certain multifunctional additives may provide both increased strength and resistance to water in asphalt concretes.

8. Asphalt concrete systems must be evaluated individually to determine the type and concentration of chemical modifier needed to meet performance requirements of specific applications.

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Development of Laboratory Oxidative Aging Procedures for Asphalt Cements and Asphalt Mixtures

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An evaluation of an oxidative aging procedure for asphalt materials is described. Test results and the effectiveness of the aging device used are presented. The study was performed by Oregon State University and the Oregon Department of Transportation. This study involved laboratory tests on field core samples as well as laboratory mixture samples and asphalt cements used in three projects constructed in Oregon. The procedure selected for aging laboratory mixtures involved using a pressure oxidation bomb (POB), a sealed container in which asphalt mixtures or asphalt samples, or both, were subjected to pure oxygen at 100 psi pressure at 60°C for periods of up to 5 days. Resilient modulus and fatigue tests were performed to measure the properties of cores and laboratory mixtures (before and after aging). The asphalt samples were aged on a Fraass plaque to achieve minimum disturbance of the sample, and the degree of aging was assessed by changes in the Fraass breaking temperature. The results of this study showed that the POB was an effective means of producing measurable changes in both mixtures and asphalt samples. However, the mixture properties were substantially different from those measured for the field core samples whereas the asphalt properties were similar. As evaluation parameters, the modulus ratio and Fraass breaking temperature are good indicators of the aging rate of mixtures and asphalt cement, respectively. The study also indicated that aging rate is a function of air voids in the mixture and asphalt properties.

Premature failure or poor performance of asphalt pavements often results from weakening of the adhesive bond between asphalt cement and aggregate particles brought on by any of all of the following causes: the action of moisture, mechanical stresses, and aging of asphalt cement. Aging causes the properties of asphalt pavements to change and usually is accompanied by hardening of the asphalt cement. Petersen *(1)* indicated that three fundamental composition-related factors govern the changes that could cause hardening of asphalts in pavements:

1. Loss of the oily components of asphalt by volatization or absorption by porous aggregates,
2. Changes in the chemical composition of asphalt molecules caused by reaction with atmospheric oxygen, and
3. Molecular structuring that produces thixotropic effects (steric hardening).

Oxidation of asphalt is generally considered a major factor contributing to the hardening and embrittlement of asphalt pavement. This oxidation occurs during the preparation and laydown of hot-mix pavements as well as because of environmental aging while in service. Excessive hardening of the asphalt cement is undesirable because it often leads to problems associated with pavement embrittlement and cracking. The rate of hardening is affected by the chemical composition of asphalt, light, aggregate properties, and ambient temperature *(2–5)*. After the development of the recovery method by Abson in 1933 provided a means of recovering the asphalt from hot plant mixtures both immediately and after periods of aging in the pavement, numerous methods of evaluating the properties of "aged" asphalt cement were developed. Many aging methods attempt to correlate short-term laboratory aging with the changes in asphalt properties that occur after long exposure in the pavement as well as during mixing operations.

Methods used by various investigators include use of high temperatures, light, chemical oxidation agents, and oxidation in solution under oxygen pressure. Welborn *(6)* summarizes the various test conditions, such as temperature and exposure time, used for the aging methods along with evaluation parameters. Most of the methods represent short-term aging due to the mixing process rather than long-term aging. Most of the evaluation parameters are measurements of consistency of the asphalt cement, such as penetration, viscosity, and ductility. For asphalt mixtures, Pauls and Welborn *(7)* used the compressive strength of the weathered mixtures as the evaluation parameter. Kemp and Predoehl *(8)* used the resilient modulus of weathered briquettes as the evaluation parameter.

The major objective of the study reported in this paper was to develop a laboratory procedure to simulate long-term aging effects. The approach selected was similar to the Lottman test for moisture effects, which has been used extensively by the authors *(9, 10)*. Fundamental properties (such as tensile strength, resilient modulus, fatigue life, and permanent deformation) of the asphalt mixture were measured before and after conditioning (i.e., either by moisture or oxygen). It was also desired to evaluate asphalt cements as well as asphalt mixtures before and after oxygen conditioning (i.e., aging in oxygen).

The purpose of this paper is to (a) present an oxidative aging procedure adopted for a recent aging study by Oregon State University (OSU) and the Oregon Department of Transportation (ODOT) and (b) discuss the test results and the effectiveness of the aging device.

SELECTION OF AGING AND EVALUATION METHODS

After review of the various aging procedures used previously, it was decided that the method adopted in the aging study should attempt to reproduce the oxidative aging that occurs after

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construction of a pavement. The research approach included comparison of artificially aged laboratory-prepared samples with cores aged in the field. The extent to which aging of laboratory mixtures could be achieved with the developed methods is emphasized in this paper. Both laboratory mixtures and asphalt samples were subjected to aging. Asphalt sample aging was included in the test program to see if the methods adopted would indicate the susceptibility of asphalt to aging. A major goal of this research was to use an aging device suitable for both types of samples.

The pressure oxidation bomb (POB), originally developed in Britain *(11)* and recently used by Edler et al. *(12)* in South Africa, was selected as the most suitable aging device and modified in the aging study. This modified device can contain two mixtures or one mixture and several Fraass samples. As reported by Thenoux et al. *(13)*, the use of Fraass samples for aging asphalt has the advantage of minimal disturbance to the asphalt, which is tested on the "container" on which it is aged.

The POB was operated in the aging study with the samples contained in a pure oxygen environment at 100 psi and 60°C. [Edler et al. *(12)* used 300 psi and 150°C.] These levels were arbitrarily selected in response to safety concerns about the use of pressurized oxygen and to preserve the shape of the mixtures.

MATERIALS TESTED

Asphalt cements and aggregates for three projects constructed in Oregon [Idylwood Street (1974) Plainview Road–Deschutes River (1980), Arnold Ice Caves–Horse Ridge (1973)] were used for the laboratory simulation aging study. These projects were from 5 to 10 years old and in each project samples of the original asphalt and representative aggregate were available. For each of the three projects the mix type was B-mix (Table 1), and the grade and optimum content of asphalt cement were AR-4000 of 7.0 percent for Idylwood Street, AR-4000 of 5.5 percent for the Plainview Road–Deschutes River project, and 120/150 pen of 6.5 percent for the Arnold Ice Caves–Horse Ridge project. Laboratory mixtures were prepared using a kneading compactor, and field cores were obtained for each project.

TABLE 1 AGGREGATE GRADATION, CLASS-B MIX

	Aggregate Gradation (% passing)		
Sieve Size	Idylwood Street	Plainview Road–Deschutes River	Arnold Ice Caves–Horse Ridge
1 in.			100
3/4 in.	100	100	97
1/2 in.	87	87	84
3/8 in.	78	74	74
1/4 in.	63	60	60
No. 4	52	52	53
No. 10	30	31	32
No. 40	12	14	15
No. 200	5	5	5

TEST PROGRAM

Cores

The ODOT testing program included tests for aggregate gradation, asphalt cement contents, air voids, and recovered asphalt cement properties. The repeated load diametral test for modulus and fatigue life of cores was performed by Oregon State University.

Laboratory Mixtures

The variables considered in the laboratory mixture preparation were

1. Compaction level: 94 percent of maximum density (100 blows at 500 psi after 20 blows at 250 psi and leveling load of 12,500 lb for 15 sec) and 88 percent of maximum density (30 blows at 100 psi and leveling load of 1,000 lb for 15 sec) and
2. Aging period: 0, 1, 2, 3, and 5 days.

Each of the variables was studied relative to the original mix design for the projects studied, which was a standard mix. Following the standard ODOT procedure *(14)* using a kneading compactor, specimens 4 in. (100 mm) in diameter and 2.5 in. (63 mm) high were fabricated for three projects (Idylwood Street, Plainview Road–Deschutes River, and Arnold Ice Caves–Horse Ridge) by using the same asphalt cement and mix design employed at the time of construction. A minimum of 10 specimens for each compaction level were prepared for each of the three projects. All 60 specimens were tested for resilient modulus and fatigue life. All diametral tests were run for tensile stress levels of 20 psi and 40 psi for 88 and 94 percent compaction level, respectively.

Asphalts

Routine asphalt tests were run by ODOT. Fraass samples were prepared and tested by OSU, and chemical composition tests were run in accordance with ASTM D 4124.

DETAILS OF TEST METHODS

For this aging study the POB was used to simulate the oxidative aging that occurs after construction. The resilient modulus and fatigue life of cores and laboratory mixture samples as well as the Fraass breaking temperature of asphalt cement were measured. The modulus ratio (the ratio of the modulus after aging to the modulus before aging) and the Fraass breaking temperature were adopted to measure the changes in the properties of laboratory mixture samples and asphalt cements, respectively.

Aging Procedure

The modified POB is a cylindrical pressure vessel made of stainless steel fitted with a screw-on cover that contains an

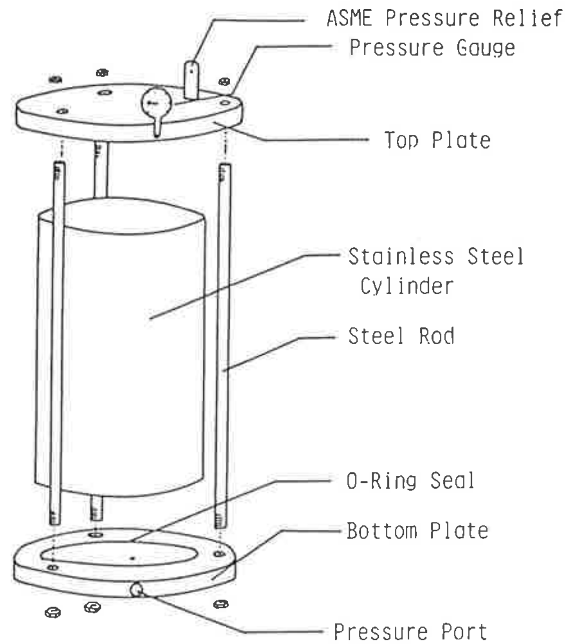


FIGURE 1 Pressure oxidation bomb.

ASME pressure relief for safety, a pressure gauge, and a stopcock. The POB was sealed with O-rings. Figure 1 shows a diagram of the POB used for this study.

The following are the main steps in the use of the POB:

1. Samples (asphalt mixtures or asphalt cement) are prepared.
2. The samples are placed in a POB.
3. Vacuum [26 in. (66 cm) Hg] is applied for 20 min.
4. The POB is filled via the stopcock from an oxygen cylinder to 100 psi (689.5 kPa). This pressure is held for 30 min to ensure leak-free joints.
5. The POB is then placed in an oven maintained at 60°C (140°F) for 1, 2, 3, and 5 days.
6. At the conclusion of the test the stopcock is opened, the cover is removed, and the aged mixtures or asphalt cement samples, or both, are cooled for 1 day and 1 hour, respectively, at room temperature.

Fraass Brittle Test

The Fraass breaking point *(11)* is the temperature at which an asphalt first becomes brittle as indicated by the appearance of cracks when a thin film of asphalt on a metal plaque is cooled at the rate of 1°C/min and flexed at a constant rate. The apparatus, sample preparation, and test are described elsewhere *(13, 15)*.

The following are the main steps used for this study:

1. The sample (0.4 g) is prepared as described in Kim et al. *(15)*.
2. A standard steel plaque [1.6 in. × 0.8 in. (41 mm × 20 mm)] is coated uniformly with a thin layer of asphalt cement [0.02 in. (0.5 mm)].
3. The steel plaque coated with asphalt cement is placed in a closed chamber, and the temperature of the plaque is lowered steadily at a rate of 1°C/min (1.8°F/min) by adding solid carbon dioxide to the acetone bath that surrounds the chamber in which the plaque is located.
4. The steel plaque is repeatedly bent a given amount in a standard time. The temperature ("brittle temperature") at which one or more cracks appear is recorded as the breaking point.

Repeated Load Diametral Test

The resilient modulus and fatigue tests were performed using the repeated load diametral test apparatus. The parameters recorded during the repeated load diametral test were the load applied, the horizontal elastic deformation, and the number of repetitions to failure. During the tests the dynamic load duration was fixed at 0.1 sec and the load frequency was 60 cycles per minute. A static load of 10 lb (4.5 kg) was applied to hold the specimen in place. The tests were carried out at 70.7°F ± 1.6°F (21.5°C ± 0.9°C). For this study the number of load repetitions to fatigue failure was defined as the number of repetitions required to cause a vertical crack approximately 0.25 in. (0.64 cm) wide in the specimens. To stop the test at the specified level of specimen deformation, a thin aluminum strip was attached to the sides of the specimen along a plane perpendicular to the plane formed by the load platen. As the deformation of the specimen exceeded a certain level, the aluminum strip broke and opened the relay, which shut off the test. The test procedures employed are described in detail elsewhere *(15)*.

TEST RESULTS AND DISCUSSION

Resilient Modulus

The average modulus values of six cores from each of three projects are given in Table 2; also given are thickness, air voids, and asphalt contents. For laboratory mixtures, two specimens were tested as compacted and the other eight specimens were tested both before and after aging at each compaction

TABLE 2 SUMMARY OF CORE DATA

Project	Thickness (in.)	Maximum Specific Gravity	Bulk Specific Gravity	Air Voids (%)	Asphalt Content (%)	Resilient Modulus (ksi)
Idylwood Street	1.9	2.459	2.17	11.8	5.9	772
Plainview Road–Deschutes River	1.4	2.497	2.29	8.3	5.8	569
Arnold Ice Caves–Horse Ridge	1.6	2.444	2.34	4.3	6.7	244

level. The aged specimens were cooled for 1 day before their moduli were measured. The modulus test results including bulk specific gravity, air voids, and maximum specific gravity (AASHTO T-209) are summarized in Table 3. The aging effect assessed by the modulus ratio (the ratio of the modulus after aging to the modulus before aging) is shown in Figure 2.

The mixtures for the Idylwood Street project (ID-ST) with 88 percent compaction level aged at a constant rate; the mixtures for the other two projects aged quite rapidly during the first two or three days. The aging rate (slope in Figure 2) of the mixtures for the Plainview Road–Deschutes River (PR-DR) project changed little after the third day, and that of the Arnold Ice Caves–Horse Ridge (AIC-HR) project still increased. Even though the same grade of asphalt cement (AR-4000) was used for both the Idylwood Street and the Plainview Road–Deschutes River project, the trend of mixture aging is significantly different. The difference is probably because the physical properties of the original and recovered asphalt cement were substantially different (Table 4) (i.e., the asphalts were probably from different sources).

The results from the mixtures compacted at the 94 percent level show a slightly different trend. Unlike the mixtures compacted at the 88 percent level, the mixtures with 94 percent compaction level for the Idylwood Street and Plainview Road–Deschutes River projects aged little during the first 2 days and then aged rapidly between the second and third days. This rapid aging was followed by a period of slow aging between the third and fifth days. This result may show that it takes some time for the oxygen to penetrate mixtures with low air voids and to react with asphalt cement. If so, the permeability of the mixture is an important factor in the aging rate as suggested by Goode and Lufesy *(16)* and Kumar and Goetz *(17)*. It is noted that the aging rates at the 94 percent compaction level for the Idylwood Street project and the Plainview Road–Deschutes River project are similar and that the aging rates of the mixtures at the 88 percent compaction level for these projects are significantly different.

One unexpected result for the mixtures at the 94 percent compaction level was that the specimens for the Arnold Ice Caves–Horse Ridge project appeared to soften during the first 3 days. This may have been caused by a slight loss of cohesion of the specimens at the high temperature used in the POB (60°C). However, the aging rate (slope) after the second day increases rapidly and then decreases like that of the other two projects.

Finally, it should be noted that the modulus results from the laboratory-accelerated aging procedure (POB) performed for 5

TABLE 3 LABORATORY MIXTURE AGING TEST DATA

Project	Asphalt Content (%)	Maximum Specific Gravity	Bulk Specific Gravity	Air Voids (%)	Days	Resilient Modulus (ksi)
Idylwood Street					0	74
					1	80
		2.275[a]	5.5[a]		2	81
					3	110
					5	118
	7.0	2.407				
					0	52
					1	53
			2.137[b]	11.2[b]	2	58
					3	59
					5	68
Plainview Road–Deschutes River					0	237
					1	241
			2.292[a]	6.6[a]	2	265
					3	366
					5	373
	5.5	2.455				
					0	149
					1	158
			2.160[b]	12.0[b]	2	214
					3	238
					5	242
Arnold Ice Caves–Horse Ridge					0	105
					1	72
			2.349[a]	4.0[a]	2	73
					3	105
					5	128
	6.5	2.447				
					0	53
					1	69
			2.202[b]	10.0[b]	2	82
					3	83
					5	94

NOTE: Pure oxygen pressure = 100 psi and aging temperature = 60°C.
[a]Samples prepared to approximately 94 percent of maximum density.
[b]Samples prepared to approximately 88 percent of maximum density.

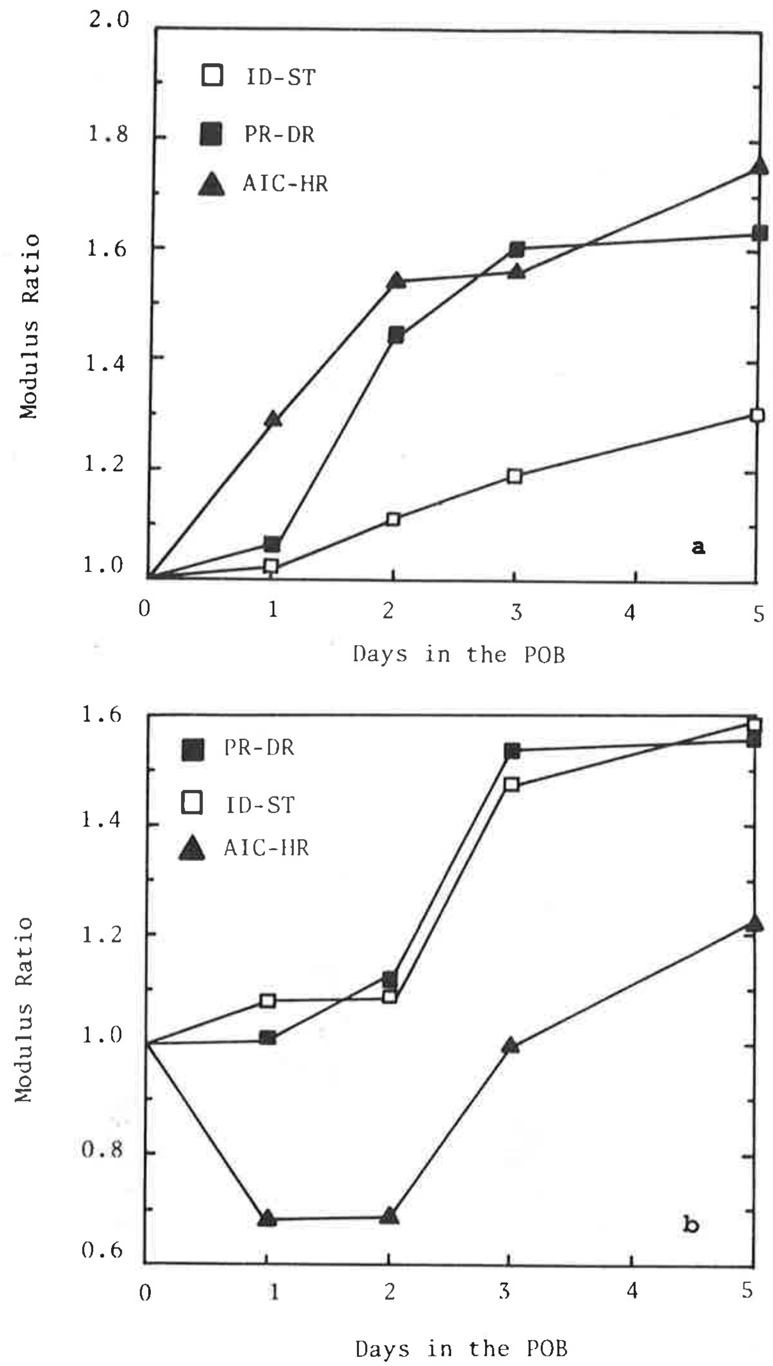


FIGURE 2 Aging modulus ratio: (a) at 88 percent compaction level and (b) at 94 percent compaction level.

days under 100 psi and at 60°C were not comparable with the modulus results determined from the cores of each project. Figure 3 shows the moduli of the laboratory-aged samples for both compaction levels after 5 days and the core modulus values. As can be seen, the core modulus values, except those for the Idylwood Street project, are approximately twice those of the low air void content laboratory samples. These results are not surprising because laboratory-compacted samples have been found to have lower moduli than field cores *(18)*. The large difference in modulus values between cores and laboratory mixtures for the Idylwood Street project results in part from the difference of asphalt content of cores (5.8 percent) and laboratory mixtures (7.0 percent). In general, mixtures with high asphalt content show low moduli.

Fatigue Life

After resilient modulus was measured, the fatigue test was run at a fixed tensile stress that ranged from 30 to 60 psi for cores. For the laboratory mixtures the fatigue test was run applying 20 and 40 psi of tensile stress for 88 percent compaction level mixtures and 94 percent compaction level mixtures, respec-

TABLE 4 PHYSICAL PROPERTIES OF ASPHALT CEMENT

	Idylwood Street			Plainview Road–Deschutes River			Arnold Ice Caves–Horse Ridge		
	O	After RTFOT	R	O	After RTFOT	R	O	After RTFOT	R
Penetration									
At 25°C (77°F)	139	66	10	80	46	22	140	66	63
At 4°C (39.2°F)	50		9	20		14	46		32
Penetration ratio (4°C/25°C)	0.36		0.90	0.25		0.64	0.33		0.51
Absolute viscosity at 60°C (poises)	1169	4306	225 129	1504	3858	13 584	762	2524	5542
Kinematic viscosity at 135°C (cSt)	353	608	3 952	368	494	885	236	393	745
Flash point, closed cup (°C)	199			252			229		
Loss on heating (%)	1.77			0.34			0.20		

NOTE: RTFOT = rolling thin-film oven test, O = original, and R = recovered.

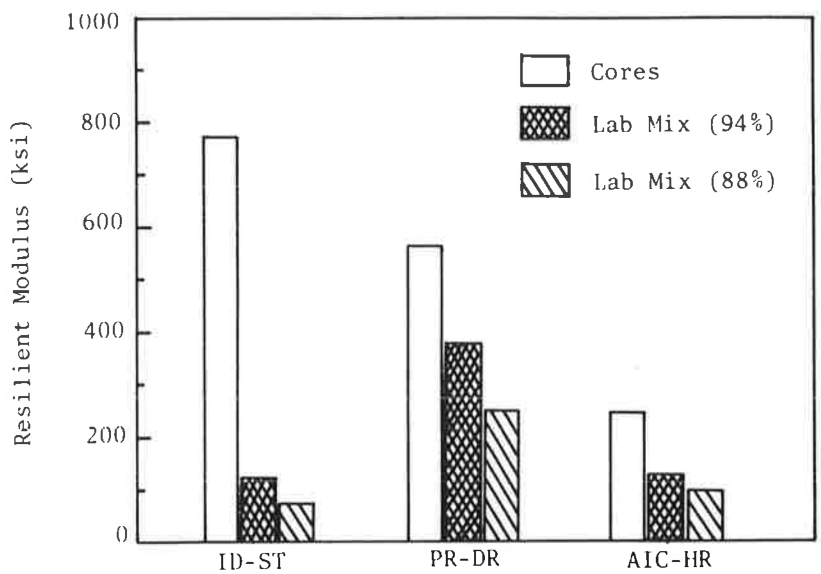
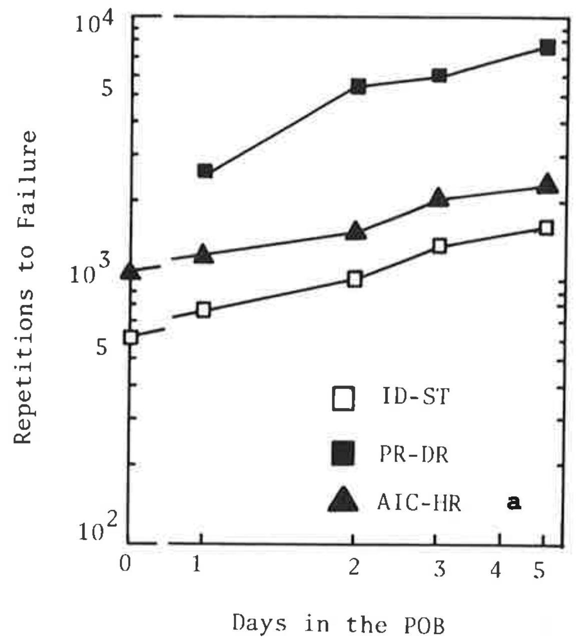


FIGURE 3 Comparison of moduli of cores and aged mixtures.

tively. Only the test results for the laboratory mixtures are discussed in this paper.

Fatigue results represent the effect of the differences in modulus presented in Table 3 and resulted in the wide variety of fatigue performance shown in Figure 4. The fatigue results show that the aged mixtures of the Plainview Road–Deschutes River project obtain the longest fatigue life for both compaction levels because this project has the highest modulus values in Table 3. In general, resistance to fatigue failure (slope in Figure 4) of mixtures decreases after the third day regardless of the difference in compaction level.

The fatigue characteristic of mixtures with different compaction levels changed slightly with aging time. For the 88 percent compaction level, the slopes of each project are relatively constant through time whereas the slopes of the mixtures compacted at the 94 percent level changed in each time interval. These results indicate that the air voids of mixtures affect their resistance to fatigue failure through their service period and that high modulus is an important factor in achieving long fatigue life. The changes of fatigue life of the Idylwood Street and the Arnold Ice Caves–Horse Ridge projects increase slightly during the first 2 days. This result can be explained by the time it takes oxygen to penetrate the mixture as discussed in the section on modulus; the actual air voids of the Idylwood Street (5.5 percent) and the Arnold Ice Caves–Horse Ridge project (4.0 percent) are much lower than that of the Plainview Road–Deschutes River project (6.6 percent). The rate of change of fatigue life of each project at both compaction levels (except the Plainview Road–Deschutes River project at the 88 percent compaction level) decreases after the third day; that is, the resistance to fatigue failure of mixtures decreases with oxidative aging time.



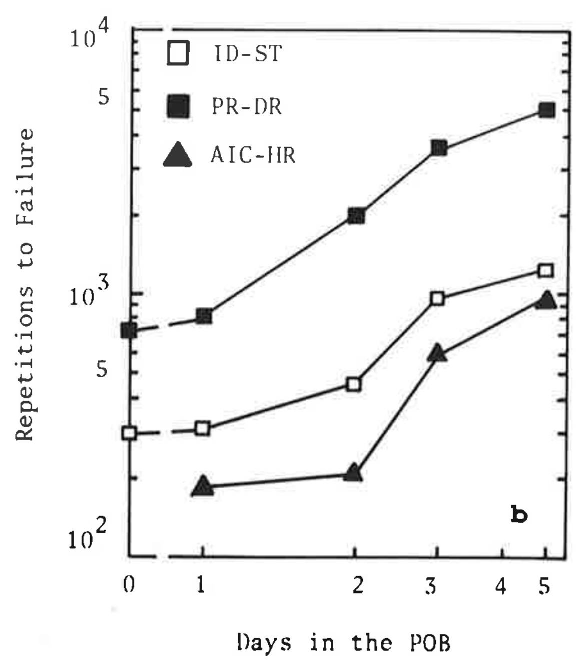


FIGURE 4 Fatigue life of aged specimens: (a) at 88 percent compaction level and (b) at 94 percent compaction level.

Fraass Breaking Temperature

The trends of increased Fraass breaking temperature (i.e., more brittle asphalt) for each project (Figure 5) are similar to those of increased modulus ratio of mixtures at the 88 percent compaction level (Figure 2). For the Idylwood Street project the breaking temperature of the asphalt cement (AR-4000) increases quite slowly as the aging period increases. For the Plainview Road–Deschutes River project, on which the same grade of asphalt cement (AR-4000) was used, the breaking temperature of asphalt cement increases much more rapidly during the first 2 days of aging. The original asphalt cement ($^{120}/_{150}$ pen) used for the Arnold Ice Caves–Horse Ridge project has almost the same breaking temperature as and an aging rate that is similar to that of the Idylwood Street project. Also, these asphalt cements have almost the same physical properties except for flash point and loss on heating (Table 4). Again, it can be seen that even though asphalt cements may have the same grade, they show significant differences in behavior.

The change in Fraass breaking temperature of an asphalt cement indicates a general change in the consistency properties of the asphalt cement that can be estimated from a bituminous test data chart (BTDC) *(19)* as shown Figures 6–8. If the temperature susceptibility of asphalt cement changed little after aging (i.e., if the lines drawn through the original and aged property data were parallel), it might be possible to predict the long-term asphalt properties with the one point of Fraass breaking temperature of asphalt aged in the POB. Hence, the Fraass breaking temperature may be a valuable indicator of the durability of asphalt cement as well as an aide in defining low-temperature consistency.

The rolling thin-film oven test (RTFOT, ASTM D 2872) is used to measure the anticipated hardening of the asphalt during hot-mix plant operations in several western States and therefore will probably not give an indication of hardening due to long-term aging. The consistency data for asphalt from each project after RTFOT (Figures 6–8) are not adequate to predict the long-term asphalt properties due to oxidation. The limited data shown in Figures 6–8 illustrate that asphalt recovered from field cores had higher consistencies than did RTFOT-aged asphalt and that POB-aged (5 days) materials may be similar to field-aged materials with regard to the Fraass point. Clearly more data are required to support this suggestion.

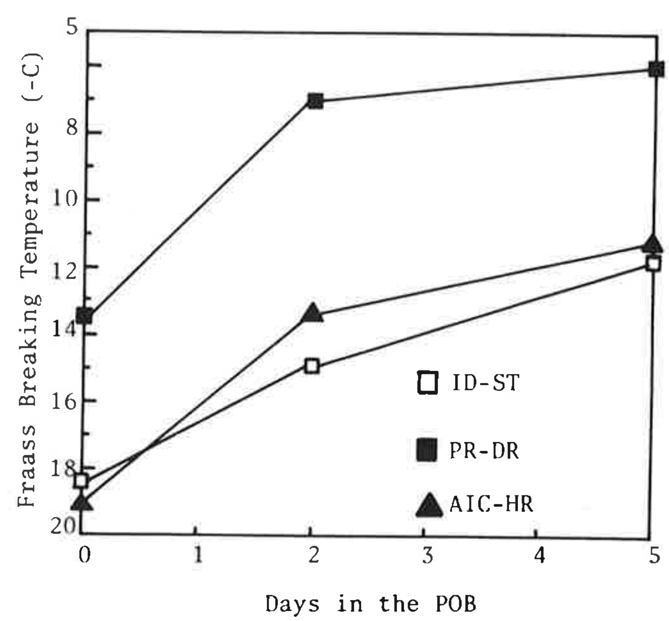


FIGURE 5 Effect of POB aging on Fraass temperature of asphalt cement.

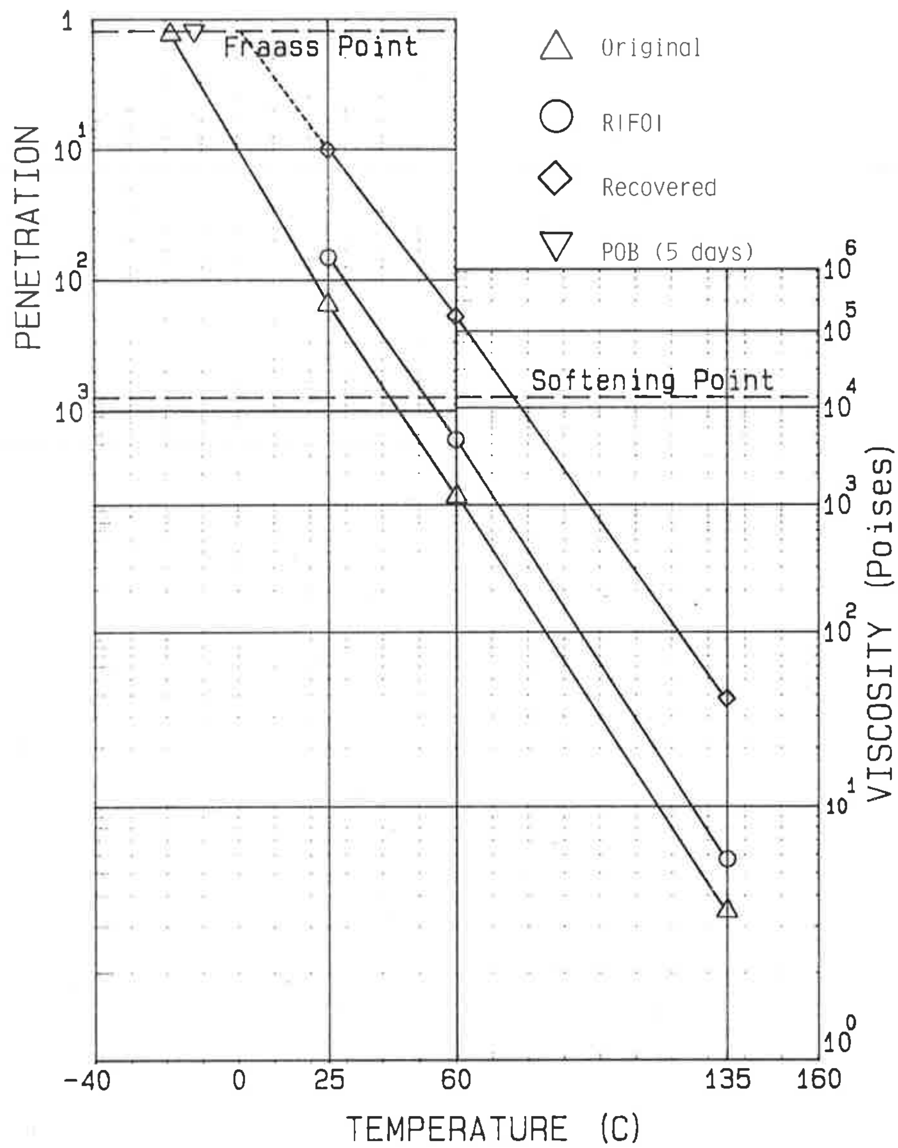


FIGURE 6 Asphalt consistency data for Idylwood Street.

For the Idlywood Street project the Fraass breaking temperature lies between the projected line (dashed line in Figure 6) of RTFOT and recovered asphalt; that is, the asphalt cement aged in the POB for 5 days was aged more than the asphalt from the RTFOT and less than that of cores. However, for the Plainview Road–Deschutes River project the Fraass breaking temperature of asphalt aged in the POB for 5 days lies close to the projected line of recovered asphalt in Figure 7. The result of the Arnold Ice Caves–Horse Ridge project is similar to that of the Plainview Road–Deschutes River project as shown in Figure 8.

Effectiveness of POB

In general, a major cause of asphalt cement hardening is oxidation, a process that occurs most readily at high temperatures and with thin asphalt films. The POB developed in England *(11)* was modified to age both asphalt cements and asphalt mixtures for the aging study at OSU. The modified POB was used with 100 psi at 60°C. Previously, higher pressures and temperatures were used *(11, 12)*, but the lower levels were adopted because of safety considerations and to preserve the shape of the mixture samples.

The modulus ratios obtained from original mixtures and weathered mixtures after 4 years aging at four different weathering sites in California *(8)* are given in Table 5. The modulus ratio ranges from 0.67 to 1.67, excluding the high air voids (7 to 12 percent) mixture using Santa Maria asphalt and nonabsorbent aggregate. Even though the modulus values of aged laboratory mixtures were substantially different from those of cores tested in the study reported herein, it can be seen that the POB causes similar changes in modulus [i.e., results in modulus ratios (Figure 2) similar to those observed in the California study (Table 5)].

The results of asphalt cement chemical composition tests *(20)* done in cooperation with this aging study indicate that the use of the POB to age the asphalt cement on Fraass plaques

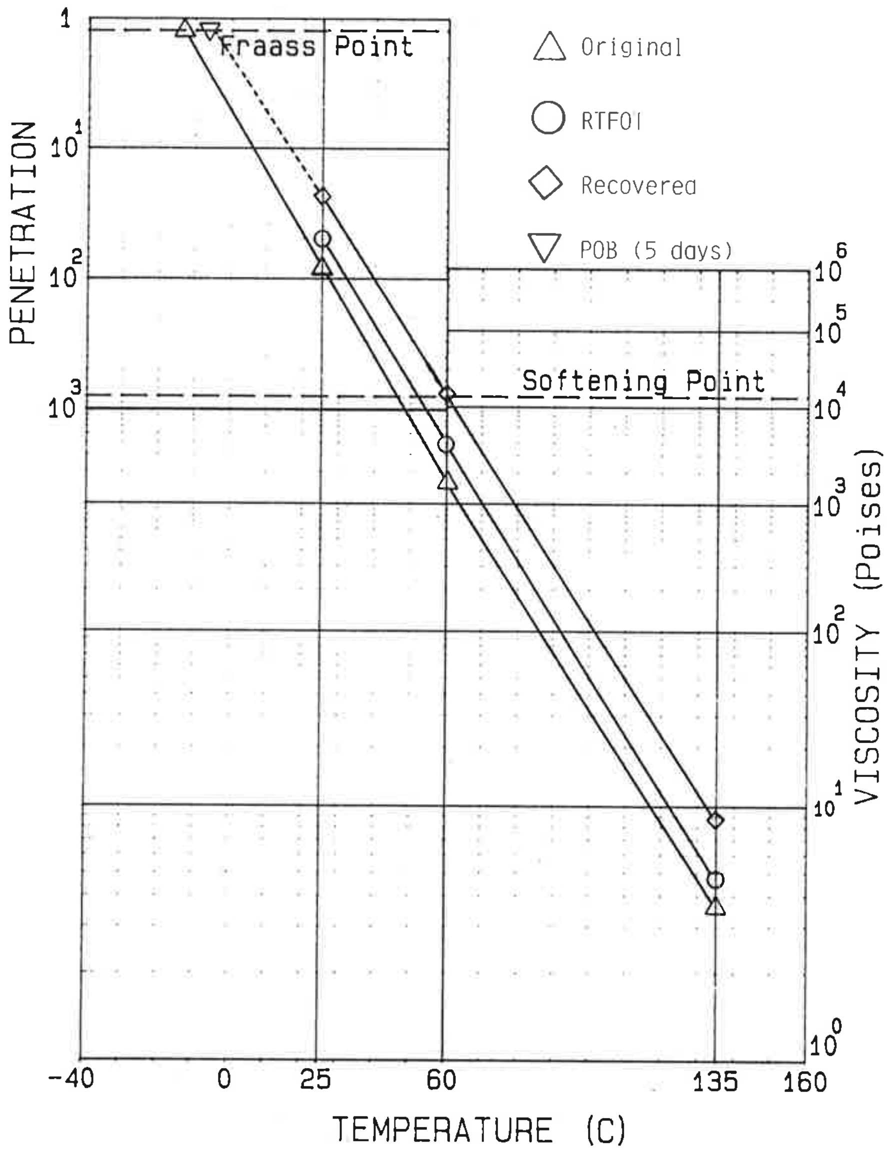


FIGURE 7 Asphalt consistency data for Plainview Road–Deschutes River.

(0.5-mm film thickness) produced a chemical composition similar to that of asphalt extracted from the cores. The results given in Table 6 show that asphalt cement aged in the POB for 5 days can have a composition similar to that of the cores. However, as is the case with the consistency data, there is a significant difference in the POB and recovered properties for the Idylwood Street project.

The component fractions of asphalt cement samples (aged for 5 days) from the Plainview Road–Deschutes River project were similar to those obtained for asphalt extracted from the cores. Also, for the Arnold Ice Caves–Horse Ridge project the fractions of asphaltenes and saturates of the asphalt cement aged for 5 days in the POB are close to those of the cores. The chemical composition of the aged asphalt (for 5 days) of the Idylwood Street project shows that the asphalt in the POB was less aged than that of cores, as discussed in the previous section. The laboratory aging period that is equivalent to field aging may vary with grade and source of asphalt cements as well as mixture properties (particularly air voids and asphalt film thickness) and environmental conditions.

The POB can be used effectively to age asphalt cements in thin films with high pressure or temperature, or both, to give an indication of the long-term asphalt properties with one point of Fraass breaking temperature on the BTDC. However, to effectively evaluate mixture aging, representative mixtures must be tested, and if possible these should be core samples obtained shortly after construction rather than laboratory-compacted mixtures that, as seen in this study and others *(21, 22)*, do not represent field mixtures. There may be large differences between the laboratory mixtures and plant mixtures because of differences in production methods and differences in the compaction method.

A Road Research Laboratory report *(11)* indicates that 1 day's aging of asphalts in the POB at a pressure of 20 atmospheres (300 psi) and at temperatures of 50°C to 60°C is equivalent to half a year on the road in Holland. However, it is

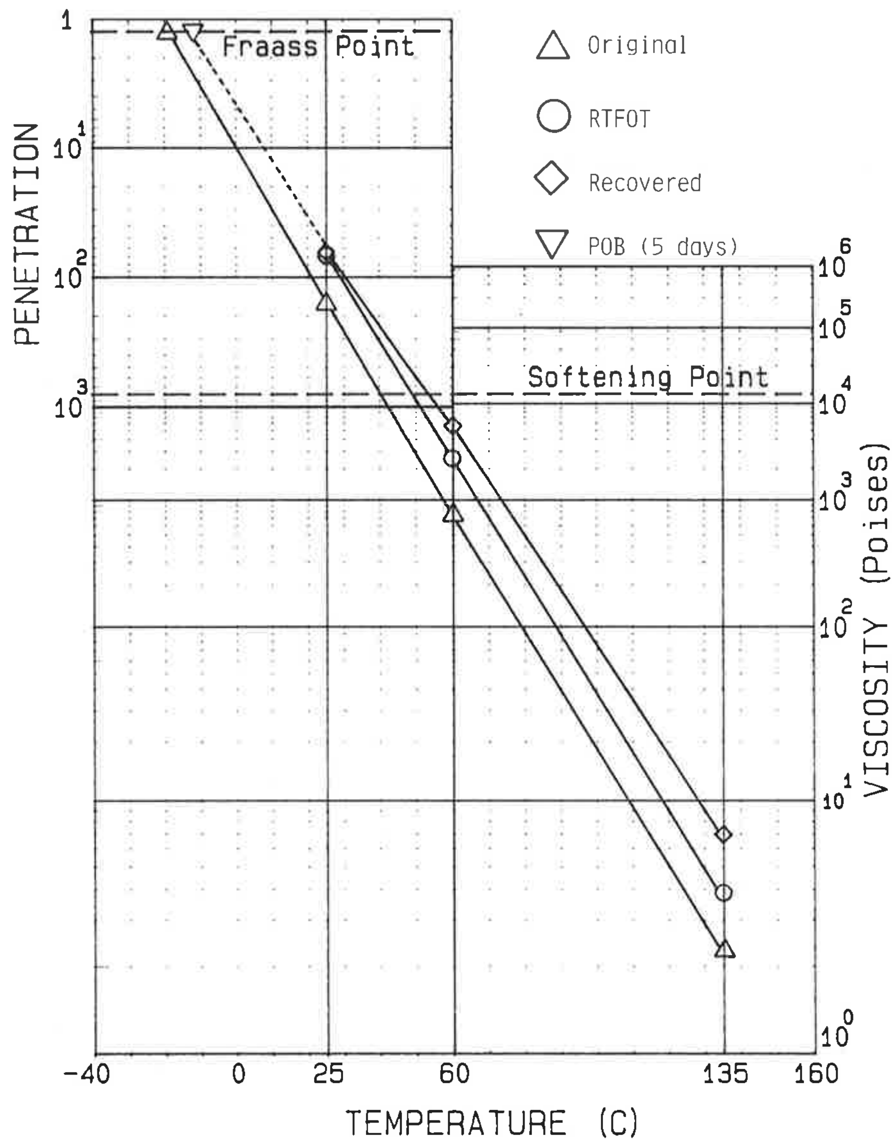


FIGURE 8 Asphalt consistency data for Arnold Ice Caves–Horse Ridge.

extremely important to fit an appropriate pressure relief device to the POB when working at these elevated levels. In this study only 100 psi at 60°C was applied because of safety concerns about the use of pressurized oxygen and in order to preserve the shape of the mixture samples.

SUMMARY

For the limited number of projects studied, the pressure oxidation bomb was found to be an effective device for oxidative aging of both asphalt cements and asphalt mixtures. The test results show that, with the exception of one project and in that case only for modulus, all of the properties changed by at least 50 percent after 5 days of POB aging. Higher pressure or temperature or longer exposure times, or all three, could be applied to accelerate the aging, but the user must follow safe operating procedures.

The modulus ratio and the Fraass breaking point are good indicators of the aging rate of mixtures and asphalt cement, respectively. The aging ratio of mixtures varies with the air voids of mixtures. Mixtures with higher air voids age more rapidly, although the aging rate also depends on asphalt properties.

The limited testing done in this study showed that asphalts aged in the POB for 5 days had composition similar to that of those recovered from field cores that were from 5 to 10 years old. This lends some confidence to the use of the Fraass samples for oxidative aging procedures.

RECOMMENDATIONS FOR FUTURE WORK

Although not evaluated in this study, the POB approach for oxygen conditioning could be combined with a Lottman moisture-conditioning approach to approximate extremely harsh

TABLE 5 MODULUS RATIO OF FIELD-WEATHERED MIXTURES [original and 4-year field weathering *(8)*]

Asphalt Source	Air Voids (%)	Original[a]	Weathering Site[b] A	B	C	D
Nonabsorbent Aggregate						
Valley	3–5	950	0.86	0.76	1.16	0.81
	7–9	880	0.85	0.84	1.05	0.82
	10–12	1,000	0.73	0.76	0.81	0.62
Los Angeles Basin	7–9	740	0.78	0.73	1.11	0.96
	3–5	330	1.21	1.21	1.24	1.67
Santa Maria	7–9	160	2.31	2.13	2.44	3.13
	10–12	150	1.93	2.47	2.13	3.40
Absorbent Aggregate						
Valley	3–5	810	0.79	0.80	1.12	0.86
	7–9	590	1.09	0.83	1.20	1.07
	10–12	730	0.92	0.70	1.11	0.95
Santa Maria	3–5	430	1.28	1.00	1.28	1.61
	10–12	270	1.37	1.44	1.63	2.07

[a]Resilient modulus of original mixture (ksi).
[b]A = Fort Bragg, B = Sacramento, C = South Lake Tahoe, D = Indio.

TABLE 6 CHEMICAL COMPOSITION OF ASPHALT CEMENT

	Idylwood Street				Plainview Road–Deschutes River				Arnold Ice Caves–Horse Ridge			
	O	2 Days[a]	5 Days[a]	R	O	2 Days[a]	5 Days[a]	R	O	2 Days[a]	5 Days[a]	R
Asphaltenes	22.7	27.2	29.0	37.3	16.1	22.4	24.3	25.3	24.9	25.6	27.8	28.0
Saturates	8.4	8.2	7.2	5.6	8.6	7.6	7.6	6.6	10.2	10.3	9.8	10.8
Naphthene aromatic	24.5	23.0	23.4	24.7	26.8	25.8	25.1	25.8	25.5	27.4	26.8	19.7
Polar aromatics	43.3	39.4	37.9	32.4	47.0	42.6	40.8	41.8	38.1	36.0	33.7	39.7
Total	98.9	97.8	97.5	100.0	98.5	98.4	97.7	99.5	98.7	99.3	98.1	98.2

NOTE: O = original and R = recovered.
[a]Aged with POB.

environmental conditions. There is some evidence that suggests that mixtures that suffer oxidative aging are moisture susceptible *(1)*, and therefore subjecting a mix to cycles of oxidation and moisture conditioning and running modulus and fatigue tests is appropriate.

Clearly, the data collected in this study were insufficient to fully evaluate the aging procedures used. More data should be collected to improve confidence in their reliability.

ACKNOWLEDGMENTS

The results from the aging portion of a Highway Planning and Research study, conducted by Oregon State Highway Division and Oregon State University in cooperation with the Federal Highway Administration, have been presented. The contribution of Bill Lien who obtained cores and materials and prepared mix designs was invaluable. The authors are also grateful to Andy Brickman of Oregon State University who helped in the development of testing equipment.

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Evaluation of Indirect Tensile Tests for Assessing Stripping of Alabama Asphalt Concrete Mixtures

FRAZIER PARKER, JR., AND FOUAD A. GHARAYBEH

Stripping in asphalt concrete has long been recognized as a cause of premature pavement damage. Yet, after many years of research and application, there are few generally accepted conclusions about the severity of the problem, the nature and causes of the process, or methods for evaluating stripping. The evolution of test procedures for assessing stripping potential appears to be progressing toward procedures that measure strength loss of moisture-conditioned mixtures compacted to 6 to 8 percent voids. Indirect tensile tests were performed on asphalt concrete mixes composed of materials common in Alabama. The purpose of these tests was to study the stripping process and to evaluate the test procedure for assessing stripping potential. Results indicate that published moisture-conditioning procedures produce variable strength loss but that either of the two procedures evaluated is acceptable. Results also indicate that asphalt cement content, as well as type of mix, has a significant influence on test results and that generalities concerning stripping potential of aggregate sources, types, and blends are not valid. Finally, test results did not distinctly delineate reported stripping and nonstripping aggregate combinations. Reasons may be that the reported performance of an aggregate combination is not valid for all mix types, or that the test is not a positive indicator of stripping potential. The former is more likely, but incorrect predictions for specific mixes are a definite possibility. Implications are that specific mix designs must be tested and results conservatively interpreted until field performance studies permit further refinements.

Stripping in asphalt concrete has long been recognized as a cause of premature pavement damage. In 1938 Hubbard wrote, "It [stripping] has been observed . . . ever since asphalt paving came into existence" *(1*, p. 239). Yet, after many years of research and application, there are few generally accepted conclusions about the severity of the problem, the nature and cause of the process, or methods for evaluating the stripping potential of asphalt-aggregate mixtures. Tunnicliff and Root *(2)*, in the latest of a series of extensive studies of stripping, found no agreement on the severity of the problem, the causes of stripping, or test procedures for predicting stripping.

This lack of consensus is not surprising given the complexity of the process. Stripping was initially considered to be the separation of asphalt coatings from aggregate surfaces *(3)*. This is still considered the dominant failure mode, but some researchers have added that a loss of cohesion in the asphalt cement may also result *(4)*. Compounding the complexity are numerous contributing factors (i.e., coarse and fine aggregate properties, asphalt cement properties, mix design, construction conditions, environment, and traffic).

The evolution of test procedures for assessing stripping potential appears to be progressing toward procedures that measure strength or modulus loss (loss of adhesion or cohesion, or both) of mixtures (particular aggregate and asphalt proportions) that have been compacted and conditioned (construction, environment, and traffic). This appears to be a logical choice because loss of strength is the most serious consequence of stripping. However, tests that attempt to measure other fundamental properties are still used. Some of these listed by Taylor and Khosla *(5)* are static immersion, dynamic immersion, chemical immersion, boiling, abrasion, simulated traffic, swell, and stress pedestal.

Evolution of loading has progressed from compression, as standardized in ASTM Method D 1075, to tensile *(2, 6–11)*. Sample preparation has evolved from standard Marshall or Hveem procedures to procedures that compact aged mix to controlled void contents. Conditioning has progressed from simple soaking to vacuum saturation followed by various combinations of freezing, thawing, and soaking.

The objective of the research reported herein was to evaluate indirect tensile tests for assessing the stripping potential of asphalt-aggregate mixtures made from typical Alabama materials. Five aggregate combinations were tested.

LABORATORY TESTS

Tests on compacted mixtures to evaluate moisture damage have historically compared strength or modulus of moisture-conditioned specimens with strength or modulus of unconditioned specimens. This comparison has been made by computing a strength or modulus ratio by dividing the conditioned strength or modulus by the unconditioned strength or modulus.

There have been numerous differences in proposed test procedures. These variations can cause differences in measured strength and modulus values and, therefore, differences in strength or modulus ratios. Consideration of these differences is necessary when evaluating test procedures and selecting limiting criteria.

Variations have been in the following general areas:

1. Sample preparation,
2. Moisture conditioning,
3. Loading conditions, and
4. Interpretation of test results.

Each will be discussed and details used in this study noted.

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Sample Preparation

Recommended sample preparation generally follows or permits the Marshall (ASTM D 1559) or some form of kneading (ASTM D 1561) or gyratory shear (ASTM D 3387) procedure. There is no evidence to indicate that strength or modulus ratios might be affected by the method of application of compaction energy. Two parameters that are thought to be important in determining moisture effects are mix aging before compaction and voids content after compaction.

Some test procedures *(7, 9, 11)* call for aging the uncompacted mix at 140°F for 15 hr before compaction. This aging allows oxidation and hardening of the asphalt cement that, in theory, simulate construction and in-service conditions. There is some agreement that high-viscosity asphalts may be more effective in resisting stripping *(2, 4, 5, 12)*, but the magnitude or significance is not well established. Aging of uncompacted mix was a variable included in this study. Condition 1 specimens were not aged, and Condition 2 specimens were aged for 15 hr at 140°F.

Additional details of the test conditions will be presented subsequently. Condition 1 was recommended by Tunnicliff and Root *(2)*, and Condition 2 is basically a modified version of the accelerated procedure developed by Lottman *(7)*. It is contained in Hazlett *(11)* as Texas Test Method Tex-531-C.

Air void content will determine the potential intensity of exposure to water. The trend in specifying void content is toward setting specific limits in order to minimize the effect of void content. Lottman *(7, 8)* recommends no specific limits, but implications are that specification voids should be achieved. Three procedures *(2, 9, 11)* recommend 6 to 8 percent voids. The Georgia procedure *(10)* recommends variable voids content depending on mix type, but the range is normally within 5 to 8 percent. All specimens for this study were compacted to void contents of from 6 to 8 percent.

Moisture Conditioning

The following parameters are critical in the moisture-conditioning phase:

1. Initial saturation achieved during vacuum application,
2. Freezing after vacuum saturation, and
3. Soaking.

Vacuum saturation is the first and possibly most important step in moisture conditioning. It determines the extent to which voids are filled with water and, thus, the intensity of exposure of the mix to water. The degree of saturation achieved depends primarily on the magnitude of the vacuum and to a lesser extent on the time of exposure. Three procedures *(7, 8, 10)* recommend a 26-in. vacuum applied for 30 min. This results in variable degrees of saturation depending on specimen properties. Several vacuum saturation schemes are reported in Kennedy and Anagnos *(9)* with a recommendation that degree of saturation be limited to from 60 to 80 percent.

Root and Tunnicliff *(2)* recommend an initial saturation of from 55 to 80 percent and the Texas procedure *(11)* an initial saturation of from 60 to 80 percent. Saturation of specimens for this study was maintained within 60 to 80 percent.

There is no consensus about what additional conditioning may be needed after vacuum saturation to promote development of stripping. Freezing, cyclic freezing-thawing, soaking, and freezing plus soaking have been tried. Data from Lottman *(7)* and Kennedy and Anagnos *(9)* plotted in Figure 1 indicate that freezing after soaking increases strength loss and that, overall, 18 freeze-thaw cycles are somewhat more severe than freezing plus soaking. However, examination of individual mixes reveals that for many the difference between freeze-soak and cyclic freeze-thaw is small. For nine mixes the freeze-plus-soak conditioning produced the greatest strength loss. The validity of using freezing to simulate the stripping mechanisms has been questioned. It is the opinion of some that the strength reduction caused by freezing may be due to damage that is unrelated to the stripping process.

Data in Figure 2 from Kennedy and Anagnos *(9)* indicate that soaking, alone or in conjunction with freezing, may have a dominant effect on strength loss. Soaking, whether alone or in conjunction with freezing, should be an integral part of a conditioning procedure. Soaking provides contact time required for stripping mechanisms to develop. For the study reported herein, Condition 1 involved soaking only at 140°F for 24 hr, and Condition 2 involved one 15-hr freeze cycle to 0°F ± 4°F followed by a 24-hr soak at 140°F.

Loading Conditions

The following loading parameters have been varied in studies of strength or stiffness loss:

1. Type of loading,
2. Rate of application, and
3. Specimen temperature.

Indirect tensile (ASTM D 4123) appears to be the loading currently favored for assessing water damage to asphalt concrete. Unconfined compression (ASTM D 1075), Marshall (ASTM D 1559), and Hveem (ASTM D 1560) loadings have been and are being used *(2)*, but they have not received extensive recent research attention. Double punch loading as proposed by Jimenez *(6)* has received little additional attention. Indirect tensile loading was used in this study.

Rate of loading and specimen temperature are variables that will affect absolute values of tensile strength and modulus but will probably not have a profound effect on strength or modulus ratios. Rates of loading have varied from 0.065 to 2.0 in./min and specimen temperature has varied from 55°F to 77°F. The study by Maupin *(13)* is often cited to illustrate insensitivity to rate of loading and temperature and to justify the use of a 2.0 in./min loading rate and a 77°F specimen temperature. These practical conditions were used for this study.

Interpretation of Test Results

Specimens were loaded to failure and vertical diametral load and horizontal diametral deformation were recorded. Indirect tensile strength was computed with the relationship

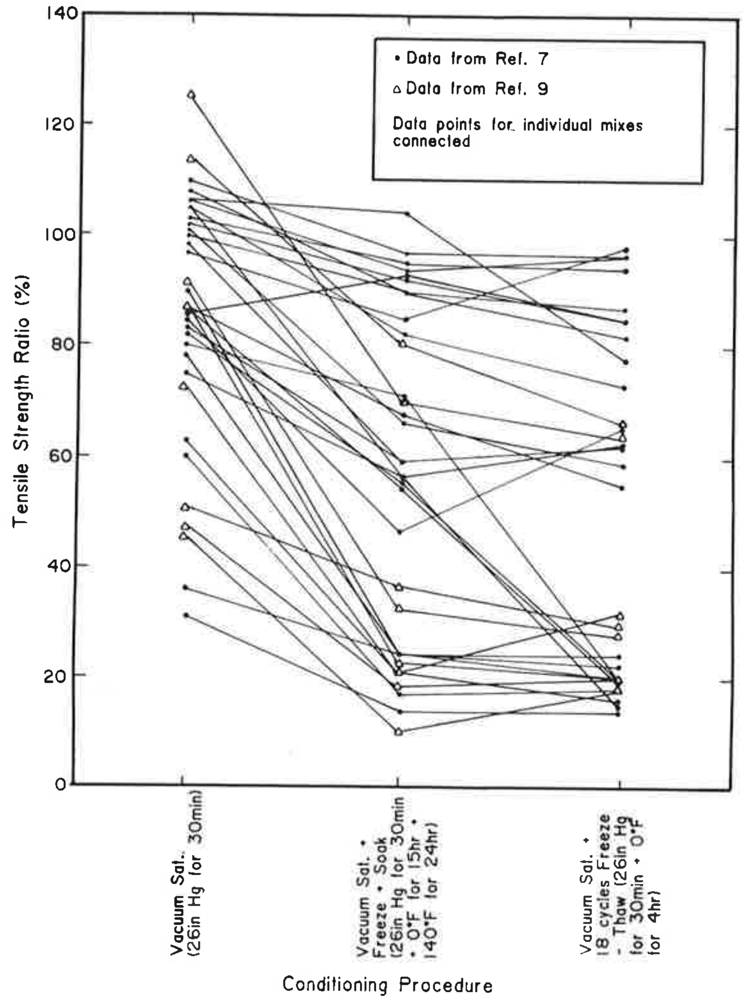

FIGURE 1 Effects of freezing on strength loss.

$$S = 2P_u/\pi td \qquad (1)$$

where

S = indirect tensile strength,
P_u = ultimate vertical diametral load,
t = specimen thickness, and
d = specimen diameter.

Tensile strength ratio (TSR) was computed by dividing the average strength of a minimum of three conditioned specimens by the average strength of a minimum of three unconditioned control specimens. Specimens were grouped for conditioning and control by sorting so that average void contents of the groups were as close as possible. All individual samples, and thus averages for the groups, have compacted void contents of from 6 to 8 percent.

Tensile modulus values were computed with the relationship

$$E = \frac{0.62}{t} \; \frac{P}{\Delta 2} \qquad (2)$$

where

E = tensile modulus,
P = vertical diametral load, and
Δ = horizontal diametral deformation at load P.

Tangent modulus values were computed using the initial slope of the load-deformation curve for P/Δ. Secant modulus values were also computed using the ultimate load and corresponding deformation.

MATERIALS

Five aggregate combinations and one asphalt cement were obtained for testing. These were selected to represent typical materials used for asphalt concrete in Alabama. The aggregate combinations were selected to provide a range of field stripping performance from good to poor. The asphalt cement was from the largest producer in Alabama. Three of the aggregate mixes were used in a study of boil and stress pedestal tests *(14)*.

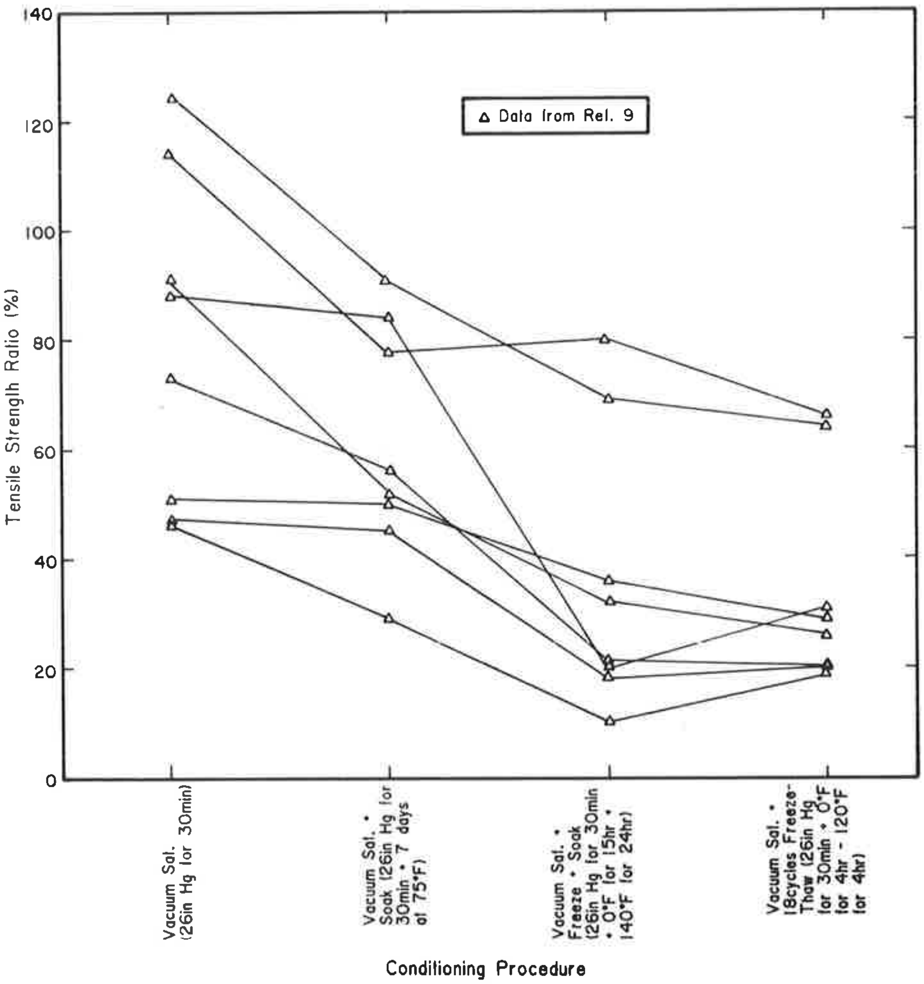

FIGURE 2 Effects of soaking on strength loss.

Asphalt Cement

The asphalt cement used for sample fabrication was grade AC-20, which meets Alabama Highway Department (AHD) specifications, from a source widely used in Alabama. The manufacturers mix crude from various sources, but at the time of sampling the majority of the crude oil was from the Gulf of Mexico.

Aggregate

Five aggregate combinations of three to five individual aggregates each was selected and arbitrarily labeled A–E. Each component of each mix was arbitrarily numbered 1–5 (e.g., the second of the three components of Combination A was assigned A2). The five aggregate combinations initially selected were real surface mixes (A–E were AHD 416 wearing surfaces and D was an AHD 411 wearing surface). Aggregate from the five combinations was also combined to produce mixes meeting base/binder specification (AHD 414 binder layer/AHD 327 bituminous base). Therefore, for each aggregate combination A–E, there will be a surface mix and a base/binder mix. The asphalt cement contents were design asphalt contents for the specific aggregate combinations. Properties of the mixes are given in Table 1.

The nature of the materials in each mix is distinctly different, but each meets specifications for a surface, binder, or base course. Different gradations of limestone for particular combinations were from the same source. Different types of gravel and coarse sands for particular combinations were from the same source, but the fine sands (C and D) were from different sources than were the gravels and coarse sands. The field performance histories of the aggregate combinations, included in the following descriptions, are a result of a limited survey of highway department personnel. They reflect general opinions regarding mixes from the primary aggregate source rather than specific observations of particular mixes.

Combination A

These are basically limestone mixes, and good performance of similar mixes with few signs of pavement distress attributable to stripping has been reported. Surface Mix A contains 85 percent crushed limestone and 15 percent natural sand and has been used for shoulder paving and leveling. Base/binder Mix A contains 100 percent crushed limestone. The limestone is

TABLE 1 MIX CHARACTERISTICS

	Surface Mix (%)	Base/Binder Mix (%)
Aggregate Combination A		
A1—screenings, dolomitic limestone	65	55
A2—natural coarse sand, quartz	15	–
A3—crushed stone, dolomitic limestone	20	45
Asphalt content[a]	5.5	4.25
Aggregate Combination B		
B1—screenings, limestone	10	10
B2—natural coarse sand, quartz, chert	20	20
B3—crushed gravel, chert	70	10
B4—uncrushed gravel, chert	–	60
Asphalt content[a]	7.5	4.5
Aggregate Combination C		
C1—natural fine sand	15	15
C2—natural coarse sand, quartz	25	–
C3—crushed gravel, quartz	60	–
C4—uncrushed gravel, quartz	–	45
C5—pit-run sand, gravel, quartz	–	40
Asphalt content[a]	6.25	4.55
Aggregate Combination D		
D1—natural fine sand	15	10
D2—coarse washed sand, quartz	50	20
D3—uncrushed gravel, quartz, chert	35	70
Asphalt content[a]	6.25	4.9
Aggregate Combination E		
E1—screenings, limestone	65	35
E2—natural coarse sand, quartz	10	10
E3—crushed stone, limestone	25	55
Asphalt content[a]	5.5	4.15

[a]Asphalt content based on weight of asphalt cement and aggregate.

dense (specific gravity ≈ 2.8) dolomitic material with an absorption of about 1 percent.

Combination B

These are basically gravel mixes with variable reported performance. Before the use of antistrip additives was required, stripping damage was severe. With the use of antistrip additives, performance has improved; however, some stripping problems are still reported. Both the surface and the base/binder mixes contain 10 percent limestone screenings and 90 percent siliceous sand and gravel. The gravel and sand are from the same source and are described as "cherty" materials (specific gravity ≈ 2.5) with relatively high absorption (3 percent). The surface mix contains crushed gravel and the base/binder mix contains basically uncrushed gravel (10 percent crushed gravel added to meet gradation requirements).

Combination C

These are siliceous gravel mixes with good reported performance. Even before the use of antistrip additives only minor stripping problems were reported. Both the surface and the base/binder mixes contain 15 percent fine sand and 85 percent coarse sand and gravel from a primary source. The coarse sand and gravel are predominately sound quartz and quartzite materials (specific gravity ≈ 2.6) with relatively low absorption (0.9 percent).

Combination D

These are siliceous gravel mixes with poor reported stripping performance. The use of antistrip additives has improved performance, but gravel mixes from this region of the state continue to be regarded as particularly susceptible to water damage. The mixes contain 10 and 15 percent fine sand and 90 and 85 percent washed sand and gravel from a primary source. The washed sand is primarily sound quartz, but the coarser particles tend to be similar to the gravel. The gravel is a highly variable cherty material (specific gravity ≈ 2.5) that includes light and porous particles. Absorption is relatively high at about 2.6 percent.

Combination E

These are basically limestone mixes with good reported stripping performance. Both the surface and the base/binder mixes contain 10 percent natural sand and 90 percent crushed limestone from a primary source. The limestone has a relatively high calcium carbonate content (≈90 percent), a specific gravity of about 2.6, and absorption of about 1 percent.

PREPARATION AND TESTING OF SAMPLES

Samples were produced following the method described in ASTM D 1559 except for the following modification. Aggregate was combined according to the percentages given in Table 1 and sieved on eight sieves to produce portions with particle sizes in the range that passes the 11/2 in. to the No. 200 sieve. Required aggregate for two samples was then combined to meet the job mix formula gradation. The preheated aggregate and asphalt cement were mixed at 300°F for 3 min. The mixture was then placed in two Marshall molds. Thereafter sample preparation continued according to the requirements given in Table 2.

Condition 1

The mixtures in molds were heated for 2 to 3 hr at the compaction temperature (285°F) for temperature stabilization. Compaction was accomplished using a predetermined number of blows in order to achieve 6 to 8 percent air voids. Specimens were cooled to room temperature (3 to 4 hr), and their bulk specific gravity was determined in accordance with ASTM D 2726 (no wax used). The maximum theoretical specific gravity of the mix was determined in accordance with ASTM D 2041, and air voids were calculated in accordance with ASTM D 3203.

TABLE 2 CONDITIONING PROCEDURES

Treatment	Condition 1	Condition 2
Mix aging	No aging	15 hr at 140°F
Curing of compacted specimens	No curing	24 hr at room temperature
Initial saturation	Between 60% and 80%	Between 60% and 80%
Freezing	No freezing	15 hr at 0°F ± 4°F
Soaking	24 hr at 140°F	24 hr at 140°F
Age of specimen at testing	2 days	4 days
Similar procedure	Tunnicliff and Root *(2)*	Modified Texas *(11)*

Compacted specimens were sorted into two groups (minimum of three specimens) such that both groups had, as nearly as possible, the same average air voids. One group was subjected to accelerated moisture conditioning and the other was used for control. Control specimens were placed in a desiccator and tested at the same time as conditioned specimens.

The conditioning procedure was similar to that described by Tunnicliff and Root *(2)*. It was accomplished in two stages, vacuum saturation and soaking. Specimens were submerged in distilled water at room temperature and a partial vacuum was applied for approximately 5 min. The vacuum level needed for saturation was dependent on the type of mix and ranged from 15 to 26 in. of mercury. In all cases the degree of saturation achieved was between 60 and 80 percent. Soaking was achieved by placing the vacuum-saturated specimens in distilled water at 140°F for 24 hr. The final degree of saturation achieved during soaking was highly variable.

Condition 2

This procedure is similar to that described by Hazlett *(11)*. After it was placed in molds, the mix was cooled to room temperature for a minimum of 3 hr. It was then aged at 140°F for 15 hr, reheated to compaction temperature (285°F) for 2 to 3 hr, and compacted using a predetermined number of blows to produce 6 to 8 percent air voids. The molded specimens were then cured at room temperature for 24 hr. Measurement and computation of specimen properties and sorting and handling of control specimens were the same as for Condition 1.

Specimens were subjected to moisture conditioning in three stages: vacuum saturation, freezing, and soaking. Vacuum saturation was the same as described for Condition 1. Freezing was started immediately after vacuum saturation and lasted for 15 hr at 0.0°F ± 4°F. Each specimen was placed in a plastic bag and this bagged specimen was placed inside another plastic bag. Ten milliliters of distilled water was added to the outer bag to form a moisture barrier and provide sufficient water for saturation. The inside bag was kept open and the outer bag was sealed. The double-bagged specimens were removed from the freezer after 15 hr, taken out of the bags, and placed in a 140°F water bath for 24 hr.

Testing Procedure

After they were soaked, specimens were taken out of the 140°F water bath and placed in another water bath at room tempera-

TABLE 3 INDIRECT TENSILE TESTS RESULTS

Aggregate Combination		Asphalt Content (%)	Moisture Condition 1				Moisture Condition 2			
			Initial Voids (%)	Final Sat. (%)	TSR* (%)	SMR* (%)	Initial Voids (%)	Final Sat. (%)	TSR* (%)	SMR* (%)
A	Surf.	5.5	6.5	89	87	-	6.2	90	81	58
	B/B	4.25	7.4	94	27	11	7.7	89	24	10
B	Surf.	7.5	7.0	100+	80	-	6.2	100+	81	53
	B/B	4.5	6.6	100+	59	30	6.4	100+	82	60
C	Surf.	6.25	7.3	80	109	-	6.9	82	88	72
	B/B	4.55	6.9	89	78	45	6.7	85	78	55
D	Surf.	6.25	7.4	88	107	76	7.5	93	98	71
	B/B	4.9	6.6	97	83	47	6.6	98	79	44
E	Surf.	5.5	6.4	82	85	59	7.4	96	70	56
	B/B	4.15	7.0	88	92	90	6.8	88	75	47

* Strength and modular ratios were computed for averages of minimum of 3 dry and 3 conditioned specimens.

ture (77°F) for 2 to 3 hr. Each specimen's final bulk specific gravity and thickness were then determined as described previously.

Immediately after thickness measurement, specimens were loaded in indirect tensile using a Marshall testing machine. Vertical diametral load was applied through ½-in.-wide loading strips by controlling vertical deformation at 2 in./min. Horizontal diametral deformation was measured with a device similar to that described by Hudson and Kennedy *(15)*. Vertical diametral load and horizontal diametral deformation were recorded with an X-Y plotter to obtain relationships for tensile strength and modulus calculations.

Calculations

Maximum theoretical mix specific gravity was calculated in accordance with ASTM D 2041. Bulk specific gravity and air voids before and after conditioning were calculated in accordance with ASTM D 2726 and ASTM D 3203, respectively. Degree of saturation, before and after conditioning, was calculated using the relationship

$$S = \frac{B - A}{V\,(B - C)} \times 100 \qquad (3)$$

where

S = degree of saturation,
A = dry weight of specimen in air,
B = weight of surface-dry specimen after saturation,
C = weight of saturated specimen in water, and
V = voids ratio of specimen.

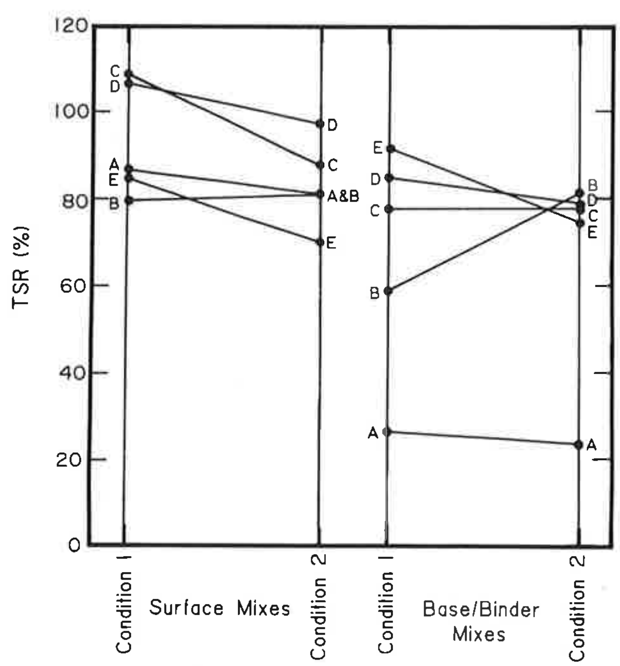

FIGURE 3 Comparison of sample preparation and moisture conditioning.

Tensile strength and secant modulus values were calculated using Equations 1 and 2, respectively. Tensile strength ratio (TSR) and secant modulus ratio (SMR), expressed as percentages, were calculated with the relationships

$$\text{TSR} = \frac{\text{Average conditioned indirect tensile strength}}{\text{Average dry indirect tensile strength}} \times 100 \qquad (4)$$

and

$$\text{SMR} = \frac{\text{Average conditioned secant modulus}}{\text{Average dry secant modulus}} \times 100 \qquad (5)$$

TEST RESULTS AND ANALYSIS

A summary of test results is given in Table 3. TSRs and SMRs are tabulated. Ratios of initial tangent modulus were also computed but are not presented because of their high variability as a result of imprecision in the initial portion of the load-deformation curves.

Effects of Sample Preparation and Conditioning Procedures

Numerous variations in sample preparation and conditioning procedures have been studied to try to simulate conditions necessary for stripping. These variations can result in significant differences in test results, as shown in Figures 1 and 2.

For certain applications, such as determining the effectiveness of antistripping agents, absolute values of strength or modulus ratios may not be critical (assuming reasonable approximations of stripping conditions are achieved). However, absolute values are important if the results are to be used for predicting stripping and the need for antistripping additives. They are important because they must be correlated with field stripping performance in order to establish limiting criteria.

Figure 3 shows the differences in TSRs observed for the two sample preparation and moisture conditions. The average difference is about 8 percent, and Condition 2 generally gives lower values. The exception is Combination B for which Condition 2 gives a larger value for the base/binder mix. Condition 2 is more severe primarily because of the freezing, but the effects are somewhat mitigated by the beneficial effects of curing and aging. Differences in SMR-values were similar.

Effects of Sample Properties

Aggregate properties are considered the dominant factor in determining mix stripping. When considering prediction of stripping potential and the associated need for antistripping additives, aggregate constituents are most often considered the controlling influence. However, specimen properties, such as void content and degree of saturation, will also affect strength retention and thereby potentially confound estimates of stripping potential. The effects of these variables are minimized by compacting specimens to standard voids (6 to 8 percent) and moisture conditioning using consistent procedures.

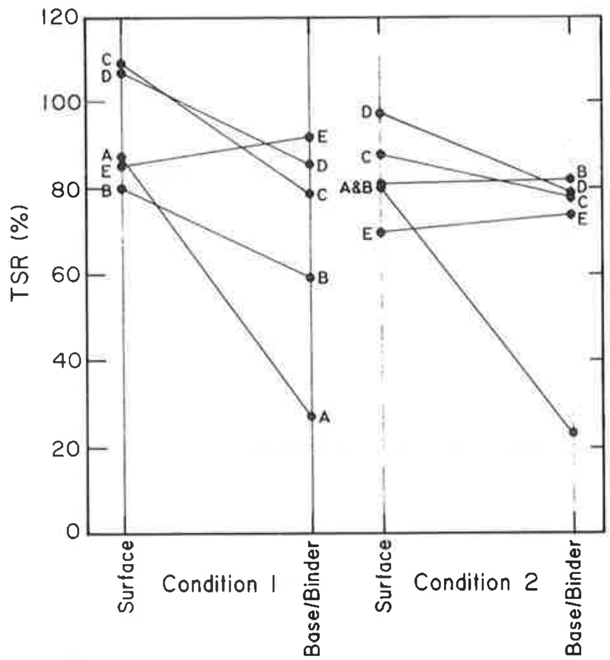

FIGURE 4 Comparison of surface and base/binder mixes.

Additional sample properties that may influence the characterization of a mix as stripping or nonstripping are asphalt cement content and aggregate gradation. They will determine asphalt cement film thickness and void size, which have the potential for influencing strength and modulus retention. Smaller film thickness will make removal from aggregate easier. Larger voids will allow easier access for water. Although void content is controlled (6 to 8 percent), coarser gradations will likely produce fewer but larger voids for a given void content.

The effect of void size on permeability was apparent during the vacuum saturation process. A trial-and-error procedure was used to achieve 60 to 80 percent saturation. Less intense partial vacuums and smaller exposure times were required to achieve the required degree of saturation for base/binder specimens than for surface specimens with the same void content.

Film thickness and void size are elusive parameters that defy accurate quantification and thus precise study. They were studied indirectly by examining surface and base/binder mixes of the five aggregate combinations. Figure 4 shows the effect of mix type. In general, base/binder mixes give lower strength retention. This would be the expected response if film thickness were smaller, as indicated by the lower asphalt content. However, base/binder mixes have coarser gradations and should require less asphalt cement for particle coating. Nevertheless, base/binder mixes do appear "leaner." The general trend is also consistent with the postulated influence of void size (i.e., coarser base/binder mixes have larger voids that provide easier access for water).

Notable exceptions to the general trend in Figure 4 are the responses of Combinations A and E. These are limestone aggregate combinations. Combination A base/binder mix has dramatically smaller (60 and 58 percent) TSR-values whereas Combination E base/binder mix has slightly larger (7 and 5 percent) TSR-values. No quantitative reasons can be formulated to explain this divergent behavior, but qualitatively the response is consistent with visual mix richness.

The influence of film thickness was examined by computing particle surface area with procedures suggested by Hudson and Kennedy *(16)*. These values, as well as the ratios of asphalt content to surface area, are given in Table 4. No tendencies were noted when these ratios were plotted versus tensile strength ratios. This is attributed to the influence of other factors such as aggregate properties, and further studies of individual mixes were conducted.

Tests were run on mixes with asphalt cement contents dif-

TABLE 4 TENSILE STRENGTH AND ASPHALT FILM THICKNESS DATA

Aggregate Combination		Asphalt Content (%)	Surface Area (m^2/kg)*	Ratio AC/SA	TSR (%)	
					Condition 1	Condition 2
A	Surf.	5.5	5.57	0.99	87	81
	B/B	4.25	4.42	0.96	27	24
B	Surf.	7.5	5.41	1.39	80	81
	B/B	4.5	4.79	0.94	59	82
C	Surf.	6.25	5.78	1.08	109	88
	B/B	4.55	4.38	1.04	78	78
D	Surf.	6.25	6.63	0.94	107	98
	B/B	4.9	4.27	1.15	83	79
E	Surf.	5.5	7.14	0.77	85	70
	B/B	4.15	4.98	0.83	92	75

*Computed with procedures from reference 16.

ferent from design asphalt contents. Base/binder Mixes B, D, and E at design asphalt contents appeared "rich," and additional tests were run at lower asphalt contents. Base/binder Mix A appeared lean at 4.25 percent asphalt content, and tests were run with 5.25 percent asphalt content. Surface Mix A at 5.5 percent design asphalt content appeared rich, and tests were run at 4.5 percent asphalt content.

Ratios of asphalt content to aggregate surface area were computed and plotted versus tensile strength ratio (Figure 5). The general trend exhibited is as expected: higher asphalt content–to–surface ratios give higher TSRs. Implications are that film thickness has a definite effect on tensile strength retention. The data cannot be used to infer that asphalt content can be used to control stripping because other mix requirements (stability, flow, voids, cost, etc.) will limit the practical range for asphalt content.

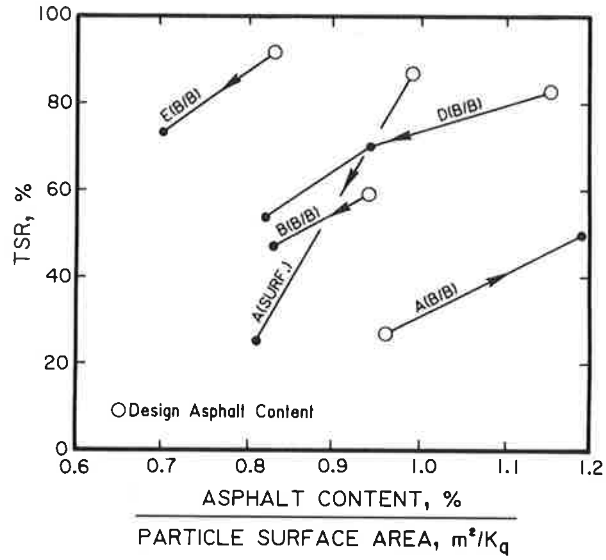

FIGURE 5 Effects of asphalt film thickness on TSR.

Prediction of Stripping

Current thinking appears to be that aggregate constituent properties are the dominate factor determining the stripping potential of asphalt concrete. However, as noted in the previous sections, asphalt content, aging, voids, and saturation can influence stripping. Some of these factors can be controlled by standardized test procedures. Others are mix specific and emphasize the necessity for testing specific mixes. Therefore, assuming that test conditions reasonably approximate field conditions and that the designation of a mix includes a design asphalt cement content, the indirect tensile test can be evaluated only as a predictor of stripping potential of specific mixes. The following analysis is somewhat weakened by the necessity of relying on the general characterization of an aggregate combination as stripping or nonstripping. The analysis is made with the knowledge that the characterization of the aggregate combination may be invalid for specific mixes.

TSR- and SMR-values from Table 3 are plotted in Figures 6 and 7, respectively. Neither TSR nor SMR differentiates between the two reported stripping combinations (B and D) and the two reported nonstripping combinations (A and E). Results for Combination C, which has variable but generally good reported stripping performance, are also similar.

The dashed horizontal line at TSR = 70 in Figure 6 represents a criterion that has been suggested *(9, 11)* for separating stripping from nonstripping mixes. Only 3 of 20 data points fall below this criterion, and stripping and nonstripping mixes are basically indistinguishable. A limiting TSR = 80 has also been

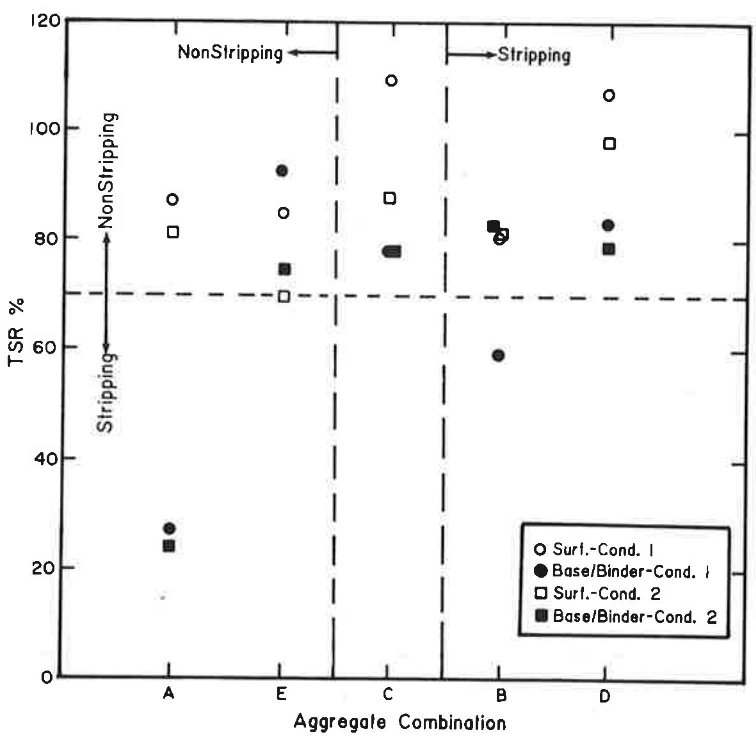

FIGURE 6 Prediction of stripping with TSRs.

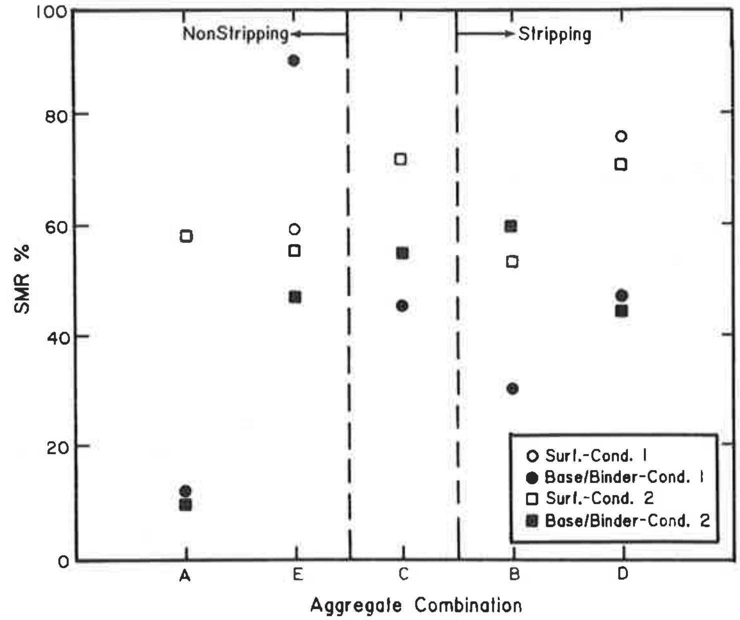

FIGURE 7 Prediction of stripping with SMRs.

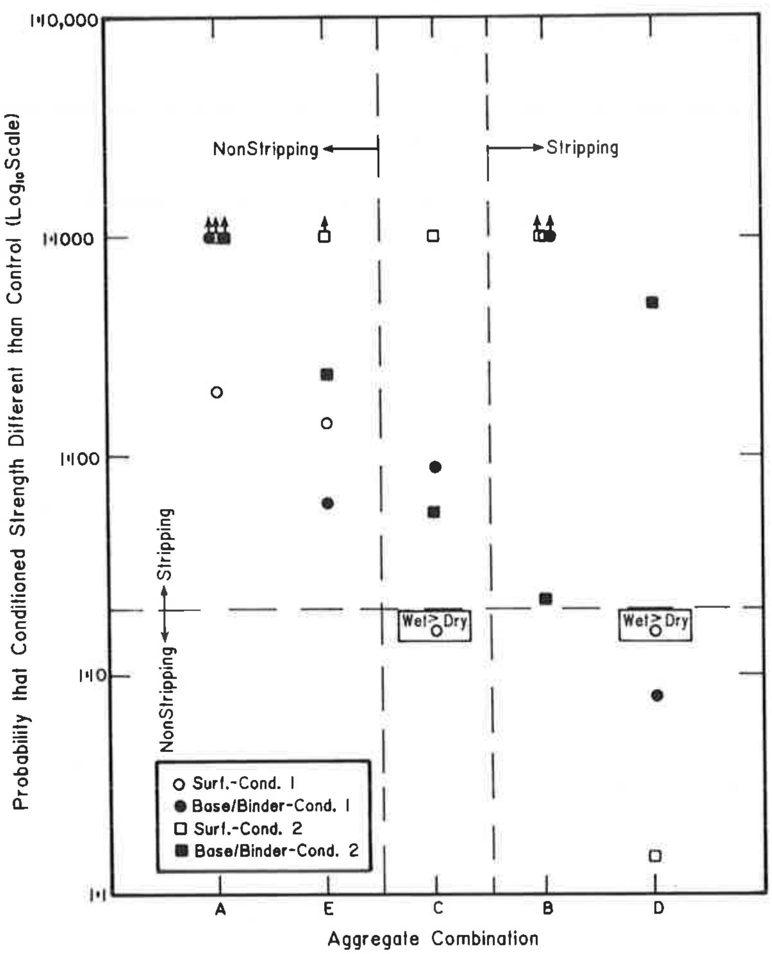

FIGURE 8 Prediction of stripping with probabilistic mean strength comparison.

TABLE 5 TSR-VALUES AND PROBABILISTIC MEAN STRENGTH COMPARISON

Aggregate Combination		Moisture Condition 1		Moisture Condition 2	
		TSR (%)	Prob. Real * Strength Diff.	TSR (%)	Prob. Real * Strength Diff.
A	Surf.	87	1:200	81	> 1:1000
	B/B	27	> 1:1000	24	> 1:1000
B	Surf.	80	1:1000	81	> 1:1000
	B/B	59	> 1:1000	82	1:25
C	Surf.	109	**	88	1:1000
	B/B	78	1:91	78	1:56
D	Surf.	107	**	98	1:2
	B/B	83	1:8	79	1:500
E	Surf.	85	1:143	70	> 1:1000
	B/B	92	1:61	75	1:235

* Two tail "t" test to compare mean tensile strength.

** Conditioned strength greater than control strength.

suggested *(8, 10)*. Examination of Figure 6 reveals that four of eight data points for nonstripping mixes and two of eight data points for stripping mixes fall below this criterion. For Combination C the base/binder mixes fall below the TSR = 80 criterion, and the surface mixes are above.

Comparison of Figures 6 and 7 reveals that patterns for SMR are similar to patterns for TSR. SMR-values do not appear to differentiate between stripping and nonstripping mixes.

Tunnicliff and Root *(2)* have suggested a criterion for predicting stripping that is based on probabilistic comparison of the mean tensile strength of conditioned and control specimens. Their criterion is "a probability of 20:1 or more that the difference between wet and dry specimens is real was used to indicate that an additive should be considered" (2, p. 18). The strength data were analyzed using the suggested criterion. The results are tabulated in Table 5 and plotted in Figure 8.

Comparison of TSRs with the probability that the mean strengths are different (Table 5) indicates general correlation but with considerable variability. Figure 8, with the suggested criterion shown as a horizontal line, indicates that 16 of 20 points fall above the line indicating potential strippers. This is contradictory to the predictions in Figure 6 where only 8 of 20 points fell in the stripping category as defined by the TSR = 80 criterion. Application of the probabilistic criterion generally indicates higher stripping potential, and application of the limiting TSR criterion generally indicates lower stripping potential.

For nonstripping mixes (A and E), the probabilistic criterion predicts that all would strip whereas the TSR = 80 criterion predicts that four of eight would strip. For stripping mixes (B and D), the probabilistic criterion predicts that five of eight would strip whereas the TSR = 80 criterion predicts that three of eight would strip.

CONCLUSIONS

Conclusions drawn from results of the testing program follow.

1. Condition 2 (vacuum saturation, freezing, and soaking) produced lower TSR-values than did Condition 1 (vacuum saturation and soaking), but either appears acceptable in light of uncertainties in modeling the field environment.
2. Base/binder mixes are somewhat more susceptible to stripping than are surface mixes of the same constituents. This may reflect the influence of asphalt cement film thickness (susceptibility to stripping increases with decreased film thickness). Differences in the size and distribution of voids resulting from differences in gradation (coarser gradation producing larger voids) may also be a factor. Larger voids would provide easier access for water.
3. The probabilistic criterion for separating stripping and nonstripping mixes as suggested by Tunnicliff and Root *(2)* is more severe than a deterministic limiting tensile strength ratio of 70 *(9, 11)* or 80 *(8, 10)*.
4. The indirect tensile test did not distinctly differentiate reported stripping and nonstripping aggregate combinations. Reasons may be that the reported stripping performance of an aggregate combination is not valid for all mix types or that the test is not a valid indicator of stripping performance. The

former is more likely, but incorrect prediction for specific mixes is a definite possibility. Implications are that specific mix designs must be tested and results conservatively interpreted until field performance studies permit further refinements.

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Significance of Various Factors in the Recycling of Asphalt Pavements on Secondary Roads

HUMBERTO CASTEDO

In this paper are presented the results of field and laboratory investigations undertaken to determine the role that material variability, mix design factors, and other parameters play in the performance of recycled asphalt pavements on secondary roads. After the pertinent literature on the economics and procedures associated with various recycling methods was reviewed, a sampling program was developed to collect cores from existing pavements. This coring program was undertaken to evaluate the variability of the materials that form the asphalt pavements of county roads and city streets in Indiana. Statistical analyses of the data obtained on these cores showed that there is practical significance in the variation of parameters, such as asphalt content, aggregate gradation, and asphalt penetration, within a section of road, between county and city asphalt pavements, and among geographic regions in the state. Characterization of these materials allowed the author to simulate in the laboratory actual conditions such as hardness of the binder, gradations of aggregates and reclaimed asphalt pavement material, asphalt contents, and other properties of the recycled mix within the range of values measured from pavement core samples. The effects of these parameters were measured by means of the Marshall and resilient modulus tests. It was found that a stable and sound pavement can generally be obtained using cold-mix recycling techniques if normal reclaiming procedures are followed and regular asphalt emulsion binders are added.

Construction, rehabilitation, and maintenance of asphalt pavements in Indiana have been done almost exclusively with virgin selected materials *(1)*. Recycling of asphalt pavements is not widely accepted as a corrective measure for structural and surface distress of asphalt pavements on primary and secondary roads.

One of the main problems faced when planning the rehabilitation of a secondary road (e.g., farm-to-market roads and residential city streets) is the lack of information on the materials that form the pavement to be recycled.

The FHWA has sponsored a number of recycling operations through Demonstration Project 39, which has the objective of advancing this technology and sharing the experience and results obtained. The economics and feasibility of this rehabilitation technique are extensively documented for demonstration projects throughout the country *(2)*. There are still, however, many questions that remain unanswered such as (a) how much variability exists in the pavement materials of secondary roads targeted for rehabilitation? and (b) what is the practical effect of this variability on the performance of a recycled pavement? These and other questions about the recycling of asphalt pavements on secondary roads in Indiana were addressed in this study.

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GENERAL CONSIDERATIONS

Recycling Process

Pavement recycling can be categorized according to the construction procedure used, the types of materials to be recycled, and the structural benefits to be gained. The main forms of asphalt pavement recycling are (a) surface recycling, (b) in-place surface and base recycling, and (c) central plant recycling. Most of the operations involved in each of these recycling techniques can be performed either cold (with materials and equipment at ambient temperatures) or hot (central plant recycling) *(3)*.

Selection of a cold- or a hot-mix recycling technique is generally based on available funds, time to completion, and other specialized efforts that are involved in either process. Hot-mix central plant recycling allows better control of materials and therefore a good-quality mix is produced. However, the costs and quality of hot-mix recycling are justified in most cases only for high-volume, first-class pavements. The normal procedure for recycling asphalt pavements on secondary roads has been the cold-mix process. The mix produced is generally used as base course material with some form of surface treatment or hot-mix asphaltic concrete overlay *(1, 4)*. For this reason, this study was concentrated on the process and materials involved in cold-mix recycling of asphalt pavements.

Factors Involved in Cold-Mix Recycling of Asphalt Pavements

There are numerous factors involved in any recycling operation. Two important economics- and materials-related factors are (a) the economics of recycling, including such items as hauling distances, energy usage, availability of virgin materials, traffic control and disruption, construction time, rural or urban environment, project size, contractor availability *(2)*, and (b) the variability in paving materials, which is related to such factors as weather effects, traffic wear, and the nature of the paving materials used (i.e., different binders and aggregate contents and compositions).

These factors, to varying degrees, influence the selection of recycling or conventional procedures for restoring a distressed pavement. The economic considerations involved in recycling secondary road asphalt pavements in Indiana are analyzed next.

ECONOMIC CONSIDERATIONS

It is widely known that one of the major criteria that determine whether or not a new process will be accepted is economics. If it can be demonstrated that recycling of asphalt pavements has economic advantages over new material construction, recycling could be accepted as a viable pavement rehabilitation alternative.

Construction costs and other economic data from asphalt pavement projects in Indiana and throughout the country were obtained for 1985 and previous construction periods in order to document the main economic differences that exist between conventional asphalt pavement rehabilitation methods and the recycling process. Current data were obtained through telephone and personal conversations with highway authorities, from the latest reports found in the literature, and by cost indexing prices before 1985. The cost index method used is the one recommended by the Engineering News Record (ENR) and described in the Texas Transportation Institute guidelines *(5)*.

Cost Components

A review of most of the reports prepared for FHWA Demonstration Project 39, as well as other information available, revealed that, on average, material cost represents 46.6 percent of the total cost of a cold in-place recycling project and that equipment and labor represent 29.7 and 23.7 percent of the project's cost, respectively *(1–4)* (Figure 1). Records from several hot-mix asphalt overlay operations in Indiana showed that materials average 90.5 percent of the total cost and that equipment accounts for 4.8 percent and labor for 4.7 percent of the cost of overlaying a pavement. Chip-and-seal or surface treatments in Indiana were found to have an average cost breakdown of 9.5 percent for labor and 90.5 percent for equipment and materials *(1)* (Figure 1).

The main economic advantage that recycling appears to have over conventional paving procedures is to be found in the raw materials required for the project because the salvaged material removed from the roadway has an intrinsic value attributable to the asphalt and aggregate components of the old pavement. However, a closer analysis of the cost data revealed that, in almost all recycling projects, the material cost was attributable to the new asphalt or recycling agent used to restore the properties of the aged binder.

Another important cost component of this type of mixture is the virgin aggregate used to upgrade the gradation of the existing aggregate to a standard particle distribution. The effects of varying amounts of virgin aggregate on the final cost per ton for asphalt mixtures, at 1985 prices in Indiana, are shown in Figure 2. As can be seen, virgin aggregate alone can, in some cases, almost triple the final cost of the mix if asphalt mixtures that contain 100 percent cold-mix recycled and 100 percent virgin materials are compared. This is because the production and hauling cost of virgin materials can be quite high in some parts of the state.

Cost Comparison and External Factors

Construction and rehabilitation of asphalt pavements in general (including cold-mix recycling) are subject to changes in price of the main material components of the asphalt mix. Studies by Schnormeier *(6)* and others *(1, 4, 5)* showed that asphalt cement price variations have the largest influence on the variation of the price of asphalt mixtures in general. Consultations with several highway agencies and local contractors throughout Indiana resulted in a list of average prices paid in 1985 (Figure 3).

From these data it can be seen that prices for new base and binder materials for Interstate projects in Indiana varied substantially more than did prices for the same materials in urban

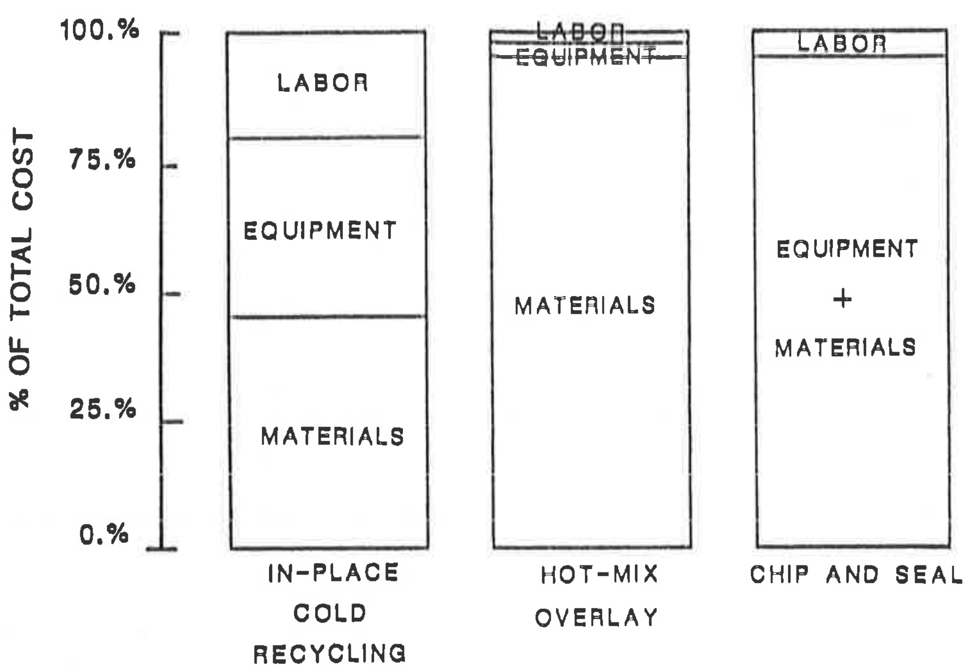

FIGURE 1 Cost breakdown for various rehabilitation alternatives.

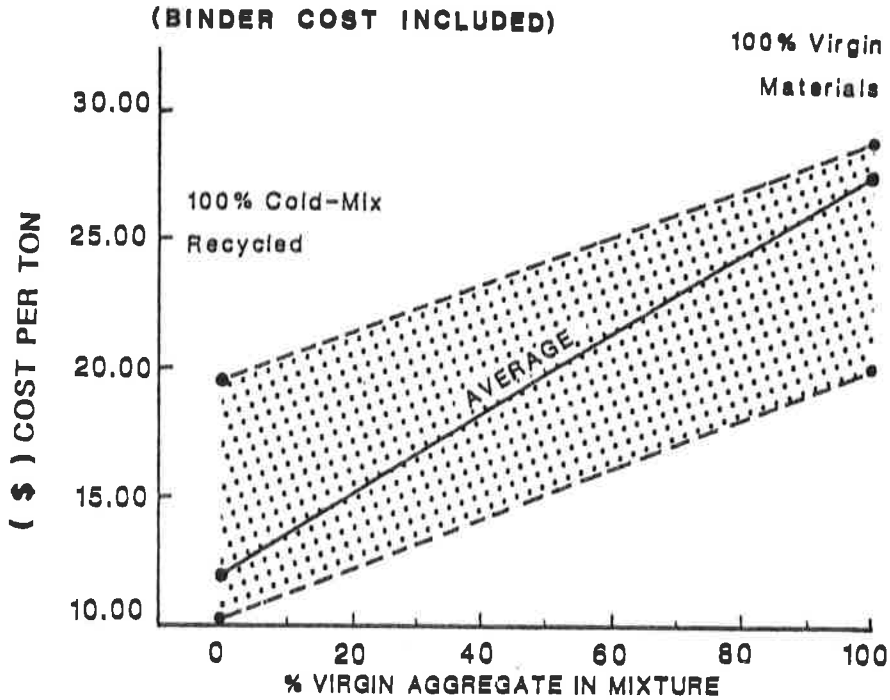

FIGURE 2 Effect of varying amounts of virgin aggregate on the cost of asphalt mixtures.

areas. This is because the cost per ton of these materials depends on, among other things, conditions such as location of projects and quantities of material required for particular projects.

When the prices paid in 1985 for hot-mix recycling (used only for high-volume pavements) are compared with those paid for virgin mix, it can be seen that hot-mix recycling was almost as expensive as new materials construction. This has created a situation in which hot-mix recycling procedures have been abandoned in favor of conventional, hot-mix asphaltic concrete operations that use virgin materials, and the various benefits associated with recycling (e.g., little or no use of virgin materials, retention of original grade) are negated thereby.

The remaining data shown in Figure 3 are the prices for cold-

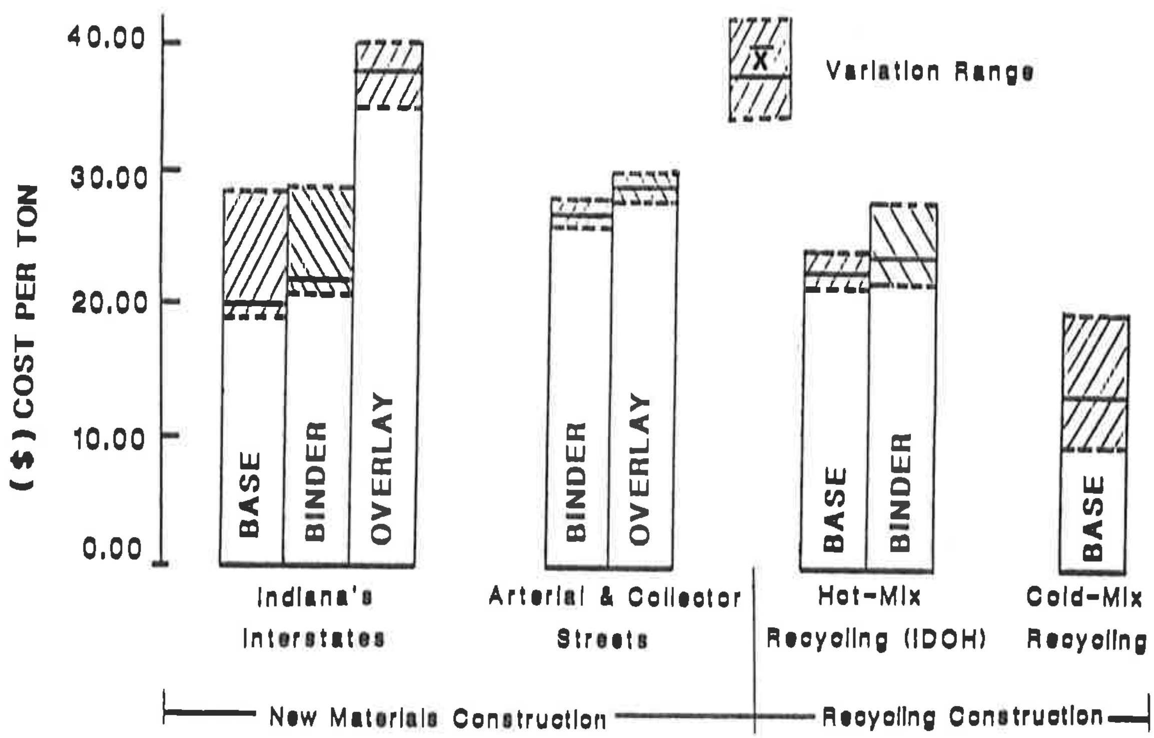

FIGURE 3 Cost of various new and used pavement materials in Indiana.

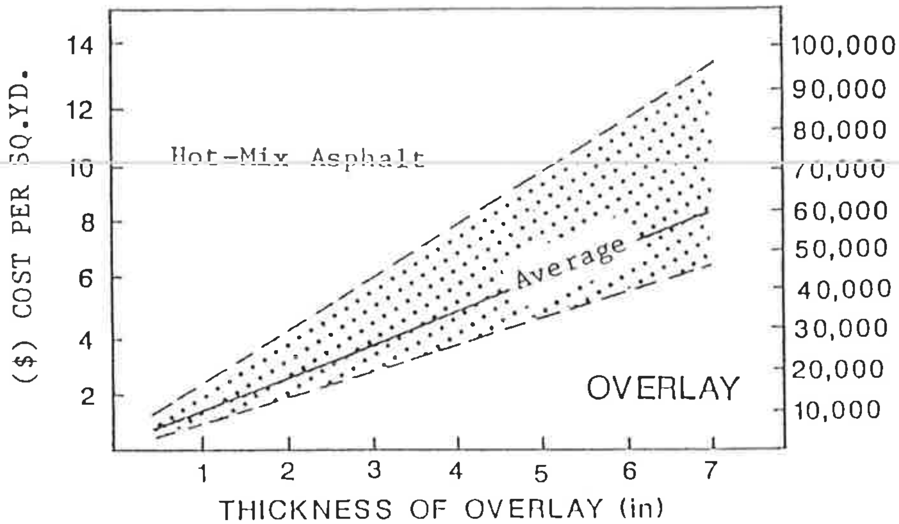

FIGURE 4 Cost of hot-mix asphalt overlay of various thicknesses *(5)*.

mix recycling materials as reported by some Indiana contractors and highway agencies that have used this process for rehabilitating county roads and residential urban streets.

Because the most common rehabilitation alternative for repairing a distressed asphalt pavement in need of structural improvement in Indiana is the placement of an asphalt overlay, cost figures were sought for this procedure in order to compare these figures with the cost of alternative cold-mix recycling.

It was found that, in general, the savings and differences in cost for these two alternatives were significant. An approximate range of prices was obtained for hot-mix overlays (Figure 4) and in-place cold-mix recycling (Figure 5). The data shown in Figure 4 were updated from figures given by the Texas Transportation Institute *(5)*, and Figure 5 was developed from data reported in the literature *(1, 2, 4)* and consultations with local contractors, highway agency personnel, and personnel from the Bureau of Local Roads and Streets of the Illinois DOT. The data presented in these figures indicate that significant savings may be realized when recycling is chosen instead of an asphalt overlay for rehabilitating an asphalt pavement on a secondary road.

There is at least one other item that has a direct bearing on the reduction or minimization of highway construction cost in general, and that is the energy used in the transportation of the materials, in the operation of equipment for processing those materials, and in manufacturing the finished product. The literature (in particular, the reports prepared for Demonstration Project 39) gives notice of the various relationships that exist between costs of a conventional asphalt paving process and recycling procedures in terms of hauling distances of virgin and reclaimed materials (aggregates as well as binder). It is reported that as the distance that new materials must be hauled increases, the advantage of recycling increases significantly *(7)*. Energy saved also increases as the proportion of reclaimed material in the final mix increases (Figure 2).

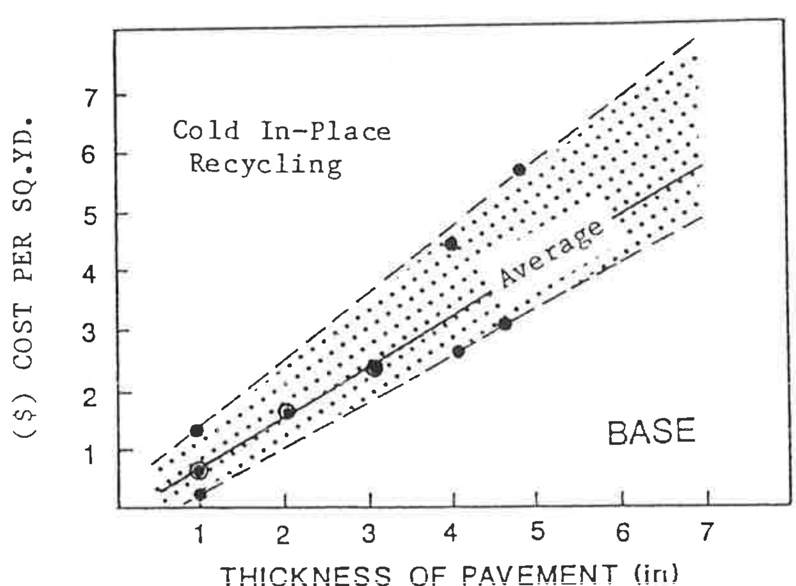

FIGURE 5 Cost of cold in-place recycled material.

Summary of Findings

The preliminary conclusions that can be developed from this information follow.

1. The cost of the new binder or recycling agent used to restore the properties of the reclaimed asphalt pavement (RAP) material accounted for more than half the total cost of cold-mix recycling projects.
2. The final cost of a recycling project is increased as more virgin materials are added to the recycled mix.
3. Significant savings in energy are possible if in-place cold-mix recycling techniques are used instead of central plant hot- or cold-mix recycling. The cost of transporting materials to and from the plant accounts for most of these savings. Savings are even more substantial when recycled mixtures are compared with conventional hot-mix asphaltic overlays.
4. The overall costs for cold-mix recycling operations are dependent on the assumptions and particular techniques adopted for each individual recycling project, as well as other factors such as traffic, weather, materials, location, and equipment.
5. The cost or energy effectiveness of cold-mix recycling cannot be determined from the information found in most of the available literature or reported here. The level and length of service to be obtained from recycled pavements have not yet been established *(1, 4)*.

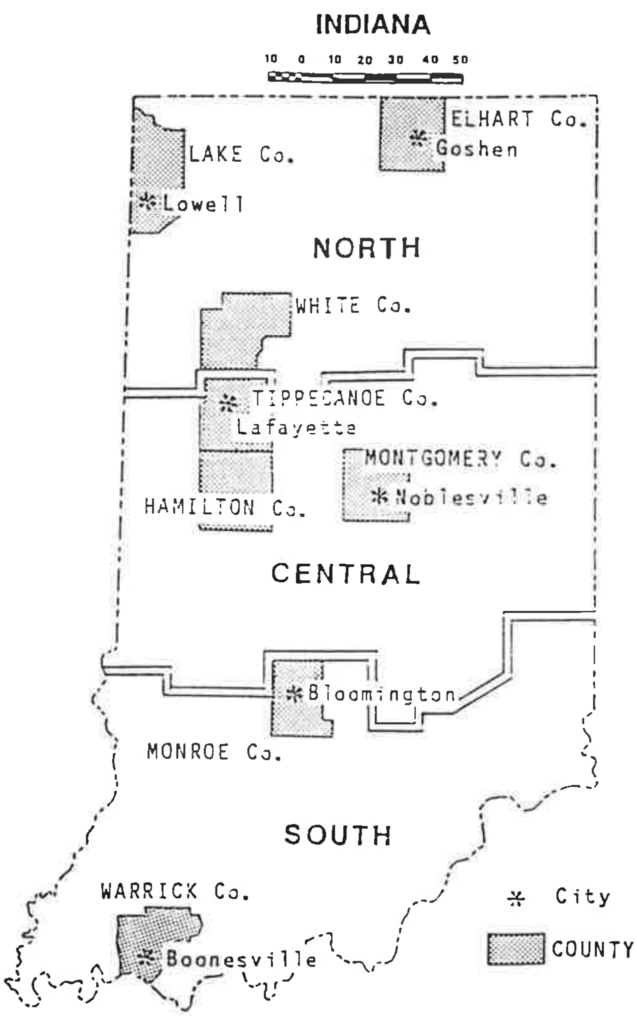

FIGURE 6 Location of pavements sampled.

TABLE 1 DEPENDENT VARIABLES

Name	Unit	Total No. of Observations
Asphalt content	%	227
Penetration	0.1 mm	159
Kinematic viscosity	cSt	159
Aggregate gradation modulus	–	227
Marshall stiffness	lb/in.	178
Layer thickness	in.	208

EVALUATION OF EXISTING PAVEMENT MATERIALS

Representative asphalt pavements from secondary roads (counties and cities) throughout Indiana were sampled at the locations shown in Figure 6. The properties of the materials that form those roads were determined in the laboratory by means of standard ASTM test procedures *(8)*. The objective of this field and laboratory work was to collect data on parameters such as asphalt content, aggregate gradation, penetration and viscosity of the recovered binder, and other important characteristics of the existing asphalt pavements given in Table 1.

These data were then evaluated using statistical analysis methods such as the analysis of variance (ANOVA) procedure *(9)* for the main factors given in Table 2. The results of these statistical analyses (Table 3) helped determine the significance of the variability found in the parameters measured. These determinations, in turn, allowed conclusions to be drawn regarding the feasibility of using the cold-mix recycling technique for rehabilitating or maintaining asphalt pavements on secondary roads in Indiana.

The information obtained from these analyses can be summarized as follows:

1. There was no significant effect that could be attributed to the geographic location of the existing asphalt pavement in terms of viscosity or penetration, or both, of the asphalt binder recovered from pavement cores. The weather in northern, central, and southern Indiana (refer to Figure 6) has, for practical

TABLE 2 MAIN FACTORS CONSIDERED

Independent Variable	Sample Size	Description
Climatological regions (refer to Figure 6)	3	Northern Indiana Central Indiana Southern Indiana
Traffic type	2	Rural Urban
Zones surveyed	14	8 counties 6 cities
Roads sampled	58	34 county roads 24 city streets
Pavement cores	117	74 from county roads 43 from city streets
Pavement layers	227	114 from county roads 113 from city streets

TABLE 3 SUMMARY OF ANOVA TESTS RESULTS

Source	Asphalt Content	Penetration	Viscosity	Gradation Modulus	Marshall Stiffness	Thickness
Traffic type (TR)						
Zones, Z (TR)	*				*	*
Roads, R(TR Z)		*	*			*
Samples, S (TR Z R)						
Layers, (L)	*				*	*
L·TR						
L·Z(TR)	+				*	
L·R(TR Z)	*	*	*	*		*
L·S(TR Z R)						

NOTE: * = statistically significant at $\alpha = 0.05$ and + = statistically significant at $\alpha = 0.10$.

purposes (according to the statistical analyses of the data), similar effects on the aging of the binders from all of the pavement cores analyzed in this study.

By simply plotting the average viscosity and penetration values obtained for as many as 227 samples of existing asphalt pavements, it can be shown that the results lie within a close range: approximately 600 to 900 cSt for viscosity and 30 to 55 dmm (0.1 mm) for penetration (Figure 7).

A statistical analysis of the penetration and viscosity data showed that the only significant differences for these two parameters were found among the layers (up to four) that form a particular road or street section. However, a closer analysis of the mean values of the viscosity and penetration of the aged binders from each layer revealed that these variations in hardness from one layer to the other appear to be minimal and of small practical importance *(1)*.

The results of the statistical analysis could then have a twofold interpretation: (a) either each pavement layer must be removed (reclaimed) separately and treated individually for mix design purposes (ideal but not always practical) or (b) the various constituent layers must be scarified, ripped or cut, and mixed together. The resulting reclaimed material would then be considered to have an average asphalt penetration and viscosity value (the average of various samples depending on the size of the project) that can be used for mix design purposes.

The results of this statistical analysis also indicated that (a) the hardness (penetration and viscosity) of extracted binders from county roads does not vary significantly from that of asphalt pavements on city streets, (b) there were no significant differences in these parameters within county or city street pavements, and (c) there were no significant differences within a particular road segment along the horizontal plane of the pavement.

2. It was found that, in general, the asphalt content of the top layer of pavement was higher than the asphalt content of the lower layers. Figure 8 shows mean asphalt content values for the top two layers of each pavement sampled in various counties and cities in Indiana. The lines connecting the data points in this graph are used only for clarity and to show the overall trend found for the asphalt content of the pavements evaluated. Even though this can be considered typical of wearing surface layers compared with binder and base course layers, it should be pointed out that almost all secondary road pavements are made of a series of asphaltic concrete layers

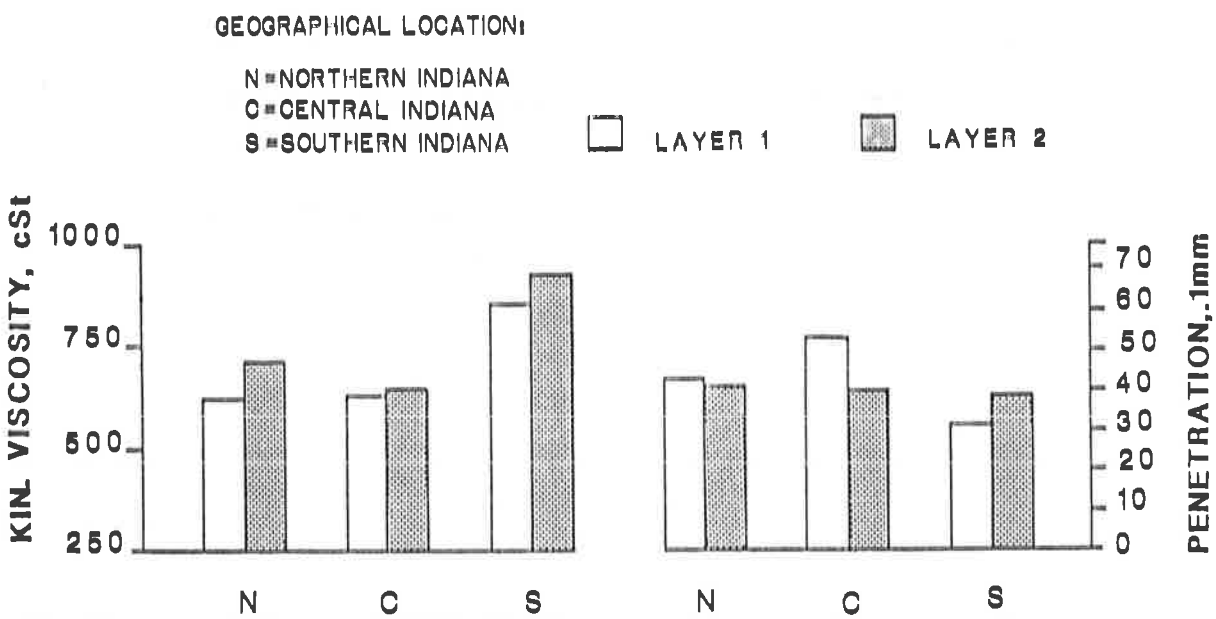

FIGURE 7 Viscosity and penetration of aged binders from various locations.

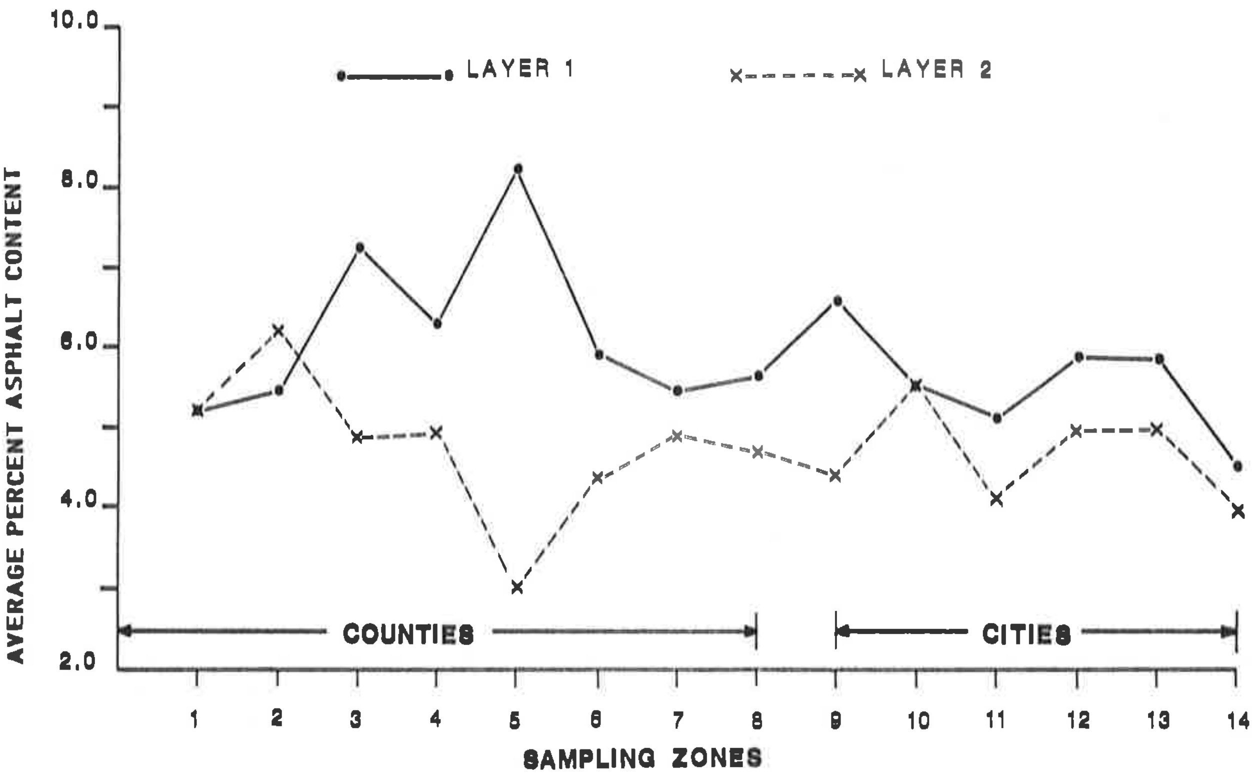

FIGURE 8 Average values of asphalt content of secondary road pavements in counties and cities.

placed, throughout the years, on top of each other (stage construction) and that the old wearing surface course layers become the base and subbase of the upper pavement layer.

The results of statistical analysis showed significant differences only among pavement layers (Table 3). In general, asphalt contents were within ±1.0 percent (by weight of total mix) of each other (except for County Zone 5 in Figure 8). These results lead to conclusions similar to those for viscosity and penetration of the asphalt pavement layers: (a) each layer can be milled off separately and the salvaged materials can be stored or used individually if the variations in asphalt content are greater than 1.0 percent and (b) for variations less than 1.0 percent, the layers can be reclaimed simultaneously and an average asphalt content value can be assumed for all of the salvaged material.

3. Asphalt pavements on county roads were found to have average aggregate gradations similar to those of city streets. Similar results were observed from samples within counties and within cities, within a section of the same road, and even among the various layers that form the pavement (Table 3). All of the gradation analysis results (227 samples) fall within the range of values shown in Figure 9.

4. Other findings were that, in general, top layers were thicker and more stable [as measured by the Marshall stability test *(8)*] than were lower layers. The variability in this parameter and in the thickness of the layer was found not to be statistically significant.

No significant differences were found between pavements of county roads and city streets nor within a particular road section for these two parameters. That top layer mixtures were found to be more stable than bottom layers was attributed to upper layers being relatively newer material and therefore less deteriorated than the lower layers they covered.

For a sound recycling design, the engineer will have to take these findings into consideration in order to account for a recycled pavement that may require a more stable subgrade than those found in this evaluation.

LABORATORY BEHAVIOR OF COLD RECYCLED ASPHALT MIXTURES

The final task of this study was an evaluation of cold-mix recycled asphalt pavement samples in the laboratory. The information gained from the results discussed in the previous section was used in the laboratory for the preparation of recycled mixtures that closely resembled an in situ cold-mix recycled asphalt pavement material.

Reclaimed asphalt pavement (RAP) material from actual city streets was used to prepare most of the laboratory specimens tested following ASTM standard test procedures *(8)*. These RAP materials were prepared with gradations, asphalt content, penetration, and other parameters within the range of in situ values.

This laboratory study covered four different RAP materials, six different recycling agents (three common asphalt emulsions and three commercial recycling agents), various RAP gradations, two artificially aged binder contents, curing time, and other mix design parameters.

The major findings obtained from the laboratory evaluation can be summarized as follows:

1. Initial mixing water contents less than 3.0 percent (by dry weight of the RAP) produced the best performing cold recycled mixes.

2. The amount of recycling agent has a significant effect on the behavior of the recycled mix. Recycled mixes with agent contents lower than 1.0 percent (by dry weight of the RAP) showed inconsistent trends of initial strength or stability properties. This apparently indicates that low agent contents are difficult to distribute uniformly throughout the mix, which yields a nonuniform paving material with unpredictable performance in the field.

On the other hand, excessive agent contents (more than 3.0 percent by dry weight of RAP) produced unstable mixes that would be expected to undergo excessive deformations and pavement bleeding in the field. The ideal range of recycling agent (liquid asphalts or modifying oils, or both, or chemicals) for the reclaimed asphalt pavement materials appears to be somewhere between 2.0 and 3.0 percent by weight of the dry RAP.

3. An aeration period before compaction and after moisture is mixed in and the recycling agent is added appears to be necessary to lower the total fluid content of the recycled mix. It was found that this initial period produced compacted recycled mixtures with high early strength and stability that allowed better handling of the test specimen in the laboratory.

4. The choice of recycling agent, modifying oil, or rejuvenating or softening agent should be based on economic considerations and the availability of these materials. All six agents considered in this laboratory study performed well in altering the hardness of the aged asphalt cement and produced recycled mixtures with characteristics similar to those of cold-mix asphalt made with virgin materials.

5. The effect of curing time following compaction was found to be significant. In the first 7 days of curing at ambient conditions (approximately 72°F), the increase in strength and stability of the mix was found to be related principally to the characteristics and amounts of agent added. Softer residue agents produced early cured recycled mixtures of low or even decreasing stabilities until evaporation of the water and other fluids began. Harder residue recycling agents yielded higher stability and strength values initially. In general, all test parameters measured for the recycled mix at various curing times increased to a maximum value and leveled off thereafter.

6. The effect of the RAP gradation (the gradation of the reclaimed material obtained after planing, crushing, ripping, or scarifying the aged asphalt pavement layers) was found to be significant at the levels used in this laboratory analysis. In most cases, however, it was found that cold recycled mixtures prepared with RAP gradations within the gradation range corresponding to those commonly produced by normal reclaiming procedures yielded mixtures with characteristics similar to those of mixtures the RAP material of which was processed to include more fines (i.e., additional processing of the RAP). The effects of RAP gradation were found to depend on the type of agent used as well as the amount present in the recycled mix.

7. Finally, recycled mixtures prepared with aged RAP mate-

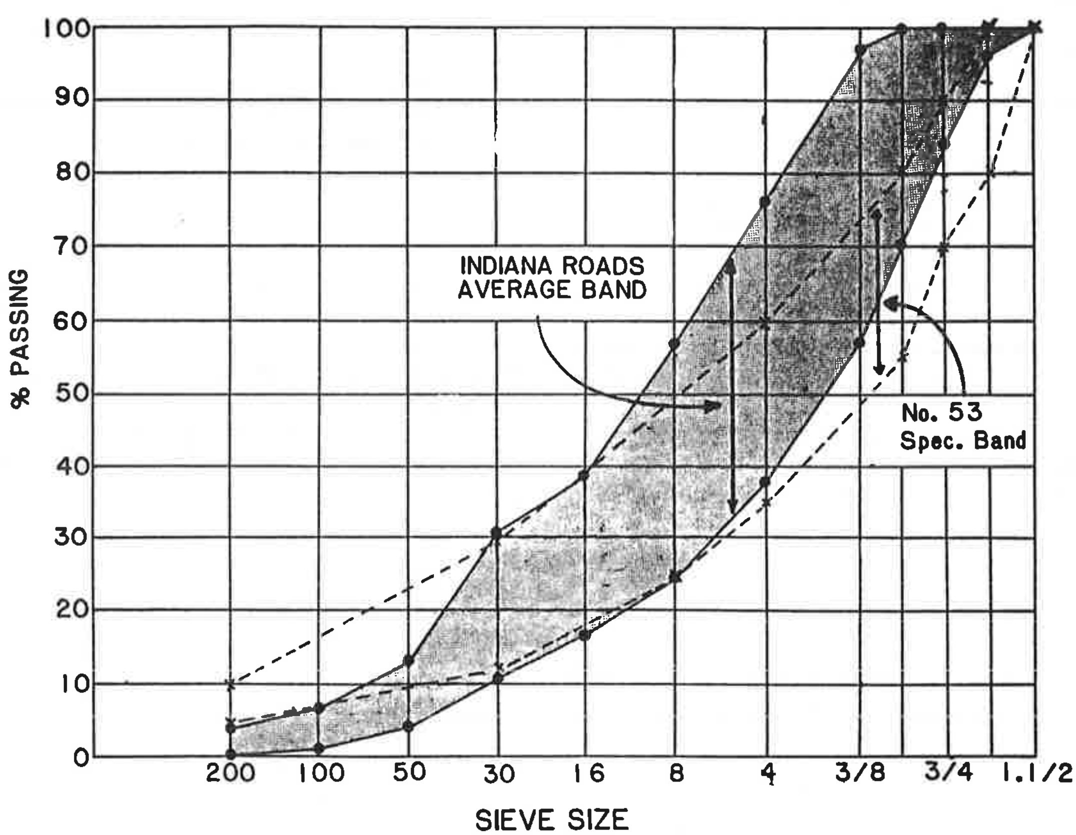

FIGURE 9 Average gradation of aggregate extracted from asphalt pavements on secondary roads.

rials of varying original asphalt contents were found to yield paving mixtures with different stabilities as measured by the Marshall test *(8)*. A combined stability-flow parameter (i.e., Marshall stability measured at room temperature divided by final flow) was found to be the most sensitive laboratory test parameter for detecting the behavior of these mixes. In most cases, a higher original binder content in the RAP yielded a lower combined stability-flow parameter for the final recycled mix.

OVERALL CONCLUSIONS

It is recommended that cold-mix recycling techniques be included in county and city highway maintenance programs in Indiana as an alternative for rehabilitating or maintaining asphalt pavements on secondary roads. Cost, energy, and mix design analyses should be used to establish whether other rehabilitation or maintenance techniques are more viable or practical than recycling.

It is believed that asphalt pavements on secondary county roads or city streets in Indiana can be rehabilitated with cold-mix recycled materials. This technique could be the least expensive alternative for restoring the original serviceability of the pavement.

Approximate costs associated with various materials and procedures (e.g., asphalt overlay, in-place recycled mix) can be used by the design engineer to estimate the approximate construction costs of the various alternatives available for a particular project. These preliminary economic evaluations can then be used as one of the factors to be considered in the selection of the most appropriate procedure for the project.

Standard construction units based on aggregate gradation and asphalt content of the material to be recycled can be established. Individualized construction units could be developed if it were demonstrated that penetration or viscosity test properties, or both, were quite different.

Whenever possible the degree of oxidation or brittleness of the original asphalt to be recycled should be known. This information can be obtained by extracting and then recovering the aged asphalt from the RAP. The harder the recovered asphalt, the softer the residue of the recycling agent must be. Common liquid asphalt emulsions were found to behave just as well as commercially available chemicals and modifying oils. The choice between these commercial products should be made initially on an economic basis. After narrowing the alternatives to two or three particular products, laboratory analysis of recycled mixes prepared with available RAP and the selected recycling agents should be performed to choose the agent that produces acceptable recycled mixtures.

The strength and stability properties of the recycled mix should be evaluated after a sufficient curing period has elapsed in order to obtain representative strength and stability values for a particular mix. Oven curing at 140°F for 3 days can be used to accelerate the curing process. A curing time of 7 days out of the mold may also be used for this purpose.

Optimum mixing moisture and recycling agent contents should be determined using trial mixtures and laboratory test procedures. Excessive (i.e., more than 3.0 percent) or deficient (i.e., less than 1.0 percent by weight of dry RAP) amounts of these two fluids should be avoided because unstable and weak mixtures, which exhibit unpredictable behavior, are produced.

The findings of the study of the effects of RAP gradations indicated that an existing asphalt pavement reclaimed or salvaged by conventional methods and equipment generally does not need extra processing.

It is also recommended that final mix designs be based on reclaimed and processed asphalt pavement material. Every recycling job should be approached with as much planning, design, and expertise as practically feasible.

ACKNOWLEDGMENT

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Structural Response of Foamed-Asphalt–Sand Mixtures in Hot Environments

AMIR F. BISSADA

This research project was conducted to investigate the ability of foamed asphalt to stabilize local marginal sand aggregates for use as base or subbase pavement materials. The variables measured were intended to give indications of the quality of performance of these mixtures as well as aid in the development of design procedures for the use of foamed asphalt as a stabilizing agent under the relatively high temperature conditions in the Arabian Gulf. Quantitative information is given about the effects on mixture response of significant factors such as sand type and gradation, quality and quantity of the fine fraction, asphalt grade and its percentage, moisture content, and curing condition. Both moisture and temperature susceptibilities of foamed-asphalt mixes were found to be significantly reduced by the addition of limestone powder to modify the fine fraction of sand. Structural response of foamed-asphalt mixes was compared with that of other hot-asphalt–sand mixes commonly used as base material in Kuwait. It appears that properly designed foamed-sphalt–sand mixes have a structural thickness that corresponds to that of hot asphalt and much less susceptibility to permanent deformations under traffic loads in hot environments.

An adequate procedure for designing foamed-asphalt–sand pavement materials cannot be developed unless the effects of all of the factors that affect mixture performance are thoroughly evaluated and clearly understood. A variety of mix design and evaluation procedures for foamed-asphalt materials has been and is being used in a number of countries *(1–7)*. This variety has led to difficulties in correlating and assessing results obtained in different environments. Therefore the initial step in developing a procedure for designing mixtures was to conduct a comprehensive laboratory study to obtain quantitative information about the effects of the properties of available materials and local environmental conditions on mixture response.

The study of foamed-asphalt stabilization of local sands was initiated in Kuwait because (a) there is a lack or limited supply of good-quality coarse aggregates and an abundance of poorly graded sands that are unsuitable for use as a base layer material and (b) asphalt base layers are subjected locally to service temperatures that vary from a minimum of 15°C to a maximum of 55°C *(8)*. Within this range of service temperatures, these layers were found to be more susceptible to deformation failure than to fatigue cracking *(9)*.

This work has two objectives. The first is to determine the significant factors that affect the structural response of foamed-asphalt mixtures compared with corresponding hot-asphalt plant mixtures. Factors such as moisture and temperature susceptibilities and elastic and creep behavior were considered. The second objective is to evaluate the potential of foamed-asphalt–sand mixtures as a structural base or subbase at relatively high local service temperatures and to compute equivalent thicknesses of these layers on the basis of their ability to dissipate vertical compressive subgrade stresses.

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EXPERIMENTAL PROCEDURE

Three grades of asphalt, classified as AC-20, AC-2.5, and a vacuum asphalt residue (VAR) with a penetration of 310, were used in this experiment. These asphalts were tested for penetration, softening point, viscosity, and specific gravity. The apparent viscosity was determined using a rotor viscometer at different shear rates and at temperatures ranging from 60°C to 165°C. The results of these tests are given in Table 1. The foaming characteristics of the three asphalt grades were investigated in terms of their foam expansion ratio and foam half-life under variable foaming temperatures and water contents. In general, the VAR with the lowest viscosity was found to possess the highest expansion ratio and half-life values at all foaming temperatures and water contents considered (Figure 1).

Two major types of locally available sands were used in this study, natural desert sand (S-2) and blow sand (S-3). In addition to these two sands, a third type of sand commonly used in hot-plant asphalt mixtures (S-1) was also considered for use. S-1 consists of natural desert sand, crusher waste aggregate, and limestone powder in a ratio of 46:46:8 by weight. The gradation, specific gravity, AASHTO T99 dry density, and optimum moisture content of the three selected sand aggregates are given in Table 2.

A foamed plant laboratory unit built by Ultra-Tec, Inc., was

TABLE 1 PROPERTIES OF ASPHALTS USED

Property	Asphalt Grade		
	AC-20	AC-2.5	VAR
Penetration at 25°C (0.1 mm)	67	135	310
Softening point (°C)	51	45	36
Viscosity (mPa/sec) at			
60°C	2.8×10^5	3.5×10^4	5.0×10^3
135°C	5.2×10^2	2.4×10^2	1.5×10^2
150°C	3.0×10^2	1.4×10^2	0.9×10^2
165°C	1.2×10^2	0.8×10^2	0.5×10^2
Specific gravity	1.030	1.010	1.005

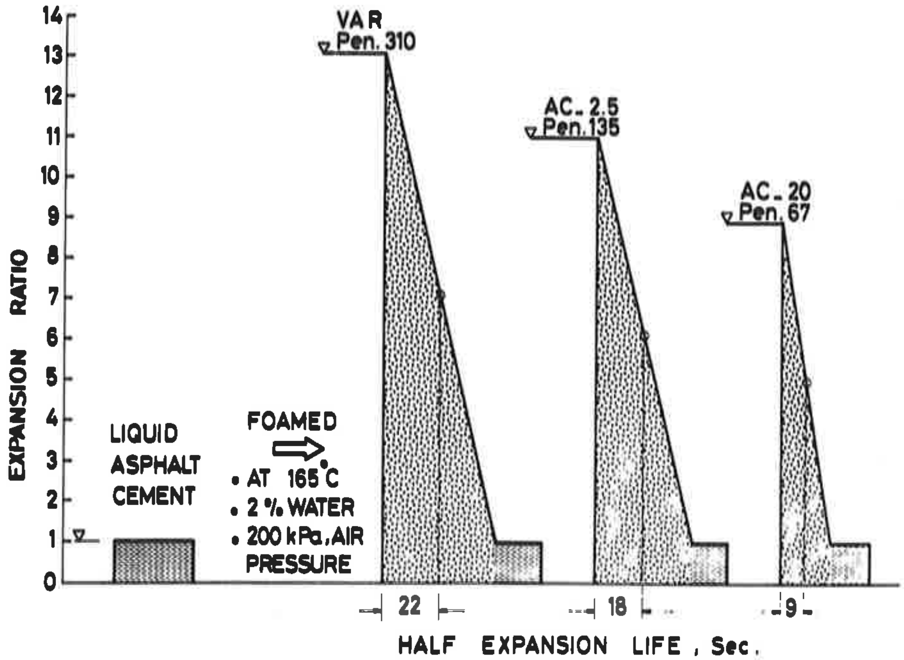

FIGURE 1 Expansion ratio and half-life of foamed asphalts.

used to produce the asphalt mixtures investigated. The electrically powered plant unit contains a foaming device that receives precise measurements of water and hot asphalt and converts them to an asphalt foam that is mixed in a paddle mixer. The plant mixes 7 kg per batch, which is enough to prepare six standard Marshall molds at once. The mixing period in the paddle mixer was 1 min after the foamed asphalt was added. Then the mixture was hand mixed in the pan for approximately 1 min.

The test specimens were molded within 45 min after the mixing operation was completed. Marshall specimens of all foamed mixes were compacted using a mechanically operated compaction hammer according to ASTM D 1559. Compacting was done at room temperature; 50 blows were applied on each side of the specimen. Specimens were carefully extruded from their molds and weighed immediately after compaction. To determine the effect of loss in moisture content, specimens were weighed periodically during curing.

TABLE 2 PHYSICAL PROPERTIES OF SANDS

Property	S-1	S-2	S-3
Gradation particle size (mm) (% passing)			
5	100	100	100
2	93	98	100
1	80	89	100
0.5	55	62	96
0.2	26	12	52
0.1	14	4	27
0.75	8	2	8
Atterberg limit	–	–	–
Bulk specific gravity	2.604	2.612	2.595
Standard proctor dry density (t/m^3)	2.062	1.915	1.736
Optimum moisture content (%)	9.4	7.8	12.0

NOTE: Dash = not provided.

Three different laboratory curing conditions, based on climatic regions of the Arab Gulf, were considered: (a) in air at room temperature (23°C), (b) in a humidity chamber (23°C and 100 percent humidity), and (c) in an oven at 40°C. The first and second curing conditions were supposed to simulate local dry and humid low-temperature seasons, respectively. The third represents the local dry temperate condition.

Foamed-asphalt mixtures were characterized by the Marshall method following ASTM D 1559 except that mechanical compaction was used to apply 50 blows on each side of the specimen at room temperature. The percentage of air voids in the compacted specimens was determined according to ASTM D 3203. Marshall specimens were conditioned according to ASTM D 1075, and moisture damage was determined by using the ratio of the Marshall stability of conditioned specimens to the Marshall stability of dry specimens.

Indirect tensile strength is determined by testing the specimen of standard Marshall size diametrically at a constant rate of 51 mm/min *(10)* for the purpose of evaluating both the moisture and the temperature susceptibility of the foamed-asphalt–sand mixtures.

The apparatus used for measuring creep stiffness was a Freundl type of loading system *(11)*. The frame load and the upper platen preloaded the specimen with 0.01 N/mm^{-2} for 1 min. The constant stress applied to the specimen was 0.10 N/mm^{-2} for 60 min.

Specimens of Marshall standard size were coated with paraffin before being immersed in a 40°C water bath for 1 hr to give them a uniform temperature before testing. The paraffin coating has the effect of avoiding any possible moisture damage to the specimens before and during testing.

A nondestructive resilient modulus (M_R) test was carried out by applying a pulsing load of 0.10-sec duration across one diameter of the cylindrical specimen while the resultant elastic response across the opposite diameter was measured *(12)*. An electrohydraulic apparatus was used to apply the selected pat-

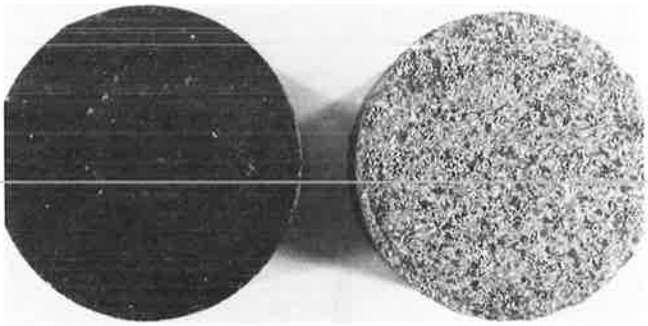

FIGURE 2 Hot-asphalt *(left)* and foamed-asphalt *(right)* specimens.

tern of dynamic loading. Temperature was controlled within 1/2°C using a thermally controlled cabin.

CHARACTERISTICS OF MIX DESIGN

Effect of Asphalt Foaming Properties

All of the foamed-asphalt–sand mixtures looked like moist sand with no visible asphalt color right after cold mixing. However, the compacted specimens after the first curing hours got darker in color, and most of the fine particles were coated with less than 0.50 mm of asphalt. Because of the partial asphalt coating, all foamed-asphalt cold mixes are lighter in color than are corresponding hot-asphalt mixes, as shown in Figure 2.

The effects of foam ratio and half-life on the stability of the foamed-asphalt mixtures were detectable. Foamed-asphalt mixes made with AC-20 with an expansion ratio of 9 and a half-life of 8 sec gave the lowest stability values of any of the mixes considered in this experiment. It was observed that foamed asphalt AC-20 did not have good mixing properties with the sands used in this study. Problems such as stickiness and lumping were encountered with this mix but were not observed in other mixes made with AC-2.5 and VAR with relatively high values of expansion ratios and half-lives. Visual examination of the foamed mixtures revealed that the VAR with the lowest viscosity exhibited the best aggregate particle coating and the most uniform dark color. Mixtures that contained AC-20 were light in color, and no asphalt was visible in those that contained 4.5 percent asphalt. Increasing the asphalt content to 6.5 percent resulted in the appearance of several balls of uncombined asphalt.

The effect of asphalt viscosity at 165°C on Marshall stability of foamed mixtures made with sand S-1 is shown in Figure 3. Foamed mixtures that contained VAR of viscosity 0.5×10^2 mPa/sec at 165°C showed higher stability values than did corresponding mixtures with AC-2.5 and AC-20 of viscosities 0.8×10^2 and 1.2×10^2 mPa/sec, respectively, at 165°C.

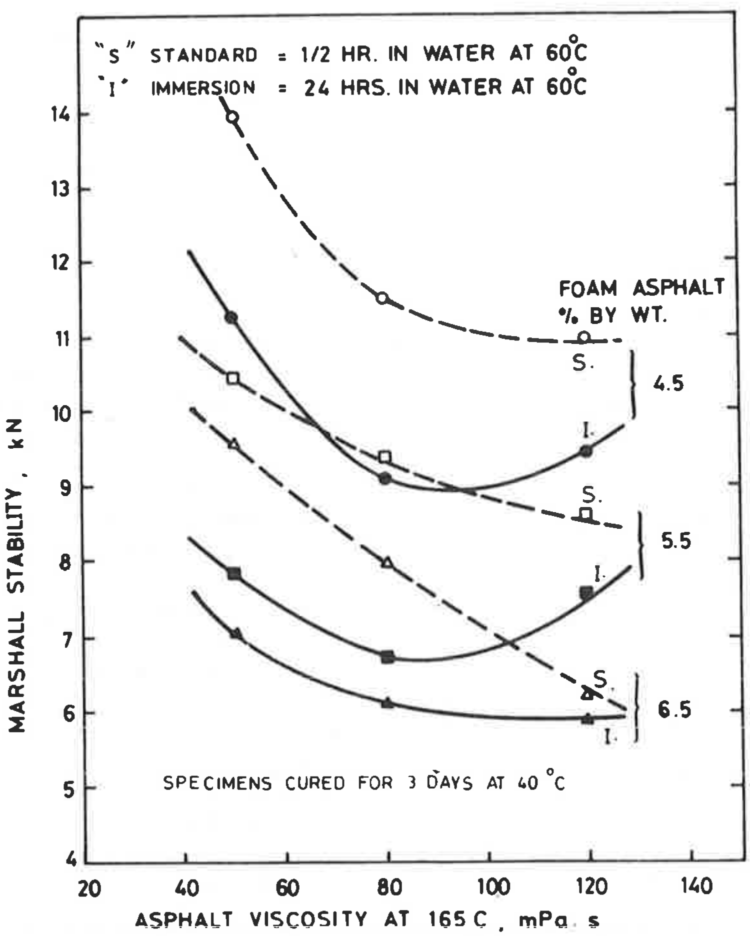

FIGURE 3 Effect of asphalt viscosity on Marshall stability.

After 3 days of curing at 40°C all tested foamed-asphalt specimens had higher Marshall stability values than did corresponding hot-asphalt specimens at lower asphalt contents (Table 3). Maximum stability of foamed-asphalt specimens was found to be at 1.5 to 2.0 percent less asphalt content than that for corresponding hot-asphalt specimens (Figure 4).

The partial asphalt coating in foamed-asphalt mixtures promoted the frictional shear component and resulted in higher air voids and lower measured flow values compared with those for corresponding hot-asphalt mixtures. These values for all tested foamed-asphalt specimens were not significantly affected by either the asphalt content within the range of variation considered or the viscosity of the asphalt cement used.

Effects of Sand Gradation and Particle Shape

Marshall test results for the three sands with grade AC-2.5 asphalt are given in Table 4. Improving the gradation, increasing the percentage of fines (less than 0.075 mm), and using crushed sand particles included in sand S-1 have resulted in significant improvements in Marshall properties before and after water immersion. Standard and immersed Marshall stability values for S-1 foamed-asphalt mixes ranged from 8.0 to 11.5 kN and from 5.1 to 9.2 kN, respectively. These stability values were higher than those obtained for S-2 and S-3 foamed-asphalt mixes.

Foamed-asphalt specimens made from natural desert sand (S-2) had a low stability value and did not withstand the moisture conditioning test. The S-3 blow sand, which includes about 8 percent natural fines of silica dust (less than 0.075 mm), produced foamed mixes of standard Marshall stability ranging from 3.4 to 5.8 kN. The relatively high values of air voids (25 to 30 percent) were related to the poor gradation of this type of sand. All foamed-asphalt specimens prepared from S-3 sand had low immersion stability values and some of these specimens collapsed before testing.

TABLE 3 MARSHALL PROPERTIES OF MIXTURES MADE WITH DIFFERENT GRADES OF ASPHALT

Asphalt Content (% by weight of aggregate)	Stability (kN)	Flow (0.1 mm)	Unit Weight (t/m^3)	Air Voids (%)
Hot Mix (AC-20)[a]				
4.5	7.1	24	2.110	13.1
5.0	7.8	24	2.169	9.9
5.5	8.0	26	2.217	7.2
6.0	10.2	30	2.208	6.9
6.5	9.8	30	2.198	6.7
7.0	8.9	33	2.198	6.0
Foam Mix (AC-20)[b]				
4.0	11.7	21	2.033	16.5
4.5	11.0	23	2.028	16.4
5.0	7.4	22	2.018	16.2
5.5	6.4	23	2.017	15.7
6.0	6.4	23	1.995	15.9
6.5	6.6	24	2.000	15.1
Foam Mix (AC-2.5)[c]				
4.0	12.7	25	2.052	15.5
4.5	11.5	25	2.040	15.9
5.0	11.0	24	2.045	15.0
5.5	9.4	25	2.027	15.1
6.0	8.1	26	2.011	15.2
6.5	8.0	28	2.010	14.6
Foam Mix (VAR)[d]				
4.0	13.8	23	2.041	15.5
4.5	14.0	25	2.052	15.4
5.0	12.8	24	2.038	15.3
5.5	10.4	26	2.020	15.4
6.0	9.9	27	2.016	15.0
6.5	9.6	24	2.000	15.1

[a]Viscosity at 165°C = 1.2×10^2 mPa/sec.
[b]Viscosity at 165°C = 1.2×10^2 mPa/sec.
[c]Viscosity at 165°C = 0.8×10^2 mPa/sec.
[d]Viscosity at 165°C = 0.5×10^2 mPa/sec.

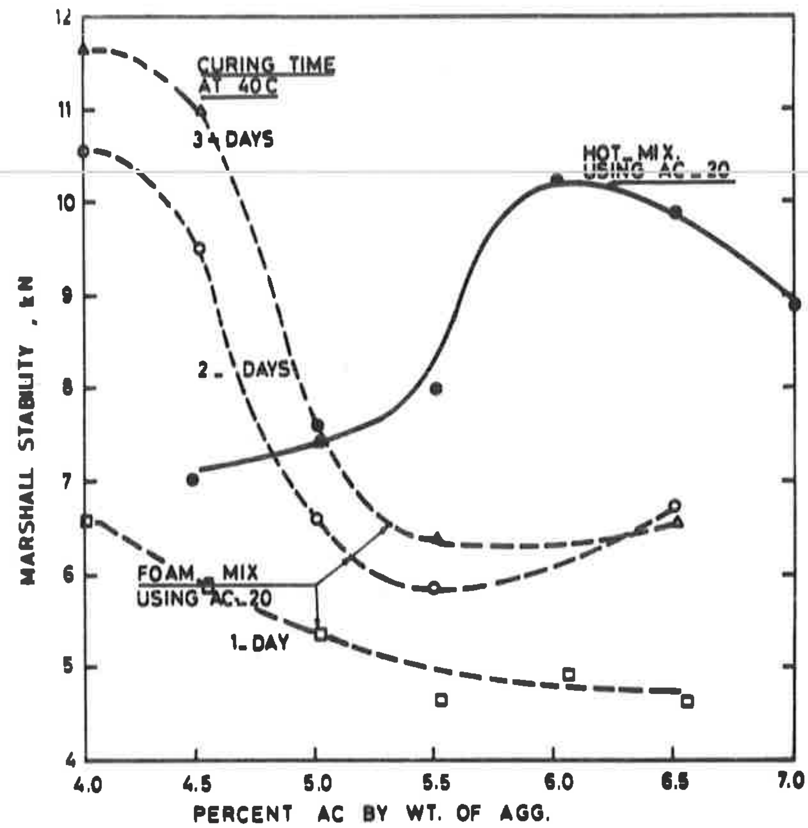

FIGURE 4 Marshall stability versus asphalt content of mixtures made with S-1.

TABLE 4 MARSHALL PROPERTIES OF MIXTURES MADE WITH DIFFERENT SANDS

	Percentage of AC-2.5 by Weight			
Property	3.5	4.5	5.5	6.5
Sand 1[a]				
Standard stability (kN)	9.2	11.5	9.4	8.0
Flow (0.1 mm)	22	25	25	28
Unit weight (t/m^3)	2.020	2.040	2.027	2.010
Air voids (%)	17.9	15.9	15.1	14.6
Immersion stability (kN)	5.1	9.2	6.8	6.2
Sand 2[b]				
Standard stability (kN)	1.7	2.1	1.5	0.9
Flow (0.1 mm)	10	14	12	18
Unit weight (t/m^3)	1.922	1.945	1.932	1.921
Avoid voids (%)	23.5	21.6	21.1	20.6
Immersion stability (kN)	0	0	0.3	0.3
Sand 3[c]				
Standard stability (kN)	3.4	5.8	4.4	3.6
Flow (0.1 mm)	25	28	29	31
Unit weight (t/m^3)	1.720	1.753	1.776	1.729
Air voids (%)	30	27.7	25.6	26.5
Immersion stability (kN)	0	0.4	0.4	0.8

NOTE: Specimens were cured for 3 days at 40°C. Immersion stability was determined after specimens were soaked for 24 hr in 60°C water.

[a]Moisture content: 7.5 percent at mixing, 0.7 to 1.3 percent cured.

[b]Moisture content: 6.4 percent at mixing, 0.7 to 1.3 percent cured.

[c]Moisture content: 9.6 percent at mixing, 2.0 to 2.9 percent cured.

Effect of Mixing Water Content

Figure 5 shows the average values of both Marshall stability (S_M) and indirect tensile strength (S_T) at different water contents for mixtures made with S-2 with 10 percent limestone. Data from this series of tests appear to indicate that the optimum mixing water content was around 9.6 percent by weight of aggregate, which corresponds to about 80 percent of optimum compaction moisture.

Specimens prepared at a moisture content of 40 percent on the dry side of the optimum value were difficult to mix with asphalt. The foamed asphalt was not uniformly distributed and the mix had a spotty appearance. Marshall stability and indirect tensile strength values dropped to about 50 percent of those obtained at the optimum mixing moisture. On the other hand, water content equivalent to 100 percent of the optimum compaction moisture resulted in the initiation of cracks during compaction. Marshall stability and indirect tensile strength decreased to about 70 and 88 percent, respectively, of the values obtained at optimum mixing water content.

The effect of varying the mixing moisture content on the percentage of retained stability and tensile strength after water immersion was found to be negligible. Within the range of mixing moisture contents applied, retained stability and tensile strength ranged from 62 to 82 percent and from 69 to 79 percent, respectively. The lower percentage values are related to the relative decrease in the density of the mix caused by the change in the mixing moisture content.

Effect of Curing Condition

The effect of the three curing conditions on the Marshall stability of the foamed-asphalt mixes and the measurements of loss of moisture are plotted in Figure 6. The following observations resulted from these tests:

- For the three curing conditions investigated, the loss of moisture content measured from the time of molding the specimen is accompanied by an increase in stability values.
- The highest rate of gain in stability was measured for 40°C oven curing and achieved a maximum value after 3 days with a moisture content of about 0.5 percent. No significant increase in stability was observed after 3 days.
- Curing in the humidity chamber resulted in the lowest rate of gain in stability; after 14 days, only 50 percent of the maximum attainable stability was achieved.
- Maximum stability values were measured for specimens cured for 21 days in air at room temperature. These stability values were equivalent to those measured for specimens oven cured for 3 days at 40°C.

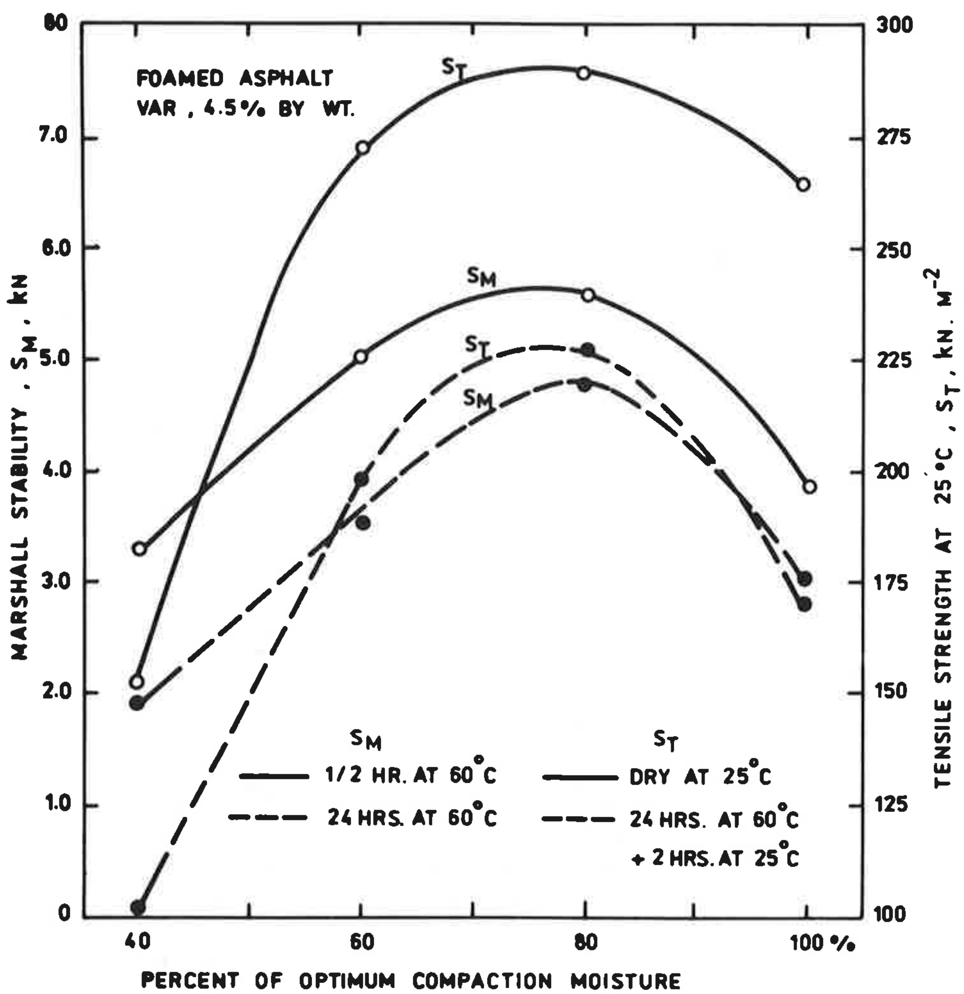

FIGURE 5 Effect of mixing water content on Marshall stability and tensile strength.

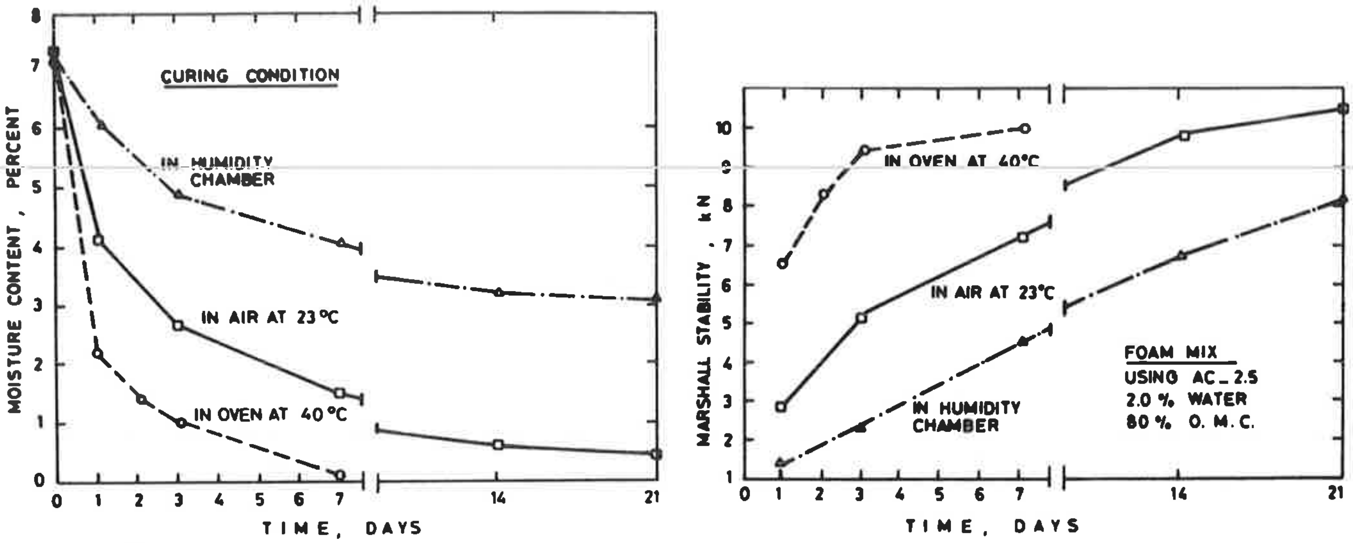

FIGURE 6 Moisture content and corresponding stability under different curing conditions.

- Examination of the data revealed no constant relationship between the loss of moisture and the consequent gain in strength under the three different curing conditions. It appears that, in addition to the effect of moisture loss, other factors such as the aging of the asphalt binder might contribute to the increase in stability of foamed mixes.
- The relations between moisture content and strength, which are related to the three curing conditions, can be used to evaluate foamed-asphalt mixes both at early-cure to determine when a road can be opened to traffic and at ultimate-cure to determine the whole service life of the foamed-asphalt layer.

Moisture Susceptibility

Moisture conditioning of test specimens that contained S-2 with different contents of fine additives resulted in a saturation

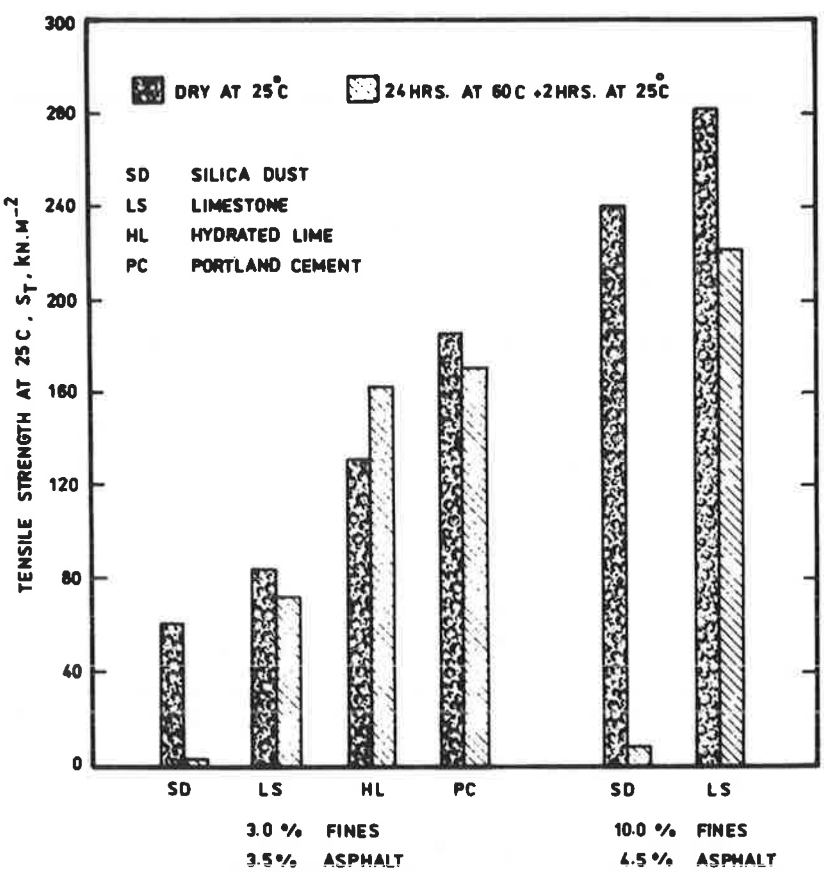

FIGURE 7 Tensile strength of mixes with different fine additives.

percentage that ranged from 40 to 60. This value was established by determining the percentage of air voids in the cured test specimen that were occupied by water after the specimen had been submerged statically in water at 60°C for 24 hr.

The silica dust is nonplastic fines normally found in natural desert sands. Such fines contributed little to improving the resistance of foamed-asphalt mixes to moisture damage. Increasing the amount of fines from 0 to 15 percent by weight of sand resulted in a maximum increase of 20 percent in retained Marshall stability. Many specimens did not withstand the moisture conditioning tests. However, the use of limestone filler (5 and 10 percent by weight of sand) resulted in retained stability values of 83 and 88 percent, respectively. Increasing the limestone filler content to 15 percent did not result in any further improvement in resistance to moisture damage.

Figure 7 shows the results of the indirect tensile tests carried out on foamed-asphalt mixes that contained S-2 with four different types of fines (minus 0.075 mm): silica dust, limestone powder, hydrated lime, and portland cement. The fines content and the asphalt content of all of these mixes were 3.0 percent and 3.5 percent by weight of dry sand, respectively. The mix that contained 3.0 percent portland cement showed the highest tensile strength values before and after moisture conditioning. Foamed-asphalt mixes that contained 3.0 percent hydrated lime had the highest percentage of retained tensile strength. Mixes with portland cement and limestone filler showed percentage-retained tensile strength values of 92 and 85, respectively. As expected, specimens that contained silica dust collapsed after moisture conditioning.

These fine additives modify the sand fines by increasing the fractions of minus 0.075 mm required to produce a stable asphalt mortar. On the other hand, the interaction of the bitumen acids with carbonate surfaces, like those of hydrated lime, portland cement, and limestone powder, results in the formation of calcium salts that are not expected to be water soluble *(13)*. This interaction could be responsible for a strong adhesive bond if these fine additives are added to the sand in a slurry.

The effect of asphalt content on Marshall stability values before and after moisture conditioning has been compared with that for corresponding hot-asphalt mixes that have the same filler content. Figure 8 shows that, at the optimum asphalt content, foamed-asphalt mixes show higher percentages of retained stability after immersion than do hot mixes. Although, in the hot-asphalt mixes, almost all aggregate particles are asphalt coated, which normally produces higher resistance to moisture damage, it appears that there exists a strong adhesive bond if the limestone filler in the foamed-asphalt mixes is added to the sand in the presence of water.

Temperature Susceptibility

Figure 9 shows the effect of increasing the percentage of fines in S-2 on the increase in tensile strength of the foamed-asphalt mixtures tested at temperatures ranging from 25°C to 50°C. The temperature susceptibility of a foamed-asphalt mixture containing 10 percent limestone powder and 4.0 percent 310-pen asphalt was compared with that of corresponding hot-asphalt–sand mixes and the results were plotted on a semilog graph (Figure 10). Both hot sand-asphalt mixtures that contained the harder asphalt grade (AC-20) and the softer asphalt grade (VAR) showed no significant difference in their slope values $\Delta \log S_T$ versus T°C within the range of test tempera-

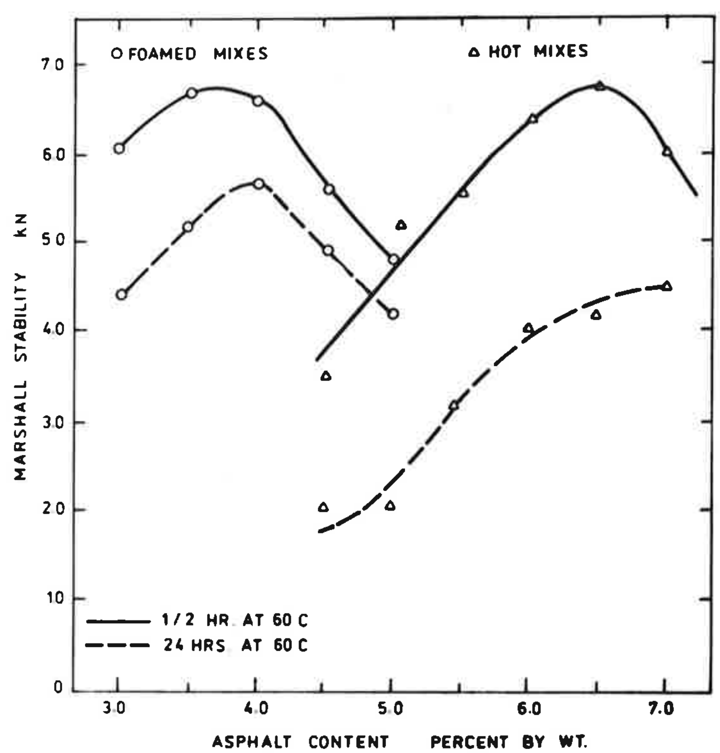

FIGURE 8 Loss in stability after immersion test (S-2 with 10 percent limestone powder).

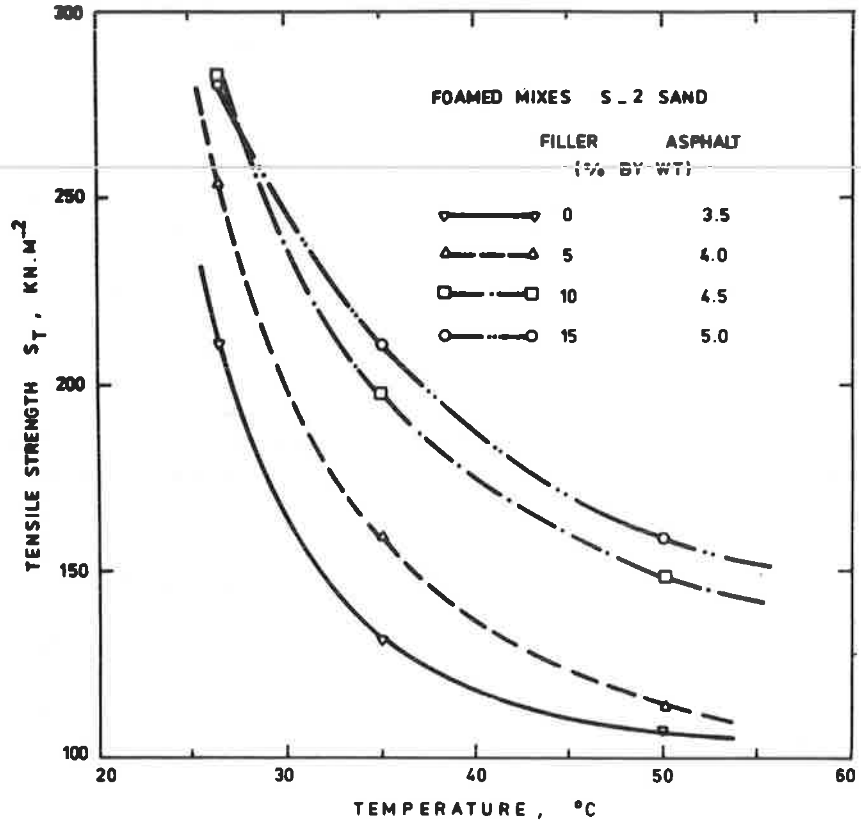

FIGURE 9 Tensile strength versus temperature (S-2 with different limestone filler contents).

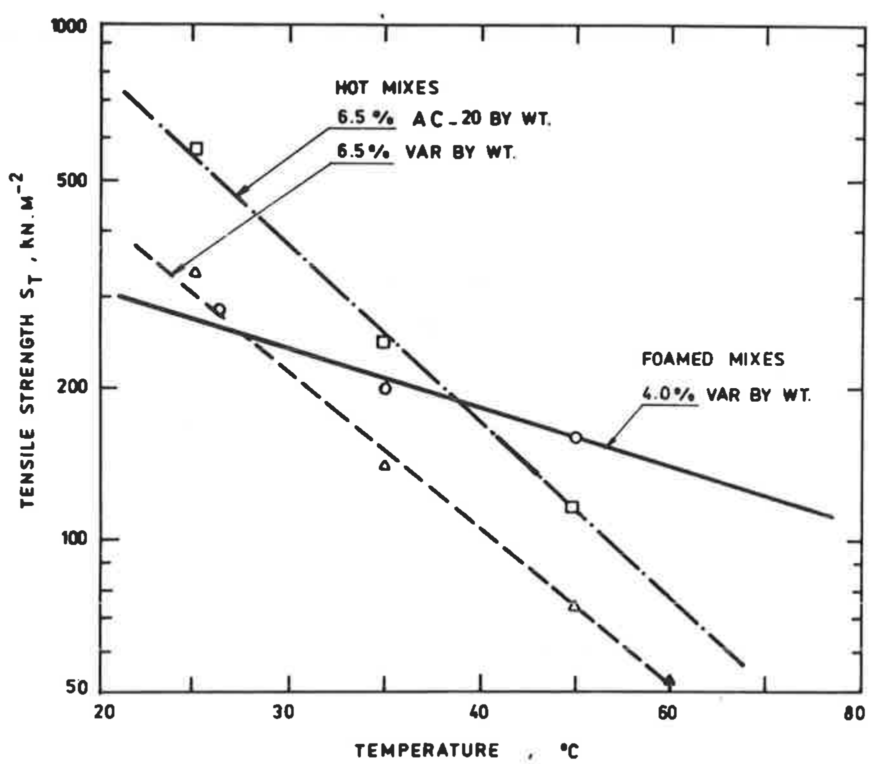

FIGURE 10 Log S_T–T°C relationship for foamed- and hot-asphalt mixes.

tures. A much smaller slope value was measured for the foamed-asphalt mix. The low-temperature susceptibility of the foamed-asphalt mixes is related to the high internal friction between the sand particles, which is due to the relatively low asphalt content.

Figure 11 shows measured resilient modulus (M_R) versus temperature for the foamed-asphalt mixtures and other hot-asphalt mixtures commonly used in Kuwait. The initial and final M_R-values of foamed mixtures were determined after curing at room temperature for 1 day and 21 days, respectively. Although the initial moduli of the foamed-asphalt mixtures over the whole range of test temperatures were quite low, the final moduli after 21 days were found to be about three times higher than the initial ones. At temperatures of 30°C and above, final M_R-values of foamed-asphalt mixtures were found to be equal to or higher than those of commonly used hot sand-asphalt mixtures.

Creep Deformation

Figure 12 shows that the stiffness value (σ/ε_t) during the creep test (60 min) increases as the percentage of fines in sand increases. On the other hand, the rate of decrease in stiffness with time was found to increase with the increase of fines content. Foamed-asphalt mix with 15 percent fines showed a rate of decrease in stiffness four times higher than that for a mix with no fines added. The effect of variations in optimum asphalt content of ±0.5 percent by weight on creep strain was also investigated. Figure 13 shows the results of this study. For all foamed mixtures the trend of increase in creep strain with the increase in asphalt content was obvious. However, there was no detectable difference in the effect of changes in asphalt content on the rate of change in creep strain.

Figure 14 shows that foamed-asphalt mixes that contained 5 percent and more fines had higher stiffness values than did hot-asphalt mixes. Stiffness of foamed-asphalt mixes containing 10 and 15 percent fines was about two- and fourfold higher, respectively, than that of hot-asphalt mixes. The complete coating of the whole sand structure with the asphalt mortar in the hot-asphalt mixes could be responsible for the relatively low resistance of these mixtures to creep deformation.

Stiffness of the mix (S_{mix}) as a function of stiffness of the asphalt used (S_A) in a log-log scale is shown in Figure 15. This presentation of the creep deformation characteristic has the advantage that the relative influence of a number of variables

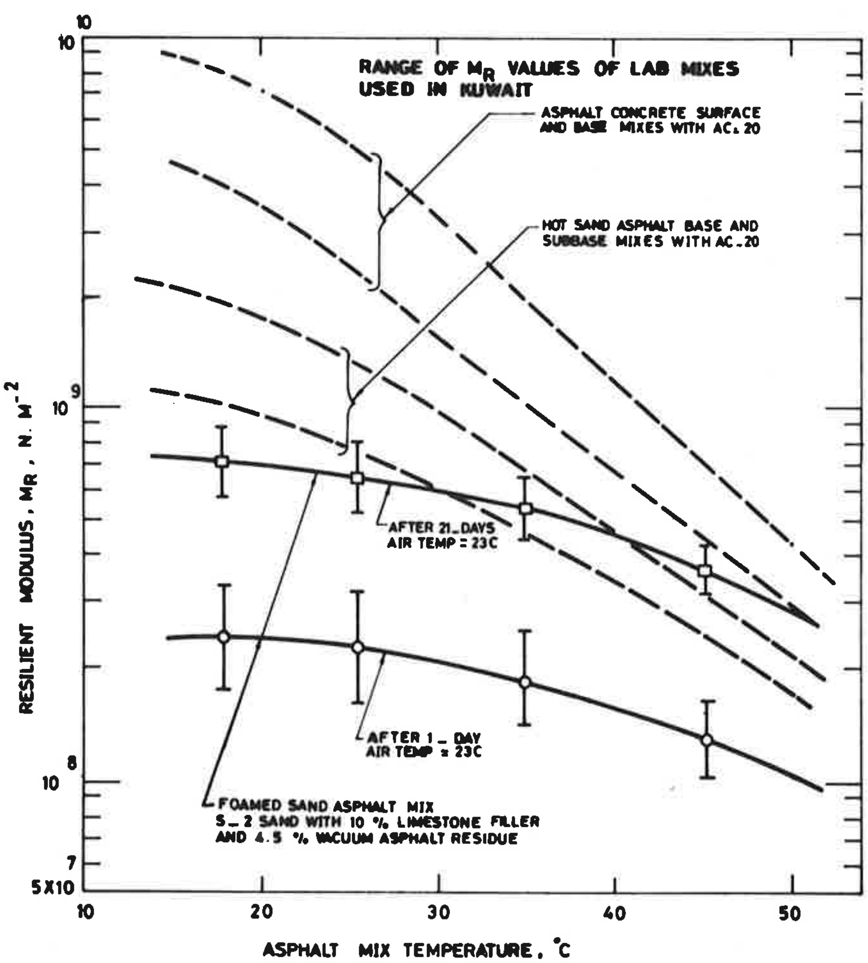

FIGURE 11 Resilient modulus versus temperature.

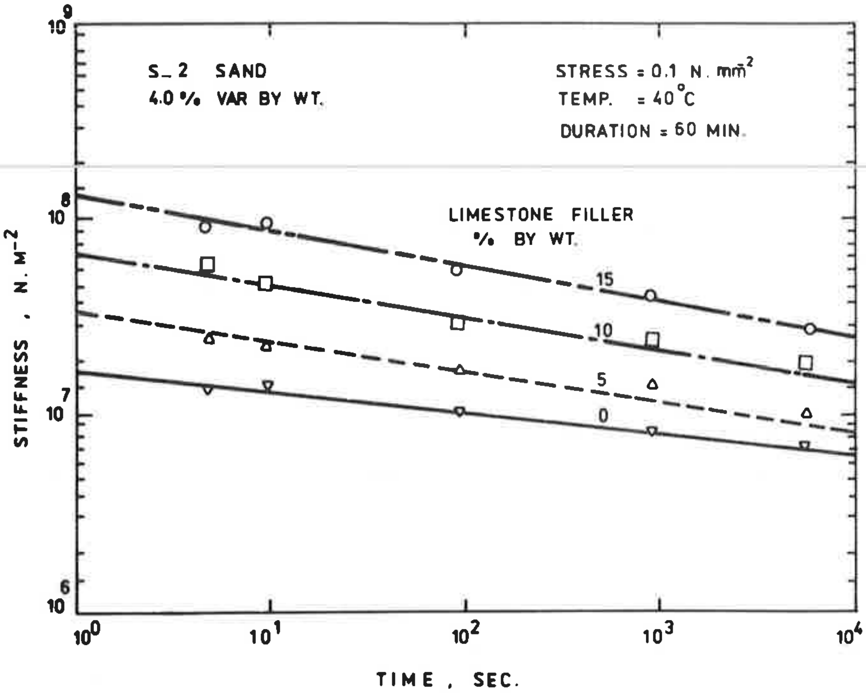

FIGURE 12 Creep stiffness for mixes with different fines contents.

can be quantified because permanent deformation is directly proportional to the reciprocal value of S_{mix} *(14)*. The stiffness of asphalt (S_A) is dependent on loading time, temperature, and asphalt properties and was determined using Van der Poel's nomograph *(15)*.

Figure 15 shows that the slope (q) of the log S_{mix} versus log S_A is equal to 0.08 and 0.18 for the foamed- and hot-asphalt mixtures, respectively. The higher the value of q, the more susceptible will be the mix to permanent deformation. This means that foamed-asphalt mixtures are less susceptible than corresponding hot-asphalt mixtures to permanent deformation. This is related to the improved low-stiffness properties of the foamed-asphalt mixture and to its relatively low asphalt content.

STRUCTURAL EVALUATION

The foamed-asphalt mixtures were evaluated in terms of their ability to perform as a pavement base or subbase layer. The structural evaluation was based on the results of tests carried

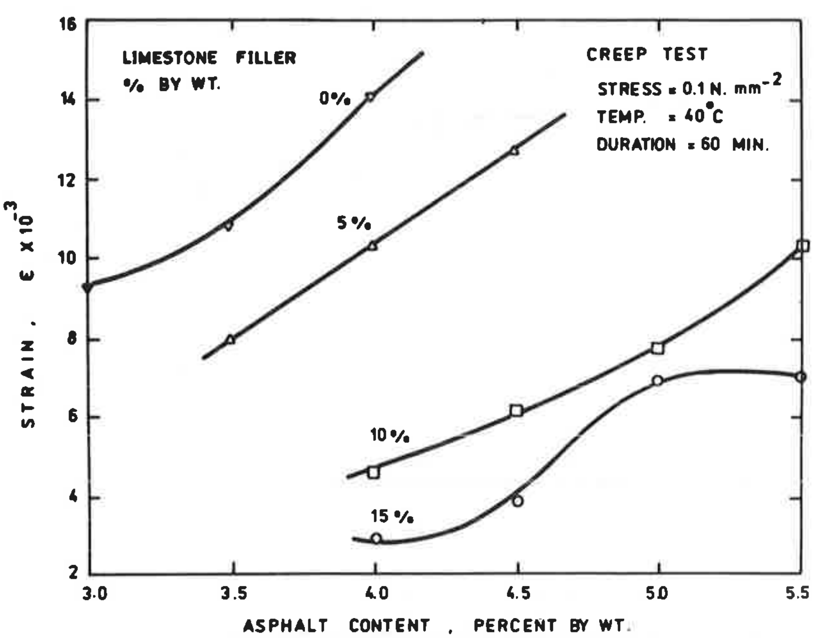

FIGURE 13 Creep strain versus asphalt content.

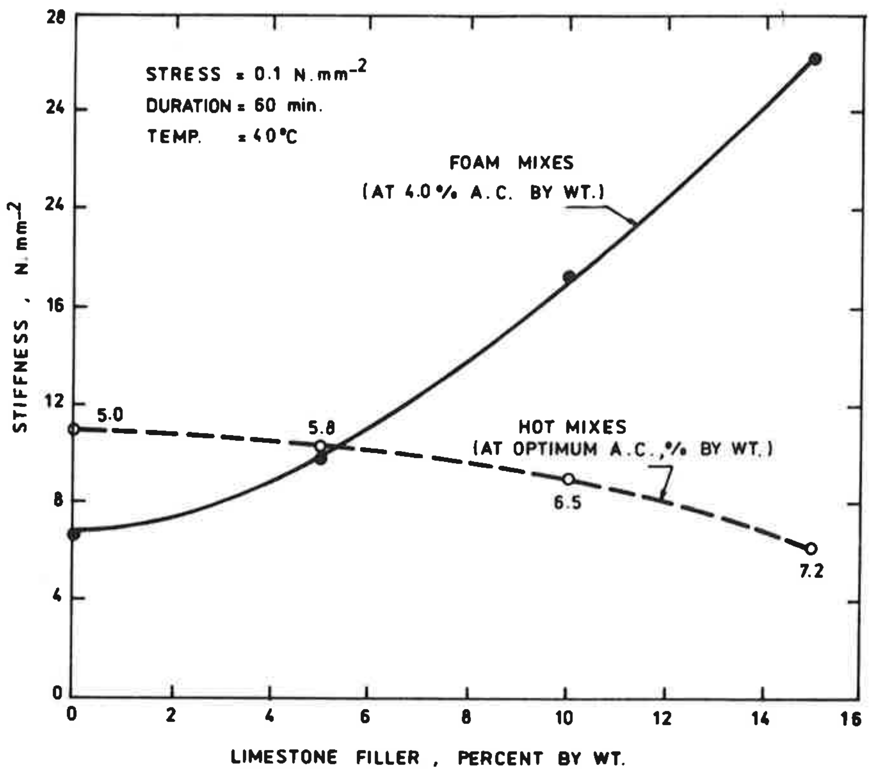

FIGURE 14 Creep stiffness of foamed- and hot-asphalt mixes.

out on mixtures at optimum percentages of foamed asphalt. Resilient modulus data for the range of local pavement temperatures were used to characterize these materials in a layered elastic model of the pavement system. The BISAR multilayered elastic computer program *(16)* and the DAMA computer program *(17)*, which analyze the pavement structure by cumulative damage techniques, were used for this purpose.

There are three criteria that affect the potential of asphalt pavement material as a structural base or subbase layer: (a) distribution of vertical stresses on the subgrade, (b) resistance to permanent deformation, and (c) fatigue life characteristics. The fatigue potential of the foamed-asphalt mixtures was found to be well below that of AASHO asphalt mixtures *(7)*. Foamed-asphalt mixtures have mechanical characteristics that fall between those of a granular structure and an asphalt-coated one. Therefore it may not be realistic to evaluate the fatigue potential of the foamed-asphalt mixtures in comparison with that of hot-asphalt mixtures. Moreover, fatigue cracking of asphalt base layers at high service pavement temperatures proved not to be the controlling criterion for pavement design life *(9)*. In this case, only subgrade deformation and resistance to permanent deformation in the pavement structural system were considered.

Thickness Equivalency

Thickness equivalencies for foamed-asphalt S-2 mixtures with 10 percent limestone powder and 4.5 percent (by weight) 310-

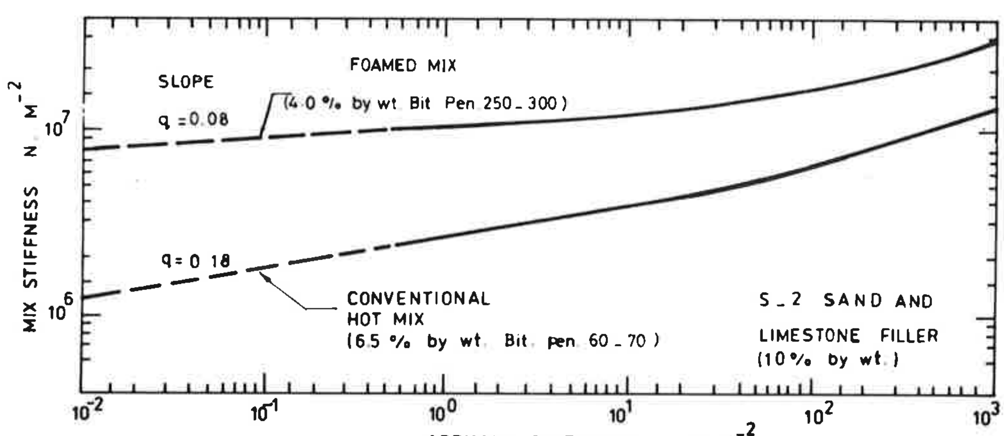

FIGURE 15 Log S_{mix}–Log S_A relationship for foamed- and hot-asphalt mixes.

pen asphalt were calculated on the basis of subgrade deformation damage. The relative ability of the foamed-asphalt layer to distribute vertical stresses and thus reduce critical subgrade strains or subgrade deformations was assessed and compared with that of other materials commonly used in pavement structures in Kuwait.

Two pavement systems were considered in this study. Pavement 1 is representative of a typical structure for a low-volume road [1,200 standard axle load (SAL) or 80 kN per month] in which the foamed-asphalt material is used as a base layer laid directly on the subgrade soil and overlaid with an asphalt concrete surface course 50 mm thick. Pavement 2 represents a medium-traffic volume road (12,000 SAL or 80 kN per month) in which the foamed-asphalt material is used as a subbase layer overlaid with 130-mm-thick base and surface courses of asphalt concrete. The thickness of the foamed-asphalt layer in both pavement systems varied between 80 and 180 mm. For each thickness the structural equivalency with hot plant asphalt concrete, hot sand-asphalt, and sand-gravel was determined. The procedure followed for computing the thickness equivalencies based on equal design lives is shown in Figure 16.

Monthly variation in temperature in Kuwait was used to account for the effect of temperature on moduli of asphalt pavement layers. For the sand-gravel materials the elastic mod-

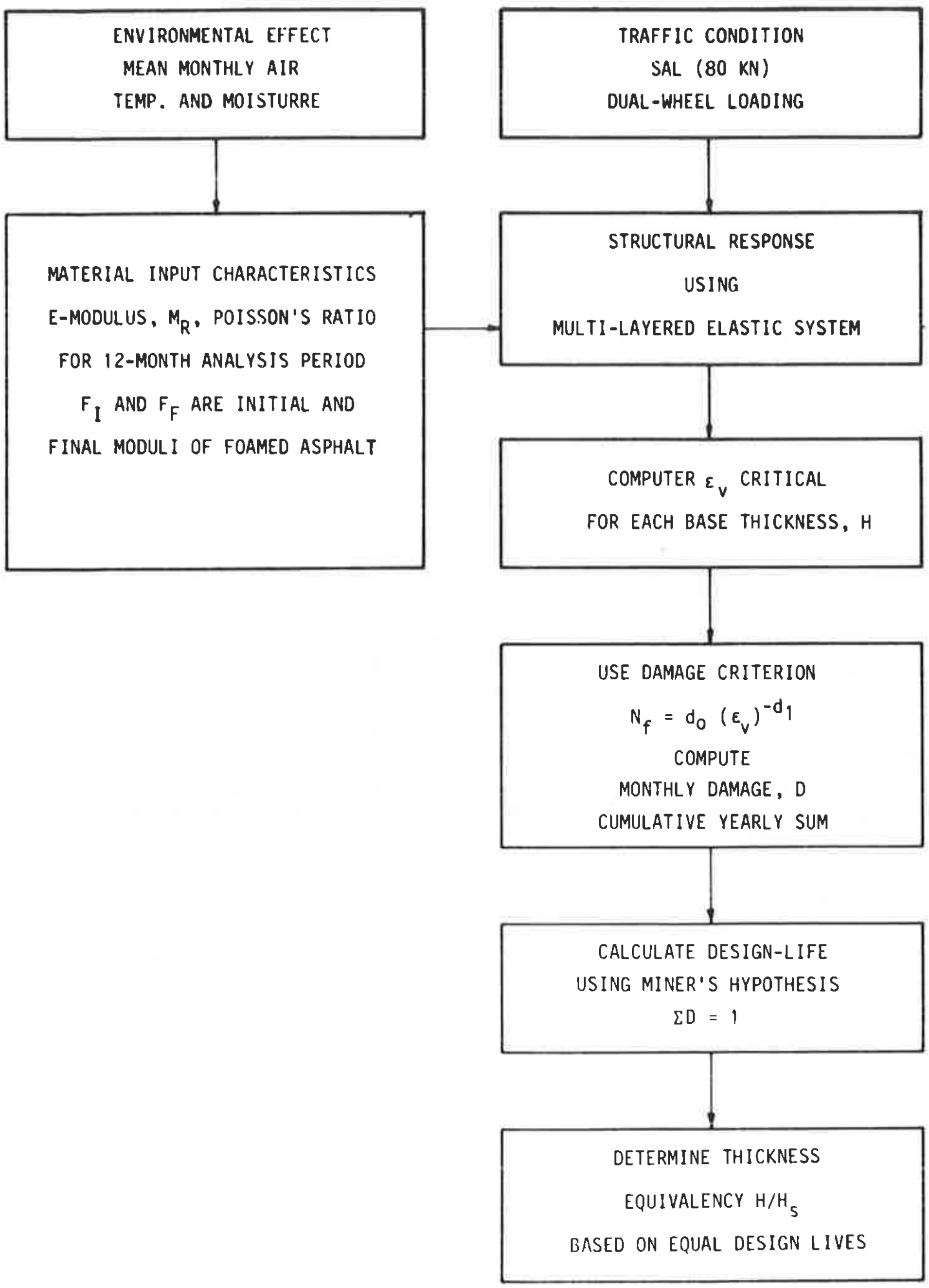

FIGURE 16 Procedure for determining thickness equivalency ratios based on subgrade deformation damage.

ulus is known to be stress dependent and is usually expressed as

$$E = K_1(\sigma_3)^{K_2} \tag{1}$$

where K_1 and K_2 are regression constants related to material type and physical properties and σ_3 is the minor principal stress (N/mm^{-2}). An indirect predictive equation, which was developed at the University of Maryland *(18)*, was used to account for the stress dependency in a simple manner.

Foamed-asphalt mixtures, like hot-asphalt mixtures, have modulus values that strongly depend on temperature. However, moduli of foamed asphalts differ in that they also are dependent on the degree of cure (loss of water). Thus, at any given temperature, the modulus will vary from some initial value (E_i) to the final modulus value (E_f) in a specified curing time. Analytically, this modulus relationship can be defined by *(19)*

$$T_{T,t} = E_{T,f} - (E_{T,f} - E_{T,i})\ (RF)_t \tag{2}$$

where

$E_{T,t}$ = modulus at temperature T and time t (N/mm^{-2}),
$E_{T,f}$ = final modulus at temperature T (N/mm^{-2}),
$E_{T,i}$ = initial modulus at temperature T(°C), and
$(RF)_t$ = reduction factor for cure at time t
= $e^{bt} = (1 - f_c)$

where

b = constant determined by the total cure period (t_c);
t = time at which RF is required (months); and
f_c = specified degree of cure, assumed to be 0.95.

The final modulus of foamed-asphalt specimens was achieved after curing outside the molds in air at 23°C for 21 days. Under field conditions, if it happened that the foamed-asphalt layer was overlaid with an asphalt concrete layer directly after construction, final modulus would be much delayed by the low degree of curing. In this analysis a total curing time of 6 months was assumed in order to achieve the final modulus value (Figure 17).

Figure 18 shows the thickness equivalencies of the foamed-asphalt layers in both pavement systems based on the subgrade damage criterion. That thickness equivalencies are a function of the geometrics of the pavement cross section and the stress distribution in the pavement system is indicated in Figure 18. Using the foamed-asphalt material as a base layer in Pavement 1 has resulted in average thickness equivalencies of 1.62, 1.00, and 0.77 compared with asphalt concrete, hot sand-asphalt, and sand-gravel base materials, respectively. However, in Pavement 2, foamed-asphalt subbase layers had average thickness equivalencies of 1.51, 0.97, and 0.60 compared with asphalt concrete, hot sand-asphalt, and sand-gravel materials, respectively.

Figure 19 shows the distribution of monthly damage to the four different subbase materials in Pavement 2 with equivalent layer thicknesses. Traffic is expected to start immediately after

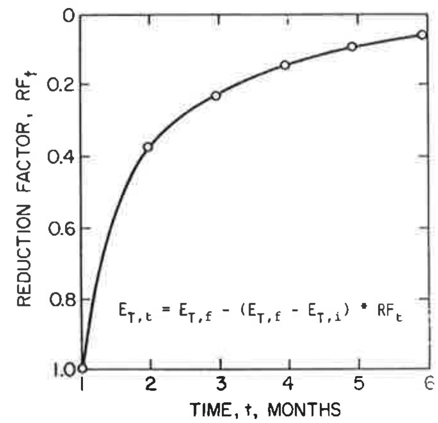

FIGURE 17 ***E*-modulus reduction factor versus curing time.**

construction in July. The relatively high damage units for the foamed asphalt during the first 2 months of curing are obvious. During the next 4 months of curing, the foamed asphalt exhibits about the same amount of damage as was computed for a sand-gravel layer of equivalent thickness. From May through September, when pavement service temperatures are highest, foamed asphalt exhibited less damage than did hot sand-asphalt and asphalt concrete layers of equivalent thickness.

Prediction of Permanent Deformation

The permanent deformation predicted within the foamed-asphalt layer was compared with that in an equivalent hot-asphalt layer for Pavements 1 and 2. With the predetermined thickness layer equivalency equal to 1.00, a thickness of 120 mm of foamed-asphalt base and subbase layer was considered. For the two candidate constructions, the permanent deformation was estimated according to the procedure outlined in the Shell Method *(20)*. There are eight steps to be followed in this procedure; the relevant quantities obtained in each step are given in Table 5.

Replacing hot-asphalt mix with foamed-asphalt mix has led to a reduction in the predicted permanent deformation from 8.30 to 5.40 mm for the base layer of Pavement 1 and from 8.25 to 4.20 mm for the subbase layer of Pavement 2. The replacement of hot-asphalt mix with a foamed-asphalt mix has resulted in a reduction in the predicted rut depth ranging from 35 to 50 percent during the whole service life of the pavement.

This analysis did not include the additional permanent deformation that may occur during curing of the foamed-asphalt layer if traffic is expected to use the road immediately after construction.

A lower asphalt grade was used in the foamed-asphalt mix than was used in the hot-asphalt mix. The reduction that occurred in the predicted permanent deformation could be related to the improved low stiffness properties of foamed-asphalt mixes at high service temperatures.

CONCLUSIONS

The foamed-asphalt process is specially suited to stabilizing locally available sands that contain a fines fraction (less than

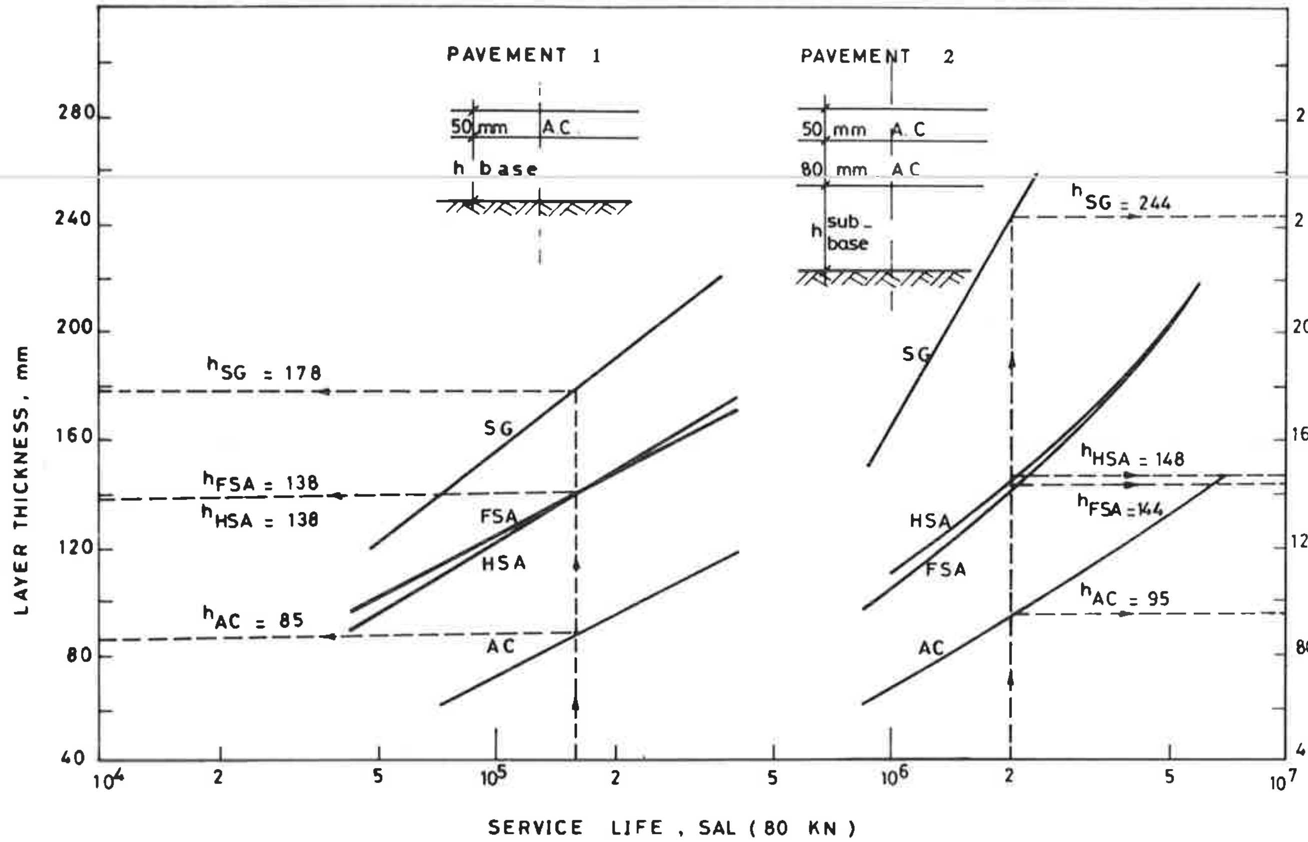

FIGURE 18 Equivalent layer thicknesses for different base and subbase pavement materials.

0.075 mm) at levels of 5 percent and above. The bitumen is concentrated in the finer fraction and forms a discontinuous random matrix of a cohesive asphalt mortar. This discontinuous nature of the foamed-asphalt binding mechanism of the sand aggregate results in mixtures that are much less temperature susceptible than are equivalent hot-asphalt plant mixtures.

The quality of sand is also important in determining its suitability for stabilization by foamed asphalt. The type and quality of fines are mainly responsible for the degree of moisture sensitivity of the foamed-asphalt mixtures. Coating sand particles with limestone powder in a slurry form initiates a subsequent bonding of the asphalt with the limestone surface rather than the surface of the original sand particles. This is quite similar to the effect of hydrated lime in improving the resistance of asphalt mixtures to moisture damage.

The effectiveness of the foaming process was found to be more pronounced at pavement service temperatures above 30°C. Foamed-asphalt mixes have higher tensile strength and resilient modulus values than those for corresponding hot-asphalt mixtures over the range of elevated service pavement temperatures.

Measured against the criterion of subgrade deformation

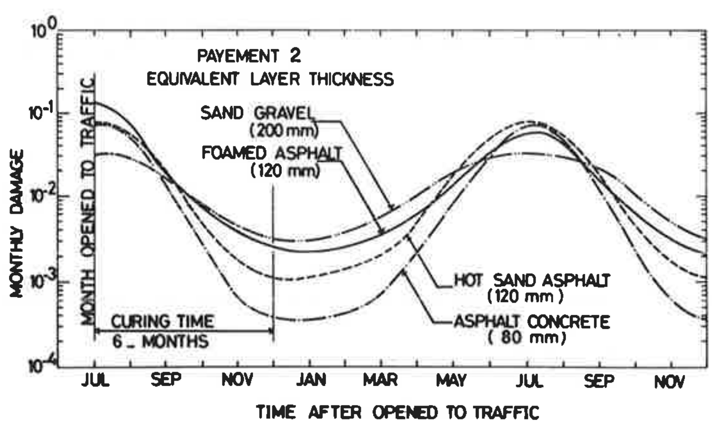

FIGURE 19 Distribution of monthly damage to different subbase pavement materials for Pavement 2.

TABLE 5 PREDICTION OF PERMANENT DEFORMATION AT MAAT = 31°C

Pavement System	$Visc_{eff}$ ($N\text{-}sec/m^2$)	T_{eff} (°C)	Creep Curve Slope (q)	E-Modulus (N/m^2)	W	$S_{bit,visc}$ (N/m^2)	S_{mix} (N/m^2)	Z	C_m	Δh (mm)
1[a]										
FSA	1.0×10^3	42	0.08	4.3×10^8	1.6×10^7	9.0×10^{-3}	8.0×10^6	0.3	2.0	5.40
HSA	2.0×10^4	42	0.18	3.8×10^8	4.5×10^5	6.6×10^6	4.2×10^6	0.3	1.6	8.30
2[b]										
FSA	1.5×10^3	40.8	0.08	4.5×10^8	1.6×10^8	1.4×10^{-3}	6.8×10^6	0.2	2.0	4.20
HSA	2.3×10^4	40.8	0.18	4.1×10^8	4.5×10^6	7.6×10^{-1}	2.8×10^6	0.2	1.6	8.25

NOTE: FSA = foamed sand-asphalt mixes, 4.0 percent by weight 310-pen bitumen; HSA = hot sand-asphalt mixes, 6.5 percent by weight 67-pen bitumen.
[a]SAL = 2.2×10^5.
[b]SAL = 2.2×10^6.

damage, foamed-asphalt base and subbase layers under the assumed local curing conditions are superior to unbound materials such as sand-gravel mix. At the local prevailing temperatures (MAAT = 31°C) foamed-asphalt mixes are structurally equivalent to corresponding hot sand-asphalt mixes. This indicates that, because of significant cost savings, sand stabilization with foamed asphalt could be an attractive alternative to either conventional hot-asphalt mixes or granular base materials.

At the local high pavement temperatures, cured foamed-asphalt base layers showed higher resistance to permanent deformations than did corresponding hot-asphalt layers. This is related partly to the improved low stiffness properties of the foamed mixes and partly to the relatively low asphalt content.

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Sulfur as a Partial Replacement for Asphalt in Pavement

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A full-scale experiment on introducing sulfur as a partial replacement for asphalt in road-paving mixtures was conducted in Kuwait. Laboratory and field testing programs were designed and carried out to analyze and investigate the results of this experiment. The optimum sulfur percentage in the sulfur-asphalt binder and the optimum binder content were established for the investigated mixtures at various curing ages. These mixtures were used in constructing two sections, each 0.67-mi long, of the driving lane of a main road. The first one, considered a control section, was constructed using the conventional asphalt concrete mixture. The second was the test section in which the sulfur-asphalt mixture was used in the entire pavement structure. The sulfur-asphalt mixture was found to significantly reduce rutting on the road. This was established in both field and laboratory investigations. Laboratory-aged samples showed longer fatigue life (indirect tensile stress control fatigue test). Inclusion of sulfur in the mixture was also shown to improve Marshall stability, indirect tensile strength, and the rate at which strength was gained during curing. On the other hand, including sulfur made the mixture more susceptible to loss of stability on immersion in water and to having a lower resilient modulus at higher temperatures (above 95°F). Field Dynaflect measurements showed higher deflections for the road test section than for the control section.

Interest in using sulfur in flexible pavement mixtures usually stems from one or both of two considerations: economy and improvement in road mixture characteristics. The first consideration reflects the availability and price of sulfur and asphalt, the percentage of sulfur used in the overall binder, and the optimum binder content of the mixture. The cost of necessary modifications to mixing plants in order to produce the new mixture is relatively minimal because of recent improvements in mixing techniques.

The second consideration, improvement in characteristics of mixtures that contain sulfur as a partial replacement for asphalt (SAPRA), needs more investigation in general and in individual projects. This is because of the different experiences and conclusions reported in the literature, particularly in regard to durability and fatigue resistance of SAPRA mixtures.

Many researchers [e.g., Gallaway and Epps *(1)*] have consistently reported that SAPRA road mixtures possess higher Marshall stability than do conventional mixtures and that this improvement becomes more noticeable with age. However, another study *(2)* reported lower Marshall stability for a SAPRA mixture after 2 years compared with a corresponding conventional mixture. A similar situation obtains for changes in the measured resilient modulus caused by using SAPRA mixtures. Meyer et al. *(3)* and Fromm and Kennepohl *(4)* separately measured higher resilient moduli for SAPRA mixtures with less susceptibility to loss of stiffness at higher temperatures. However, this was not observed by others *(5)*.

Research findings about temperature- and fatigue-related cracking also differ. Many field studies reported slightly higher susceptibility of SAPRA mixtures to cracking [e.g., Munoz *(6)*] because of the increased brittleness of these mixtures caused by the presence of sulfur. Other studies [e.g., Kennedy et al. *(7)* and Meyer et al. *(3)*] reported better fatigue life expectancy for the SAPRA pavements on the basis of laboratory fatigue tests.

These differences in conclusions among various research efforts may be attributed to factors related to the technique of sulfur inclusion in the mixture, mix design considerations, types of materials, environmental conditions, and testing and evaluation procedures and criteria.

The SAPRA road mixtures were found, almost unanimously, to be less susceptible to rutting. This advantage provided by SAPRA is especially important where heavy slow-moving wheel loads prevail in warm climatic conditions.

A pilot project was constructed in Kuwait to test SAPRA application. A laboratory testing and design program was conducted, complemented by a follow-up field testing and evaluation plan, to investigate the performance of the introduced mixture. The project consisted of two 0.62-mi sections of a driving lane on a major six-lane highway. The first 0.62-mi section was constructed as a control section using conventional asphalt concrete. SAPRA mixtures were used in the construction of all pavement layers in the other 0.62-mi test section of the driving lane. The pavement structure was the same in both sections. It consisted of a 1.6-in. wearing surface (Type III), a 2.4-in. binder course (Type II), a 3.9-in. base course (Type I), and a 3.9-in. sand mix subbase course conforming to the Kuwait ministry of public works specifications *(8)*.

The purpose of this paper is to present the results of the laboratory and field research conducted on this project.

EXPERIMENTAL PROGRAM

The experimental program can be divided into two phases, laboratory and field. The laboratory phase was conducted in two stages. The first stage included preliminary studies to establish mix design and procedure and thus determine optimal mixing technique, optimum sulfur percentage in binder, and optimum binder content. Marshall mix design and stability were used as the comparison criteria. Marshall stability tests were run on samples after curing (setting) periods of 1, 3, and

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10 days. Asphalt cement of 60/70 penetration and 99 percent pure sulfur were used in the mix with crushed limestone aggregate gradation (Type III) as shown in Figure 1. Liquid sulfur, at 280°F, was thoroughly premixed with asphalt, at 300°F, for 30 sec to form sulfur-extended asphalt binder. This binder was then mixed with preheated aggregate, at 320°F, for another 30 sec. This method of preparing SAPRA mixtures was used in both the field and the laboratory.

In the second stage the experimental performance of the SAPRA binder and SAPRA concrete mixture was compared with that of asphalt cement and conventional asphalt concrete, respectively. The following laboratory tests were performed:

1. Penetration (ASTM D 5), viscosity (ASTM D 2171), and softening point (ASTM D 2398). These tests were run for sulfur-to-asphalt ratios of 0/100, 15/85, 30/70, 40/60, and 50/50 at 1, 3, 10, 30, and 90 days of age.
2. Effect of water submersion on strength of SAPRA mixtures (ASTM D 1075). Two sulfur-to-asphalt ratios, 0/100 and 40/60, were used for mixture preparation. At each ratio, three aggregate fillers were used: 7 percent limestone filler (LF), 6 percent LF plus 1 percent hydrated lime (HL), and 7 percent LF plus 2 percent portland cement (PC) premixed with aggregate. Adding 1 percent HL or 2 percent PC, or both, improved mix resistance to water damage *(9)*. These additives were used to study their effect in the presence of sulfur.
3. Fatigue life tests on laboratory-prepared samples. Samples, 4 in. in diameter by 2.5 in. thick, were prepared of mixtures with sulfur-to-asphalt ratios of 0/100, 15/85, and 40/60 and tested after 3, 7, and 30 days of curing. They were tested under 1-Hz haversinusoidal dynamic indirect tensile stress until failure.
4. Laboratory creep tests on multilayered cores obtained from the field. Deformation measurements were made separately for each layer. These tests were performed at room temperature (77°F).
5. Laboratory resilient modulus tests on multilayered cores obtained from the field. Deformation measurements were made for single pavement layers in the cores. These tests were run at various temperatures in the range of 77°F to 131°F and under haversinusoidal dynamic load at frequencies of 1 and 8 Hz.

Details of test procedures for fatigue, creep, and resilient modulus are given elsewhere *(9, 10)*.

Field experimentation and measurements involved

1. Field rutting measurements at two intersections in the test lane, one on the control section and the other on the section in which SAPRA mixtures had been used. The measurements were taken 1 year after the road was opened to traffic.
2. Dynaflect deflection measurements during the year after construction; pavement surface temperature varied from 41°F to 131°F.

ANALYSIS AND DISCUSSION OF DATA

Rheology of SAPRA Binder

The standard penetration test results (Table 1) did not show a distinct trend of variation in the penetration values with sulfur-

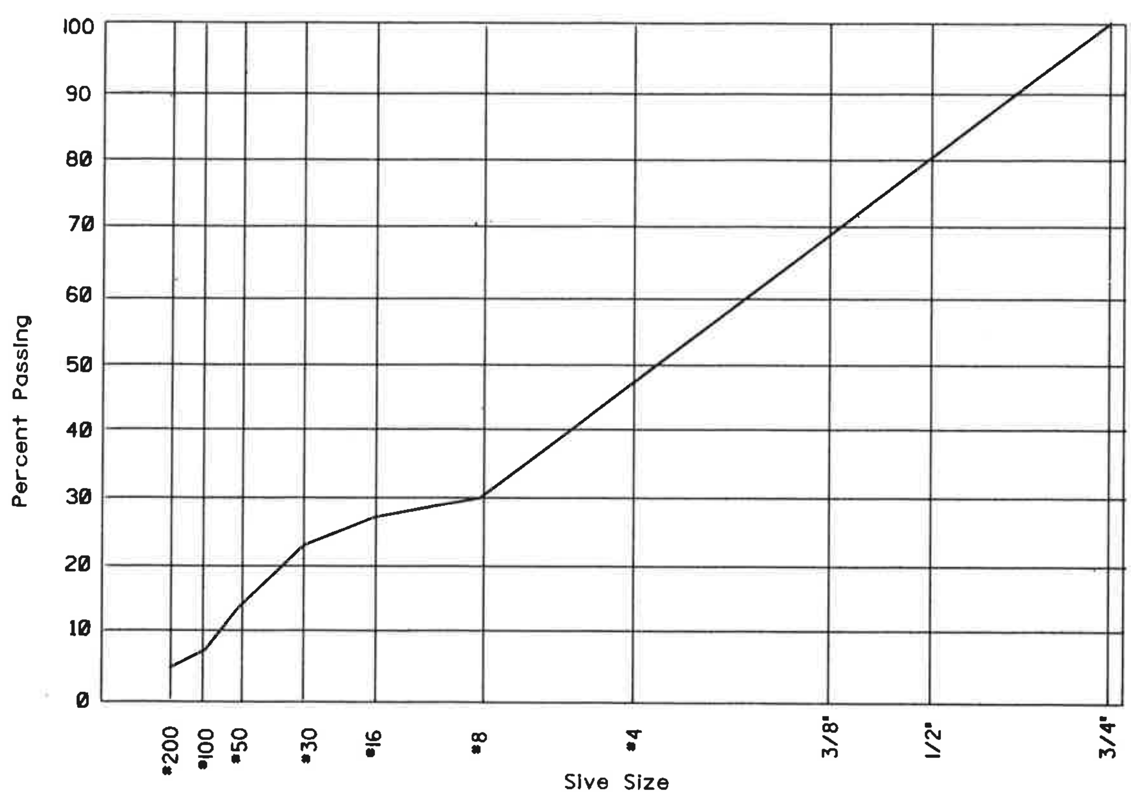

FIGURE 1 Aggregate gradation for surface course (Type III).

TABLE 1 STANDARD PENETRATION TEST RESULTS

S/A Ratio	Age 1 Day	3 Days	10 Days	1 Month	3 Months
0/100	60	53	53	52	33
15/85	81	90	70	56	60
30/70	51	56	75	82	41
40/60	65	55	109	97	91
50/50	61	72	106	89	61

to-asphalt (S/A) ratio or binder age. This might be because it was not practically possible to obtain a homogeneous binder mix at the standard test temperature of 77°F. At high temperatures, the free sulfur globules settled with time and formed sulfur crystals during cooling; this created an inhomogeneous binder. In that case, the measured penetration value normally was not reproducible and depended on the existence of sulfur crystals at or near the penetration needle. Therefore the results were inconclusive and the penetration test, in its standard format, is neither indicative of nor suitable for the needed comparison.

The viscosity tests were run at 140°F. The results are shown in Figure 2. The inclusion of sulfur decreased the binder's viscosity until the dissolving point of sulfur was reached; beyond that point viscosity was higher for higher S/A ratios. The dissolving point of sulfur is the maximum percentage of sulfur that dissolves in asphalt cement. In the present study it was an S/A ratio of approximately 20/80. Sulfur added beyond this ratio existed in the SAPRA binder in the form of free sulfur globules. Similar results have been reported by other researchers *(11)*.

Results of softening point tests are given in Table 2. The results showed that SAPRA binders had lower softening points than did pure asphalt cement. However, it appeared that the S/A ratio did not significantly affect the amount of decrease in the softening point.

TABLE 2 SOFTENING POINT TEST RESULTS

S/A Ratio	Age 1 Day	3 Days	10 Days	1 Month
0/100	55	55	53	57
15/85	48	49	50	50
30/70	57	52	50	50
40/60	50	51	47	50
50/50	55	51	51	49

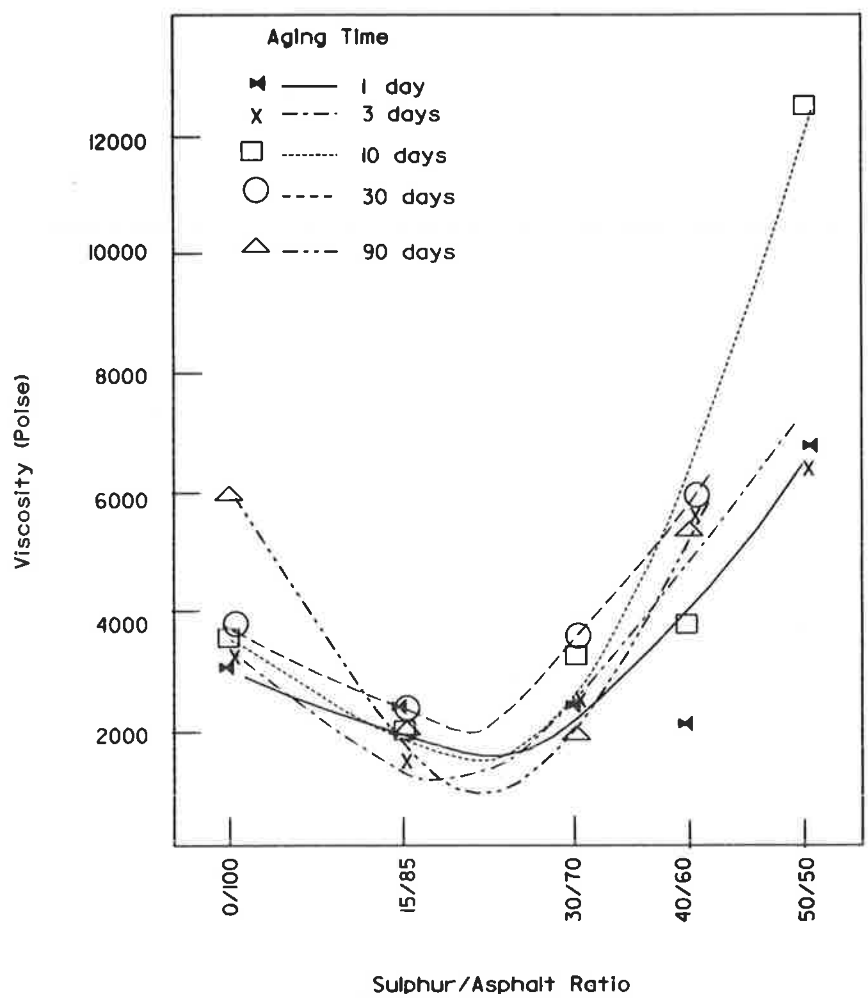

FIGURE 2 Viscosity of SAPRA binder at 140°F.

TABLE 3 SUMMARY OF MARSHALL MIX DESIGN FOR DIFFERENT S/A RATIOS

Mix Property	Curing Time (days)	S/A Ratio 15/85	30/70	40/60	50/50
Optimum binder content (% by weight)	1	5.3	5.5	5.8	6.0
	3	5.2	5.4	5.7	5.7
	10	5.1	5.2	5.6	5.6
Marshall stability (lb)	1	2,100	2,650	3,000	4,700
	3	2,500	2,650	4,000	4,900
	10	2,950	3,240	4,300	5,000
Bulk specific gravity	1	2.390	2.413	2.390	2.400
	3	2.395	2.370	2.400	2.378
	10	2.399	2.363	2.390	2.370
Void ratio (%)	1	2.6	2.1	3.2	4.1
	3	2.4	3.9	3.3	4.3
	10	1.8	3.4	3.5	5.2
Flow (0.01 in.)	1	20	17	19	25
	3	22	17	21	22
	10	22	19	24	24

Mixture Design

Table 3 is a summary of the Marshall mix design results for the SAPRA mixtures with S/A ratios of 15/85, 30/70, 40/60, and 50/50, respectively. Figure 3 shows a typical set of curves obtained for the 15/85 S/A ratio. In recognition of the importance of the effect of curing time *(5)*, the design curves were obtained at 1, 3, and 10 days. Optimum values of mix design parameters after 1 day are shown in Figure 4. The following observations were made about the mix design results (Table 3):

- The optimum binder content for maximum Marshall sta-

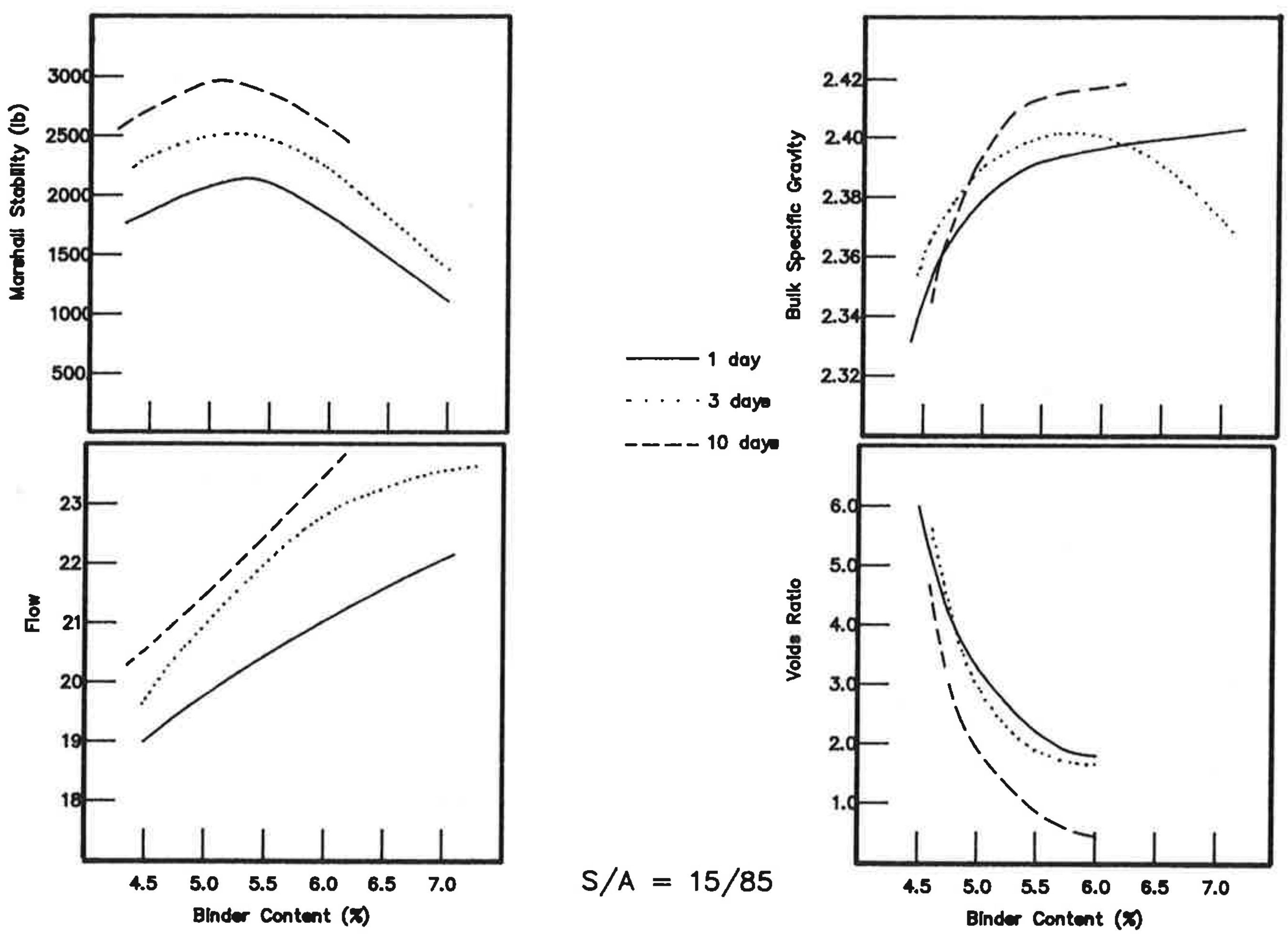

FIGURE 3 Marshall mix design curves for S/A = 15/85.

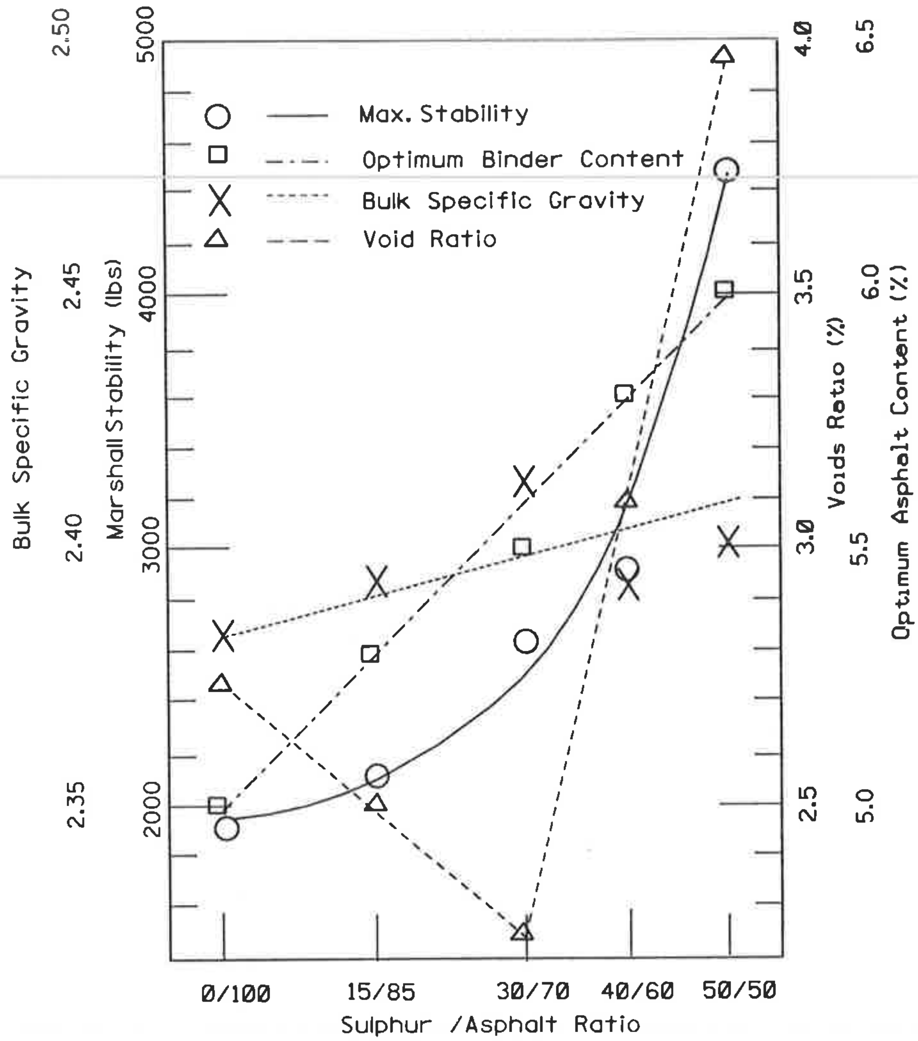

FIGURE 4 Optimum values of mix design at 1 day.

bility was higher for higher S/A ratios (Figure 4). It increased from 5 percent for a ratio of 0/100 to 6.0 percent for a 50/50 ratio. However, the optimum binder content tended to be lower than Marshall mix design might require for more aged SAPRA mixtures.

- The 1-day maximum Marshall stability values increased with increasing S/A ratio. These values also increased with the age of the SAPRA mixtures for all S/A ratios investigated. The rate of this increase, however, was different for various ratios; the highest rate was observed at 40/60.
- The flow of the samples appeared to increase when the S/A ratio was above 30/70. The SAPRA mixture with S/A = 15/85 showed lower flow values. The relationship between flow and binder content, however, did not appear to maintain a particular trend for various S/A ratios or mixture ages.
- There was no discernible trend of variation in the bulk specific gravity or in the voids ratio with age of the mixture.
- There was a different trend of variation in the values of the void ratio, at 1-day optimum binder contents, below and above S/A = 30/70 (Figure 4). Although it was lower up to S/A = 30/70, it shows a shift with significant increase above this ratio. This might indicate higher susceptibility of the SAPRA mixture, with an S/A ratio above 30/70, to rutting. On the other hand, this could be offset by the stiffness contributed by the increased sulfur content of the mixture.

Indirect tensile strength was measured on samples prepared at optimum corresponding binder contents for S/A ratios of 0/100, 15/85, and 40/60. The samples were tested under a load applied parallel to and along a vertical diametral plane that produced a tensile stress perpendicular to the direction of the applied load plane. This tensile stress ultimately caused the specimen to fail by splitting along the vertical diameter. Tensile strength (S) was computed as

$$S = 2P/\pi Dt \tag{1}$$

where

P = vertical load at failure,
t = thickness of the sample, and
D = diameter of the sample.

The results of the test are shown in Figure 5. Indirect tensile strength increased with increase in the S/A ratio.

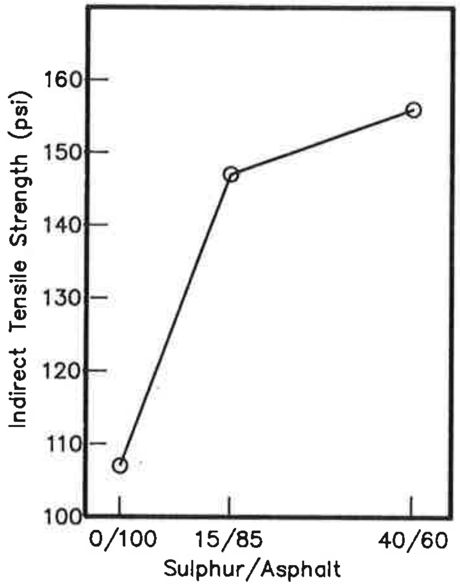

FIGURE 5 Indirect tensile strength of SAPRA mixtures.

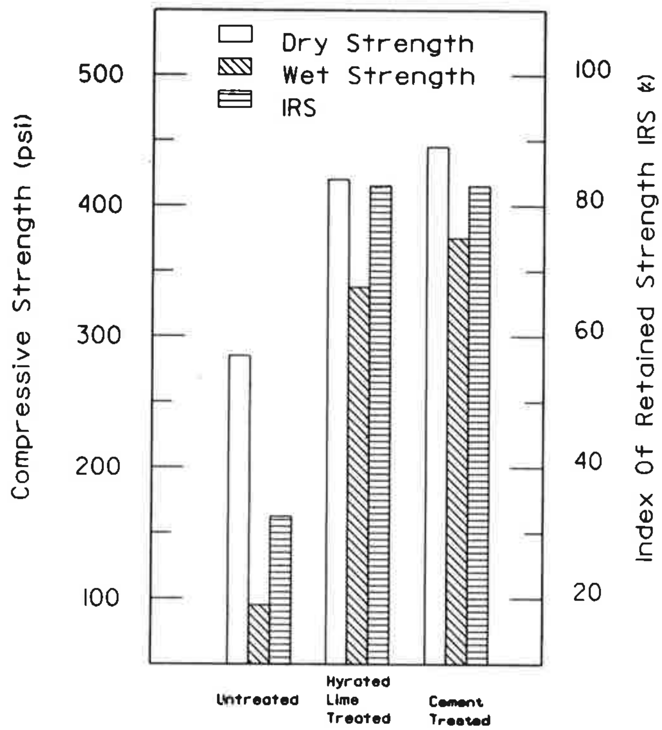

FIGURE 6 Dry and wet compressive strength of SAPRA mixtures (S/A = 40/60).

Effect of Submersion in Water

Sample groups to be subjected to the standard test for the effect of water on cohesion of compacted bituminous mixtures (ASTM D 1075) were prepared with binder content of 5.8 percent for an S/A ratio of 40/60. The results are shown in Figure 6. The following observations were made:

- The cement-treated mixture provided the highest dry and wet compressive strength, followed by the hydrated lime–treated mixture.
- The index of retained strength (*IRS*) defined as

$$IRS, \% = (C2/C1) \times 100 \qquad (2)$$

where $C1$ is compressive strength of dry specimen and $C2$ is compressive strength of immersed specimen. *IRS* is higher for the cement-treated mixture (83 percent) and for the hydrated lime–treated mixture (82 percent) than for the untreated mixture (31 percent).
- The SAPRA mixture was more prone to moisture-induced damage than were conventional mixtures. This can be seen in Figure 6. The *IRS*-values for the SAPRA mixture tend to be at the lower side of the corresponding range for conventional mixtures.

Resilient Behavior

Resilient Modulus

The values of resilient modulus were determined for field cores by two methods, creep test (assuming viscoelastic behavior of mixtures) and dynamic loading. The first testing method consistently resulted in lower values for resilient modulus in the range of from 30 to 60 percent compared with the corresponding values produced by the second testing method. Only the values of resilient modulus produced by dynamic testing will be discussed further in this paper.

Figures 7–9 facilitate the comparison of the values of resilient modulus of SAPRA and conventional mixtures at two dynamic load frequencies and over a range of road surface temperatures for the surface course (Type III), binder course (Type II), base course (Type I), and sand mix subbase materials, respectively. The following observations can be made:

- The resilient modulus is higher for all SAPRA mixtures than for conventional mixtures for both load frequencies in the low-temperature range of 74°F to 90°F. The only exception was the sand mix in which the SAPRA mixture showed inferior behavior. However, no specific conclusion can be drawn with respect to the sand mix because the available results are only for field cores of sand mix subbase. No laboratory samples of this layer were tested.
- The rates of temperature softening of SAPRA mixtures were higher than those of conventional mixtures. This led to equal or higher values of the resilient modulus for conventional mixtures at higher temperatures. This observation may not agree with previous reports *(4, 7)*. However, it provided an explanation for the behavior of the road mixtures under field Dynaflect testing, which will be discussed in a later section of this paper.

Fatigue Life

Laboratory fatigue tests were performed at 77°F with load control mode under different load intensities. The results are shown in Figure 10 in the form of the stress versus fatigue life

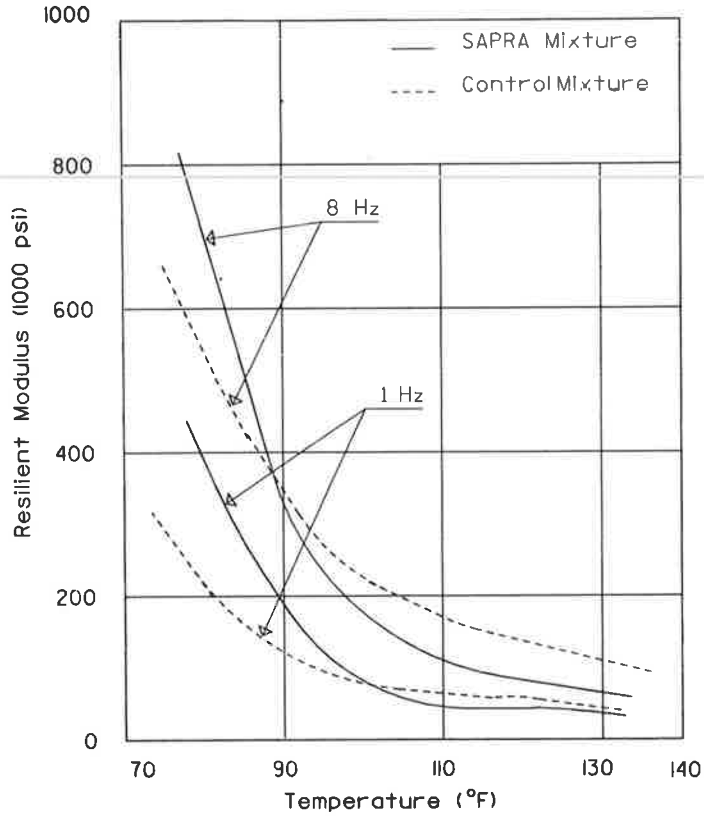

FIGURE 7 Resilient modulus of SAPRA and control mixtures, field cores of surface course.

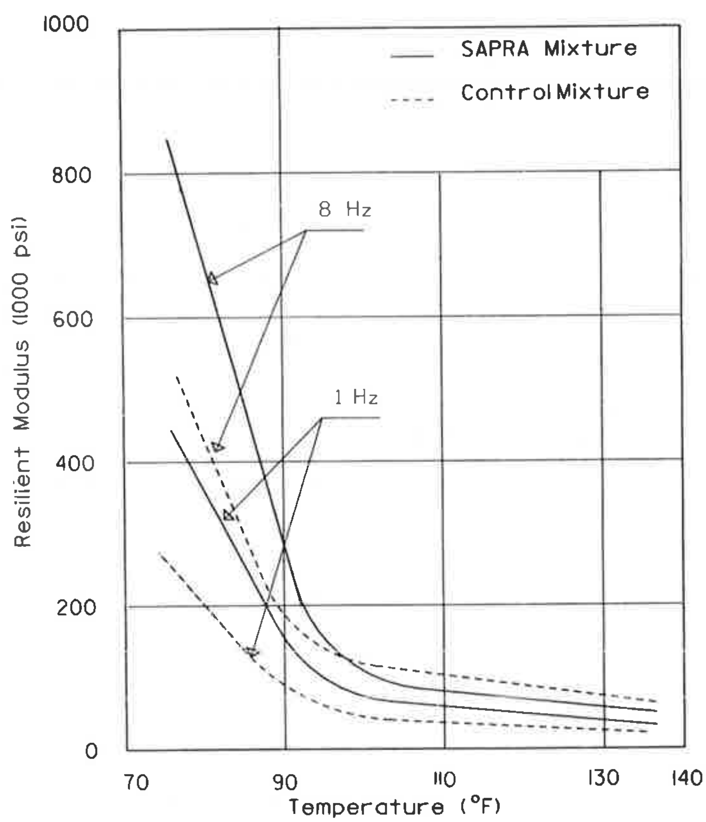

FIGURE 8 Resilient modulus of SAPRA and control mixtures, field cores of binder course.

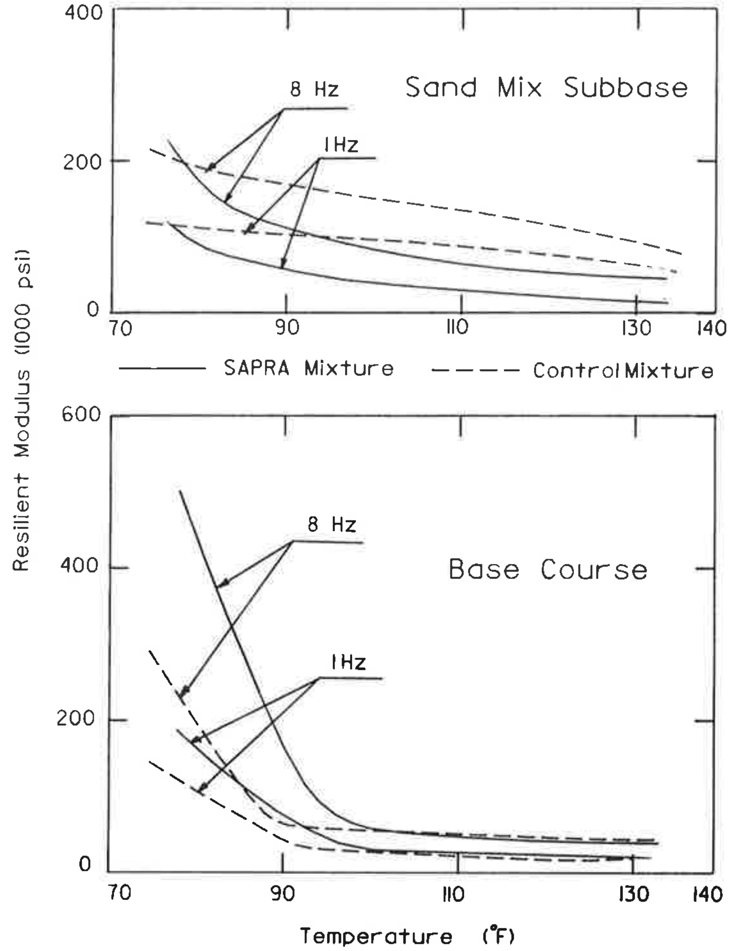

FIGURE 9 Resilient modulus of SAPRA and control mixtures, field cores of base course and subbase sand mix.

relationship (S-N diagram). The following observations can be made:

- Fatigue life increased with curing time. The rate of increase appeared to be variable and to depend on the stress level and percentage of sulfur in the mixture. In general, the rate of increase in fatigue life is higher for the SAPRA mixtures.
- After 3 days of curing, the conventional mix exhibited longer fatigue life than did the SAPRA mixtures. As the curing period was extended, the SAPRA mixtures showed longer fatigue life. The difference was more pronounced at lower stress levels.
- The results appear to suggest that there is an optimum sulfur content at which maximum fatigue life is obtained after a "reasonable" curing period. This might be apparent from the better fatigue life performance of the S/A = 15/85 mixture compared with the S/A = 40/60 mixture.

Field Deflection Measurements

Dynaflect resilient deflection measurements were taken on the test and control sections of the experimental field project. Five deflection geophone readings were taken at the test load ($W1$) and at 12-in. intervals thereof along the deflection basin ($W2$, $W3$, $W4$, and $W5$, respectively). The following deflection parameter definitions are used in presentation of the data:

- Maximum deflection ($W1$) mainly reflects the behavior of the overall pavement structure; higher deflection means weaker pavement.
- Surface curvature index [$SCI = (W1 - W2)$] mainly represents the structural characteristics of the upper portion of the pavement; higher values indicate weaker surface layer or layers.
- Spreadability [$SP = (W1 + W2 + W3 + W4 + W5) \times 100/5\ W1$] is an indicator of the relative stiffness of the pavement components and the load distribution characteristics of the pavement system; higher values (closer to 100 percent) indicate more uniform pavement.

Figure 11 shows the variation in the representative values of the deflection parameters with road surface temperature for both the test and the control sections of the road. Statistical regression analysis led to the following relationships, which represent the test results for the SAPRA test section:

$$\log W1 = -0.4586 + (8.7 \times 10)\ T^2 \qquad R = 0.93 \qquad (3)$$

$$\log (SCI \times 100) = 0.6732 + 0.00022\ T^2 \qquad R = 0.90 \qquad (4)$$

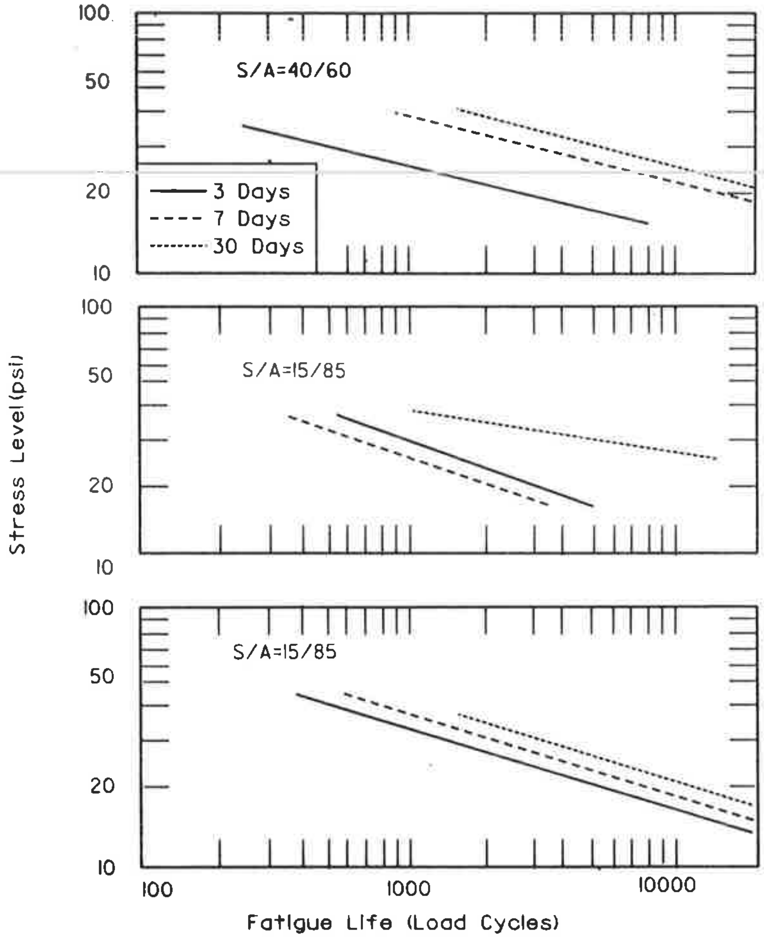

FIGURE 10 Fatigue test results for various S/A ratios.

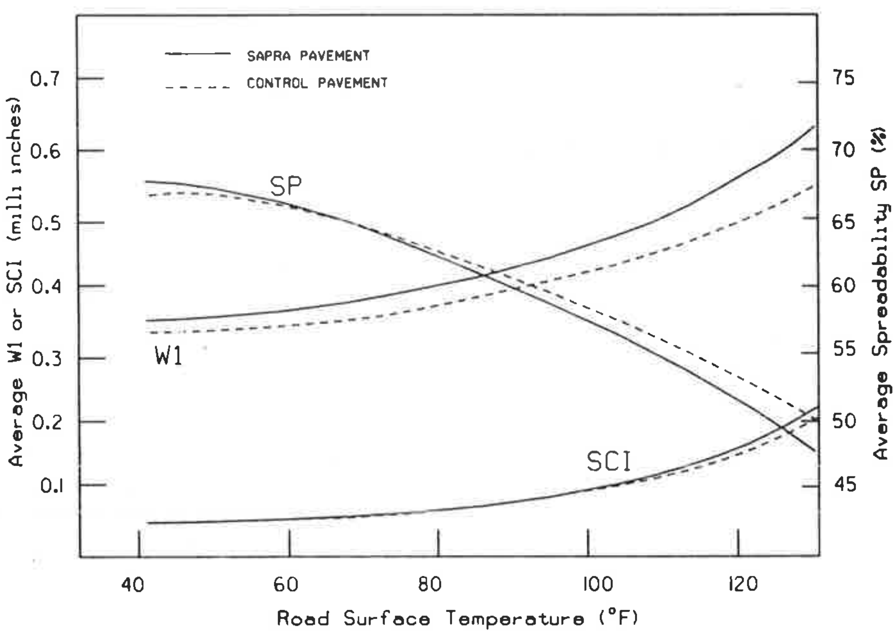

FIGURE 11 Variation of *W1*, *SCI*, and *SP* with road surface temperature.

$$\log SP = 1.8330 - (4.97 \times 10)\, T^2 \qquad R = 0.93 \qquad (5)$$

Those for conventional control sections are

$$\log W1 = -0.4825 + (7.30 \times 10)\, T^2 \qquad R = 0.81 \qquad (6)$$

$$\log (SCI \times 100) = 0.6802 + 0.00021\, T^2 \qquad R = 0.91 \qquad (7)$$

$$\log SP = 1.8278 - (4.31 \times 10)\, T^2 \qquad R = 0.94 \qquad (8)$$

where

T = (5/9) (Tf − 32),
Tf = surface temperature (°F), and
R = statistical correlation coefficient.

The SAPRA test section showed higher deflections ($W1$) than those measured for the control section of the road. This might indicate an overall weaker SAPRA pavement that might result in earlier fatigue cracking in the test section. However, 1 year after construction, this had not happened.

Both sections of the road produced almost identical values of *SCI* except in the high-temperature range in which the SAPRA pavement exhibited higher *SCI* indicating weaker behavior of the surface layer or layers. However, the difference in this range was not considered significant. The lower values of the spreadability *(SP)* of the SAPRA section at high temperature indicated more nonuniformity in that pavement. This might be attributed to the high-temperature softening susceptibility of the subbase sand mix. That is, the poor performance of the subbase, according to the laboratory results, might be a major contributor to the overall inferior performance of the SAPRA pavement.

These observations lead to the conclusion that, with the exception of the sand mix layer, SAPRA mixtures performed almost as well as or insignificantly less well than conventional mixtures from the standpoint of resilient behavior at high temperature. At lower temperature levels, the performance of SAPRA and conventional pavements was virtually the same.

Residual Behavior

Creep Under Static Loads

Table 4 gives the results of a representative creep test in which both SAPRA and conventional mixtures were under the same load at 77°F. The results clearly show superior performance of the SAPRA mixtures. They exhibited between 15 and 48 percent less deformation than did conventional mixtures.

Permanent Deformation in the Field

Two sites were selected for comparison of permanent deformation, a signalized intersection along the control section and a signalized intersection along the test section of the same driving lane. At intersections the permanent deformation of a pavement surface is the sum of the accumulated static creep due to stopped vehicles and rutting (dynamic creep) due to moving vehicles. The general nature of the static creep and dynamic creep curves is similar *(9)*. However, the mechanism and the material behavior are different in each case. It was practically impossible to differentiate between the two mechanisms in the field measurements. Therefore the objective at this stage of the study was limited to comparing the overall permanent deformation of SAPRA and conventional mixtures.

TABLE 4 CREEP STRAINS (10^{-6} in./in.) FOR FIELD CORES (77°F)

Course	SAPRA Mixture	Conventional Mixture
Surface (Type III)	587	1,123
Binder (Type II)	341	403
Base (Type I)	270	511

Figure 12 shows permanent deformation profiles in the left and right wheel track before and partly through the two intersections. It was noticed that the control section had suffered significant permanent deformation whereas the SAPRA test section showed much less deformation. Examination of cored samples from both sections indicated that most of the permanent deformation of the pavement occurred in the binder and base courses, with minor residual deformation in the surface course and minimal change in thickness in the subbase and subgrade layers. In other words, it appeared that most of the permanent deformation of the pavement could be attributed to the bituminous mixtures of the binder and base course layers.

Permanent deformation was considerably higher at the 33-ft spot where the rear wheels of a loaded truck are normally when the truck is stopped at the signalized intersection. Therefore it appeared that static creep might have contributed more to the overall permanent deformation at that spot. The SAPRA pavement showed more resistance to this heavy static loading than did the conventional pavement.

CONCLUSIONS

The following conclusions about SAPRA in flexible pavement, within the limitations and range of the present study, can be drawn.

1. SAPRA pavement projects need individual project study that reflects the particular conditions of the project.
2. SAPRA mixtures were found to have the following advantages over conventional mixtures: (a) lower susceptibility to permanent deformation, (b) higher optimum Marshall stability and a higher rate of stability gain with curing time, (c) higher indirect tensile strength, (d) higher resilient modulus at lower temperatures (73°F to 91°F), (e) longer fatigue life after extended curing time and at the optimum S/A ratio, and (f) equivalent field deflection performance.
3. SAPRA mixtures were found to have the following disadvantages or concerns that need further study: they are (a) more prone to water immersion damage and (b) more susceptible to

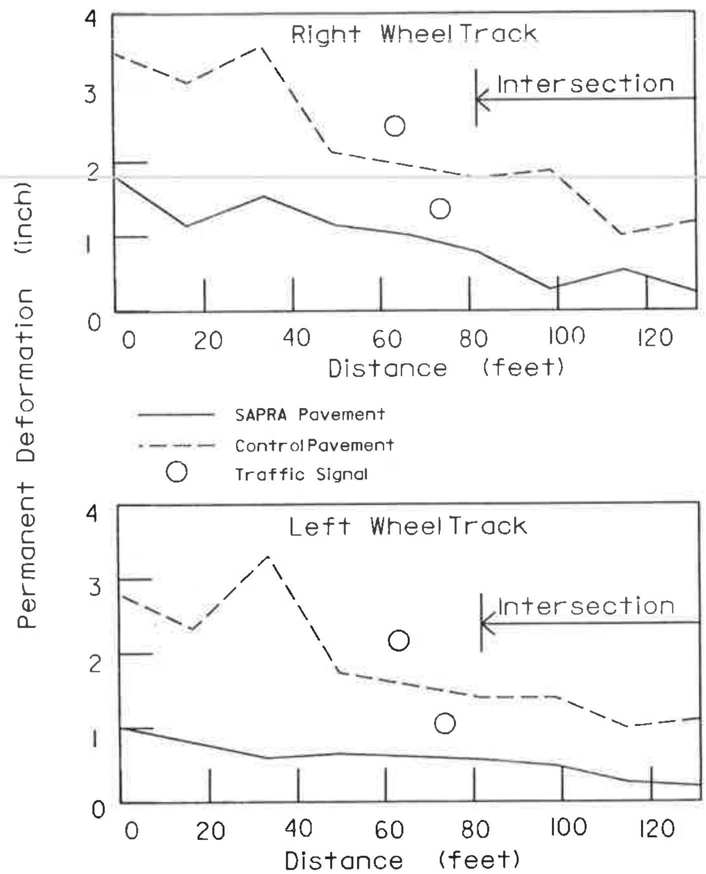

FIGURE 12 Permanent deformation measurements.

loss of resilient strength at higher temperatures. However, this conclusion is based only on tests performed on field cores. More laboratory tests are recommended to reach a firm conclusion.

4. The penetration test in its standard format was unsuitable for investigation of SAPRA binders. Viscosity tests provided better results for identification and comparison of SAPRA binders.

5. The optimum binder content was higher for higher S/A ratios. However, it tends to be less when mix design curves at extended curing periods are considered.

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Field Evaluation of Sulfur-Extended Asphalt Pavements

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Twenty-six sulfur-extended asphalt (SEA) paving projects, constructed between 1975 and 1982 in 18 states, were surveyed to measure the incidence and severity of major, visible types of pavement distress. Present condition indices were calculated for each of the SEA pavements and for each pavement in a control group of closely matched, conventional asphalt concrete pavements. Analysis of the evaluation results indicates that the presence of 20 to 40 percent by weight of sulfur in the paving binder had no deleterious effect on the overall performance of SEA pavement but yielded not significant improvement compared with the control group. Within the limits of the analysis, the measured types of distress and their severity were not significantly affected by variation in the sulfur content of the paving binder.

The term "sulfur-extended asphalt" (SEA), applied to a paving binder, paving mixture, or pavement, denotes the replacement of a significant portion of the conventionally used asphalt with elemental sulfur. Typically, 20 to 40 percent by weight of the asphalt is replaced with sulfur.

The initial development of SEA technology was carried out in the early 1970s by several Canadian petroleum companies that had accumulated millions of tons of elemental sulfur recovered from sour natural gas. Interest in the United States in the SEA technology was spurred by the 1974 and 1979 disruptions of the petroleum market. Public and private agencies saw SEA as a possible means of conserving petroleum stocks through a reduction in asphalt usage in highway construction. At the time, elemental sulfur was considerably less expensive than asphalt cement; thus a savings in construction costs was also anticipated.

Laboratory research on SEA-aggregate mixtures [see, for example, Saylak and Conger *(1)* and the references therein] had indicated that significant benefits might be realized from the use of SEA paving mixes. Of particular interest was the possibility of employing a softer grade of asphalt than would normally be used in a particular situation in conjunction with the sulfur. At high ambient temperatures the sulfur was expected to stiffen the binder sufficiently to resist deformation, but at low temperatures the asphalt properties, which reduce the probability of thermal cracking, would predominate.

Interest in the potential cost savings and engineering benefits of SEA binders led to the construction of more than 75 SEA pavement projects in the United States between 1975 and 1985. These projects involved new construction as well as overlays on existing pavements. A variety of methods of blending sulfur and asphalt were employed *(1, 2)*. Usually liquid (molten) sulfur was used, but some later projects investigated the introduction of the sulfur in solid, prilled form. SEA projects were built in every U.S. climatic zone and on all types of highway facility, from farm-to-market roads to Interstate highways.

In the 1980s the price of sulfur on the world market rose sharply, and surplus supplies were drawn down. The economic incentive for SEA use disappeared, and there was little information available on improved pavement performance to encourage the use of SEA.

Recently, increased attention has been paid to the use of modifiers to enhance the performance of asphalt paving mixes. As a potential asphalt modifier, sulfur is unique in that it has been extensively tested in pavement construction and is relatively inexpensive (as a modifier or additive, not as an extender). A comprehensive evaluation of existing SEA pavement performance, which would serve as a basis for estimating the effects of lesser amounts of sulfur added to paving mixtures as a modifier, is lacking. Although many of the SEA projects have received some degree of postconstruction evaluation, there has been no organized study of the overall performance of SEA pavements in the United States.

In 1985 the FHWA organized a task force to conduct a comprehensive SEA field evaluation study. The study objectives were to compare the field performance of a representative group of SEA pavements with that of a control group of conventional asphalt concrete (AC) pavements and determine what differences in performance and durability existed between the two groups. This paper is a summary of the important results and conclusion of that study. For a complete account of the experimental procedures, results, and analyses, the reader is referred to the full FHWA report *(2)*.

PROJECT SELECTION

Available information on the 75 SEA projects in the United States, including construction reports, postconstruction evaluations, and similar material, was reviewed to identify a representative set of projects for detailed evaluation by the task force.

The following factors were considered in the selection of the SEA projects:

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1. Geographic location and climatic zone;
2. Ratio of sulfur to asphalt in the binder [expressed as S/A where S is the percentage by weight of sulfur and A is the percentage by weight of asphalt (e.g., 30/70, 20/80)];
3. Existence of a satisfactory AC control pavement for comparison;
4. Variety of uses (base course, surface course, overlay, etc.);
5. Variety of sulfur/asphalt blending methods;
6. Sulfur form; and
7. Project age at time of evaluation.

Twenty-six SEA projects located in 18 states were identified for evaluation by the task force. The location and age at the time of evaluation of each of these projects are given in Table 1.

TABLE 1 SUMMARY OF SEA PROJECT

State	Review Section Number	Location	Age (Years)[a]	Freezing Index
AZ	850401	Glendale Ave., Phoenix	5.2	0
CA	850601	I-15, West of Baker	3.2	0
CA	860601	Lincoln Ave., Anaheim	4.3	0
CA	860602	Lincoln Ave., Anaheim	4.3	0
DE	851001	US 13 in Greenwood	6.4	0
FL	861201	Southwest 16th Ave. in Gainesville	6.9	0
FL	861202	I-75 North of Gainesville	5.4	0
GA	861301	Bainbridge Bypass (US 27 & US 84)	4.6	0
ID	851601	State Route 14, East of Golden	4.0	500
LA	862201	State Route 22, near Darrow	6.0/7.2[b]	0
ME	852301	I-95, 30 miles so. of Bangor	4.1	1000
ME	852302	I-95, 90 miles no. of Bangor	6.2	2000
MN	862701	Trunk Highway 63, no. of Rochester	7.0	1700
MS	862801	State Route 15, so. of Phila.	4.4	0
NV	853201	US 93-95, no. of Boulder City	8.9	0
NV	853202	US 50 Alternate, near Fernley	5.3	200
NM	853501	US 62/180, near Carlsbad	3.7	0
ND	853801	US 2-82, west of Minot	5.2	2500
PA	854201	Emmaus Ave., near Allentown	4.4	250
TX	854801	I-10, near Fort Stockton	4.2	0
TX	854802	MH153 in College Station	7.4	0
TX	854803	Loop 495, north of Nacogdoches	5.2	0
TX	854804	US 59, near Lufkin	3.2	0
WA	855301	US 2, west of Pullman	6.2	200
WI	865501	State Highway 29, west of Tilleda	3.6	1500
WY	865601	State Route 225, west of Cheyenne	3.7	1250

[a]Age of pavement at time of evaluation.

[b]Ages varied for two design sections.

In general, each SEA project that was evaluated is composed of one or more SEA pavements and one or more contiguous AC pavements built simultaneously with and to the same specifications as the SEA pavement or pavements. The AC pavements are designated as controls for comparative purposes. The evaluation discussed here is a "snapshot" of the condition of matched pairs of SEA and AC pavements during late 1985 and early 1986. The types and severity of pavement distress that occur in the SEA pavements may be compared directly with those found in the AC pavements, and the overall condition of the SEA pavements may be contrasted with the condition of those in the AC control group.

An essential feature of this effort is the strict reliance in each SEA project on comparison of pairs of SEA and AC pavements, constructed contemporaneously with virtually identical mix designs and structural sections; this allows factors other than the presence or absence of sulfur to be filtered out of the analysis. Of necessity, the analysis considers only in a peripheral way the effects of such factors as age and climate on pavement performance; possible variability in construction quality and material properties is assumed to be small and reasonable within each SEA project.

EXPERIMENTAL PROCEDURES

A standardized method for identifying and measuring different types of pavement distress was required to assure the validity of the evaluation results. The method needed to be comprehensive, sensitive to differences in distress level from project to project and within a given project, and objective. Objectivity is especially important because measurements made on different pavements by different evaluation teams are directly compared.

The distress evaluation methodology contained in the Highway Pavement Distress Identification Manual for Highway Condition *(3)* was chosen because of its widespread use and because it describes types of distress along with their primary causes. Levels of severity are also defined; measurement criteria are given; and typical photographs of each type of distress and level of severity are provided.

In addition, a method was needed to derive a pavement condition rating from the discrete distress measurements obtained during the evaluation. This rating would allow comparison of the overall condition of the pavements reviewed. The method developed by Carpenter et al. *(4)* was used to determine a present condition index (*PCI*) for each pavement surveyed. The *PCI* is determined through a comprehensive evaluation of visible pavement distress.

The following procedure was employed to evaluate each SEA project and determine specific distress factors and the *PCI* for each SEA and AC pavement section in the project. A team composed of two or three task force members drove over the entire SEA project length at a low rate of speed. The project was observed to determine if any portion or portions deviated significantly from the general observed condition. Planned survey locations were adjusted as necessary to obtain a representative survey of the overall project condition. Selected sections of the projects (or ideally the entire project when its length permitted) were then surveyed on foot by the evaluation team. All visible pavement distress was categorized, measured, and classified by level of severity. For example, lineal feet of transverse cracking were measured with a tape measure and classified by crack width to determine feet of cracking at low, medium, and high levels of severity.

Distress data in the form of total lineal feet of cracking, total square feet of rutting, and so forth were converted mathematically to deduct values for each type of distress and level of severity present in the pavement section. The values represent the relative effect of the specific type of distress and its severity and density on the structural or operation condition of the pavement, or both. The *PCI* for the pavement section was in turn calculated from the individual deduct values. The *PCI* represents the overall rating of the structural integrity and operational condition of the pavement.

In practice, the *PCI* for a pavement section is calculated by the equation

$$PCI = 100 - CDV$$

where *CDV* is the total corrected deduct value; the *CDV* represents an adjustment of the total individual deduct values for the pavement to account for the observation that the overall impact of several types of distress on pavement condition is less than the linear sum of the deduct values.

Each of the 26 SEA projects evaluated has been assigned in Table 1 a unique review section number that codes the project location and its review chronology. Each SEA project contains one or more SEA pavements and one or more AC control pavements. In many cases SEA projects were constructed with a variety of pavement sections with varying sulfur contents, structural sections, asphalt cement grades, and so forth. Each review section was further divided into design sections. The design sections of the least complicated projects differ only in the sulfur content of the paving binder. On other projects, additional design sections were needed to account for differences in typical structural section within the project or the use of several AC grades.

In general, the entire length of the SEA and AC control pavements in each SEA project was surveyed and the distress measured. Some projects, such as in Idaho (review section 851601), were many miles in length; in such cases each design section was divided into samples that differed only in station location within the project. Each sample represents at least 0.1 mi of pavement; the samples for evaluation were chosen by using a random number table.

Ideally, each review section should be divisible into pairs of design sections that differ only in the presence or absence of sulfur in the paving binder. This is generally the case; however, for a few projects this pairing could not be completely achieved because of the circumstances of the original project design; this required comparison of a single AC control section with two or more SEA sections that had varying sulfur/asphalt ratios.

The data requirements for each SEA project were organized in a three-tier arrangement that approximated the division of each SEA project into a review section, design sections, and samples. Tier 1 contains overall project data, Tier 2 data on the design sections, and Tier 3 the field distress survey results for the design sections or samples. The large volume of data, including both historical records and distress survey results, was entered into a computer data base developed for the study. A hard copy of the full data base is available elsewhere *(2)*.

RESULTS OF PROJECT EVALUATION

Tables 2–4 give mean values of the *PCI* and the combined cracking and rutting distress values measured for each SEA project. The means are calculated over all design sections in the project with equal sulfur content. Comparisons of the mean *PCI*-values and the cracking and rutting deduct values over all of the SEA and AC design sections are presented graphically in Figures 1 and 2.

In reviewing these results, it should be recalled that a pavement free of any significant visible distress will have a *PCI* of 100. A distress deduct value of zero indicates that the type of distress is not present in the pavement sample surveyed. Thus, Figure 1 shows that both the SEA and the AC pavements surveyed, considered as a group, are in quite satisfactory condition. Combined cracking (i.e., the total of the transverse, longitudinal, and joint reflective cracking found in a pavement) is the predominant type of distress in both the SEA and the AC pavement groups with rutting a distant second. Alligator cracking is also found in the SEA pavements to a much greater degree than in the AC control pavements, but its incidence is small in both types of pavement. Other types of distress occur to a minor degree in both pavement groups. A full enumeration of all types of distress and deduct values found for each SEA project is available elsewhere *(2)*.

A sulfur/asphalt ratio of 30/70 is found in 21 of the 26 SEA

TABLE 2 MEAN *PCI* SUMMARY

State Code/ Review Section Number	Mean Present Condition Index for Each Sulfur/Asphalt Ratio Represented					
	0/100	20/80	25/75	30/70	35/65	40/60
AZ 850401	95.0			98.0		
CA 850601	100.0	100.0				100.0
DE 851001	90.0			83.5		
ID 851601	96.6			100.0		
ME 852301	87.0	92.0		84.0		
ME 852302	88.0			82.0		
NV 853201	85.0			88.5		
NV 853202	87.0		89.5			
NM 853501	94.0			100.0		
ND 853801	83.5		80.0	83.0		
PA 854201	95.5			90.0		
TX 854801	100.0			100.0		
TX 854802	57.0			80.0		
TX 854803	85.0				80.0	
TX 854804	82.0			82.0		
WA 855301	90.0			89.0		85.0
CA 860601	97.0			93.0		
CA 860602	100.0			100.0		
FL 861201	96.0			97.0		
FL 861202	90.0			74.0		52.0
GA 861301	86.7			90.4		
LA 862201	90.0			90.0		87.0
MN 862701	65.5					79.0
MS 862801	100.0			100.0		100.0
WI 865501	71.0			85.3		
WY 865601	82.0	84.0				
Mean	88.2	92.0	84.8	90.0	80.0	83.8

TABLE 3 MEAN COMBINED CRACKING DEDUCT VALUE SUMMARY

State Code/ Review Section Number		Mean Combined Cracking Deduct Value for Each Sulfur/Asphalt Ratio Represented					
		0/100	20/80	25/75	30/70	35/65	40/60
AZ	850401	10.0			0.0		
CA	850601	0.0	0.0				0.0
DE	851001	29.0			51.5		
ID	851601	0.0			0.0		
ME	852301	47.0	28.0		44.0		
ME	852302	8.0			1.7		
NV	853201	43.0			18.0		
NV	853202	33.7		10.5			
NM	853501	6.0			0.0		
ND	853801	8.5		8.0	4.0		
PA	854201	0.0			6.0		
TX	854801	0.0			0.0		
TX	854802	58.0			24.0		
TX	854803	24.0				13.0	
TX	854804	35.0			34.0		
WA	855301	30.0			21.0		24.0
CA	860601	3.0			2.0		
CA	860602	0.0			0.0		
FL	861201	2.5			3.0		
FL	861202	10.0			39.0		0.0
GA	861301	16.3			10.2		
LA	862201	5.0			5.0		20.0
MN	862701	55.0					61.0
MS	862801	0.0			0.0		0.0
WI	865501	13.0			22.3		
WY	865601	44.0	35.5				
Mean		18.5	21.2	9.2	13.6	13.0	17.5

Note: Sum of the individual deduct values measured for transverse cracking, longitudinal cracking and joint reflective cracking.

projects. Examination of the data in Tables 3 and 4 fails to uncover any remarkable trends in the observed occurrence of combined cracking and rutting distress with variation in sulfur/asphalt ratio from 20/80 to 40/60.

Laboratory testing of SEA mixtures has indicated that increased sulfur content in the binder is reflected in increased binder stiffness, all other factors being equal. This trend might be expected to be translated into increased pavement cracking and decreased rutting with increasing sulfur content. The data presented here fail to substantiate these expected trends in the pavements surveyed during this study. Indeed, the data in Tables 2–4 and Figures 1 and 2 suggest that performance of the SEA pavements surveyed was comparable to that of the AC control group regardless of the sulfur content of the binder. This observation will be tested more rigorously in the next section.

ANALYSIS OF RESULTS

Statistical methods of analysis were employed to test the significance of observed differences in performance factors (*PCI*

TABLE 4 MEAN RUTTING DEDUCT VALUE SUMMARY

State Code/ Review Section Number		Mean Rutting Deduct Value for Each Sulfur/Asphalt Ratio Represented					
		0/100	20/80	25/75	30/70	35/65	40/60
AZ	850401	0.0			0.0		
CA	850601	0.0	0.0				0.0
DE	851001	0.0			0.0		
ID	851601	3.2			0.0		
ME	852301	0.0	0.0		0.0		
ME	852302	11.0			26.3		
NV	853201	6.0			6.0		
NV	853202	0.0		0.0			
NM	853501	0.0			0.0		
ND	853801	12.0		19.0	14.0		
PA	854201	2.5			1.0		
TX	854801	0.0			0.0		
TX	854802	31.0			9.0		
TX	854803	0.0				0.0	
TX	854804	0.0			0.0		
WA	855301	0.0			0.0		0.0
CA	860601	0.0			0.0		
CA	860602	0.0			0.0		
FL	861201	1.5			0.0		
FL	861202	0.0			0.0		0.0
GA	861301	0.0			0.0		
LA	862201	0.0			0.0		0.0
MN	862701	0.0					0.0
MS	862801	0.0			0.0		0.0
WI	865501	31.5			7.0		
WY	865601	15.0	0.0				
Mean		4.4	0.0	9.5	3.0	0.0	0.0

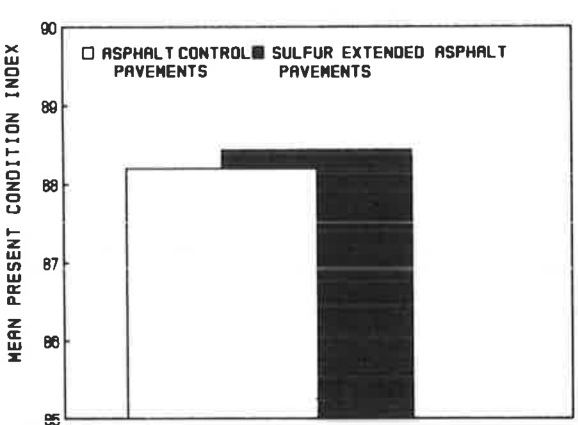

FIGURE 1 Comparison of the *PCI* for SEA and AC pavement sections.

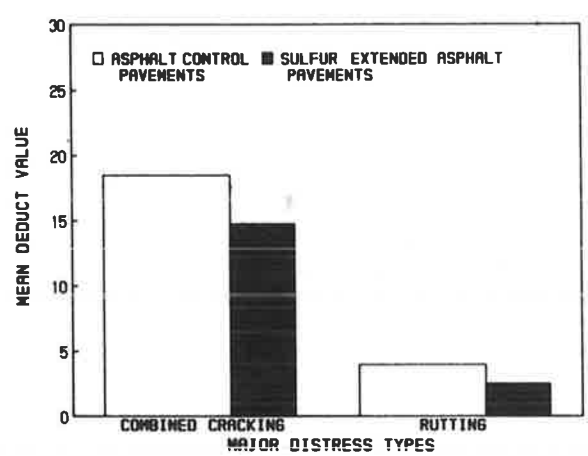

FIGURE 2 Comparison of mean distress deduct values for SEA and AC pavement sections.

and distress deduct values) between the SEA and AC pavement groups. That every SEA pavement may be matched with a control AC pavement that is presumed to be identical to it except for the sulfur content of the binder aided the analyses. Random, variable effects of construction quality, traffic volume, age, and so forth and the precision of the evaluation procedure are also minimized by provision of the AC control group. The analysis may therefore be concentrated on the main item of interest, the effect of sulfur on the performance and durability of the pavements.

The Student's *t*-test was employed to estimate the significance of the observed differences among the SEA and AC pavement groups. This test is described in detail in any textbook of statistical methods (*5*, p. 194 ff). The analysis is predicated on testing a null hypothesis; in this case, the null hypothesis states that observed differences between the SEA and the AC pavement groups are not statistically significant. The magnitude of the *t*-statistic allows acceptance or rejection of the null hypothesis at a desired level of significance.

Table 5 gives the results of the analysis of the observed

TABLE 5 *t*-TEST RESULTS: COMPARISON OF SEA AND AC PAVEMENT PERFORMANCE FACTORS

Performance Factor	n	t	Accept or reject null hypothesis[a] at $\alpha = 0.005$
Present Condition Index, all design sections	43	0.437	Accept
Rutting deduct value, all design sections	43	-1.336	Accept
Combined cracking deduct value, all design sections	43	-0.965	Accept
Present Condition Index, 30/70[b] design sections	29	0.959	Accept
Rutting deduct value, 30/70 design sections	29	-1.238	Accept
Combined cracking deduct value, 30/70 design sections	29	-0.632	Accept
Present Condition Index, 20/80 and 25/75 design sections	5	0.848	Accept
Rutting deduct value, 20/80 and 25/75 design sections	5	-0.444	Accept
Combined cracking deduct value, 20/80 and 25/75 design sections	5	-2.168	Accept
Present Condition Index, 35/65 and 40/60 design sections	9	-0.526	Accept
Rutting deduct value, 35/65 and 40/60 design sections	9	N.A.[c]	N.A.[c]
Combined cracking deduct value, 35/65 and 40/60 design sections	9	0.468	Accept

[a]The null hypothesis states that the observed difference between the two groups is not statistically significart.

[b]S/A = Sulfur to asphalt weight ratio, e.g. 30/70, 20/80, etc.

[c]N.A.: Not applicable. The rutting deduct values for all nine SEA and AC design sections were 0.0.

differences of *PCI*, combined cracking deduct value, and rutting deduct value between the SEA and the control AC groups at a level of significance of 0.005. Insofar as possible, data for unique pairs of SEA and AC design sections that differed only in sulfur content of the binder were compared; however, for a few projects, because of their original design, a single control AC design section was compared with more than one SEA design section. Also, for design sections with multiple samples, the mean parameter value for the samples was used in the analysis.

The results given in Table 5 indicate that the null hypothesis may not be rejected for the *PCI* or either distress deduct value. This indicates that no significant differences exist between the SEA and the control AC pavement groups within the limits of this analysis. In all respects, the analysis concludes that the two types of pavements have performed comparably.

Table 6 gives the results of an analysis of observed differences of *PCI* and distress deduct values among SEA pavements with sulfur/asphalt ratios of <30/70, 30/70, and >30/70. For all cases, the *t*-test indicates that the null hypothesis may not be rejected at a level of significance of 0.005. In all respects, the analysis concludes that the performance of the SEA pavement was not significantly influenced by the sulfur content of the paving binder (within the range of sulfur/ asphalt ratios in the SEA pavement group).

Table 7 gives the results of the determination of the correlation coefficient (r) for the pavement performance factors compared with pavement age and climatic exposure expressed as freezing index (2, Appendix 1) for both the SEA and the control AC pavement groups. The sample correlation coefficient (5, p. 321 ff) was used to estimate what percentage of variation (100 r^2) in one set of observations may be accounted for by the variation in another set of observations. The calculated values of r and 100 r^2 are low, which implies a lack of evidence for causal relationships between age or freezing index and the *PCI* and distress deduct values measured for the pavements.

FINDINGS AND CONCLUSIONS

The statistical analysis carried out on the pavement evaluation data gathered in this study of 26 SEA projects built between

TABLE 6 *t*-TEST RESULTS: COMPARISON OF SEA PAVEMENTS WITH DIFFERENT SULFUR/ASPHALT RATIOS IN THE BINDER

Performance Factor	n	t	Accept or Reject null hypothesis[a] at α = 0.005
Present Condition Index:			
30/70[b] vs. 20/80 and 25/75	32	0.103	Accept
30/70 vs. 35/65 and 40/60	36	1.685	Accept
20/80 and 25/75 vs. 35/65 and 40/60	12	0.851	Accept
Rutting Deduct Value:			
30/70 vs. 20/80 and 25/75	32	-0.393	Accept
30/70 vs. 35/65 and 40/60	36	-2.078	Accept
20/80 and 25/75 vs. 35/65 and 40/60	12	-0.473	Accept
Combined Cracking Deduct Value:			
30/70 vs. 20/80 and 25/75	32	0.265	Accept
30/70 vs. 35/65 and 40/60	36	1.570	Accept
20/80 and 25/75 vs. 35/65 and 40/60	12	1.388	Accept

[a]The null hypothesis states that the observed difference between the two groups is not statistically significant.

[b]S/A = Sulfur to asphalt weight ratio in SEA binder, e.g., 30/70, 20/80, etc.

TABLE 7 CORRELATION AMONG PAVEMENT PERFORMANCE FACTORS, PAVEMENT AGE, AND FREEZING INDEX

For Correlation Between:	Correlation Coefficient, r	Percent Variation Explained, 100r
PCI (30/70[a]) and Age	0.439	19.3
PCI (20/80 and 25/75) and Age	0.576	33.2
PCI (35/65 and 40/60) and Age	-0.770	59.3
PCI (AC) and Age	-0.412	17.0
PCI (30/70) and Freezing Index	-0.357	12.8
PCI (20/80 and 25/75) and Freezing Index	-0.716	51.2
PCI (35/65 and 40/60) and Freezing Index	-0.112	1.3
PCI (AC) and Freezing Index	-0.404	16.3
Rutting Deduct Value and Freezing Index (all design sections:		
SEA	0.644	41.5
AC	0.477	22.8
Combined Cracking Deduct Value and Freezing Index (all design sections:		
SEA	0.181	3.3
AC.	0.714	51.0

[a]S/A = Sulfur to asphalt weight ratio in SEA binder, e.g., 30/70, 20/80, etc.

1977 and 1982 in 18 states indicates that the overall performance and susceptibility to distress of the SEA pavements are not significantly different than those of the closely matched, control AC pavement group. Furthermore, for the SEA pavements studied, the level of sulfur in the paving binder did not have a significant effect on pavement performance or measured levels of distress. Correlation of SEA and AC pavement *PCI* and distress deduct values with pavement age and freezing index was poor, which indicates a lack of important causal relationships.

The SEA projects evaluated here were generally built to study only the effect of the presence or absence of sulfur on pavement performance. Sulfur was substituted for from 20 to 40 percent of the asphalt in the mix; the binder content was adjusted to maintain equal binder volume in the mix; and in general the typical structural section of the pavement was unchanged. Rarely was the asphalt grade altered in the SEA binder to test the practical consequences of changes in binder consistency, temperature susceptibility, and stiffness noted in the laboratory on introduction of sulfur into asphalt.

It is noteworthy that, in the 26 SEA projects evaluated, the use of sulfur in substantial quantities in the paving binder appears to have had little, if any, deleterious effect on pavement condition and, by extension, pavement performance and durability. Only one SEA project (Florida, review section 861202) was found to be in poor condition, and this situation appeared to be the result of severe moisture damage to which the presence of sulfur was one of several contributory factors.

The results of this study imply that in most circumstances the use of sulfur as an extender in asphalt paving mixtures is innocuous and that SEA pavements should perform in a satisfactory manner if they are constructed to proper design and with adequate attention to detail. Given the 1987 cost of elemental sulfur compared with that of asphalt, the use of SEA is difficult to justify because no significant improvement in pavement performance attributable to the incorporation of sulfur was found.

The observation that SEA performance as measured in this study was not significantly affected by variation in the sulfur content of the binder within wide limits is surprising. On the basis of past research *(1)*, some significant if perhaps small effect would have been expected. The field results reported here may indicate that in practical terms the effects of variation in binder and mixture properties are masked by the inherent variability of the construction process.

The results of this study suggest two areas in which further research may be useful. First, because the use of sulfur as an asphalt extender did not have detrimental effects, investigation of the use of elemental sulfur as an additive for asphalt modification may be worthwhile, if only because elemental sulfur is relatively inexpensive compared with other proposed modifiers. Second, the entire SEA data set collected in this study *(2)* may be analyzed further to determine if factors such as traffic influenced SEA pavement performance at specific locations in a manner not discerned in the comparative analyses reported here.

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Recycling Waste Roofing Material in Asphalt Paving Mixtures

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The technical feasibility of using waste roofing products in asphalt concrete paving mixtures is addressed. Approximately 9 million tons of roofing waste are generated annually in the United States. Disposal costs are significant. Recycling represents an economical and, perhaps, environmentally attractive alternative to placing these wastes in landfills. The relatively large quantities of asphalt cement and "aggregate-type" materials present in roofing waste suggest that these materials have potential as a partial substitute for asphalt cement or aggregate, or both, in a paving mixture. A study that arrived at the following conclusion was conducted. (a) Acceptable paving mixtures, which contain up to 20 percent by volume of roofing waste, can be produced; (b) proper selection of binder type and quantity is critical to the performance of the mixture and depends on the type and quantity of the roofing waste in the mixture; (c) improved asphalt cement extraction and recovery processes need to be developed to effectively determine the properties of the asphalt cement in the roofing waste; (d) the total "active binder" content, depending on the effectiveness of the recycling agents, should be considered when designing asphalt concrete mixtures; (e) the gradations of conventional aggregates and roofing wastes should be considered when designing paving mixtures; and (f) the long-term field performance of paving mixtures containing roofing waste needs to be established.

Approximately 9 million tons of roofing waste are generated annually in the United States (*1*, p. 2). As is the case with large amounts of any type of waste, disposal costs are significant. Recycling represents an economical alternative to placing these wastes in landfills. Recycling of roofing waste into paving materials is an alternative.

Typical roofing waste products, such as old shingles or built-up roofing, contain about 36 percent asphalt cement; 22 percent hard rock granules (minus No. 10 to plus No. 60 sieve size); 8 percent filler (minus No. 100 sieve size material); and smaller amounts of coarse aggregate (approximately 1 in. in size), cellulose fiber felt, glass fiber felt, asbestos felt, and polyester films. The relatively large quantities of asphalt cement and "aggregate-type" materials present in roofing waste suggest that it has potential as a partial substitute for asphalt cement or aggregate, or both, in a paving mixture. This paper contains the results of a preliminary investigation to establish the feasibility of using roofing waste products in asphalt concrete paving mixtures.

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BACKGROUND

Technically, a wide variety of waste products and by-products, including old pavement materials, waste glass, battery cases, polypropylene containers, old tires, fly ash, bottom ash, and slag, can be successfully incorporated into paving materials (*2–7*). From a national perspective and based on both a technical and an economic viewpoint, the reuse of old pavement materials, old tires, fly ash, and slag has been successful. Literature that addresses the technical and economic feasibility of using roofing wastes in asphalt paving materials is not available.

Several technical items will have to be addressed before widespread use is made of roofing waste in asphalt concrete mixtures. These items include

1. The nature and quantities of the material in roofing waste including the properties of the asphalt cement and the grain size distribution of the solid material;
2. The quantity of roofing waste that can be introduced into a paving mixture without adversely altering the engineering properties of the mixture;
3. The quantity and type of asphalt cement or aromatic-type oils, or both, needed to soften the aged roofing asphalt to an appropriate paving grade asphalt cement;
4. The techniques for introducing the processed roofing waste into the asphalt concrete mixing and paving process without creating adverse environmental effects;
5. Establishing the long-term performance characteristics of asphalt concrete containing roofing waste by an extensive laboratory and field testing program; and
6. Determining the local economics of using this waste material in a paving mixture.

A test program that addresses Items 1 through 3 has been conducted. A more extensive research effort, including additional and more sophisticated laboratory testing and field studies, will be required to define Items 1 through 6 in sufficient detail to gain acceptance by the contracting and engineering communities.

TEST PROGRAM

Roofing wastes from five sources have been obtained and subjected to the test program shown in Figure 1.

Figures 2 and 3 show and Table 1 gives a further description of the asphalt concrete mixture test program. The first three parts of the mixture-testing program shown in Figure 2 were performed to establish the source of roofing waste, the range of

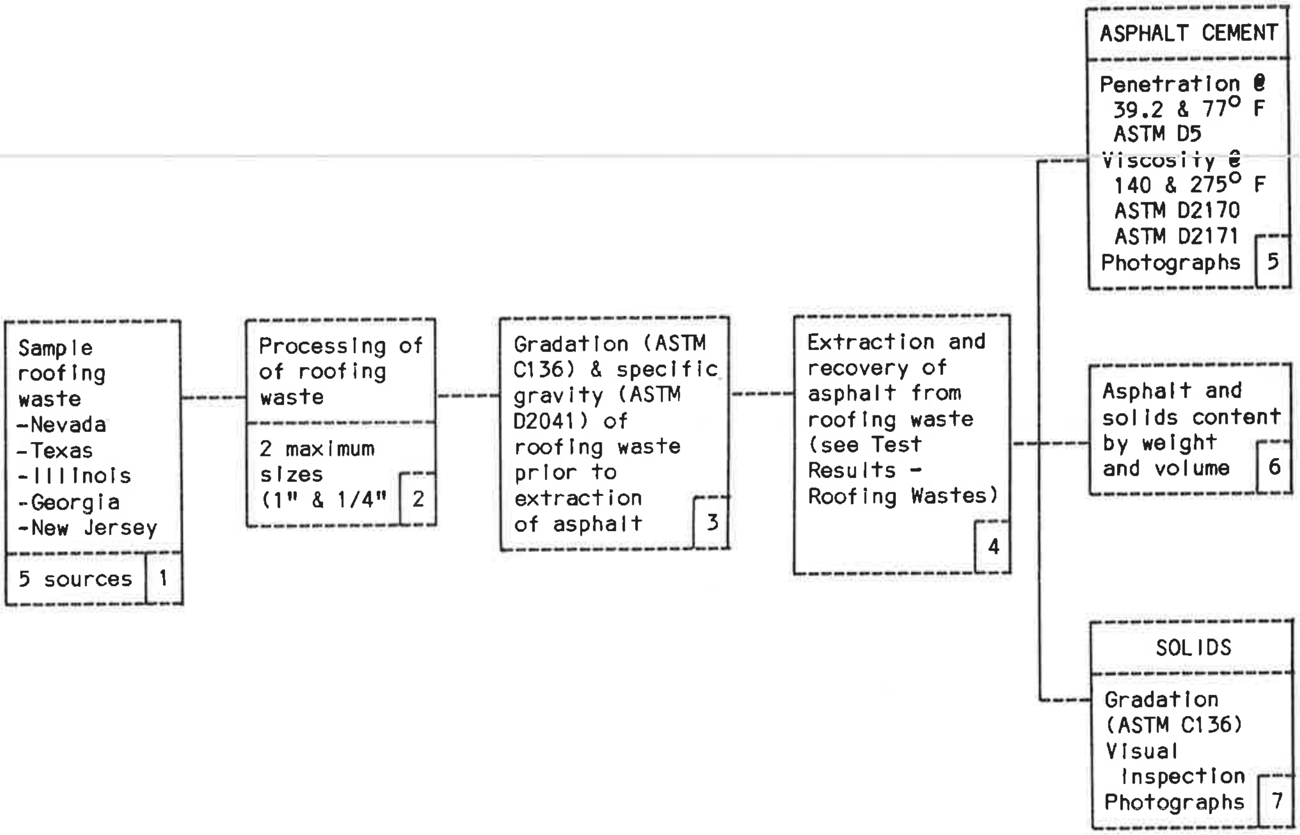


FIGURE 1 Test sequence for defining properties of roofing wastes.

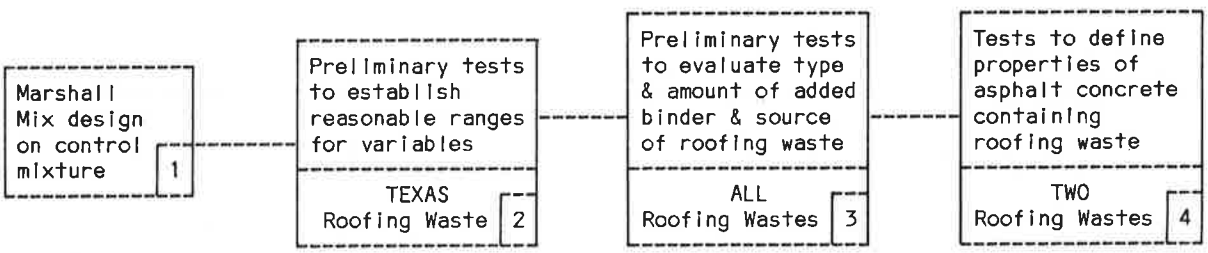
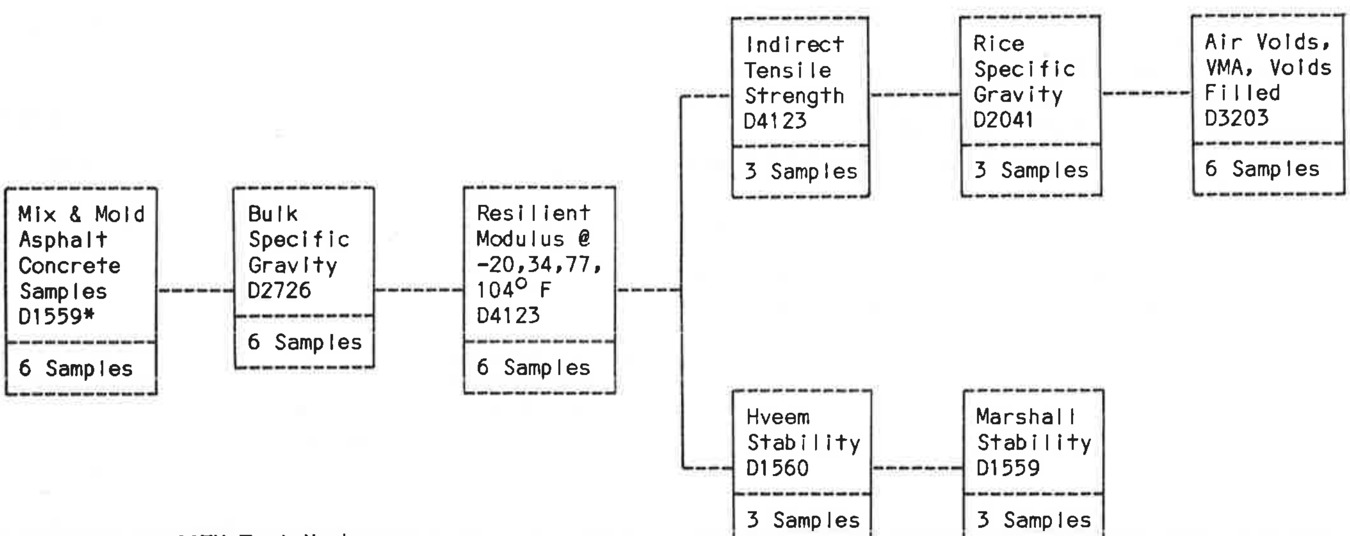


FIGURE 2 Test sequence for defining properties of asphalt concrete that contains roofing waste.



* Refers to ASTM Test Numbers.

FIGURE 3 Test program identified in Table 1.

TABLE 1 TEST MATRIX FOR MIXTURE-TESTING PROGRAM

Type and Source of Roofing Waste	Quantity of Roofing Waste (% by volume)	Type of Binder: RA-5[a], Quantity of Added Binder (% by weight) 3	4	5	RA-75, Quantity of Added Binder (% by weight) 3	4	5	AR4000, Quantity of Added Binder (% by weight) 4	5	6
1 in. minus, New Jersey	10									
	20		x			x			x	
	30									
1/4 in. minus, New Jersey	10				x	x	x			
	20	x	x	x	x	x	x	x	x	x
	30				x	x	x			
1/4 in. minus, Nevada	10					x				
	20		x		x	x	x		x	
	30					x				
1 in. minus, Nevada	10									
	20		x			x			x	
	30									

[a]RA = recycling agent.

quantities of roofing waste, the type of added binder, and the quantity of added binder to be used in the fourth and major part of the study. The major portion of the mixture-testing program was performed on two roofing wastes as indicated in Table 1.

All asphalt concrete mixtures were subjected to the test methods shown in Figure 3. Each mixture was placed in an oven at the specified 275°F compaction temperature for 1 1/2 hr after the completion of mixing. This was done to simulate mixing at the hot plant, transportation to the job site, and laydown elapse time. In addition, this extra time would allow the recycling agent to digest the aged waste asphalt cement. Each mixture was then remixed before compaction.

Air voids were calculated by ASTM D 3203 using Rice specific gravities corrected for absorption. The presence of roofing waste material did not affect the air void analysis. The magnitude of the correction for absorption was consistent with that for mixtures containing the same absorptive aggregate and no roofing waste.

MATERIALS

Asphalt Cement

An AR4000 asphalt cement was used in the control mixture and in mixtures containing roofing wastes. The physical properties of this asphalt cement are given in Table 2. This California Valley asphalt cement is used in Arizona, California, Nevada, and Oregon and has been used in several research projects in the western states.

Recycling Agents

Two recycling agents were used in mixtures containing roofing wastes. These products meet Pacific Coast specifications for RA-5 and RA-75 recycling agents and have been used on recycled asphalt mixture projects in the western states. The physical properties of these materials are given in Table 2. These lower viscosity materials have the capability of softening harder asphalts. The chemical composition of these materials has been established to ensure compatibility with a wide range of paving-grade asphalt cement types.

Aggregates

Aggregate was obtained from a pit located in Sparks, Nevada. The aggregate is a subrounded gravel, partly crushed and washed, from an alluvial deposit. The aggregate is considered an absorptive aggregate with absorption capacities in the range

TABLE 2 PHYSICAL PROPERTIES OF BINDERS

Physical Property	AR4000	RA-5[a]	RA-75
Penetration at 39.2°F, 100 g/5 sec (0.1 mm)	14	–	–
Penetration at 77°F, 100 g/5 sec (0.1 mm)	54	–	–
Viscosity at 140°F	2180 P	260 cSt	8000 cSt
Viscosity at 275°F	270 cSt	13 cSt	60 cSt
Ductility at 77°F, 5 cm/min	100 +	–	–
Ring-and-ball softening point (°F)	123	–	–
Flash point, Cleveland open cup (°F)	–	400 min	450 min
Specific gravity	0.98–1.02	0.98–1.02	0.98–1.02

[a]RA = recycling agent.

TABLE 3 PROPERTIES OF ROOFING WASTES

Property	Reno, Nevada	Dallas/Fort Worth, Texas	Oakbrook, Illinois	Savannah, Georgia	New Jersey
Apparent specific gravity[a]	1.37	1.26	1.36	1.28	1.13
Asphalt content[b] (Method 1)	37.2	39.1	33.2	35.4	37.6

[a]Before extraction.
[b]Percentage by total weight of waste.

of from 3 to 4 percent. The gradation of the aggregate meets the Nevada Department of Transportation's specification for Type-2 dense-graded plant mix and road mix. The maximum aggregate size is ³/₄ in.

TEST RESULTS—ROOFING WASTES

Specific Gravity

The specific gravities of the wastes are given in Table 3. The specific gravities of these materials are considerably below those of conventional aggregates, which typically have values between 2.55 and 2.70. Thus the percentages of roofing wastes in mixtures expressed as a percentage by total volume or a percentage by total weight will be considerably different. Asphalt concrete mixture design concepts are based on volume concepts but are commonly expressed on a weight basis for construction convenience.

Extraction and Recovery

Two methods were used to extract and recover the asphalt cement from the roofing waste. Method 1 used ASTM D 2172 (Method B) to extract and a Buchi Rotavapor distillation apparatus to recover the asphalt cement from the roofing waste. Method 2 was performed by the Manville Service Corporation. This method used a centrifuge process and a cyclohexane solvent for extraction and a hot-plate evaporation system for recovery. A more detailed description of Method 2 is found in Appendix A of Paulsen et al. *(8)*.

Asphalt Cement Content

The asphalt content expressed by the total weight of the roofing wastes is given in Table 3. Data in Table 3 were determined from extraction and recovery Method 1. However, when Method 2 was used, asphalt contents for the Nevada and New Jersey roofing wastes were 20 and 26 percent. Method 2 uses a cyclohexane solvent, which may have dissolved only a portion of the asphalt and therefore resulted in low asphalt contents. The black color of the mineral residue after extraction tended to confirm this suspicion.

Penetration (ASTM D 5)

Penetrations at 39.2°F and 77°F, 100 g, and 5 sec were obtained for the asphalt cement extracted and recovered by Method 2. These test results are given in Table 4. Roofing waste asphalt cements extracted and recovered by Method 1 were not suitable for penetration testing. Air bubbles could not be eliminated in the recovered asphalt; thus penetration results were erratic.

Viscosity (ASTM D 2170 and D 2171)

Viscosity tests were performed at 140°F and 275°F. Test results from the asphalt cement extracted and recovered by Method 2 are given in Table 4. Blank data entries are the result of the recovered asphalt cements having viscosities greater than available viscosity tube ranges. All asphalt cements recovered by Method 1 had viscosities that exceeded available viscosity tube ranges.

The data contained in Table 4 suggest that the asphalt recovered from the Nevada roofing waste, by Method 2, is considerably softer than are asphalts recovered from other wastes. All data in Table 4 relate to Method 2. However, this same recovered asphalt appeared to have the highest viscosity when extracted and recovered by Method 1. This observation was completely subjective because actual viscosity data from Method 1 were unattainable. The nature of this highly weathered asphalt, possibly produced with a ferric chloride

TABLE 4 RECOVERED ROOFING WASTE ASPHALT CEMENT PROPERTIES (Method 2)

Source of Roofing Waste	Penetration at 39.2°F, 100 g/5 sec (0.1 mm)	Penetration at 77°F, 100 g/5 sec (0.1 mm)	Viscosity at 140°F (poise)	Viscosity at 275°F (cSt)	Pen-Vis No.
Reno, Nevada	104	189	27	52	–2.97
Dallas/Fort Worth, Texas	11	24	149 000	2 630	0.73
Oakbrook, Illinois	5	12	–[a]	3 880	5.53
Savannah, Georgia	3	7	–[a]	53 100	2.68
New Jersey	4	11	–[a]	31 500	2.67

[a]Recovered asphalt cement viscosity exceeded available tube ranges.

catalyst, may have imparted different solubility characteristics in various solvents. All recovered asphalts from Method 1, which uses a hot extraction process and a trichloroethylene solvent, were very hard. This supports experimental evidence that recovered asphalt may have higher viscosity values when recovered from solutions obtained by hot extraction methods. The recovered asphalt from the Nevada roofing waste was very soft when Method 2 was used. This may be because only a portion of the asphalt was dissolved when extraction Method 2 was used. It is the authors' opinion that Method 1 more accurately represented the asphalt contents and relative stiffnesses of the asphalts in the roofing wastes. In any case, improved extraction and recovery procedures for roofing wastes need to be developed.

Gradation

A typical comparison of roofing waste gradations before and after extraction of the asphalt cement is shown in Figure 4. A comparison of materials before and after extraction is shown in Appendix B of Paulsen et al. *(8)*. The ¼-in.-minus roofing waste from Nevada has a finer gradation except at the No. 200 sieve than do the wastes from other sources. The percentage passing the No. 200 sieve for the roofing wastes processed ranged from 9 to 23 percent. It should be noted that the aggregate gradation for each asphalt concrete mixture was adjusted for the quantity and type of roofing waste in the mixture so that the final mix gradation was equivalent to the gradation in the control mixture, which did not contain roofing waste.

TEST RESULTS—ASPHALT CONCRETE MIXTURES

The reader is urged to note that all roofing waste quantity percentages are by volume of the mixture and that added binder contents are by weight of the mixture.

Control Mixture

The Marshall mixture design method as defined by the Asphalt Institute was used to select an asphalt content for the control mixture. The control mixture contained aggregate from a Sparks, Nevada, pit and an AR4000 California Valley asphalt cement. The design asphalt cement content was 5.9 percent by total weight of mixture.

Preliminary Tests—Range of Variables

Preliminary tests were performed on mixtures to establish reasonable ranges for the study variables. The processed roofing waste from Texas was used in all mixtures because it alone was available during the early stages of the study. Quantities of roofing wastes ranged from 10 to 50 percent by total volume of the mixture. The RA-5 and RA-75 recycling agents were used in percentages that ranged from 3.0 to 5.5 percent by total weight of mixture. The test sequence shown in Figure 3 was followed.

The desirable ranges of the various test parameters are given in the following table (*9* and researchers' unpublished data).

Parameter	*Range*
Resilient modulus	200,000 to 800,000 psi at 77°F
Indirect tension	75 to 250 psi
Hveem stability	Minimum of 30 for light traffic
	Minimum of 35 for heavy traffic
Marshall stability	Minimum of 750 for light traffic
	Minimum of 1,500 for heavy traffic
Marshall flow	8 to 20 for light traffic
	8 to 16 for heavy traffic
Air voids	3 to 5 percent

Mixtures that contain the higher percentages of recycling agents at 10, 30, and 50 percent roofing wastes have values that are slightly below or at limiting values. These same mixtures have low tensile strength, Hveem stability, and air voids. Mix-

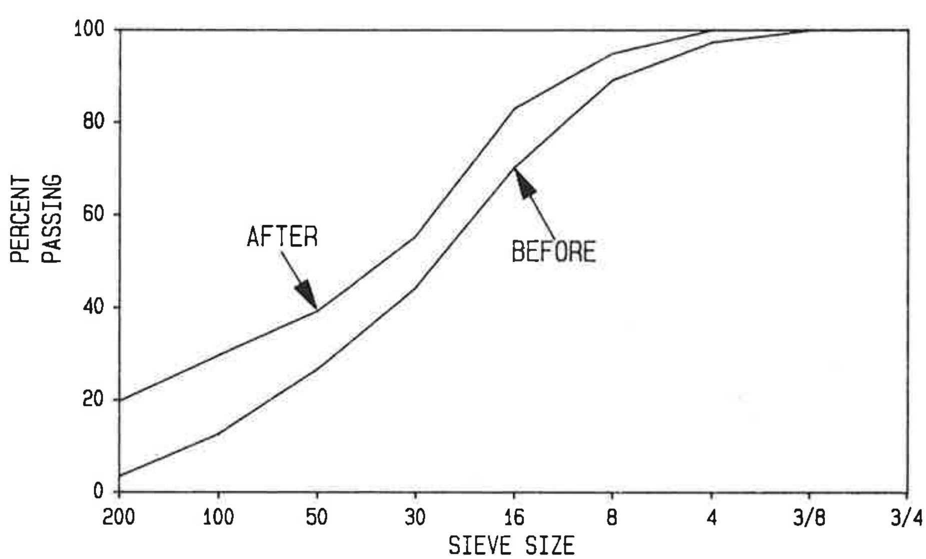


FIGURE 4 Typical grading curves of roofing waste before and after extraction.

tures that contain 30 and 50 percent roofing wastes have low air voids, which suggests that excessive additional binder was used or that the viscosity of the binder was too low, or both.

Mixtures that contain these high percentages of roofing wastes might also contain higher than generally accepted percentages of minus No. 200 material depending on how much roofing waste asphalt is digested by the recycling agent. On the basis of experience with recycling asphalt concrete mixes, hot-mix plant air quality problems, associated with smoke generation, can be expected at approximately 50 percent wastes during field mixing operations. Large waste percentages cause a decrease in new aggregate contact and an increase in mixing temperatures. Both of these factors will result in smoking of the asphalt in both drum and batch mixing plants.

This preliminary testing program indicated that mixtures with acceptable properties can be made with roofing wastes. However, in general, the content of additional binder needs to be decreased or its viscosity needs to be increased, or both, to increase the resilient modulus, tensile strength, Hveem stability, and air voids. Marshall flow values need to be decreased for some mixtures.

Preliminary Tests—Type of Roofing Waste

Preliminary tests were performed on mixtures prepared from the five roofing wastes. Ten and 30 percent roofing waste by total volume of mixtures with recycling agents RA-5 and RA-75 were evaluated. The test sequence shown in Figure 3 was used.

Mixtures that contained 10 percent roofing waste and 4.75 percent recycling agent RA-75 were, in general, low in resilient modulus, tensile strength, Hveem stability, and air voids. The binder content should be reduced or the viscosity of the binder increased, or both.

Mixtures that contain 30 percent roofing wastes and 2 percent recycling agent RA-5 have properties that are within acceptable ranges with the exception of some Hveem stabilities.

Previous research conducted on conventional mixtures indicates that values of resilient modulus reflect the stiffness of the binder contained in the mixture. The resilient modulus results shown in Figure 5 and the asphalt stiffness parameters given in Table 4 do not exhibit the expected relationship. Mixtures produced from the roofing wastes from Nevada and Texas have the highest resilient moduli, and mixtures made with the roofing wastes from Illinois and New Jersey have low values of resilient modulus. This conflicting behavior is partly due to the inability of extraction and recovery Method 2 to properly characterize the binder properties of waste roofing asphalt. Differences in the relative digestion of the waste roofing asphalt by the recycling agent or "active" binder content may also contribute to this inconsistency.

The results from the preliminary tests were used to select the source and amount of roofing waste as well as the type and amount of binder to be used in the test matrix of Table 1.

Properties of Mixtures

The test sequence shown in Figure 3 was used for mixtures identified in the test matrix of Table 1. The Nevada and New Jersey roofing wastes were selected to represent mixtures with a wide range of resilient modulus, tensile strenth, stability, flow, and air void contents.

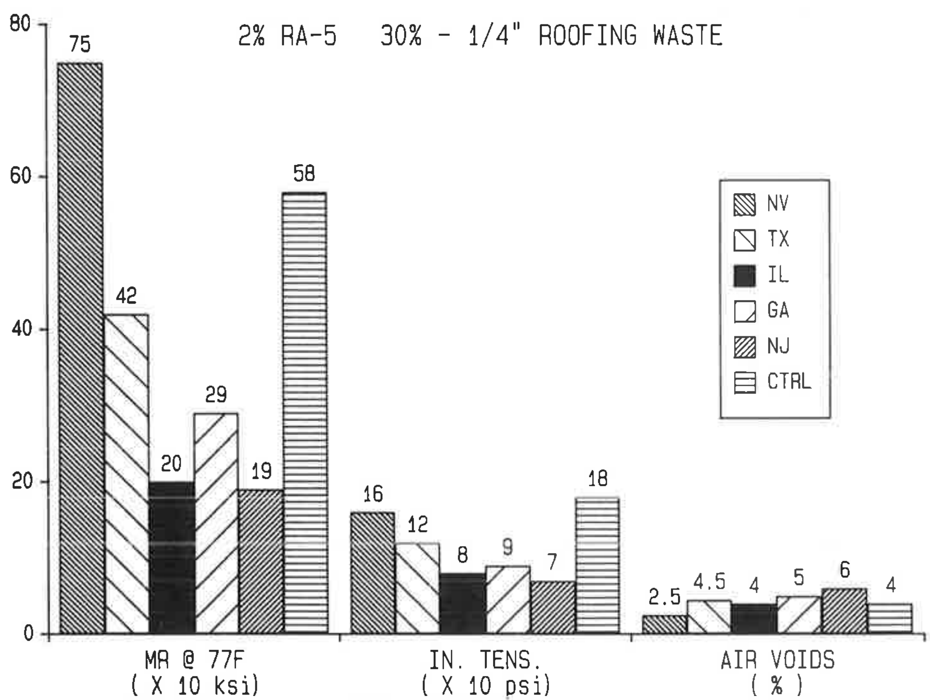


FIGURE 5 Effect of type of roofing waste—preliminary testing.

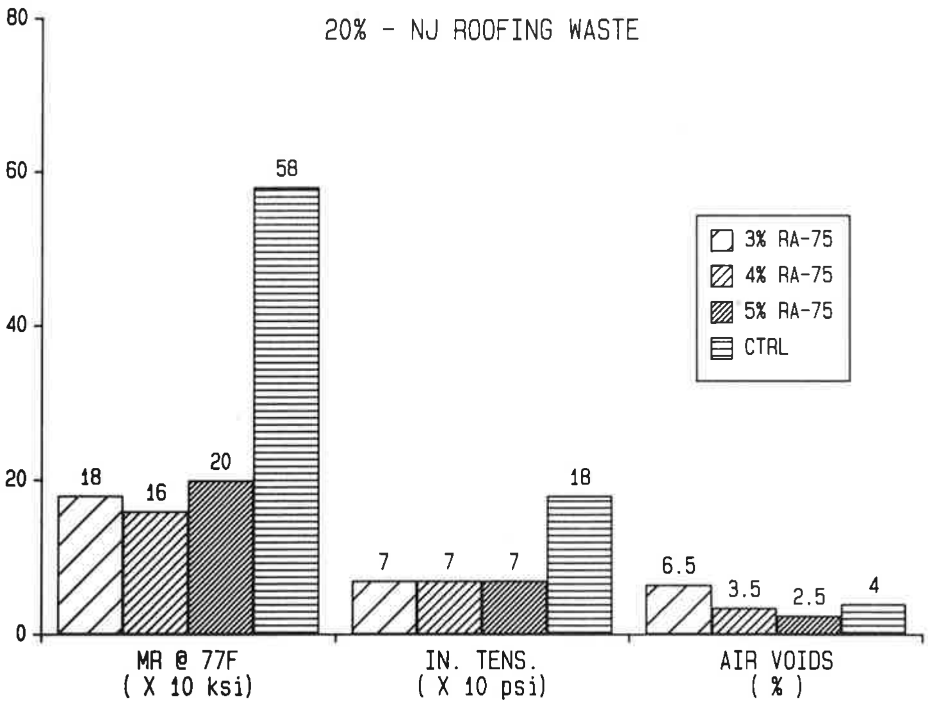


FIGURE 6 Effect of added binder content—New Jersey roofing waste and RA-75—on resilient modulus and tensile strength.

Quantity of Binder

Figures 6–11 show some of the effects of the quantity of added binder on the properties of mixtures. The quantity and type of binder, to a large degree, control the properties of a mixture. Mixtures prepared with the New Jersey roofing waste and recycling agent RA-75 (Figures 6 and 7) had low resilient modulus, tensile strength, and Hveem stabilities and high flows. At higher binder contents, air voids were also low.

Figures 8–11 suggest that acceptable mixtures can be prepared at binder contents in the range of 3 to 4 percent when the roofing waste contents are 20 percent by volume. The mixture containing 20 percent New Jersey waste and 4 percent AR4000 binder has good resilient modulus and tensile strength, but

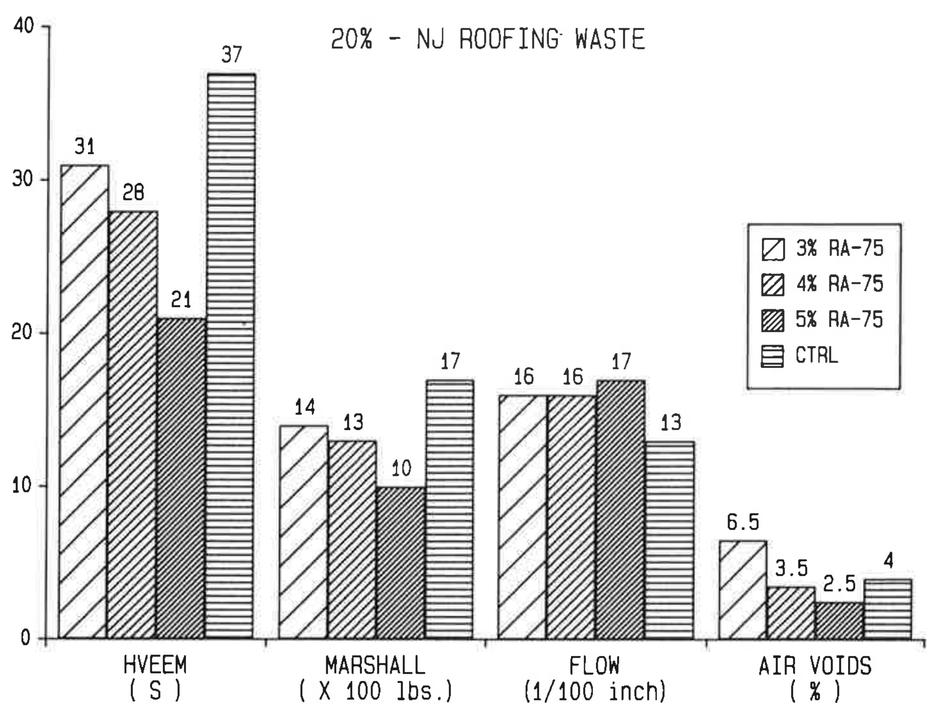


FIGURE 7 Effect of added binder content—New Jersey roofing waste and RA-75—on stability and flow.

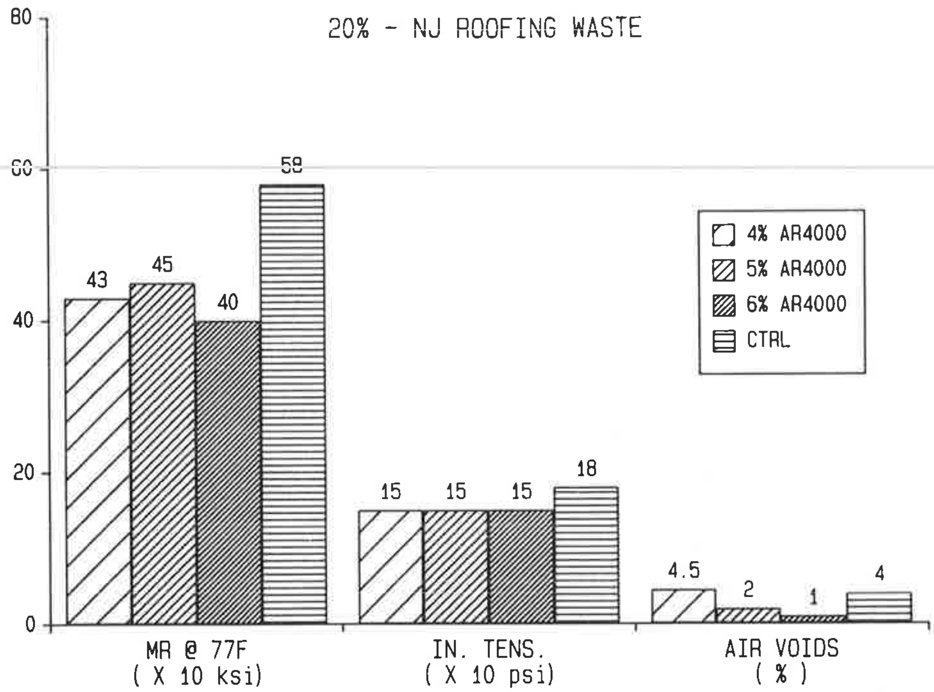


FIGURE 8 Effect of added binder content—New Jersey roofing waste and AR4000—on resilient modulus and tensile strength.

stability values are slightly outside acceptable ranges. A reduction to 3.5 percent binder would probably produce an acceptable mixture.

An acceptable mixture is possible at the 3 percent binder level when the Nevada roofing waste is used at the 20 percent level with recycling agent RA-75.

Type of Binder

Figures 6–11 also show some of the effects of the type of binder on the properties of the mixtures. Because the asphalt in the Nevada roofing waste appeared to be more viscous than the asphalt in the New Jersey roofing waste (observations from

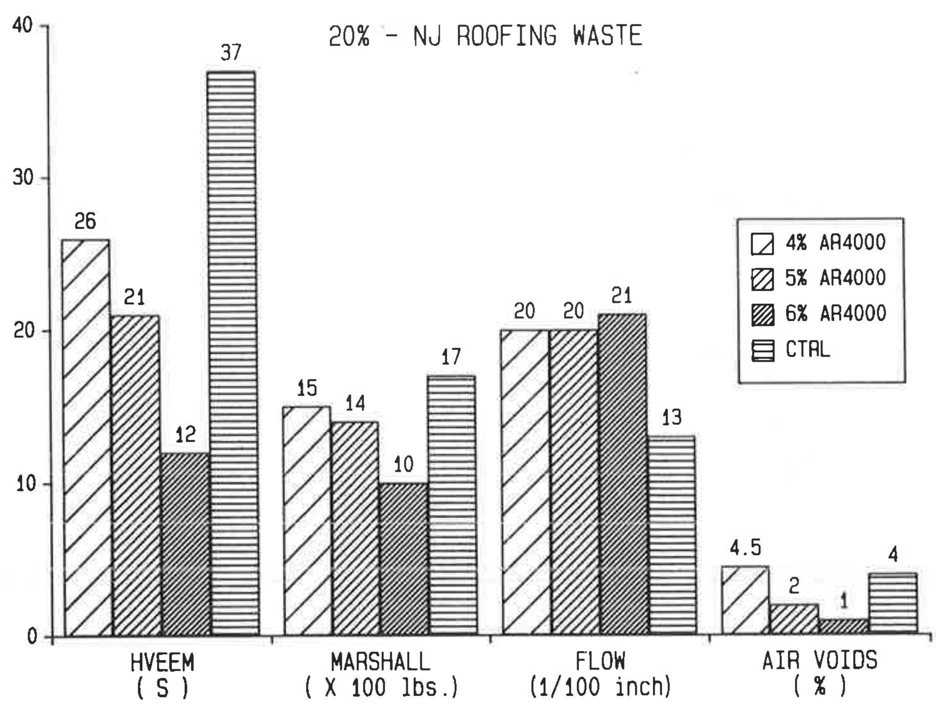


FIGURE 9 Effect of added binder content—New Jersey roofing waste and AR4000—on stability and flow.

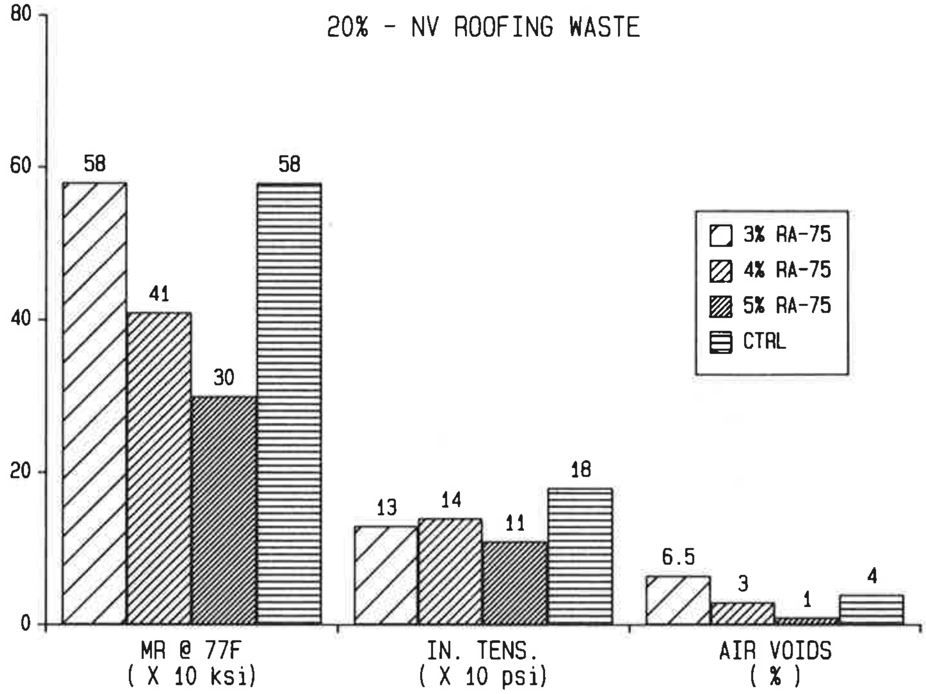


FIGURE 10 Effect of added binder content—Nevada roofing waste and RA-75—on resilient modulus and tensile strength.

extraction and recovery Method 1), an acceptable mixture can be prepared with recycling agent RA-75 and the Nevada waste. A harder binder (AR4000) is required to produce an acceptable mixture with the New Jersey roofing waste. The characteristics of the new binder have to be matched with the properties of the binder in the roofing waste and, perhaps, with the physical characteristics of the roofing waste solids.

Quantity of Roofing Waste

Figures 12–15 show some of the effects of the quantity of roofing waste. Unfortunately, the selected binder (RA-75) was too soft to produce mixtures with desirable properties from the New Jersey roofing waste. With the exception of Hveem stability, a mixture of suitable properties was produced with the

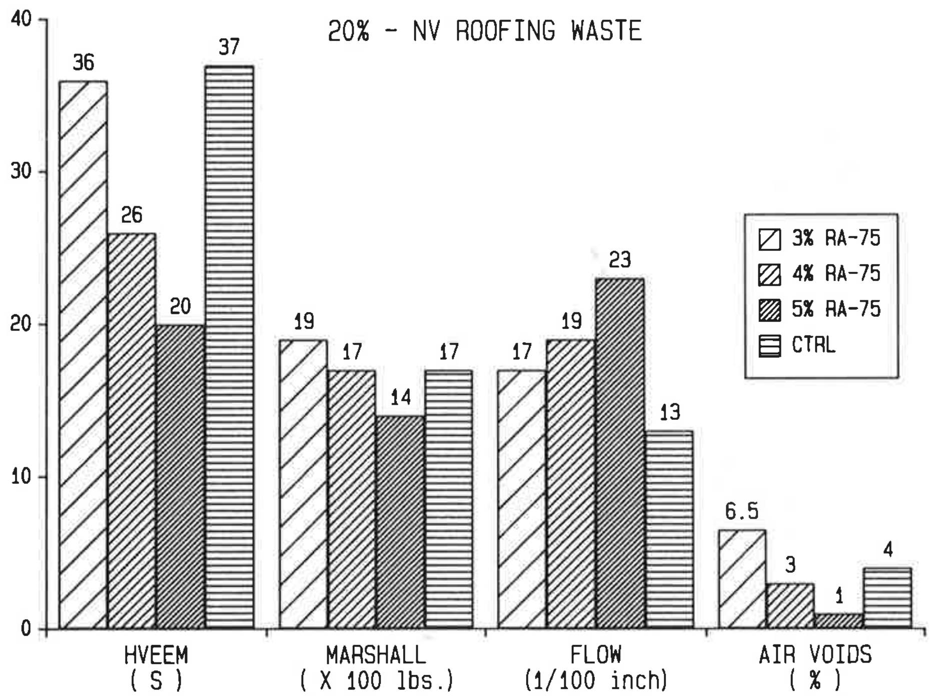


FIGURE 11 Effect of added binder content—Nevada roofing waste and RA-75—on stability and flow.

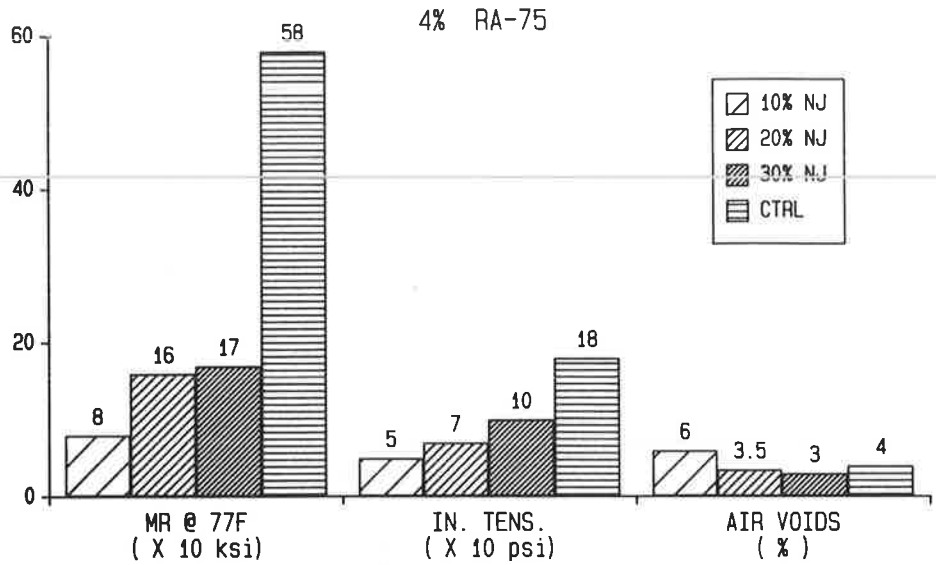


FIGURE 12 Effect of roofing waste quantity—New Jersey roofing waste and RA-75—on resilient modulus and tensile strength.

Nevada roofing waste up to the 20 percent level. Proper selection of the type and amount of binder should allow mixtures that contain 30 percent roofing waste to be produced.

Size of Roofing Waste

Test results reported by Paulsen et al. *(8)* indicate that the effect of the size of the roofing waste cannot be determined by the limited data collected in this study. In general, higher quantities of the finer sieve size materials are produced when hammer mill processing produces 1/4-in. material.

ECONOMIC CONSIDERATION

A limited economic study, which indicates that cost savings of 20 percent may be realized by using paving mixtures that contain roofing wastes, has been conducted. Adequate coverage of this study cannot be given here. Interested readers

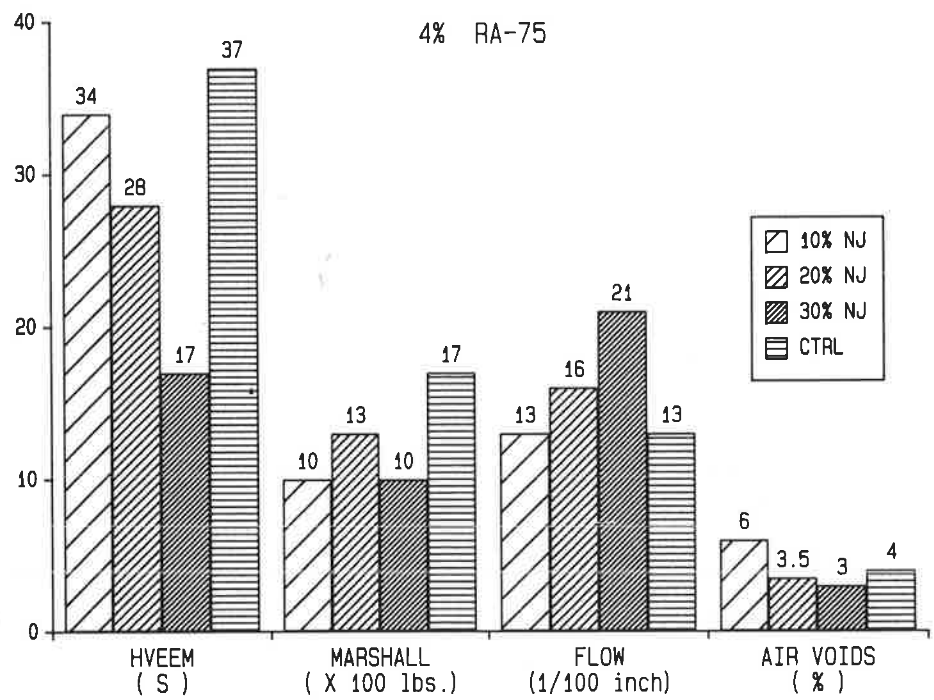


FIGURE 13 Effect of roofing waste quantity—New Jersey roofing waste and RA-75—on stability and flow.

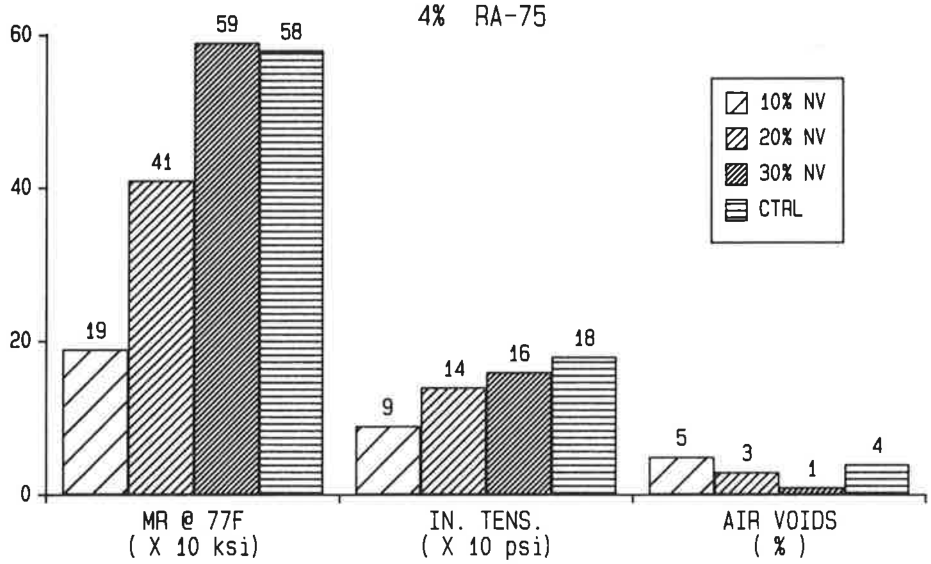


FIGURE 14 Effect of roofing waste quantity—Nevada roofing waste and RA-75—on resilient modulus and tensile strength.

are encouraged to contact the authors for a copy of the report on that study *(10)*.

CONCLUSIONS

Results from this study indicate that acceptable paving mixtures that include roofing waste can be made. However, because roofing asphalts are generally air-blown asphalts and are highly weathered, asphalt concrete mixtures that contain roofing waste may experience durability problems. Thus the application of these mixes may be limited to surface courses of lightly trafficked roads or to the lower layers of pavement sections. Long-term field performance needs to be established.

It was seen that different extraction and recovery methods can produce quite different asphalt contents and binder properties for the same roofing waste. This is an important factor to consider when designing asphalt concrete mixtures that contain roofing wastes. In addition, the relative digestion of the roofing waste asphalt by the recycling agent must be considered. Al-

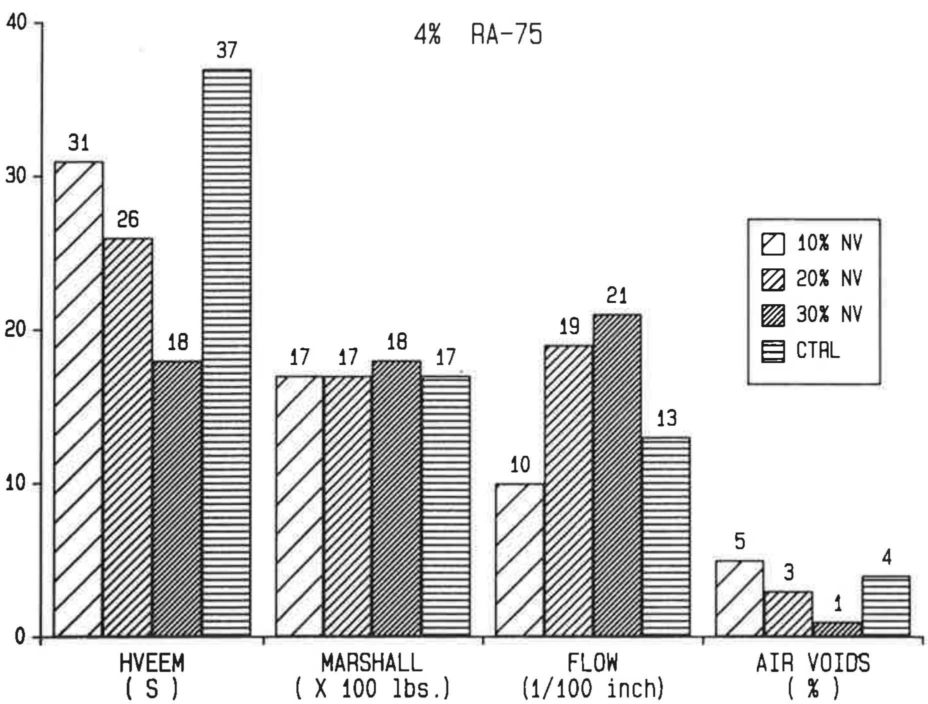


FIGURE 15 Effect of roofing waste quantity—Nevada roofing waste and RA-75—on stability and flow.

though difficult to quantify, the amount of roofing waste asphalt that becomes an "active" binder in the mix will also affect the properties of the asphalt concrete mixture.

Laboratory test results obtained in this preliminary study indicate that

1. Acceptable paving mixtures that contain 20 percent by volume roofing waste can be produced. With proper selection of binder type, binder quantities, and aggregate gradations, acceptable mixtures containing roofing waste quantities to, and perhaps beyond, the 30 percent level can probably be prepared.
2. The type of binder selected for use in a mixture containing roofing waste should be based on the stiffness (penetration and viscosity) of the asphalt cement in the roofing waste.
3. Improved asphalt cement extraction and recovery processes need to be developed for roofing wastes in order to effectively determine the properties of the asphalt cement in the roofing waste.
4. Gradations of conventional aggregates and roofing wastes should be considered when designing paving mixtures.

ACKNOWLEDGMENT

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Effect of Additives on Bituminous Highway Pavement Materials Evaluated by the Indirect Tensile Test

R. J. SALTER AND F. RAFATI-AFSHAR

The use of the indirect tensile test to investigate the effects of ethylene vinyl acetate, polypropylene fiber, rubber, and sulfur as additives on the fatigue and strength characteristics of bituminous mixtures is discussed. The test apparatus, preparation of specimens, and selection of binder contents using the Marshall procedure are described. Experimental results on the effects of stress level, binder content, and additive on the mean load cycles to failure are given and compared with the work of other investigations. Relationships between fatigue life and applied stress difference are developed and compared with experimental results. The addition of additives in most cases increased fatigue life; significant improvements were observed when the bitumen incorporated ethylene vinyl acetate and when a 50/50 sulfur/bitumen blend was used. Some improvement in fatigue properties was noted for rubber-modified mixes and 20/80 sulfur/bitumen mixes. Fiber-reinforced specimens had lower fatigue lives because of sample preparation difficulties.

Progress in the structural design of highway pavements has led to a need to characterize highway pavement materials by laboratory testing. This in turn has made it possible to investigate the effect of mix composition and additives on such important characteristics as fatigue resistance and creep in the laboratory before full-scale road trials are carried out.

SCOPE OF RESEARCH

The work described in this paper used the indirect tensile test employed extensively by Adedimila and Kennedy *(1)* to investigate the effects of ethylene vinyl acetate, polypropylene fiber, rubber, and sulfur on fatigue and strength characteristics. The objects of the research program were

- To use the indirect tensile test to obtain relationships between the number of load repetitions to failure and the resilient characteristics of bituminous mixtures and
- To investigate the effects of incorporating additives in bituminous mixes.

Mixes used in the experimental program were made from hard limestone aggregate with the continuous grading shown in Figure 1; the binder was a 50-pen bitumen supplied by Croda Hydrocarbons, Ltd.

Schools of Civil Engineering, University of Bradford, England.

PREPARATION OF SPECIMENS

Ethylene vinyl acetate was Evatane 18-150 supplied by Imperial Chemical Industries in the form of pellets approximatey 2 to 3 mm in diameter. Polypropylene fiber was manufactured by Don Fibres (Scotland), Ltd.; the 500-denier fiber staples had a length of 50 mm and a tensile strength of from 4.5 to 5.5 g per denier. Melting point of the fiber was given as 167°C with some softening to be expected at 150°C; 10 g of fiber were incorporated into each of the fiber-modified specimens.

Rubber additive was in the form of Pulvatex rubber powder supplied by Rubber Latex, Ltd., an unvulcanized rubber powder manufactured from concentrated natural rubber latex with 60 percent natural rubber and 40 percent by mass of a separator to keep the rubber particles from agglomerating. The rubber/bitumen ratio was 5/95 by mass.

The aggregate and filler were heated for a minimum of 2 hr at 165°C before mixing. For the specimens containing fiber it was noted that the fibers were susceptible to temperatures above 125°C, and mixing and compacting were achieved without exceeding this temperature. In the preparation of the sulfur-modified specimens, the heated sulfur powder was mixed with the bitumen before addition to the aggregate.

Compaction of specimens was carried out in standard Marshall molds, and compaction was achieved by 50 blows of the standard Marshall compaction hammer. Samples were cured for 3 weeks before testing. It has been suggested that with end-compacted specimens differences in density may occur along the specimen length. To determine the extent, if any, of these differences, the compacted specimens were cut into three equal discs and their densities determined. Four specimens were divided in this way, and the maximum density difference observed was 4.1 percent.

TESTING OF SPECIMENS

The indirect tensile testing of specimens was carried out by applying a repeated compressive load that acted parallel to and along the vertical diametral plane. Load was transmitted to the specimen through a 20-mm-wide curved loading strip of the same curvature as the specimen being tested.

The repeated compressive loads were applied to the specimen by a hydraulic ram, and the duration and value of the applied load were determined by signals from an electrical

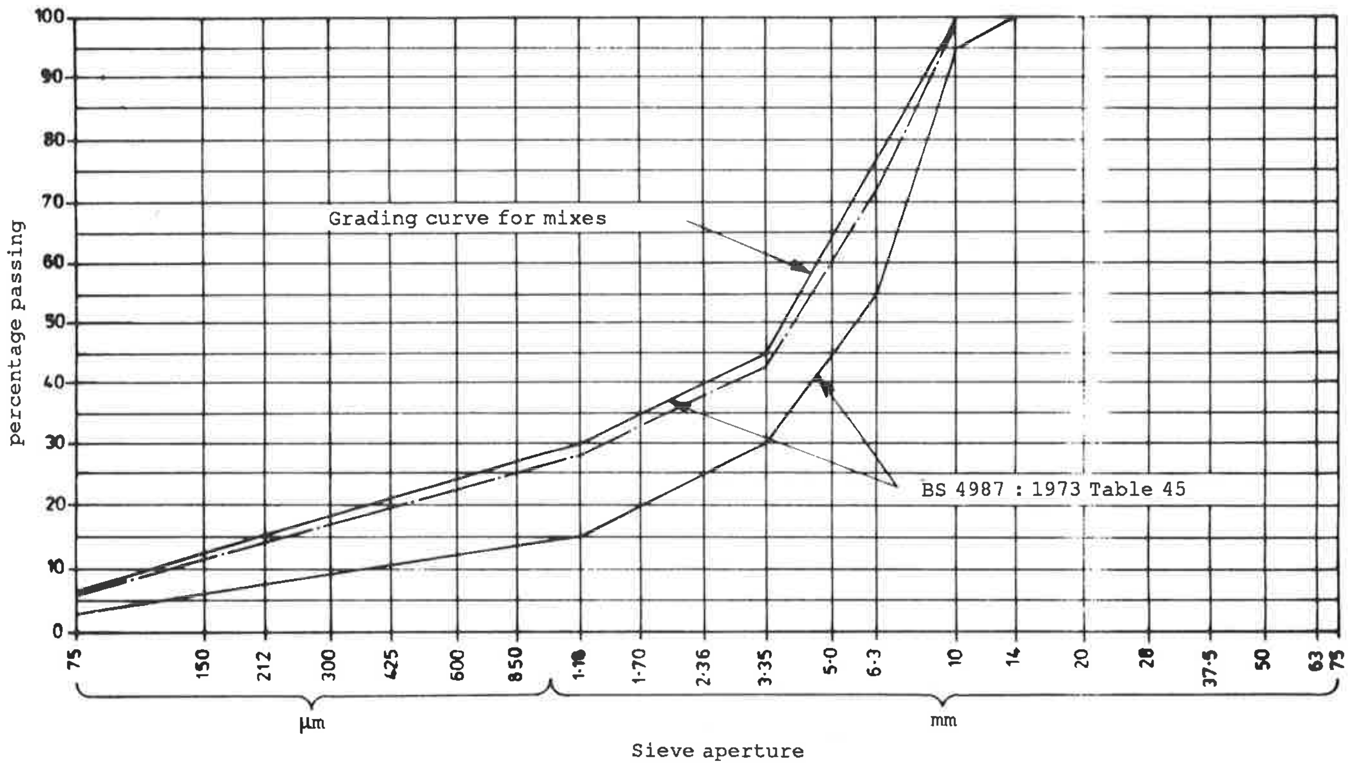

FIGURE 1 Aggregate grading curve for test specimens.

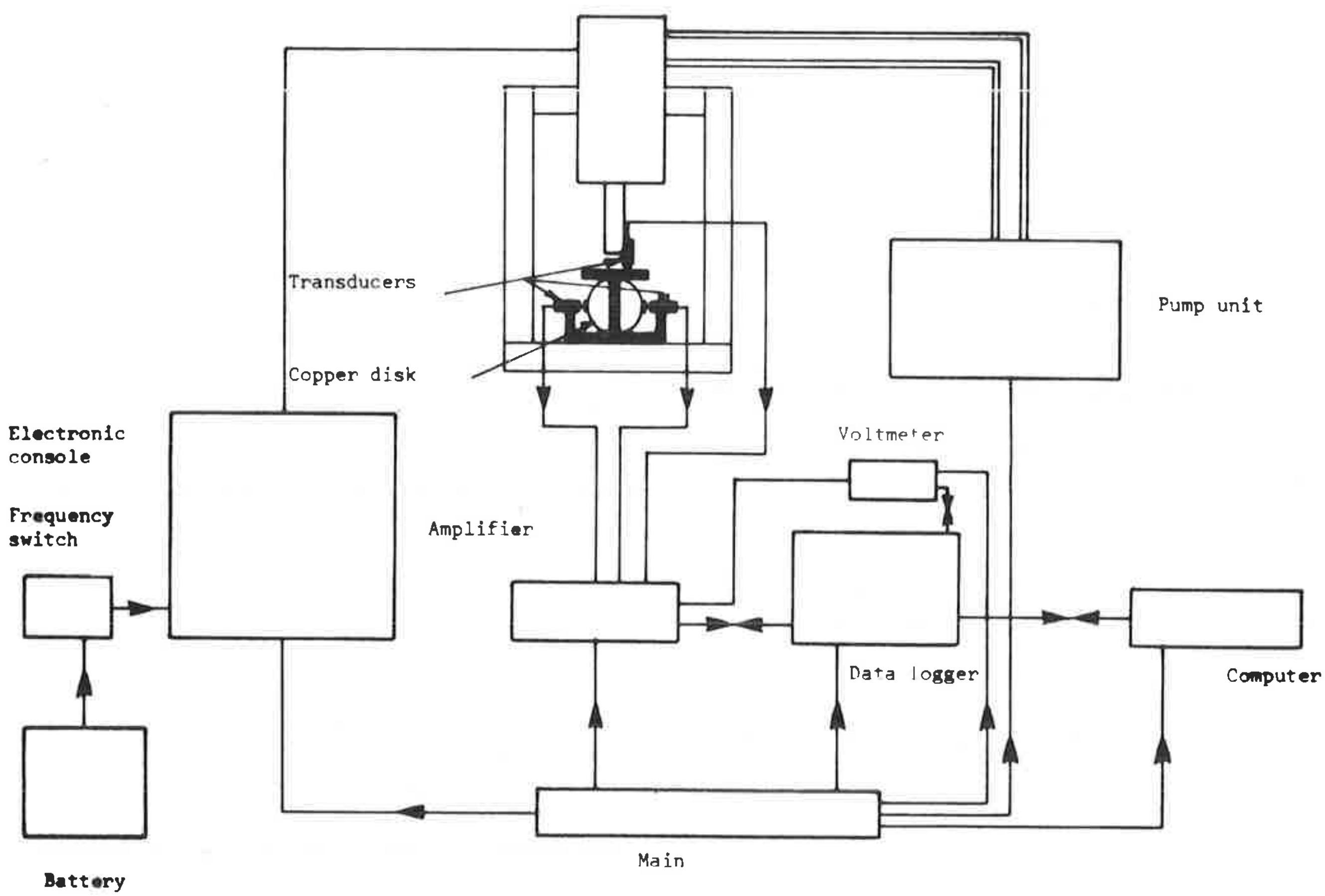

FIGURE 2 Setup of test equipment.

TABLE 1 TEST SPECIMENS USED IN EXPERIMENTAL PROGRAM

Bituminous Specimens			Bituminous Specimens Containing Fiber			Bituminous Specimens Containing Evatane (copolymer)			Bitumen Specimens Containing Sulfur (20 percent of bitumen weight)			Bituminous Specimens Containing Sulfur (50 percent of bitumen weight)			Rubber-Modified Bituminous Specimens (5 percent of bitumen weight)		
Bitumen Content (%)	No. of Speci-mens	Stress Level (kPa)	Bitumen Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)
5	3	283	5	3	283	5.0	4	283	5.5	4	283	5.0	4	283	4.8	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
6	3	283	6	3	283	6.0	4	283	6.0	4	283	6.0	4	283	5.2	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
7	3	283	7	3	283	6.5	4	283	6.5	4	283	6.5	4	283	5.6	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
8	3	283				7.0	4	283	7.0	4	283	7.0	4	283	6.0	4	283
	3	496					4	496		4	496		4	496		4	496
	3	662					4	662		4	662		4	662		4	662
						8.0	4	283									
							4	496									
							4	662									

TABLE 2 MARSHALL TEST DATA: DENSITY AND MARSHALL QUOTIENT AT OPTIMUM BINDER CONTENT FOR BITUMINOUS AND MODIFIED BITUMINOUS MIXES

Type of Specimen	Optimum Binder Content (%)	Stability (kN)	Flow (mm)	Density (g/mL)	Percentage of Voids	QM[a] (kN/mm)
Bituminous	6	10.2	5.0	2.40	3.8	2.04
Bituminous containing fiber	6	9.7	5.6	2.26	9.1	1.72
Evatane-modified bituminous (5:95 EVA:bitumen ratio)	6.5	13.6	4.3	2.42	2.4	3.21
Sulfur S20–modified bituminous (20:80 sulfur:bitumen ratio)	5.5	10.0	3.1	2.42	1.3	3.22
Sulfur S50–modified bituminous (50:50 sulfur:bitumen ratio)	6.5	18.4	3.0	2.42	3.7	6.13
Rubber-modified bituminous (5:95 rubber:bitumen ratio)	5.2	9.5	3.0	2.40	3.6	3.22

[a]QM = Marshall quotient.

console that was part of the data acquisition and control system.

Plotting of the load and the horizontal and vertical displacement patterns, and recording values of load and strain, was carried out by an HP 85A computer. Readings and graphic plots were made at preset time intervals under computer instruction. A schematic layout of the apparatus is shown in Figure 2.

Testing was carried out in a controlled-temperature laboratory with temperature maintained in the 22°C to 24°C range for all tests. Loading frequency was at a rate of one cycle per second with a load duration of 0.4 sec and a rest period of 0.6 sec.

Previous research *(2, 3)* has indicated that fatigue life, modulus of elasticity, tensile strength, and Poisson's ratio are affected by stress level and mix composition. In the present study the effect of binder content and stress level on the fatigue characteristics of mixes modified by additives was investigated. A range of binder contents, chosen with reference to the optimum binder content as determined by the Marshall testing procedure, was selected. Details of the mixes selected are given in Table 1, and the results of the Marshall test procedure are given in Table 2.

To carry out a fatigue test all measuring devices were calibrated before testing, the specimen was placed in the test rig, and the upper platen was brought into light contact with the specimen. Vertical and horizontal transducers were fixed in place and a 222.5-N (50-lb) preload was applied to remove any slack between the upper platen and the specimen. The HP 85 was loaded with the control program and its first action zeroed the transducer and load cell readings. The additional load required to produce the required stress level for the test (283, 496, or 662 kPa) was then applied at a frequency of one cycle per second. Load and vertical and horizontal strain patterns were accurately traced by taking 20 readings per cycle. These were made at preset time intervals as shown in Figure 3. Testing continued until complete fracture occurred.

Experimental Results

Details of the fatigue test results for all the specimen types obtained by indirect tensile testing are given in Table 3 and shown graphically in Figures 4–6. It can be seen that polymer-modified binders offer an improvement in fatigue life compared with mixes that contain the other additives evaluated or unmodified mixes.

Fatigue and Applied Stress Relationship

The relationships between the logarithm of applied stress and the logarithm of fatigue life for bituminous and additive-modified bituminous specimens were investigated in this program. As in previous studies, a linear relationship is found to exist between the logarithm of applied stress and the logarithm of fatigue life, which was expressed as

$$Nf = K_2\left(\frac{1}{\sigma}\right)^{n_2} \tag{1}$$

where

Nf = fatigue life,
σ = applied stress, and
K_2, n_2 = constants that depend on mixture properties and temperatures.

The values of the constants K_2 and n_2 for all types of specimens are given in Table 4. The K_2-values range between 4.50×10^{10} and 4.03×10^{13}. The n_2-values for all fatigue test results vary between 2.62 and 3.75.

Table 5 gives typical values of K_2 and n_2 obtained by Pell and Cooper *(2)*, Monismith et al. *(4)*, Kennedy and Moore *(5)*, and Raithby and Sterling *(6, 7)*.

Table 6 gives typical values of K_2 and n_2 and K_1 and n_1 obtained by Adedimila and Kennedy *(1)* for the repeated-load indirect tensile test.

Figure 7 shows typical logarithmic stress–fatigue life relationships for a variety of mixtures tested under different conditions by a number of investigators (including this research group) using various types of fatigue tests; Figure 7 shows typical values of K_2 and n_2. It can be seen that there are large differences in fatigue lives obtained in the various studies especially for the indirect tensile test. A comparison of K_2- and

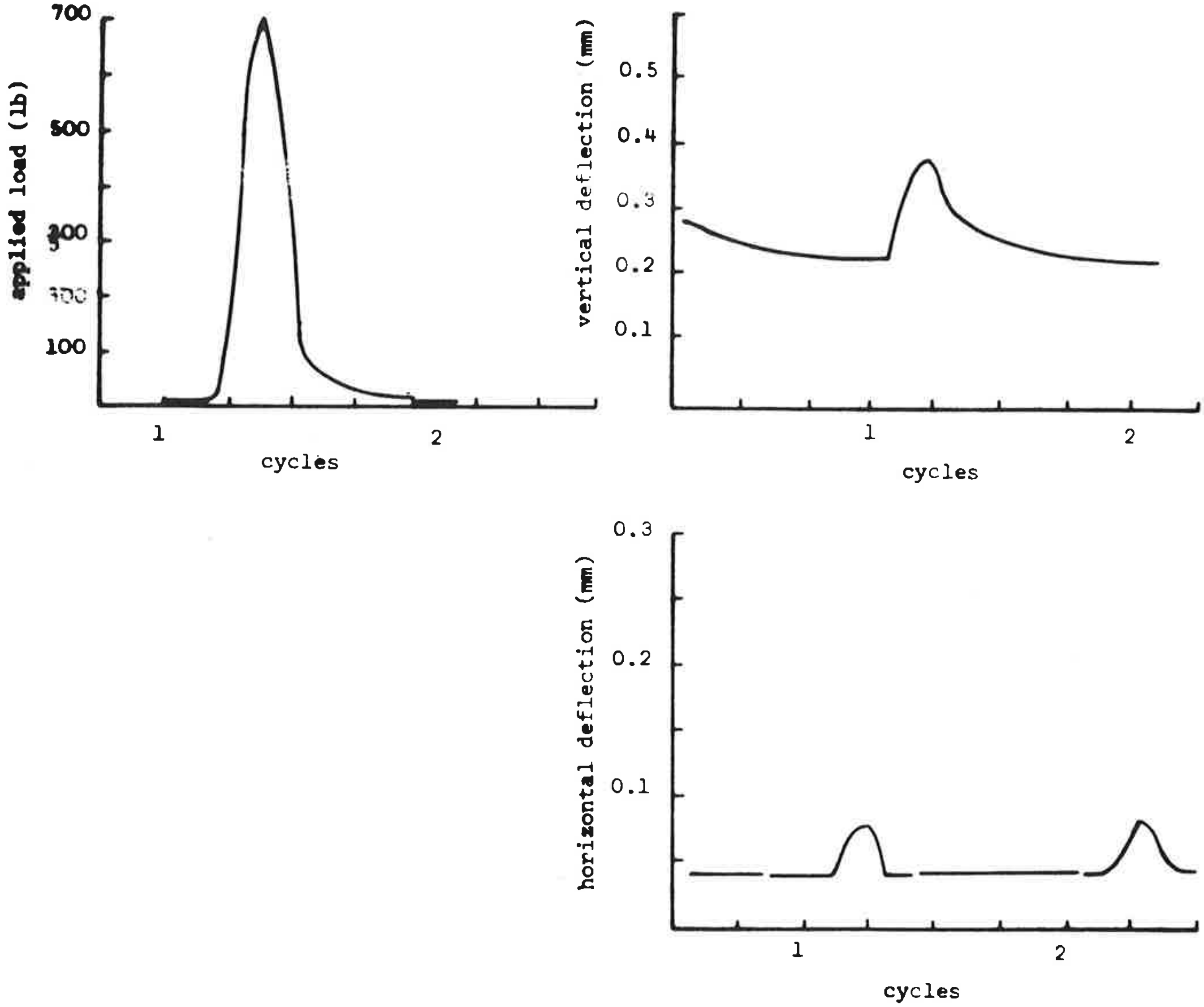

FIGURE 3 Typical load and vertical and horizontal deflection during the repeated-load indirect tensile test.

n_2-values indicates that the differences are mainly in K_2-values and that the values of n_2 are approximately the same. Values of K_2 range from 6.19×10^5 to 2.24×10^{21}, with the lower values associated with the repeated-load indirect tensile test (6.19×10^5 to 1.9×10^{13}) and with higher testing temperatures. Values of n^2 ranged from 2.56 to 6.43.

It is thought that the differences in the fatigue results obtained by different test methods are due to differences in loading and environmental testing conditions and the composition of the specimens.

Previous findings *(8)* have indicated that, because of the biaxial state of stress developed in the indirect tensile specimen, fatigue life should be evaluated in terms of stress difference, which for the indirect tensile test is equal to $4\sigma t$. Expressing fatigue life in terms of stress difference changes the values of K_2. The relationship can be expressed as

$$Nf = K_2'\left(\frac{1}{\Delta\sigma}\right)^{n_2} \quad (2)$$

where $\Delta\sigma$ is the stress difference; K_2' is a constant, dependent on mixture properties and testing temperature; and n_2 is as previously defined.

The relationships between the logarithm of applied stress and the logarithm of fatigue life have been shifted to the right so that values of K_2' are larger than values of K_2. Values of K_2' ranged from 1.69×10^{12} to 7.00×10^{15} (Table 4). The obtained values of K_2' are generally smaller than those reported for flexural and axial load tests for other mixtures *(4, 8)*. Many researchers have found that temperature change and different load durations and types of binder and aggregate affect the values of K_2', and K_2. Adedimila and Kennedy *(1)* have reported that factors such as temperature and aggregate bitumen content influence values of K_2 and K_2'. Increased test temperature has the main effect on values of K_2' and K_2. These values generally decrease with an increase in temperature. For this research program the test temperature, type of aggregate, and type of bitumen used have been kept constant for all specimens. The main mix variable is the bitumen content, which has a limited effect on the value of K_2 and K_2'. The combined effect

TABLE 3 MEAN, STANDARD DEVIATION, AND COEFFICIENT OF VARIATION OF FATIGUE LIFE FOR BITUMINOUS SPECIMENS

Type of Specimen	Bitumen Content (%)	Stress Level (kPa)	No. of Specimens	Mean Cycles to Failure	Standard Deviation	CV (%)
Bituminous	5	283	3	18,193	12237	67.0
	5	496	3	4,740	779	16.4
	5	662	3	2,050	292	14.2
	6	283	3	22,322	10113	45.3
	6	496	3	2,863	742	25.9
	6	662	3	1,247	54	4.3
	7	283	3	27,956	9503	34.0
	7	496	3	4,790	714	14.9
	7	662	3	1,380	212	15.4
	8	283	3	20,519	3229	16.0
	8	496	3	2,930	453	15.5
	8	662	3	1,130	61	5.4
Bituminous containing fiber	5	283	3	26,826	1861	6.9
	5	496	3	3,500	320	9.1
	5	662	3	880	162	18.4
	6	283	3	17,200	2040	11.9
	6	496	3	4,200	1407	33.5
	6	662	3	1,536	418	27.3
	7	283	3	21,941	1846	8.4
	7	496	3	3,105	740	23.8
	7	662	3	1,288	423	32.8
Bituminous containing 5% EVA	5	283	4	39,100	3158	8.0
	5	496	4	5,195	1388	26.7
	5	662	4	2,560	723	28.2
	6	283	4	46,030	3210	7.0
	6	496	4	5,790	1151	20.0
	6	662	4	2,750	633	23.0
	6.5	283	4	41,300	2786	6.7
	6.5	496	4	6,100	729	12.0
	6.5	662	4	2,020	396	19.5
	7	283	4	35,731	2439	6.8
	7	496	4	5,160	386	7.5
	7	662	4	1,855	253	13.6
	8	283	4	27,700	3558	12.8
	8	496	4	3,570	620	17.4
	8	662	4	1,497	105	7.0
Bituminous containing 20% sulfur/80% bitumen	5.5	283	4	18,457	1770	9.6
	5.5	496	4	3,354	453	13.5
	5.5	662	4	1,101	195	17.7
	6	283	4	20,596	2938	14.3
	6	496	4	3,707	463	12.5
	6	662	4	1,215	232	19.1
	6.5	283	4	17,819	1935	10.8
	6.5	496	4	3,492	416	11.9
	6.5	662	4	1,441	188	13.0
	7	283	4	15,976	3535	22.1
	7	496	4	3,315	662	20.0
	7	662	4	1,297	451	34.7
Bituminous containing 50% sulfur/50% bitumen	5.5	283	4	24,025	2731	11.4
	5.5	496	4	4,393	1041	23.7
	5.5	662	4	1,375	294	21.4
	6.0	283	4	26,277	2895	11.0
	6.0	496	4	4,900	1631	33.3
	6.0	662	4	1,645	420	25.5
	6.5	283	4	36,956	2575	7.0
	6.5	496	4	5,290	1347	25.5
	6.5	662	4	2,390	960	40.0
	7.0	283	4	31,500	3286	10.4
	7.0	496	4	4,012	603	15.0
	7.0	662	4	1,850	263	14.0
Bituminous containing rubber	4.8	283	4	20,533	2856	14.0
	4.8	496	4	3,578	577	16.0
	4.8	662	4	1,350	209	15.5
	5.2	283	4	23,474	3437	14.6
	5.2	496	4	3,232	477	14.7
	5.2	662	4	1,422	168	11.8
	5.6	283	4	26,540	2777	10.5
	5.6	496	4	4,202	737	17.5
	5.6	662	4	1,600	366	22.6
	6.0	283	4	22,181	3613	16.3
	6.0	496	4	3,820	1285	33.6
	6.0	662	4	1,410	320	22.7

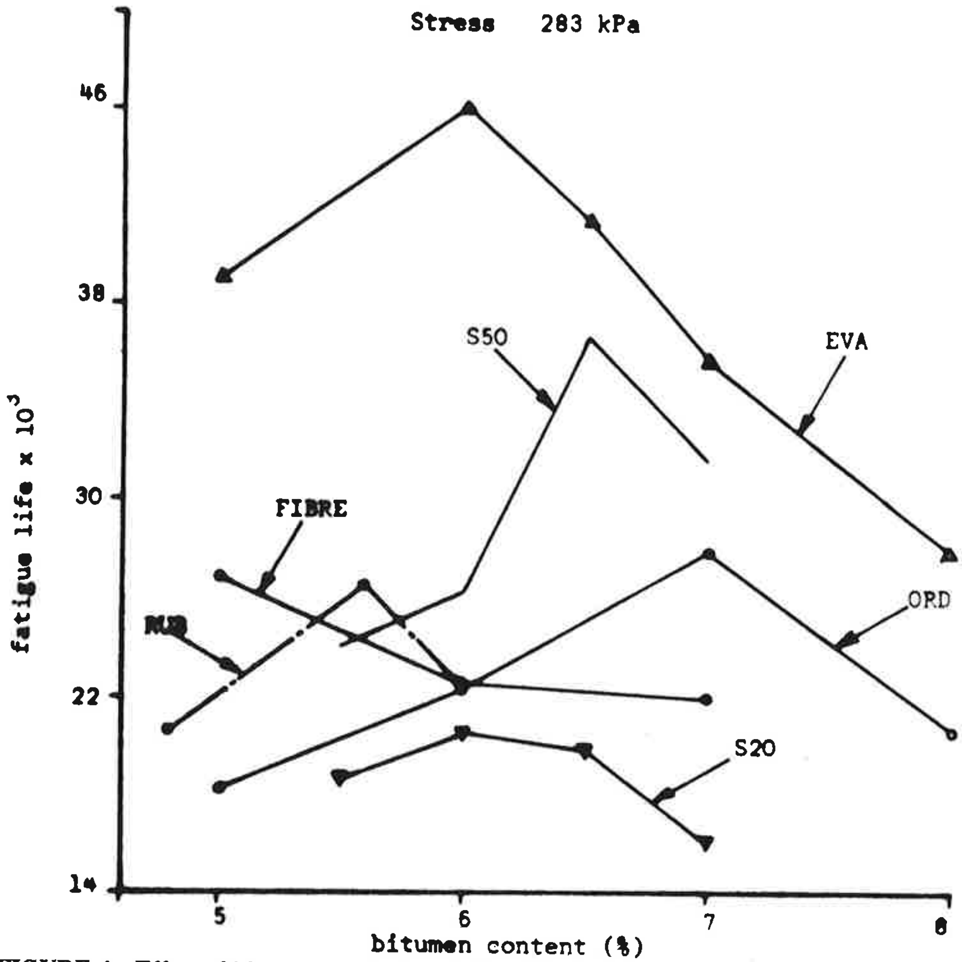

FIGURE 4 Effect of bitumen content on fatigue life (stress = 283 kPa).

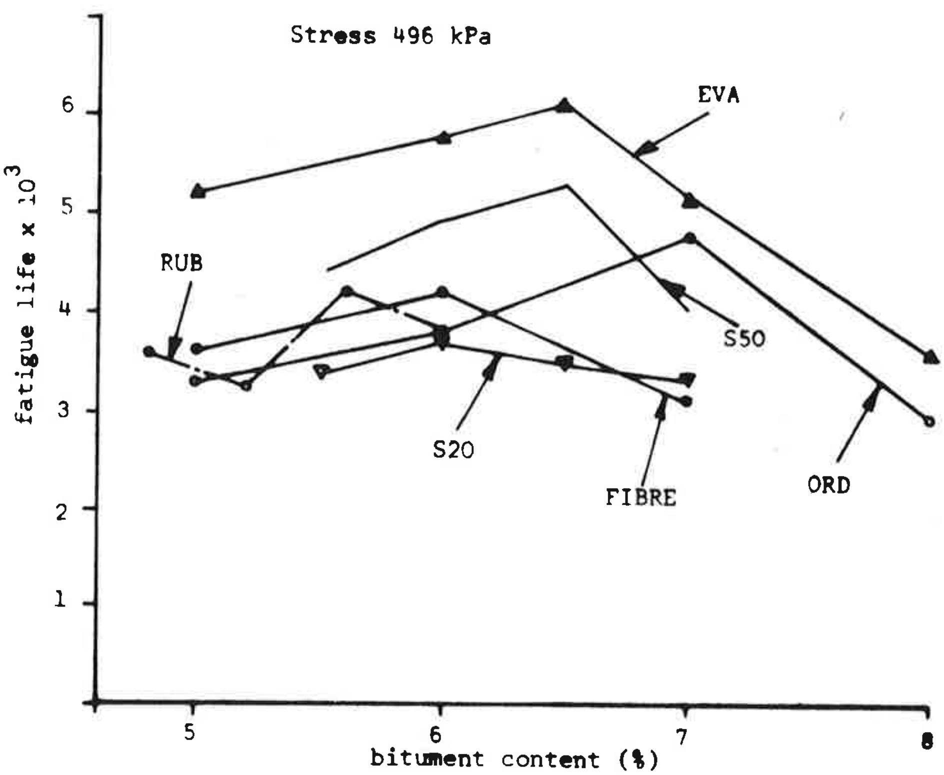

FIGURE 5 Effect of bitumen content on fatigue life (stress = 496 kPa).

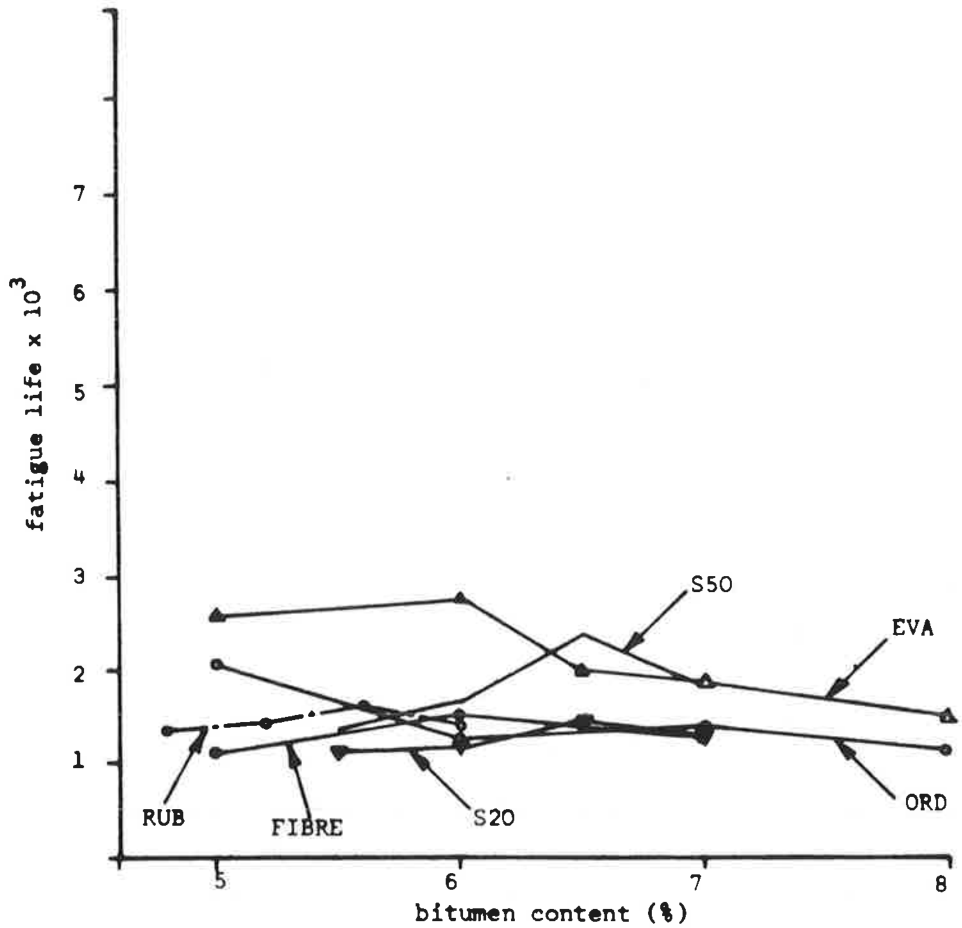

FIGURE 6 Effect of bitumen content on fatigue life (stress = 662 kPa).

TABLE 4 EXPERIMENTAL VALUES OF K_2, K_2', AND n_2

Type of Specimen	Bitumen Content (%)	No. of Specimens	K_2	K_2'	n_2	Correlation Coefficient
Bituminous	5.0	9	4.50×10^{10}	1.69×10^{12}	2.62	0.98
	6.0	9	3.54×10^{12}	3.73×10^{14}	3.34	0.99
	7.0	9	9.97×10^{12}	1.24×10^{15}	3.48	0.99
	8.0	9	3.65×10^{12}	3.74×10^{14}	3.36	0.99
Fiber-modified bituminous	5.0	9	4.03×10^{13}	7.00×10^{15}	3.74	0.99
	6.0	9	1.22×10^{12}	9.73×10^{13}	3.15	0.98
	7.0	9	4.54×10^{12}	4.69×10^{14}	3.39	0.98
Polymer-modified bituminous	5.0	12	4.11×10^{12}	3.90×10^{14}	3.28	0.99
	6.0	12	8.05×10^{12}	8.56×10^{14}	3.37	0.99
	6.5	12	2.00×10^{13}	2.66×10^{15}	3.54	0.99
	7.0	12	1.18×10^{13}	1.45×10^{15}	3.47	0.99
	8.0	12	7.38×10^{12}	8.53×10^{14}	3.44	0.99
Sulfur-modified bituminous (20/80 sulfur/bitumen ratio)	5.5	12	2.11×10^{12}	1.97×10^{14}	3.28	0.99
	6.0	12	2.47×10^{12}	2.42×10^{14}	3.29	0.99
	6.5	12	3.05×10^{11}	1.79×10^{13}	2.95	0.99
	7.0	12	2.93×10^{11}	1.82×10^{13}	2.96	0.98
Sulfur-modified bituminous (50/50 sulfur/bitumen ratio)	5.5	12	3.63×10^{12}	3.61×10^{14}	3.33	0.98
	6.0	12	1.81×10^{13}	2.40×10^{14}	3.60	0.98
	6.5	12	3.75×10^{12}	3.60×10^{14}	3.27	0.99
	7.0	12	6.78×10^{12}	7.65×10^{14}	3.40	0.99
Rubber-modified bituminous (5/95 rubber/bitumen ratio)	4.8	12	1.43×10^{12}	1.10×10^{14}	3.20	0.99
	5.2	12	2.82×10^{12}	3.14×10^{14}	3.30	0.99
	5.6	12	3.46×10^{12}	3.52×10^{14}	3.31	0.99
	6.0	12	1.93×10^{12}	1.69×10^{14}	3.24	0.98

TABLE 5 FATIGUE DATA FOR VARIOUS TESTS AND INVESTIGATIONS

Test and Investigator	Mixture	Binder Content (%)	Binder Penetration	Temperature (°F)	K_2	n_2
Flexure, Monismith et al. *(4)*	British 594	7.9	40–50	68	1.36×10^{11}	2.87
	California	6.0	85–100	68	1.55×10^{11}	3.51
	California	6.0	60–70	68	7.29×10^{12}	4.21
	California	6.0	40–50	68	1.97×10^{15}	4.93
	California	6.2	60–70	68	6.01×10^{10}	3.24
	Gonzales lab surface	6.0	85–100	40	1.78×10^{16}	5.09
	Surfaces 1 and 2	4.9	85–100	40	1.16×10^{18}	5.71
Axial loading Raithby and Sterling *(6, 7)*	Pell's mix G	6.5	38	50	2.59×10^{13}	4.11
	Pell's mix G	6.5	38	50	1.72×10^{19}	6.43
	Pell's mix G	6.5	38	50	2.24×10^{21}	5.97
	Pell's mix G	6.5	38	50	2.13×10^{17}	5.28
	Pell's mix G	6.5	38	77	3.65×10^{11}	3.87
	Pell's mix G	6.5	38	77	5.78×10^{13}	4.76
	Pell's mix G	6.5	38	77	2.49×10^{13}	4.09
Rotating cantilever, Pell and Cooper *(2)*	HRA base	6.0	40–50	50	3.7×10^{16}	5.40
	HRA base course A13	6.8	40–50	50	1.1×10^{12}	3.50
	AC wearing course B6	6.0	60–70	50	3.9×10^{15}	4.90
	DBH base	4.7	90–110	50	3.0×10^{12}	3.90
	DTH base course	6.0	40–60	50	7.5×10^{14}	6.40
Repeated-load indirect tension, Kennedy and Moore *(5)*	Limestone	4.0	88	75	6.19×10^{5}	2.56
	Limestone	5.0	88	75	8.11×10^{6}	2.84
	Limestone	6.0	88	75	6.9×10^{7}	3.23
	Limestone	7.0	88	75	4.76×10^{8}	3.88
	Limestone	8.0	88	75	5.88×10^{7}	3.42
	Gravel	4.0	88	75	2.74×10^{6}	3.24
	Gravel	5.0	88	75	9.4×10^{6}	3.14
	Gravel	6.0	88	75	6.62×10^{7}	3.34
	Gravel	7.0	88	75	3.56×10^{8}	3.80
	Gravel	8.0	88	75	1.9×10^{7}	3.13

TABLE 6 FATIGUE DATA FOR REPEATED-LOAD INDIRECT TENSILE TESTS REPORTED BY ADEDIMILA AND KENNEDY *(1)*

Mixture	Binder Content[a] (%)	Temperature (°F)	K_2	n_2	K_1	n_1
Limestone	6	50	4.59×10^{10}	3.74	1.29×10^{-11}	3.73
Gravel	6	50	1.9×10^{13}	5.14	5.65×10^{-17}	5.19
Limestone	4	75	9.36×10^{5}	2.67	9.97×10^{-8}	2.67
Limestone	5	75	8.75×10^{6}	2.86	4.49×10^{-8}	2.86
Limestone	6	75	5.63×10^{7}	3.14	3.67×10^{-9}	3.20
Limestone	7	75	4.29×10^{8}	3.84	5.48×10^{-11}	3.84
Limestone	8	75	4.20×10^{7}	3.33	6.24×10^{-9}	3.33
Gravel	4	75	1.62×10^{6}	3.06	2.10×10^{-9}	3.06
Gravel	5	75	5.43×10^{6}	2.96	6.82×10^{-9}	2.97
Gravel	6	75	1.09×10^{8}	3.46	9.31×10^{-11}	3.49
Gravel	7	75	8.68×10^{8}	4.11	2.9×10^{-11}	4.13
Gravel	8	75	1.86×10^{7}	3.13	3.71×10^{-9}	3.13
Limestone	6	100	5.27×10^{5}	2.66	5.01×10^{-7}	2.66
Gravel	6	100	3.26×10^{5}	2.72	5.75×10^{-8}	2.72

[a]The binder was an AC-10 asphalt cement with a penetration value of 88.

of aggregate grading and binder content and the manufacture of specimens may explain the variations in K_2-values. It has been reported by Monismith and Epps *(9)* that in controlled tests the greater the stiffness the flatter the slope (high n_2-values) of the regression line and the longer the fatigue life.

Relationship Between Fatigue and Initial Strain

Fatigue life relationships are often expressed in terms of initial strain for controlled-stress tests and repeated strain for controlled-strain tests. The relationships between fatigue life and initial strain for bituminous specimens and additive-modified bituminous specimens are shown in Figure 8. A regression analysis is used to obtain values of the constants K_1 and n_1 for the general equation .

$$Nf = K_1\left(\frac{1}{\varepsilon_{mix}}\right)^{n_1} \tag{3}$$

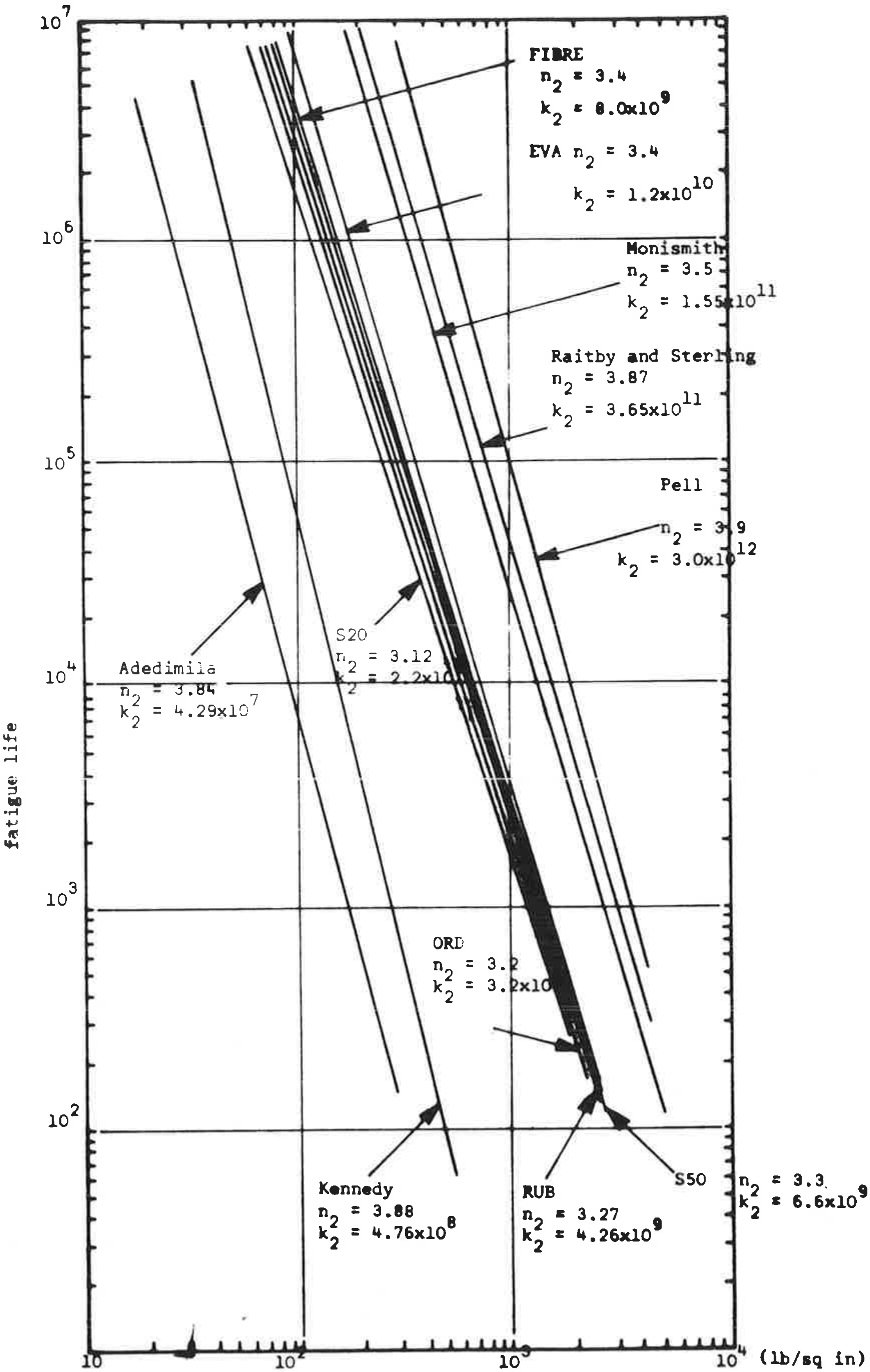

FIGURE 7 Typical stress-fatigue relationships for various tests.

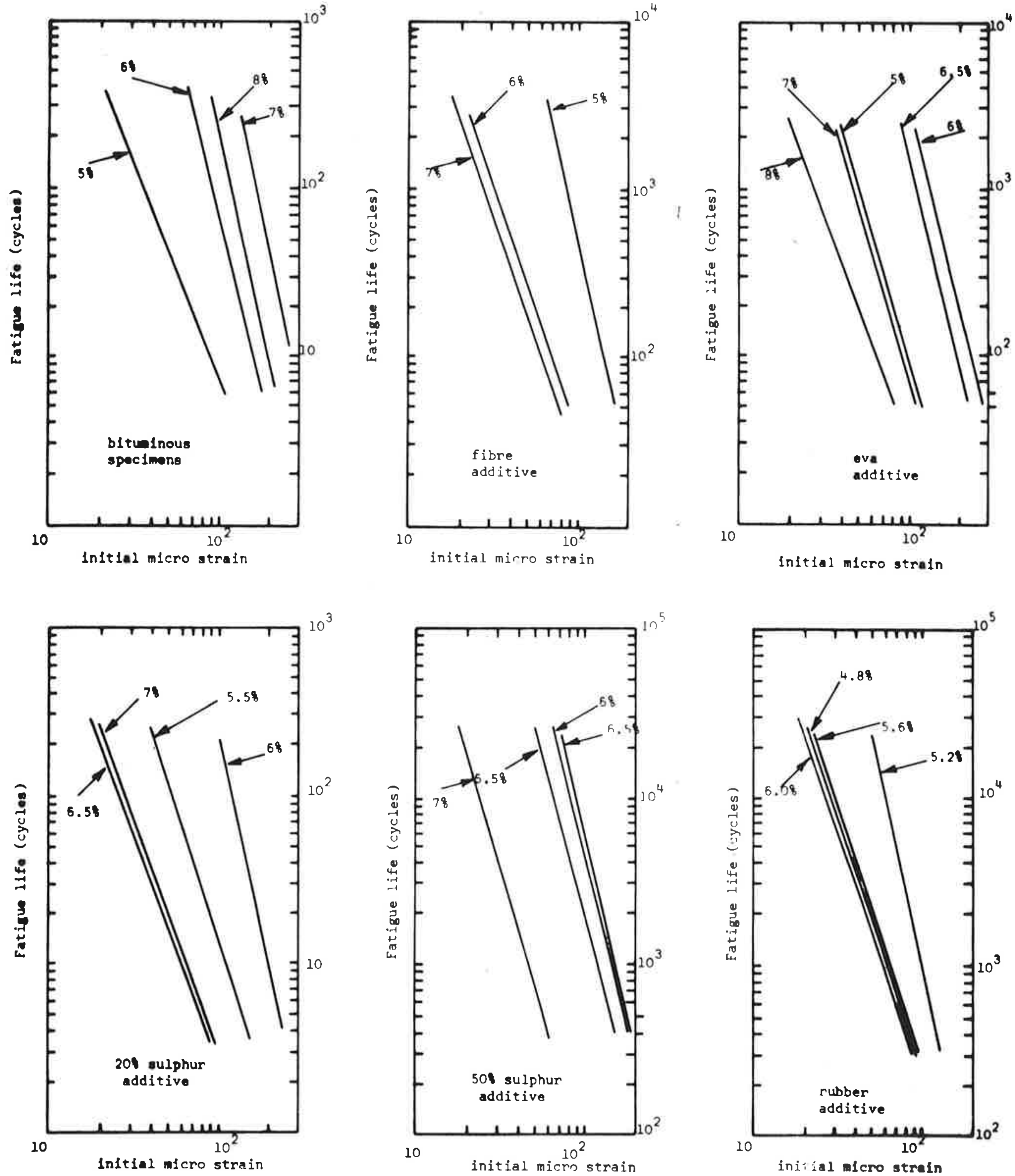

FIGURE 8 Relationships between fatigue life and initial strain.

where

Nf = fatigue life,
ε_{mix} = initial strain in the mixture, and
K_1, n_1 = constants.

A summary of values of K_1 and n_1 is given in Table 7. Values of K_1 range from 7.93×10^{-7} to 3.10×10^{-4} for bituminous and modified bituminous mixtures. Values of n_1 ranged from 2.61 to 3.74 for bituminous and modified bituminous specimens.

It is rather difficult to compare the values of K_1 and n_1 obtained in this research program with those obtained from previous flexural and axial load and dynamic indirect tensile tests because the mixtures, bitumen contents, testing temperatures, and testing procedures are different for each type of test.

Adedimila and Kennedy *(1)* conducted dynamic indirect tensile tests on bituminous mixtures that contained gravel or limestone. The reported values for K_1 and n_1 for the part of their test program that dealt with similar mixtures with the same bitumen contents and the same test temperature can, however, be compared with the values of K_1 and n_1 in Table 7. Their K_1-values ranged from 2.9×10^{-13} to 4.49×10^{-8}, and n_1 ranged from 2.67 to 4.13.

CONCLUSION

To predict fatigue life with respect to applied stress and mixture strain, regression analysis was used to obtain the best-fit values of the constants K_1, n_1, K_2, and n_2 in Equations 1 and 3. To compare the predicted and the actual mean fatigue values, the percentage of error for each bitumen content was calculated. Values of the actual mean fatigue life and the predicted mean fatigue life, along with the percentage of error for all types of mixtures, with respect to applied stress and mixture strain were determined, and the percentage of error for the bituminous and the modified bituminous mixes indicated that, in general, the difference between the actual and predicted mean fatigue life values were small; no particular trend was noticed in the degree of error with increase in binder content. Increased applied stress had little effect on the percentage of error. Some isolated cases for which the percentage of error reached 15 percent can be associated with the difficulty in maintaining high consistency in the manual preparation and manufacture of specimens.

The difference between the predicted values of applied stress and mixture strain were small. The percentage of error varied with binder content for both sets of analysis.

Marshall stability tests were used to determine optimum binder contents for fatigue testing, and it was noted that the addition of 20 percent sulfur to the binder did not increase stability compared with that of a normal bituminous specimen but did decrease the flow, which resulted in an increased Marshall quotient. Fiber specimens produced disappointing test values because of low densities caused by mixing and compacting difficulties. Mixtures with rubber-modified binders also had low stability values but decreased flow values. The two most successful mixes were those with ethylene vinyl acetate binders and those with binder modified by 50 percent sulfur.

An analysis of the indirect tensile test fatigue results indicated low coefficients of variation that exhibited a general decrease with an increase in binder content. Tests carried out at different binder contents showed that several mixes had an optimum binder content for maximum fatigue life. These binder contents were within ± 1 percent of the optimum derived by the Marshall test.

TABLE 7 EXPERIMENTAL VALUES OF K_1 AND n_1

Type of Specimen	Bitumen Content (%)	No. of Specimens	K_1	n_1	Correlation Coefficient
Bituminous	5.0	9	6.35×10^{-4}	2.61	0.98
	6.0	9	2.76×10^{-6}	3.34	0.99
	7.0	9	7.93×10^{-7}	3.48	0.99
	8.0	9	1.23×10^{-5}	3.36	0.99
Fiber-modified bituminous	5.0	9	1.59×10^{-6}	3.74	0.99
	6.0	9	6.18×10^{-5}	3.14	0.98
	7.0	9	4.41×10^{-5}	3.38	0.98
	7.0	9	4.41×10^{-5}	3.38	0.98
Polymer-modified bituminous	5.0	12	6.32×10^{-6}	3.26	0.99
	6.0	12	2.67×10^{-6}	3.37	0.99
	6.5	12	1.14×10^{-6}	3.55	0.99
	7.0	12	5.33×10^{-6}	3.48	0.99
	8.0	12	2.44×10^{-6}	3.44	0.99
Sulfur-modified bituminous	5.5	12	3.13×10^{-5}	3.28	0.99
(20/80 sulfur/bitumen ratio)	6.0	12	2.29×10^{-5}	3.30	0.99
	6.5	12	3.10×10^{-4}	2.94	0.99
	7.0	12	4.71×10^{-4}	2.95	0.98
Sulfur-modified bituminous	5.5	12	2.28×10^{-6}	3.32	0.98
(50/50 sulfur/bitumen ratio)	6.0	12	2.87×10^{-6}	3.26	0.98
	6.5	12	3.46×10^{-6}	3.27	0.99
	7.0	12	3.62×10^{-6}	3.41	0.99
Rubber-modified bituminous	4.8	12	5.51×10^{-6}	3.20	0.99
(5/95 rubber/bitumen ratio)	5.2	12	2.33×10^{-6}	3.32	0.99
	5.6	12	3.40×10^{-6}	3.32	0.99
	6.0	12	9.70×10^{-6}	3.22	0.98

In these tests at optimum binder content, the use of ethylene vinyl acetate as a binder additive produced the highest fatigue life improvement followed by the use of a 50 percent sulfur additive. Rubber-bitumen mixtures and 20 percent sulfur also gave an improvement in fatigue properties in these tests at some stress levels. Difficulty was experienced in the preparation of fiber-reinforced specimens; this resulted in low densities, high air void contents, and shorter fatigue lives.

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Development of Spray-Reducing Macadam Road Surfacings in the United Kingdom, 1967–1987

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Increases in traffic speeds and intensities, associated with the development of the motorway system in the United Kingdom since the late 1950s, have accentuated the problem of vehicle-generated spray. Although some reduction in spray dispersion can be achieved by improved mudguard design, a more effective solution is to use a permeable wearing course that acts both as a sponge and as a draining layer. Pervious macadam with 20-mm aggregate was developed for roads from the 10-mm nominal size "friction course" that was developed in the 1950s to minimize aquaplaning on airfield runways. The evolution of pervious macadam has been primarily the result of a number of road trials that have led to specification trials by the Department of Transport. The resulting specification for pervious macadam is included in the latest revision of British Standard 4987. The material can be expected to have effective spray-reducing properties for 3 years for a traffic flow of 7,000 commercial vehicles per day (cvd) per lane or for 6 years for 2,500 cvd. To improve the long-term durability of the material and its economic viability, a road trial was started in 1984 to study the performance of pervious macadams with polymer-modified binders.

The introduction of higher speed aircraft in the 1950s gave rise to the need for runway surfacings that were free from standing water capable of inducing aquaplaning and subsequent loss of control of aircraft during braking. The solution to this problem was found in a material, known as "airfield friction course," that provided a free-draining surfacing. The maximum size aggregate used in this application was 10 mm so that damage to jet engines by any loose chippings was minimized. This concept was subsequently adapted for roads because increasing traffic flows and speeds gave rise to the hazard of reduced visibility from spray.

In the United Kingdom roads are wet for between one-quarter and one-half of the year, depending on location. Estimates of accidents in which spray is a contributory factor vary from 1.3 to 10 percent of all wet-road accidents *(1, 2)*. Szatkowski and Brown *(3)* have estimated the cost of accidents caused by spray to be about one-third of that of accidents caused by skidding on wet roads. There are other detrimental aspects of spray that are less easily quantifiable; these include stress on drivers on high-speed roads, particularly when overtaking, and nuisance to cyclists and pedestrians in urban areas.

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Limited spray reduction can be achieved by improved design of mudguards on commercial vehicles, but a more effective solution is to use a water-permeable, pervious wearing course that acts as a sponge and a draining layer. To be effective, a pervious wearing course must retain a free-draining structure. In general, this is more difficult to achieve on roads than on aircraft runways because of the heavily channelized nature of most road traffic and accumulations of dust and detritus. General factors that contribute to these properties in a material are a stable, mostly single-sized aggregate matrix and a stiff but flexible durable binder. In the application of pervious materials as road surfacings it is possible to use larger aggregate particles than the 10-mm aggregate used in the airfield application with the consequent potential of larger sized voids that are less likely to become blocked. The viscosity of the binder has to be increased to make the mixture more resistant to rutting, but, because the surfacing is permeable, the binder is subjected to weathering throughout its depth; this renders it susceptible to hardening and subsequent embrittlement. When the binder has hardened to a viscosity of about 20 pen, surface fretting followed by disintegration occurs. To reduce these effects of weathering, present research is aimed at modifying the binder so that the film thickness can be increased without the risk of binder drainage and the onset of embrittlement can be delayed by improving the low-temperature rheology.

The evolution of this type of surfacing has been primarily the result of a number of road trials because it has not been possible to simulate accurately in the laboratory the long-term effects of traffic on the effective spray reduction of the materials and of natural weathering of the binder on the durability of the surfacing.

The greatest benefits from a spray-reducing surfacing will be realized on the most heavily trafficked sites, but, of course, these provide the most difficult environment in which to obtain acceptable performance from this type of material. Spray measurements determined from a vehicle-mounted optical backscatter measuring device suggest that, when the road is very wet, pervious macadam generates only about 10 percent of the spray level that is generated from the hot-rolled asphalt surfacing used in the United Kingdom. An additional benefit of pervious macadam is that it generates about 6 to 8 dB(A) less noise under wet conditions than rolled asphalt *(4, 5)*. In noise-sensitive areas the quieter surfacing may be a more acceptable solution than the suppression of noise in dwellings or compensation payments. Structurally, 40 mm of pervious macadam is equivalent to about 16 mm of rolled asphalt or 20 mm of dense bitumen macadam *(6)*.

EARLY ROAD TRIALS

The first pervious macadam wearing course was laid on Motorway M40 in 1967 *(7)*. The material used was a 19-mm nominal size wearing course with a grading that complied with the then current British Standard 1621 for an "open-textured" bitumen macadam, and a 100-pen bitumen that contained 4 percent natural rubber as binder. The performance of this material was carefully monitored for more than 10 years during which time the traffic increased from 600 cvd to 1,900 cvd. Jacobs *(8)* reported that the initial surface texture was well over 2 mm and that it subsequently dropped and remained reasonably constant at about 2 mm. The draining properties were markedly reduced after 2 years, but in all other respects the material remained quite effective as a road surfacing and produced less tire noise than other surfacings with a similar surface macrotexture.

Warwickshire County Council experimented with the 10-mm friction course at about the same time on a similarly trafficked site, and its conclusions about the performance of this material were similar to those reported by Jacobs for the 20-mm materials.

Brown *(9)* carried out an experiment with a 10-mm friction course and a 20-mm pervious macadam using two bitumens of 100 pen and 200 pen from different crude oil sources, as well as a 100-pen bitumen modified by the addition of natural rubber. On this much more heavily trafficked site, which carried 4,500 cvd for the first 16 months and 2,300 cvd subsequently, it was found that the coarser 20-mm material with the harder bitumen (100 pen) retained its spray-reducing properties for more than 6 years. A surprising finding was the difference in the reduction of permeability measured in similar materials containing bitumen from different crude sources. The 20-mm macadam made with a 100-pen bitumen from a Venezuelan crude source showed a reduction of 80 percent in 2 years whereas that made using a similar grade bitumen from a Middle Eastern source showed a reduction of only 33 percent.

In 1973 the opportunity was taken to lay pervious macadam (20-mm nominal size aggregates and 100-pen bitumen) on the southbound carriageway of Motorway M1, which carried about 7,000 cvd in the nearside lane *(10)*. Effective spray reduction was achieved for 3 years under the heaviest motorway traffic. It was also observed at this site that after 2½ years the permanent deformation in the nearside wheeltrack was about 1 mm for the pervious macadam; the conventional surfacing for this site, rolled asphalt, had deformed by 3 mm *(3)*.

TABLE 1 SPECIFIED AGGREGATE GRADINGS FOR A38 BURTON BYPASS

British Standard Sieve (mm)	Percent by Mass Passing	
	Grading 1	Grading 2
20	100 – 5	100 – 5
14	65 ± 15	60 ± 10
6.3	25 ± 5	20 ± 5
3.35	10 ± 5	10 ± 5
75 μm	4.5 ± 1.5	4.5 ± 1.5

As a result of these experiments, the most suitable composition of pervious macadam was found to be 20-mm nominal size aggregate with a grading similar to Grading 1 in Table 1 using 100-pen bitumen binder; subsequently the Department of Transport carried out a number of specification trials to prove the material, and this specification is included in the latest revision of British Standard 4987. The surfacing is expected to have effective spray-reducing properties for 3 years under a traffic flow of 7,000 cvd and for 6 years with a traffic flow of 2,500 cvd.

It was thought that there should be benefits from the use of elastomeric additives even though the trials using natural rubber had not produced conclusive results.

In 1975 an experiment was started at Buckden, Cambridgeshire, on Trunk Road A1, which carried 2,300 cvd, to provide information on the performance of a range of wearing-course materials including open-textured and pervious macadams. The results after 7 years of traffic have been summarized in Table 2 *(5)* in terms of the relative performance of the materials with respect to a number of properties.

The pervious macadam showed advantages in retention of skid resistance at high speed, spray suppression, and noise reduction and disadvantages in durability and cost.

1984 ROAD TRIAL

This trial was laid on Trunk Road A38, Burton Bypass, in Staffordshire with the objective of comparing the performance of pervious macadams made with a range of conventional and polymer-modified bitumens. The performance of the materials may be improved by

1. Reducing binder drainage in the interval between mixing and laying to enable a higher binder content to be used; the

TABLE 2 RELATIVE PERFORMANCE OF SURFACINGS AT BUCKDEN AFTER 6 YEARS

Aspect of Performance	Open-Textured Macadam	Pervious Macadam	Dense Bitumen Macadam	Rolled Asphalt	Surface Dressing
Rutting	* * *	* * *	* * * *	* * * *	NA
Durability	* * *	* *	* *	* * * *	* * *
Skid resistance					
Low speed	* * * * *	* * * * *	* * * * *	* * * *	* * * * *
High speed	* * * *	* * * * *	* * * *	* * * *	* * * * *
Riding quality	* * * *	* * *	* * * * *	* * *	*
Spray suppression	* * *	* * * * *	*	* *	* * *
Noise reduction	* * * *	* * * *	* * *	* *	*
Ease of application	* * *	* * *	* * *	* *	* * * *
Area/unit cost	* *	* *	* *	*	* * * * *

NOTE: Ratings range from * * * * * (very good) to * (poor).

TABLE 3 A38 BURTON BYPASS: DESIGNATION OF SECTIONS

Section	Aggregate Grading	Binder	Target Binder Content (percent ± 0.3)
1	1	70-pen bitumen	3.7
2	1	100-pen bitumen[a]	3.7
3	1	Shell bitumen + epoxy resin[a]	3.7
4	1	100-pen bitumen + Inorphil[b]	5.0
5	1	100-pen bitumen + 5.0% 18-150 EVA[c]	4.2
6	1	100-pen bitumen + 5.0% 18-150 EVA[c]	3.7
7	1	Mobil Mobilplast grade C1	4.2
8	1	200-pen bitumen + 5.0% 18-150 EVA[c]	3.7
9	1	100-pen Esso bitumen + 5.0% modified EVA	4.2
10	1	200-pen Philmac bitumen + SBS[d]	4.2
11	1	200-pen Shell bitumen + SBS[d]	4.2
12	1	100-pen BP bitumen + SR[e]	5.0
13	2	100-pen bitumen	3.7
14	1	100-pen bitumen + Pulvatex NR[f]	5.0
15	1	100-pen bitumen (control)	3.7

[a] 4.5 percent limestone filler (no hydrated lime).
[b] Inorphil fibers (alumino magnesium silicate) added at 9 percent of binder percentage (no hydrated lime).
[c] Ethylene vinyl acetate, 18 percent vinyl acetate content, 150 melt flow index (ICI).
[d] Styrene-butadiene-styrene block copolymer.
[e] Synthetic rubber.
[f] Pulvatex NR (natural rubber) added at 8.3 percent of binder percentage; equivalent to 5.0 percent of natural rubber in binder.

resulting thicker binder film should delay the onset of embrittlement; and

2. Rendering the macadam more resistant to deformation and closing-up by adjusting the aggregate grading and by improving the rheological properties of the binder.

Fifteen trial surfacings were laid in August and September 1984; the traffic flow is about 3,500 cvd in the nearside lane and about 400 cvd in the offside lane.

Aggregate

The coarse aggregate was specified to have a minimum polished stone value of 60, a maximum aggregate crushing value of 16, a maximum aggregate abrasion value of 12, and a flakiness index not exceeding 20. The main aggregate grading used was Grading 1 (Table 1), which is similar to that used in the earlier trials *(3)*. One section of the slightly coarser Grading 2 (Table 1) was laid; this grading was the result of cooperative

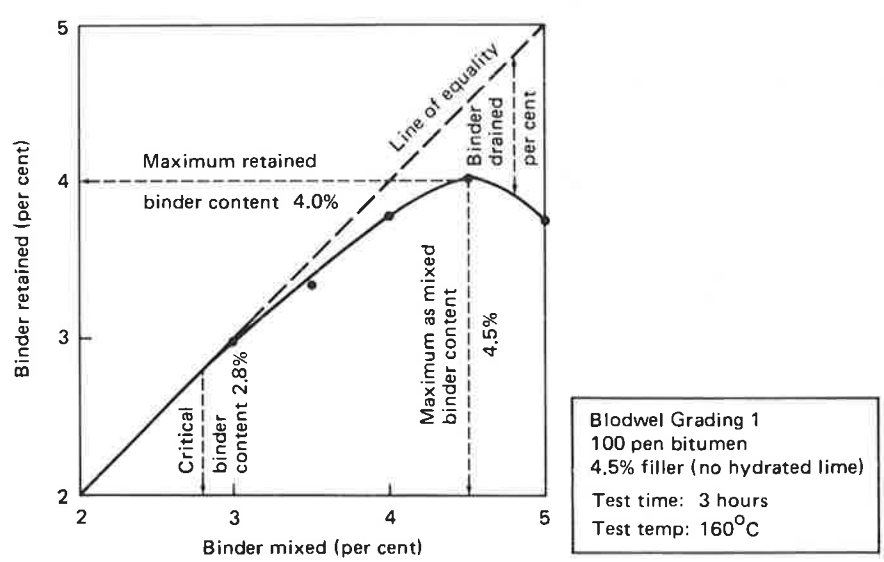

FIGURE 1 Typical binder drainage test result.

work involving the British Aggregate Construction Materials Industries, the Refined Bitumen Association, and the Transport and Road Research Laboratory.

For all except three of the materials used in the trial (Sections 2–4), the filler was specified to contain 2 percent (by mass of the aggregate) hydrated lime to act as an antistripping agent and as a binder stiffener.

Binders and Binder Contents

The control binder was 100-pen bitumen; the full list of the materials used in the experiment is given in Table 3. Hitherto the binder contents for pervious macadams in the United Kingdom have been based on experience; in the United States some qualitative and semiquantitative approaches have been used to assist in the specification of binder contents *(11–13)*.

The design binder contents in this work were determined by a recently developed quantitative binder drainage test on the total mix. In this test *(14)* a sample of pervious macadam is placed in a perforated basket and then put in an oven over a preweighed tray. The weight of material draining onto the tray is determined at the end of the test period. This drained material is essentially a mixture of binder and filler, and it is assumed that the ratio of filler to binder in the drained material is the same as that in the original mixture. The amount of binder draining and hence the binder content remaining in the mixture can then be calculated. By carrying out the test over a range of binder contents, a curve relating binder content retained to initial binder content can be drawn, as shown in Figure 1. Surprisingly, a peak is reached beyond which the binder content retained in a mix actually reduces rapidly with increasing initial binder content. From Figure 1, the highest practical binder content consistent with a minimum of drainage

TABLE 4 RESULTS OF BINDER DRAINAGE TESTS

Section No.	Binder type	Binder content per cent			
		Critical (approx)	Max. retained	Max. as-mixed	Trial target ±0.3
1	70 pen bitumen	3.5	3.5	3.5	3.7
2	100 pen bitumen (no hydrated lime)	2.8	4.0	4.5	3.7
3	Shell bitumen + epoxy resin††	4.0	>5.0	>5.0	3.7
4	100 pen bitumen + Inorphil	>7.0	>7.0	>7.0	5.0
5	100 pen bitumen+5.0% 18-150 EVA	4.0†	4.2†	4.5†	4.2
6	100 pen bitumen+5.0% 18-150 EVA	4.0†	4.2†	4.5†	3.7
7	Mobil Mobilplast C1	3.5	3.8	4.0	4.2
8	200 pen bitumen+5.0% 18-150 EVA	3.5	3.9	4.0	3.7
9	100 pen bitumen**+5.0% Esso modified EVA	3.5	3.8	4.2	4.2
10	200 pen Philmac bitumen+SBS	4.0	4.3	4.5	4.2
11	200 pen Shell bitumen+SBS	4.5	>5.0	>5.0	4.2
12	100 pen BP bitumen+SR	>5.0	>5.0	>5.0	5.0
13	100 pen bitumen (grading 2)	3.4	3.4	3.5	3.7
14	100 pen bitumen +Pulvatex	>5.5	>6.0	>7.0	5.0
15	100 pen bitumen	3.0	3.6	4.0	3.7

Note: 3 hours at $160^{\circ}C$

** BP bitumen used in drainage test. Esso bitumen used in trial

† 2 hours at $160^{\circ}C$

†† 2 hours at $120^{\circ}C$

TABLE 5 A38 BURTON BYPASS: INITIAL SURFACE MEASUREMENTS

Section no.	Texture depth (mm)*	Relative hydraulic conductivity≠ s^{-1}		Skid resistance s.f.c.**
		n.s. lane	o.s. lane	
1	2.5	0.38	0.38	0.66
2A	3.1	0.21	0.18	) 0.62
2B	5.5	0.72	0.62	)
3	3.1	0.50	0.41	0.66
4	3.6	0.24	0.16	0.63
5	3.3	0.33	0.23	0.63
6	3.1	0.32	0.28	0.64
7	3.0	0.23	0.31	0.57
8	2.6	0.24	0.44	0.62
9	2.5	0.25	0.41	0.62
10	3.2	0.40	0.47	0.62
11	3.7	0.41	0.44	0.62
12	3.4	0.19	0.19	0.58
13	3.1	0.41	0.58	0.62
14	2.7	0.17	0.26	0.63
15	2.9	0.42	0.37	0.62

* Nearside lane September 1984

** November 1984 nearside lane

≠ Reciprocal for outflow time of 0.00125 m^3, corrected for apparatus constants.

can be deduced. The binder drainage test, mostly performed at 160°C for 3 hr, was carried out on all the binders used in this trial; the values for the critical binder content (the lowest level at which binder drainage appears), the maximum retained binder content, and the maximum binder content of the as-mixed material are given in Table 4.

The relationship between the laboratory test and practice was investigated in a practical trial that was carried out before the test sections were laid. In this trial, lorry loads of pervious macadam traveled 80 km and stood for an additional 2 ± 1/2 hr. Samples were taken from the tops of the lorry loads for analysis both at the mixing plant and after the journey and standing time. Although there was no visible drainage, analysis results showed reductions of binder content of up to 0.9 percent that confirmed the trend of the binder drainage test result, although the trial results showed greater drainage than the test predicted *(14)*.

Nevertheless, this test formed a rational basis for the specification of the binder contents used in the test sections. In specifying a binder content, it would be inappropriate to set a level greater than the maximum as-mixed value, which would indicate appreciable drainage, or below the critical binder content, which would reduce durability. Because of such factors as specification tolerance on target binder content, the precision of the drainage test, and the desirability of reducing the number of variables in a full-scale trial, small differences in the binder drainage test results were ignored. These factors and, in some cases, the proprietor's option supported the selection of three groups with target contents of 3.7, 4.2, and 5.0 percent.

Observations on Laying the Trial Sections

The materials were laid by two pavers operating in echelon (except for the epoxy resin material); this made possible a well-compacted permeable and invisible longitudinal joint. In general, there was a 3- to 5-hr delay between mixing and laying; however, this did not result in load temperatures being appreciably lower than the target mixing temperatures. With the epoxy resin material, laying took place as soon as the lorries arrived on site, and in this case a satisfactory but visible longitudinal joint resulted.

Because the target binder contents were set at values only slightly higher than the critical binder contents given in Table 4, a small reduction in measured binder contents for site samples would be expected. Only 3 of the 15 sections showed some evidence of binder drainage; in those 3 cases average binder contents for site samples were more than 0.3 percent below the target values.

Before the road was opened to traffic, measurements of texture depth and relative hydraulic conductivity were made in the nearside wheelpath of the nearside lane. Measurements of

TABLE 6 RESULTS OF TESTS ON CORES OF WEARING COURSE

Section no.	Binder	Specific gravity	Voids open to water (%)	Rate of tracking (mm/h)
1	70 pen bitumen	2.404 2.378	24.4 18.6	* 0.6
2	100 pen bitumen (no hydrated lime)	2.414 2.370	24.4 20.3	0.9 1.1
3	Shell bitumen + epoxy resin	2.400 2.400	26.3 24.5	0.3 0.2
4	100 pen bitumen + Inorphil	2.335 2.344	18.3 22.0	* *
5	100 pen bitumen +5.0% 18-150 EVA	2.362	21.0	2.4
6	100 pen bitumen +5.0% 18-150 EVA	2.371 2.333	19.2 19.2	0.8 0.3
7	Mobil Mobilplast C1	2.325 2.265 2.365 2.327	19.1 17.9 19.8 16.6	0.5 0.2 0.7 0.6
8	200 pen bitumen +5.0% 18-150 EVA	2.320 2.347 2.366 2.356	23.1 18.5 24.1 19.1	* 1.9 1.0 0.2
9	100 pen Esso bitumen +5.0% Esso EVA	2.342 2.382	17.1 19.0	0.9 0.3
10	200 pen Philmac bitumen+SBS	2.412 2.307	18.3 19.6	1.4 0.8
11	200 pen Shell bitumen+SBS	2.351 2.352	20.7 17.1	0.3 0.3
12	100 pen BP bitumen+SR	2.282 2.317	16.8 17.3	0.3 1.4
13	100 pen bitumen (grading 2)	2.336 2.404	19.9 *	5.2 *
14	100 pen bitumen+ Pulvatex	2.268 2.280	18.9 14.7	2.2 0.4
15	100 pen bitumen	2.370 2.388	18.2 24.4	0.4 1.0

Note: Cores taken from nearside wheel track of nearside lane
* Core disintegrated

skid resistance [sideway force coefficient (sfc)] were made after 6 weeks of trafficking. Results are given in Table 5. Cores were removed from both lanes of each section and various parameters, including voids open to water and resistance to permanent deformation, were measured. These results are given in Table 6; details of the methods of measurement used have been given by Daines *(14)*.

The compositions of the polymer-modified binder could not be checked against the specifications because standard procedures are not available for the analyses. Infrared spectroscopy can be used with some polymers, but this method cannot be used with all of the polymers used in this trial. The contributors assured the compositions of their proprietary binders and the laboratory's staff supervised the blending of the other polymers. Work is continuing to develop methods of measuring the polymer content of binders.

Future Measurements

Regular measurements are being made of surface texture, hydraulic conductivity, profile, skid resistance, and visual condition. Spray suppression and noise levels are also included in the program. Surface texture and profile are measured using infrared lasers as sensors *(15)*; skid resistance is measured by the long-established sideway-force method; and spray suppression is measured by the more recently developed optical backscatter device *(16)*.

After 2 years of trafficking there are some signs of differences in the performance of the various materials, but it is too early to draw any conclusions about the relative merits of the additives used.

GENERAL CONCLUSIONS

Pervious macadam surfacings have been shown to have advantages over other types of surfacing, in particular suppression of spray, lower noise levels, and maintenance of high-speed skid resistance. The major disadvantage is the relatively short life of the material on heavily trafficked sites where the advantages are most apparent to the road user.

Efforts have been made to improve the durability of the material by using polymers and mineral fibers that make it possible to use higher binder contents. If the life of the surfacing can be increased without significant loss in other properties, this type of material will become more economically attractive and may be more widely used.

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Use of Friction Course Mixes in Ontario

KAI K. TAM AND DANIEL F. LYNCH

The dramatic increase in the volume and speed of automotive vehicles in the 1950s and 1960s in Ontario resulted in the need for safer and better quality highways. The increased traffic also created rapid deterioration of frictional and durability characteristics of the road system that was originally designed for lighter traffic. The need to upgrade urban freeways and to rehabilitate other highways led to the development of friction course mixes. In the process of formulating policy for safer and better quality highways, in 1974 a major installation of 18 test sections of bituminous overlays on one of the most heavily trafficked roads in Ontario was undertaken, and a task force was set up to review the performance of friction courses and make recommendations on their use. The developments that led to the adoption of a policy of using friction course mixes in Ontario are outlined. The design, construction, and results of the experimental sections and the findings of the task force on friction course mixes are discussed. The experience gained and the annual program of friction courses are also discussed. One important finding of the experiment is that the most effective way to improve the level of friction on a wearing course is by using harsh, angular, fine aggregates such as traprock or slag screenings and a sufficient proportion of crushed good-quality coarse aggregate in the bituminous mixes to maintain the micro- and macrotexture of a surface. These characteristics can best be provided by open friction course and dense friction course mixtures. Friction courses are accepted by the Ontario Ministry of Transportation and Communications as the most suitable surface course mixtures for freeways and accident "black spots." The volume of friction course mixes laid each year verifies this commitment.

Advances in automobile technology in the 1950s resulted in the need for safer and better riding quality roads to accommodate high-speed transportation. The dramatic increase in the volume of traffic on the roads that was brought about by the postwar increase in the ownership of automobiles and the change to the transportation of goods and commodities by truck challenged the highway engineer to construct and maintain better highways.

Such need clearly existed on a major multilane urban commuter freeway in metropolitan Toronto, the Highway 401 Toronto Bypass. The rehabilitation and upgrading of this section of freeway initiated the search for better techniques for improving driving qualities and for a hot-mix resurfacing system that would produce a long-wearing surface with good friction properties under high volumes of traffic.

The developments that led to the use of open friction course (OFC) and dense friction course (DFC) on a major installation of 18 test sections are outlined, and the findings of a task force on the performance of these friction courses are presented. In addition, the annual OFC and DFC program and the rehabilitation techniques are discussed.

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BACKGROUND

Use of OFC and DFC was sparked by the need to rehabilitate Highway 401. Planned, constructed, and reconstructed in stages during the past 40 years, Highway 401 (Figure 1), or the MacDonald-Cartier Freeway as it is also called, has become the main artery across southern Ontario. It runs from the Ontario-Quebec boundary on the east to the Ontario-Michigan boundary on the west, for a total distance of 820 km (510 mi). Commencing in 1947, sections of this major route were constructed to bypass major centers. It runs with Highway 427 for 50+ km (30 mi) through metropolitan Toronto and is the main east-west commuter route serving a population of 3 million people. The average annual daily traffic (AADT) for the busiest location is 238,400, and there are 1,800 vehicles per lane per hour at peak periods on several sections.

Construction of Highway 401 as an Urban Freeway

In the vicinity of Toronto, Highway 401 was first constructed as a four-lane divided rural freeway through largely agricultural land. Construction began in 1952, and by 1958 the basic four-lane divided controlled-access highway was completed. It was built as a conventional flexible pavement with gravel shoulders, core-type construction, and a wide depressed median.

By 1960 the bypass was being taxed beyond capacity because, within a relatively short period of time after completion, numerous residential subdivisions and industrial areas had been constructed adjacent to the freeway. A major increase in car and traffic volume resulted (Table 1).

The reconstruction of the original facility as a multilane, urban commuter, concrete pavement in the vicinity of Toronto took place between 1963 and 1973 when the original four-lane asphalt pavement was widened to a twelve-lane system of collector and express lanes. Concrete pavement was chosen for the highway because it was assumed to have the best potential for providing a long service life under the anticipated heavy traffic conditions.

Performance of the Concrete Pavement

It was originally expected that the portland cement concrete pavement would be maintenance free for 25 years. However, within less than 10 years after construction it was evident that rehabilitation would be required sooner than expected because of emerging problems. The main and earliest problem to develop was the marked increase in the multicar wet weather

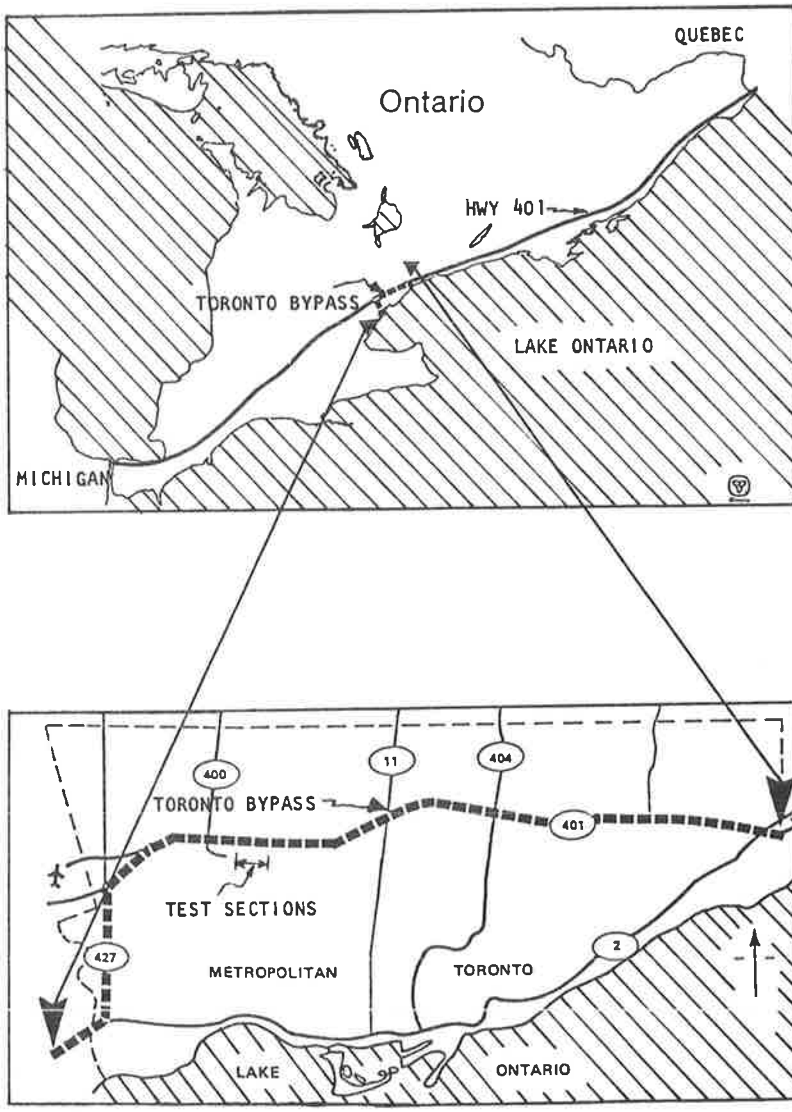

FIGURE 1 Location of test sections (1).

accident rate. The ministry tried grooving the existing concrete pavements to improve their frictional properties. That technique, although somewhat effective for a while, created excessive tire noise that was unacceptable to both the driving public and nearby residents. Problems were also encountered with the performance of the joints: transverse cracking began to appear between the joints, and spalled areas of varying depth developed at quite a number of joints and cracks.

During the 10-year period after the construction of the first section of concrete pavement there was a marked increase in traffic volume above that anticipated as well as a dramatic increase in allowable truck weights and axle loadings (Table 1).

TABLE 1 TRAFFIC VOLUMES AND TRUCK WEIGHTS

Year	AADT	Gross Vehicle Weight (tonnes)	Legal Axle Weight (tonnes)
1963	77,000	32.0	8.6
1976	199,000	63.5	10.6

DEVELOPMENT OF POLICY ON SURFACE COURSE MIXES

The low-friction condition was created because the original burlap drag and broom textured concrete pavement surface was severely polished by the heavy traffic and studded tires (banned after 1971). The texture depth was reduced to about one-tenth of what is desirable. The friction value as determined by the ASTM brake-force trailer at 100 km/hr (62 mph) varied between 19 and 22.

Bituminous Mix Test Sections

In an effort to determine the best rehabilitation methods for restoring the driving qualities of the concrete pavement, the ministry designed and placed 18 bituminous overlay test sections in 1974. Dense- and open-graded hot mixes, sand-asphalt mixes, and mastic mixes both with and without asbestos were used. The emphasis in the evaluation of the test sections was on long-term frictional characteristics and improvement of ride quality. Another consideration was that the rehabilitation method chosen should be low cost. The main points of the experiment are described next.

Test Location and Traffic

The test sections were on the westbound core lane between Allen Expressway and Jane Street (Figure 1). The traffic count had increased dramatically from 111,000 AADT in 1967 to 179,000 in 1974 and 260,600 in 1985. Truck traffic averaged about 17 percent.

Layout of Test Sections

The 18 test sections were each 137 m (448 ft) long and 11.3 m (37 ft) wide. Sections 1–10 and 17 were 38 mm (1½ in.) thick overlays whereas Sections 11–16 were only 25 mm (1 in.) and Section 18 consisted of two lifts of 38 mm (1½ in.) each.

Mix Design and Composition

Because a surface with good frictional properties must possess sufficient microtexture or harshness (evaluated per ASTM D 3319) and suitable macrotexture or stone projections (measured by the ASTM E 770 method) *(2)*, the mixtures were designed to have various degrees of micro- and macrotextures. The test sections included both dense- and open-graded types of mixes with a variety of coarse and fine aggregates including traprock, steel slag, and blast furnace slag (Table 2).

Test Sections 1–10 consisted of HL-1 mixes in which the coarse aggregate content was progressively increased to obtain a greater macrotexture [i.e., more stone particles at the surface (Figure 2)]. The standard HL-1 mix is a dense-graded mixture with good-quality coarse aggregate [50 percent retained on No. 4 (4.75-mm) sieve] and local fine aggregate. The modified HL-1 mixes tested are mixtures with good-quality fine aggregate (i.e., DFC by definition), or a blend thereof, instead of just local natural sand.

Mixes in Sections 11 and 12 were sand-asphalt mixes with traprock screenings that contained small percentages of coarse aggregates and asbestos fiber filler for greater flexibility and impermeability.

Sections 13 and 14 were open-graded mixes designed to have high permeability [e.g., >10^{-5} cm^2, test method of Saylak et al. *(3)*] characteristics that facilitate rapid drainage of surface

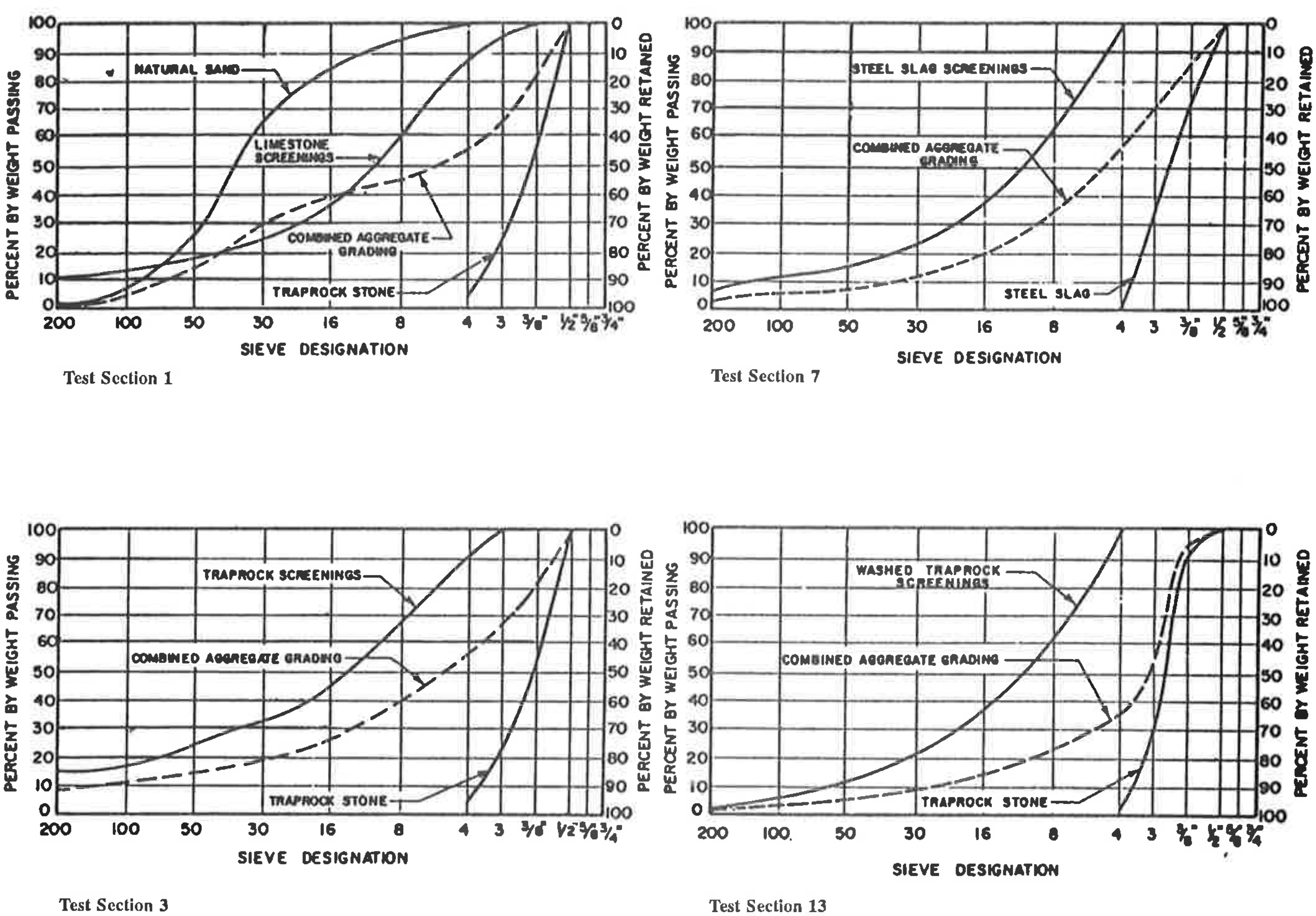

FIGURE 2 Gradations of four of the trial mixes *(2)*.

TABLE 2 COMPOSITION AND DESIGN DATA ON BITUMINOUS MIXES

Test Section	Type of Mix	Composition: Coarse Aggregate Retained on No. 4 Sieve: Type	Percentage	Fine Aggregate Passing No. 4 Sieve: Type	Percentage	Filler Material: Type	Percentage	Percentage Retained on No. 4 Sieve[a]	Asphalt (percentage by weight of mix)	Characteristic: Marshall Stability (lbf)	Marshall Flow (in.)	Voids in Mineral Aggregate (percentage by volume)	Voids (percentage by volume)
1	HL-1												
	L	TR	45	NS	41			43.8	5.4	1,625	10.3	16.9	2.8
	F			LS	14				5.4	2,614	11.6	17.5	3.7
2	HL-1												
	L	TR	45	NS	41			48.2	5.4	1,625	10.3	17.2	3.0
	F			TRS	14				5.3	2,870	13.4	15.0	0.8
3	HL-1												
	L	TR	45	TRS	55			47.5	4.1	3,185	16.5	12.9	1.2
	F								4.0	3,407	15.3	13.5	2.0
4	HL-1												
	L	TR	55	NS	34			54.1	4.8	1,730	10.2	16.0	2.7
	F			LS	11				4.8	3,027	12.3	14.2	1.2
5	HL-1												
	L	TR	60	NS	28	ASB	2	58.1	5.6	1,800	15.0	14.2	1.0
	F			LS	10				5.7	1,903	22.5	16.0	0.9
6	HL-1												
	L	TR	60	TRS	38	ASB	2	62.3	5.3	2,190	22.7	15.2	1.0
	F								5.4	2,570	18.7	18.6	3.4
7	Modified HL-1												
	L	SL	45	SLS	55			46.8	5.3	3,520	16.0	18.5	2.7
	F								5.2	3,640	13.2	18.7	3.3
8	Modified HL-1												
	L	SL	50	NS	38			47.1	5.7	2,160	12.9	17.3	2.3
	F			LS	12				5.7	2,990	13.7	17.7	1.9
9	Modified HL-1												
	L	BF	45	BFS	55			43.2	8.0	2,830	12.8	24.0	6.9
	F								7.8	3,326	14.9	23.1	6.1
10	Modified HL-1												
	L	BF	40	NS	45			40.5	6.8	1,975	10.4	17.9	2.9
	F			LS	15				6.5	3,155	9.7	17.0	2.2
11	Sand												
	L	TR	14	TRS	84	ASB	2	5.4	7.1	2,084	27.2	17.7	0
	F								7.0	2,175	40.2	19.5	0.2
12	Sand												
	L	TR	9	TRS	89	ASB	2	6.9	7.0	2,650	19.2	16.8	0
	F								7.2	1,885	45.1	20.7	1.1
13	Open graded, F	TR	67	TRS	33			60.5	5.9	1,458	12.7	20.6	4.7
14	Open graded, F	TR	67	TRS	31	ASB	2	71.7	5.8	1,691	11.9	19.6	4.0
15	Open graded, F	TR	30	TRS	70			29.3	5.6	2,678	19.2	16.3	0.7
16	Open graded, F	TR	30	TRS	68	ASB	2	31.4	6.6	2,116	30.5	18.6	0.2
17	Mastic, F	TR	70	TRS	19	MF	9	75.2	7.5	1,887	56.7	21.0	1.0
18[b]	HL-1												
	L	TR	45	NS	41	ASB	2	47.4	5.4	1,625	10.3	16.9	2.8
	F			LS	14				5.4	2,825	14.5	14.6	0.4

NOTE: Adapted from Ryell et al. *(2)*. L = laboratory mix designs, F = field laboratory tests, TR = traprock, LS = limestone screenings, NS = natural sand (glacial deposit), TRS = traprock screenings, ASB = short-fiber asbestos, SL = steel slag screenings, BF = blast furnace slag, BFS = blast furnace slag screenings, and MF = mineral filler (finely crushed limestone).

[a]Based on field and laboratory extraction tests.

[b]Same as Section 1 but constructed over a 38-mm-thick bituminous base course.

water into, and laterally through, the surface course layer (i.e., OFC mixes). The mixes used a large proportion (67 percent) of single-sized coarse aggregate and a small amount of washed fine aggregate (Figure 2).

Sections 15 and 16 were also considered OFC mixes but with 30 percent coarse aggregate and washed fine aggregate as in Mixes 13 and 14. Section 17 was a mix called "Mastiphalt," which is a kind of mastic asphalt derived from the German "Gussasphalt" technology and modified so that the material can be mixed and placed with conventional equipment.

Construction

Night paving was necessary because of heavy traffic. The core lanes at the contract location were closed and traffic was diverted to the collector lanes via a transfer lane. Construction was problem free, except that normal mixing time was extended for mixes with asbestos because lumps of asbestos appeared in the mix.

Frictional Performance of Test Sections

The frictional characteristics were measured using the ASTM brake-force trailer. A previous report by Ryell et al. *(2)* detailed the results obtained up to 1978. Thereafter, some sections had physically deteriorated to the degree that complete resurfacing of all of the test sections was warranted in 1985. Before this work was done, a final evaluation was carried out on the sections and a report is being prepared on the evaluation of the performance of these test sections after 11 years of service. The updated results to 1985 (Figure 3) are summarized next.

- HL-1 and modified HL-1 mixes: Sections 7 (Figure 4) and 9, which are considered DFC mixes (i.e., consisting of steel slag or blast furnace slag coarse and fine aggregates, respectively), had maintained a friction number (FN), formerly skid number, in excess of 31 *(4)*. In comparison, Section 8, which contained steel slag coarse aggregate and a blend of natural sand and limestone screenings as the fine aggregate, and Section 10, with similar fine aggregate and blast furnace slag coarse aggregate, exhibited significantly lower FNs. These fine aggregates had a similar effect on the frictional properties of the HL-1 mixes that contained traprock coarse aggregate, particularly in the driving lane (e.g., Section 3 versus Section 1). Test Sections 3 and 6 that had traprock screenings as the fine aggregate have FNs just on the 31 level.
- Open-graded mixes: Test Sections 13 and 14 had maintained high FNs with good macrotexture appearance (Figure 5). However, Sections 15 and 16, which contained a reduced proportion of coarse aggregate, had a reasonable initial FN but the level declined to below 30. The interconnected voids in Mixes 13 and 14 appeared to be quite efficient at removing water from the tire-road contact area and thus enhancing the frictional characteristics as well as reducing splash and spray in wet weather. These voids tend to clog up after 4 to 5 years of service, but the permeability is still relatively higher than that of the other mixes (Table 3). It appears that the pumping action of the tires helps to clean out some of the debris.
- Other mixes: The two sand mixes (11 and 12) remained at about FN = 30, but these mixes are considered not quite suitable for high-speed traffic because of the lack of macrotexture on the surface; the lower initial FN-value for Section 12 is because of the higher AC content (Table 2). The mastic mix (Section 17) had the lowest initial friction value of all of the test sections, and after 3 years the values are approaching 20.
- FN versus speed: Figure 6 shows that the OFC (Section 13) has the smallest drop in FN of all of the mixes. The sand mix (Section 11) has the greatest. It is clear that the two DFC mixes consisting of slag materials (Sections 7 and 9) yield the

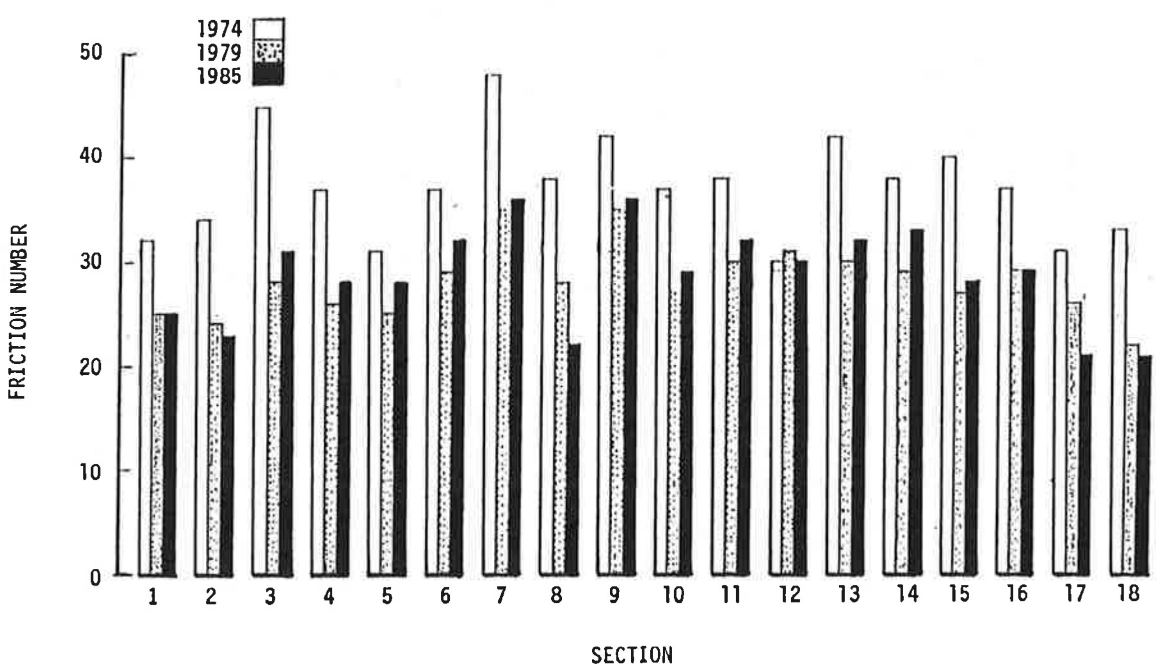

FIGURE 3 Friction number (at 100 km/hr) of the driving lane of test sections.

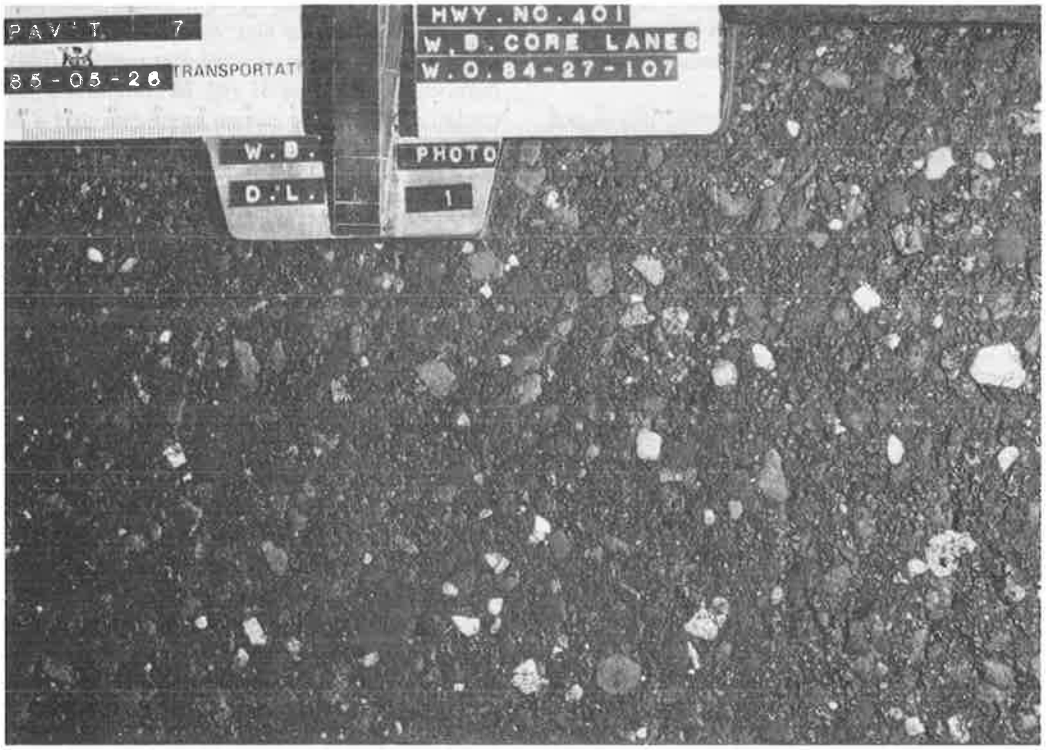

FIGURE 4 Close-up of wheelpath of driving lane of Section 7 (steel slag coarse aggregate, steel slag screenings fine aggregate) after 11 years of service; friction is excellent, and macro- and microtexture are good.

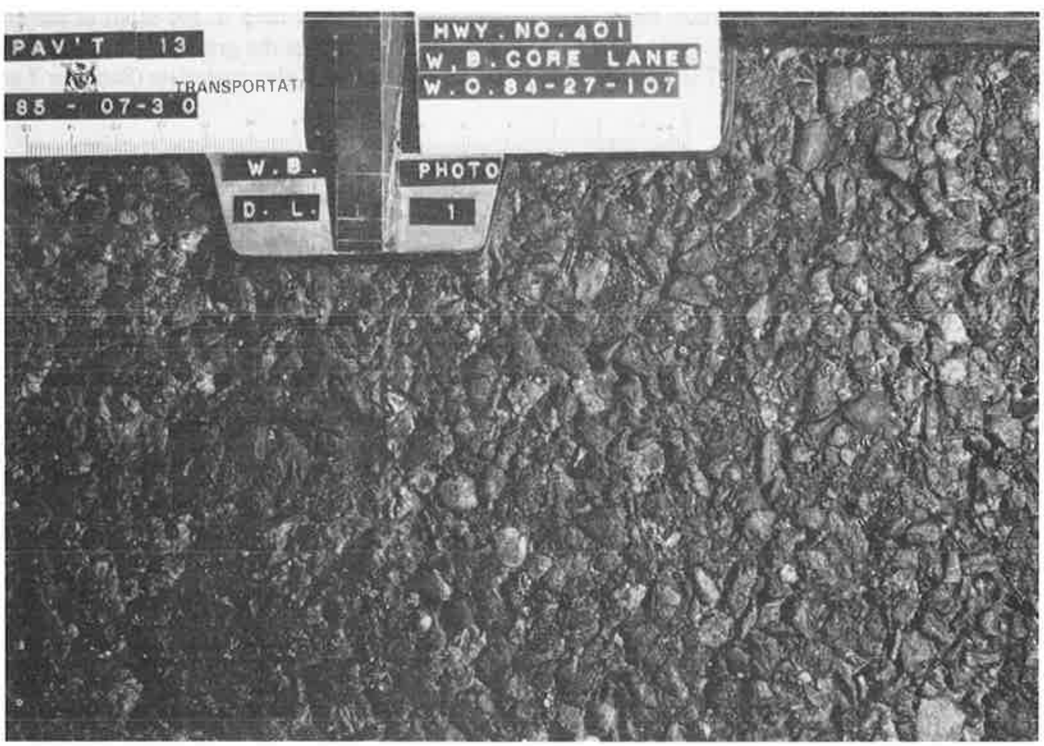

FIGURE 5 Close-up of wheelpath of driving lane of Section 13 (traprock stone coarse aggregate, traprock screenings fine aggregate) after 11 years of service; friction is excellent, and macrotexture is well developed.

TABLE 3 PERMEABILITY RESULTS

Section	Water Permeability (mL/min) at Age (years)				Air Permeability (cm) at 11 years		
	0	2	5	7	Passing Lane	Center Lane	Driving Lane
1	–	1.1	0	0	imp	imp	8.6E-12
2	–	1.7	–	0	imp	imp	2.2E-11
3	70	3.2	2.5	0	6.6E-10	1.3E-9	1.9E-9
4	–	1.2	–	0	imp	imp	imp
5	–	1.9	–	0	6.4E-10	imp	imp
6	–	50	–	3	3.0E-9	6.0E-9	1.3E-9
7	–	4	0	0	10.0E-10	2.6E-10	3.2E-11
8	–	4	–	0	imp	imp	imp
9	vh	30	5.5	0	1.9E-9	2.4E-9	1.5E-10
10	22	2	0	0	imp	imp	imp
11	–	2.8	–	0	1.6E-9	imp	7.1E-11
12	–	2.4	–	0	6.6E-11	3.1E-11	–
13	vh	78	20	7	1.3E-9	1.3E-10	–
14	vh	150	–	13.5	6.6E-9	1.2E-9	1.0E-8
15	6	3	–	0	6.7E-12	imp	imp
16	22	2.5	–	0	1.4E-10	imp	imp
17	–	4.3	–	0	imp	imp	1.0E-9
18	–	2.4	–	0	3.4E-11	imp	imp

NOTE: Dash = results not available, vh = very high permeability, imp = impermeable, and E = × 10⁻.

highest FN of all of the mixes although they maintain a friction-speed gradient that is similar to that of the traprock DFC mix (Section 3). The dense-graded mixes (Sections 1 and 4) and the mastic asphalt (Section 17) appear to have an FN-speed gradient that is similar to that of the DFC mixes. A 10 percent increase in stone content (e.g., Section 4 versus Section 1) of the same type of aggregate tends to reduce the FN-speed gradient. The graph clearly suggests that although the FN-speed gradient is governed by the macrotexture, the level of FN achievable at a given speed is a function of the types of aggregate used, which determine the level of microtexture available in a mix.

• General: Almost all of the mixes provided substantially better friction characteristics than did the existing concrete. The mixes appear to have reached their equilibrium friction level after 2 or 3 years of service (Figures 3 and 7). Also, mixes

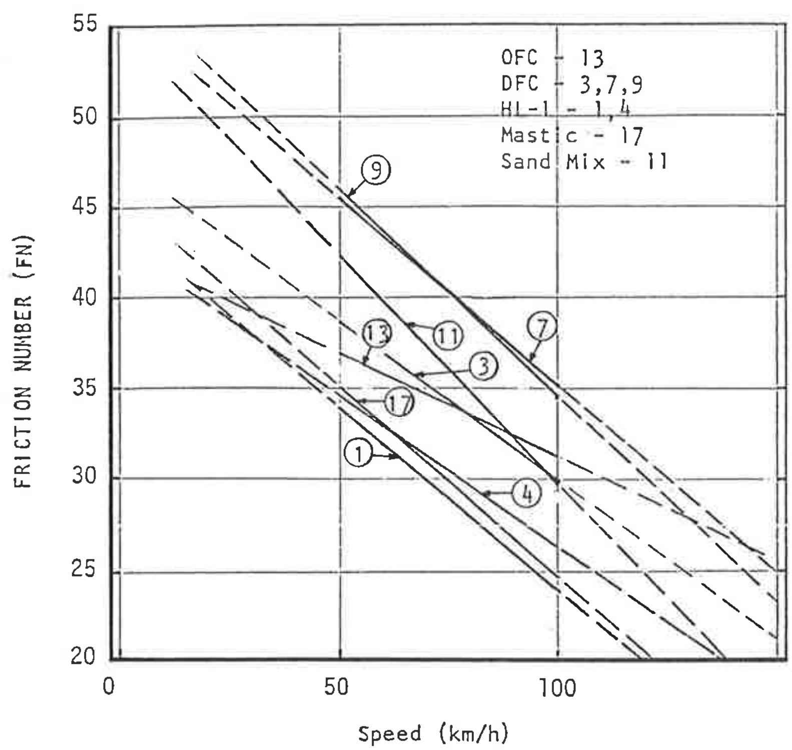

FIGURE 6 Influence of speed on FN of mixes with various degrees of macrotexture after 11 years of service.

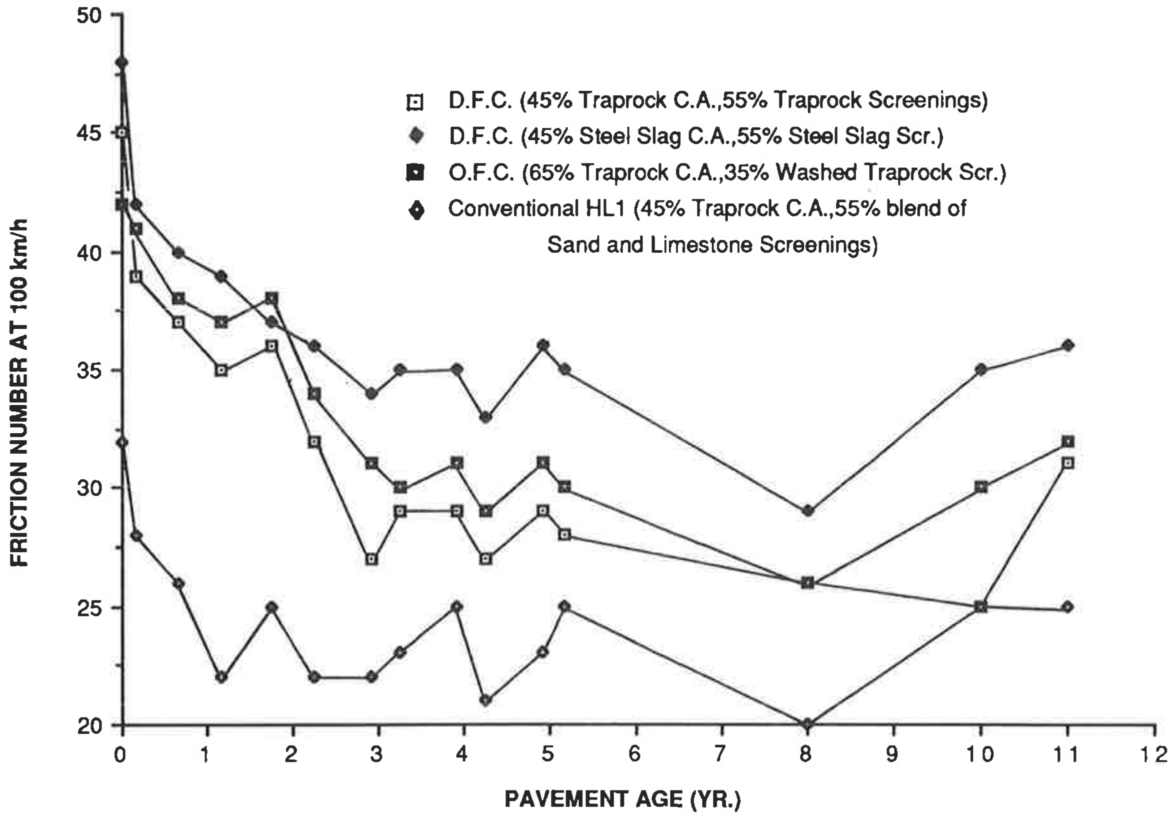

FIGURE 7 Change in FN with time for selected test sections.

that have a high initial friction value tend to take longer to arrive at their equilibrium (e.g., Sections 7, 13, and 3 versus Section 1). In the driving lane that carries 3,700 commercial vehicles per day, the mixes that provide an FN above the value of 30 are (a) dense-graded mixes with both coarse and fine aggregates consisting of traprock, steel slag, or blast furnace slag (DFC) and (b) open-graded mixes with traprock coarse and fine aggregates with high stone contents (OFC).

Durability

The durability of the sections is compared by using the "overall rating" derived by multiplying the length or the area of the appropriate distress by the weighting factors assigned for the severity and type of distress as outlined in the *Manual for Condition Rating of Flexible Pavements (5)*. The data in Table 4 indicate that Sections 1, 9, and 17 have overall ratings of 57, 54, and 58, respectively, whereas the other sections have overall ratings between 25 and 37. The friction course mixes (e.g., Sections 3, 7, 9, and 13) performed well relative to the dense-graded mixes, except Section 9 that was built with all blast furnace slag aggregate. The deterioration of Section 9 took the form of raveling and delamination. This is because of the highly absorptive nature of the blast furnace slag aggregate that absorbs the asphalt concrete (AC) and reduces the effective AC available for holding the aggregate in place as well as for adhering the mat to the existing pavement surface. The transverse cracking was reflection cracking from the underlying concrete pavement.

Permeability

Permeability tests were performed on the driving lane wheelpath of each test section using the John Manville outflow

TABLE 4 DURABILITY PERFORMANCE OF TEST SECTIONS

	Cracks						
Section	Transverse	Longitudinal	Raveling	Patching	Delamination	Rutting	Overall Rating
1	4	44	6	0	0	3	57
3	0	1	18	6	4	3	32
4	12	4	6	0	0	3	25
7	16	2	6	9	1	3	37
9	5	7	18	6	14	5	54
11	34	0	0	0	2	0	36
13	6	2	6	14	5	0	33
17	26	5	18	8	2	0	58

permeameter *(2)*. However, by the 11th year, the wheeltrack matrix of all of the sections had closed up to the extent that it required a much longer time (>1 hr) to obtain a water permeability reading. Hence, cores taken from the test sections were tested in the laboratory for permeability using the air permeability method described in the ASTM D 3637.

The results (Table 3) indicate that the OFC test sections (13 and 14) had high initial permeability but that the voids closed up after 7 years of traffic compaction, reducing the permeability to about 10 mL/min [and that of the DFC to zero (Figure 8)]. However, the air permeability of the OFC in the 11th year on the driving lane is still relatively higher than that of the other mixes (e.g., 10^{-8} cm versus $<10^{-9}$ cm, respectively). In general, the lower air permeability in the driving lane confirms the belief that traffic compaction in the driving lane wheeltracks is greater than in the passing lanes because of heavier traffic volume.

Policy on Use of Friction Course Mixes

Because of the performance of these test mixes, a new ministry policy was introduced in 1978 governing the selection of surface course mixes for main highway facilities. The new policy specifies the use of OFC mixes *(6)* as the surface layer for urban freeways and DFC mixes *(7)* for other heavily trafficked main highways carrying traffic in excess of 5,000 AADT per lane. The DFC mixes are also used in accident "black spots" with lower traffic volumes. Other factors to be considered are

- Projected traffic volumes,
- Types and percentage of trucks (accident rate), and
- Highway classification.

Special provisions were drawn up for both the OFC (SP 311) and DFC (SP 321) mixes. The OFC consists of from 65 to 70 percent coarse aggregate [retained on No. 4 (4.75-mm) sieve] with 30 to 35 percent washed screenings. Both the coarse and the fine aggregates have to be obtained from the same source of traprock or steel slag.

The DFC mix consists of 55 percent coarse aggregate [retained on No. 4 (4.75-mm) sieve], and both coarse and fine aggregates have to be selected from the "List of Designated Sources" but do not have to be from the same source. The fine aggregate is unwashed screenings.

Task Force on Performance of Bituminous Friction Course

The commitment of the ministry to constructing more friction course pavements made it essential that the field performance of the new mixes be evaluated and necessary changes be made to optimize performance. A task force was therefore set up with a mandate to review, determine, and recommend the most suitable driving qualities of pavement, with specific emphasis on improving frictional characteristics. The task force considered the following topics:

- General performance,
- Winter performance,
- Frictional characteristics,
- Safety,
- Riding quality,
- Noise levels,
- Performance of zone paint,
- Open friction mixes in other jurisdictions,
- Observations, and
- Recommendations.

The task force evaluated 11 contracts (5 OFC and 6 DFC mixes) between February 1979 and July 1981. The findings of the task force *(8)* in regard to each of the topics are summarized in the following subsections.

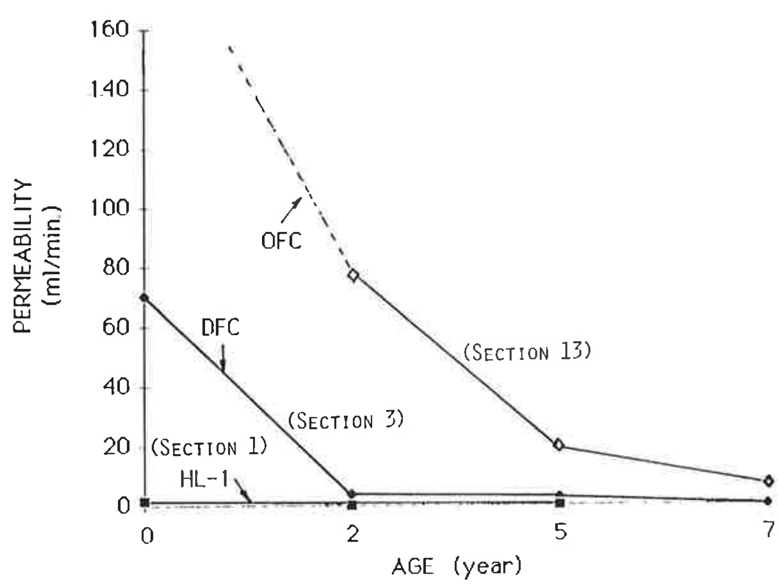

FIGURE 8 Water permeability and age of pavement.

General Performance

On review of the contracts and observations, the pavements were considered to be performing satisfactorily. The OFC mix that had been placed over concrete pavement without sawing and sealing the concrete joints showed reflection cracks that were raveling extensively. All of these cracks were routed and sealed. Reflection cracks on some of the DFC pavements on the other hand were not raveled.

Winter Performance

Visual observations made during the winter of 1978–1979 indicated that

- The OFC mixes appeared to provide the best frictional characteristics.
- There were no problems observed in clearing the OFC surfaces. The "broadcasting" technique of salt application has worked quite effectively to clear the OFC pavements.
- Ice buildup in the DFC mixes has been reported. This problem can be related to the method of spreading deicing salt, (i.e., windrowing versus a recommended broadcasting application). The permeable nature of the DFC mix allows the brine solution to penetrate the pavement before the windrowed deicing salt has a chance to be spread by traffic. It is recommended that broadcasting of deicing salts at a slightly higher application rate for winter snow and ice control be required for both OFC and DFC mixes.

Frictional Characteristics

The task force monitored some 48 sites in southern Ontario that had been laid with the DFC and HL-1 mixes. A friction survey in 1976 showed that mixes with fine aggregate that consists predominantly of natural sand do not provide as good a friction number as do mixes with screenings as fine aggregate (i.e., DFC, Figure 9).

Safety

Collisions on wet pavement decreased by between 42 and 81 percent after rehabilitation. However, alignments had been improved, additional lanes had been added, and lane-change tapers had been lengthened at some locations. Undoubtedly, such improvements contributed to the reduction in the number of accidents on wet pavement.

Riding Quality

The results of the Mays meter readings (in inches of roughness per mile) indicate that the majority of the friction courses fall in

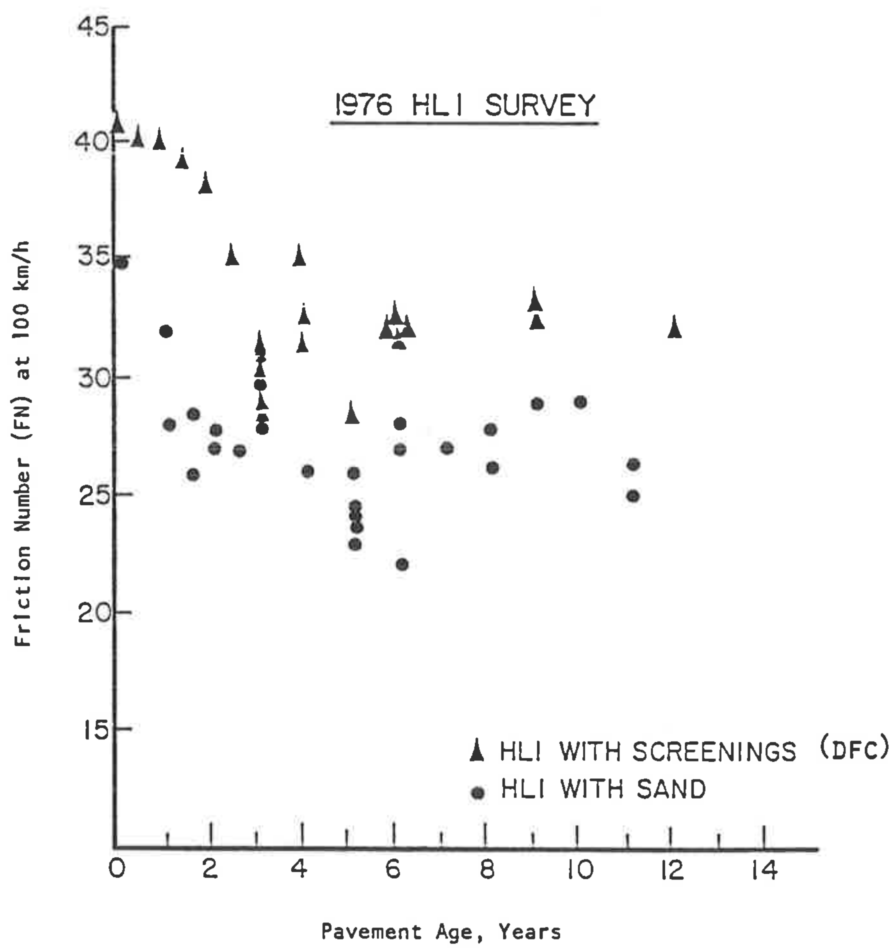

FIGURE 9 Friction survey of HL-1 mixes laid in southern Ontario.

the fair ride range of 80 to 100 in./mi. Two of the contracts are in the good to excellent ride range of less than 80 in./mi.

Noise Levels

Roadside noise measurements indicated that the OFC was 2.1 dB(A) quieter than the standard HL-1 mix.

Performance of Zone Paint

Monitoring of test sections in the core lane of Highway 401 indicates that normal mixes and open and dense friction course mixes have to receive a buildup of traffic paint in order to have a durable appearance. The traffic paint on the bituminous pavements had a minimum of 25 percent remaining with an average of 70 percent, whereas paint on the concrete pavement adjacent to the test sections was on the order of 15 percent remaining, and this was mainly in the grooves. Zone markings on fresh OFC and DFC pavements appear to require the same amount of repainting as do those on standard surface course mixes.

Problems were encountered with adhesive-backed plastic strips not sticking to the OFC mix. It would appear that this material must be placed on the hot mix before compaction so that the mix and the plastic strip can be rolled as one.

Observations

The contracts monitored have not shown any major problems. Small overasphalted areas were observed on some OFC pavements. These areas were caused by asphalt draining down in the mix that was mixed too hot. This problem was eliminated by reducing the mix temperature to below 135°C at discharge.

Recommendations

The task force made the following recommendations:

- The ministry should continue the policy of using OFC and DFC mixes to surface urban freeways and main highways.
- Late fall paving with OFC and DFC mixes should be avoided, and DFC mix should be placed to depths of not less than 30 mm (1.2 in.). Resurfacing should include a leveling course (particularly for DFC mixes) to reduce the chance of thin lifts and subsequent raveling.
- Cheaper local aggregates may be used in OFC mix for shoulders.
- Mix temperatures at discharge into the truck at the plant should be between 122°C (250°F) and 135°C (275°F) (present specification has a maximum of 140°C and placement within 1 hr of discharge) in order to avoid asphalt drainage.
- Tandem paving on multilane facilities should be used wherever practical to reduce joint raveling.
- Dump boxes should be cleared after each discharge onto the paver.
- Maintenance patrollers should be informed of the location of OFC and DFC surface courses and the need to broadcast deicing salts on these surfaces.

RESURFACING PROGRAM

On the basis of the performance of the Highway 401 test sections, a hot-mix overlay system consisting of a binder course 40 mm (1½ in.) thick followed by an OFC surface course 25 mm (1 in.) thick was selected as the best method of rehabilitating the freeway.

In 1976 the ministry embarked on a program to rehabilitate the 50-km bypass around Toronto. The program is to be completed by 1995. A budget of from $3 million to $5 million has been allowed annually for this program. The rehabilitation involves five main areas:

- Additional capacity,
- Repairs to structures,
- Environmental improvements,
- Pavement and shoulder rehabilitation, and
- Construction staging or scheduling of the resurfacing.

On other concrete pavements that carry high volumes of traffic, the normal resurfacing design has been

- 25-mm (1-in.) sand leveling course,
- 50-mm (2-in.) open-graded (porous) binder course,
- 40-mm (1½-in.) binder course, and
- 40-mm (1½-in.) surface course.

These concrete pavements, however, have been allowed to deteriorate to a point where they require more substantial treatment, whereas the rehabilitation treatment, in comparison, on the Toronto bypass consists only of

- 40-mm (1½-in.) binder course and
- 25-mm (1-in.) OFC mix surface course.

It can be seen that the challenge of a high-volume road with many overhead clearance problems has been met with an overlay system that is far less expensive than the normal approach.

Performance of the Rehabilitation System

Pavement Ride

The placing of the new binder and OFC surface course mixes on the two contracts indicates an initial improvement in ride as determined by the Mays meter (Table 5). It is thought that the roughness figures may be improved even more by the use of automatic screed controls with skis on both sides of the paver when laying the hot-mix binder course in the future.

Surface Durability

The surface course OFC shows a small number of flushed areas due to overasphalting (drain down in truck boxes) and some

TABLE 5 ROUGHNESS MEASUREMENT BY MAYS RIDE METER

	Original			New			Change		
Contract	$\bar{x}$	σ	n	$\bar{x}$	σ	n	$\bar{x}$	σ	n
1	110	24	5	73	3	5	–38	17	5
2	141	19	7	72	14	7	–70	14	7

minor winter snowplow scuffing at grade points such as crowns and shoulders, but, all things considered, the durability of the OFC has been exceptionally good to date.

Annual Friction Course Resurfacing Program

The general effect of the OFC mix overlay on improving the driving qualities of the Toronto bypass has been substantial. In addition to the commitment to the overlay program on the bypass, there was a gradual increase in tonnage laid on freeways from 1978 to 1981 (Table 6). The steady volume of OFC and DFC mixes constructed each year confirms the commitment. The OFC and DFC mixes are specified for use on some 1500 km of main highway across the province (the Ontario Ministry of Transportation and Communications is responsible for 21 500 km of roadway), and most of it is used in southern Ontario. The tonnage is relatively small: less than 20 000 tonnes of OFC and 100 000 tonnes of DFC compared with an annual average total of about 2.5 million tonnes of hot mix.

TABLE 6 ANNUAL VOLUME OF FRICTION COURSE MIXES (tonnes)

Year	OFC	DFC
1976	5 080	27 400
1977	40 130	104 800
1978	11 098	104 800
1979	8 147	129 271
1980	7 357	113 698
1981	50 108	112 708
1982	10 531	73 096
1983	18 371	43 829
1984	36 213	76 488
1985	11 138	95 186
Total	198 173	881 276

In general, there is a misconception about the high cost of the special single-sized (-3/8 in.) washed coarse aggregate and washed screenings. Also, on the supplier's side, high stone content in the mix poses the problem of having to find an outlet for the surplus screenings. However, based on coverage per square meter, the cost of OFC (laid 25 mm thick, 24 kg/m²/10 mm) turns out to be about the same as or slightly less than that of the standard dense surface mix (HL-1) for medium-high traffic. Compared with DFC, its cost is about 35 percent lower (Table 7).

The aggregate used before 1982 for the OFC and DFC was mainly traprock. However, factors such as availability and good frictional characteristics of the slag aggregates (both steel and blast furnace slag) led to the increasing use of the slag mixes in later construction.

CONCLUSIONS

OFC and DFC mixes are accepted by the ministry as the most suitable mixtures for treatment of urban freeways and accident black spots where heavy traffic volumes and a large number of commercial vehicles prevail.

The sequence of events that led to the use of OFC and DFC is as follows:

1947—Construction of sections of Highway 401 bypass,
1952—Initial construction as a four-lane divided highway,
1963—Reconstruction of original facility to multilane facility,
—Widening to a 12-lane system of collector and express lanes,
—Marked increase in multicar wet weather accidents,
1974—Placement of 18 bituminous overlay test sections,
1976—Use of OFC and DFC in various contracts begun,
1978—Task force set up to review and recommend the use of friction course mixes for future construction,
—Policy on use of OFC and DFC formalized.

The task force's findings on friction courses follow:

1. General performance is satisfactory with raveled joints at reflection cracks.
2. Winter performance

   a. OFC provides the best friction properties;
   b. Salt should be broadcast.

TABLE 7 COMPARATIVE COST (1983) OF FRICTION COURSE MIXES (OFC and DFC) AND STANDARD MIX (HL-1) (1986 cost about $1.50/tonne higher)

	Cost ($Cn)				Percentage Above (below) HL-1		
Unit	HL-1	DFC	OFC	OFC	DFC	OFC	OFC
mm	40	40	40	25	40	40	25
Tonne	34.00	42.00	50.00	50.00	24	47	47
m²	3.50	4.30	5.15	3.20	23	47	(9)

3. Friction characteristics are superior to those of standard dense mixes.
4. Safety: greater than 40 percent reduction in accidents after rehabilitation.
5. Riding quality is rated good to excellent.
6. Noise level is 2.1 dB(A) quieter than standard dense mixes.
7. Zone painting requires the same quantity as standard dense mixes, but adhesive-backed strips must be rolled in when mix is hot.
8. Construction

   a. Avoid late fall paving;
   b. Cheaper local aggregates may be used in OFC mixes for shoulders;
   c. Discharge temperature at the plant should be between 122°C (250°F) and 135°C (275°F);
   d. Tandem paving should be used to reduce joint raveling;
   e. Dump boxes should be cleaned after each discharge.

To be effective in improving the levels of friction, bituminous friction course mixes must contain harsh, angular, fine aggregate such as traprock or slag fines and a sufficient proportion of crushed good-quality coarse aggregate (50 to 70 percent retained on 4.75-mm sieve) to maintain the micro- and macrotexture on the traveled surface. These characteristics can be provided by the open friction course and the dense friction course.

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FAA Mixture Design Procedure for Asphalt-Rubber Concrete

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A mixture design procedure was developed to allow the use of asphalt-rubber binders in concrete for flexible airport pavements. The rubber material considered in this project included only rubbers produced by grinding scrap tires. Such materials are widely available across the United States and have been used in seal coat and interlayer construction for almost 25 years. However, only limited experimentation has been done using this type of asphalt-rubber in concrete for flexible pavements. The asphalt-rubbers chosen for use in this project were produced in the field. The materials used in this study were shown to be similar to materials produced in the laboratory from the same combination of ingredients. The results of this study include a suggested laboratory procedure for producing asphalt-rubber for use in the asphalt-rubber concrete mixture design procedure. A suggested setup of equipment is also included along with vendors who have these items on the shelf. The mixture design procedure includes modifications to the standard FAA procedure included in the Asphalt Institute MS-2 manual. These modifications mainly involve mixing and compaction temperatures and gradation changes to accommodate the solid rubber particles added to the asphalt.

Charles H. McDonald, Consulting Engineer, Phoenix, Arizona, is considered to be the father of the asphalt-rubber systems developed in the United States. McDonald's laboratory work, which was initiated in 1963, resulted, in the mid-1960s, in the development of a patented patching material that consisted of 25 percent ground scrap vehicle tire rubber and asphalt cement blended at approximately 375°F for 20 min.

McDonald continued his experimental work with the city of Phoenix and initiated research efforts with Atlos Rubber, Inc. Several experimental test sections were placed at Phoenix Sky Harbor International Airport (1966) and on US-80 near downtown Phoenix. Sahuaro Petroleum Asphalt Company (Sahuaro) became interested in the product and cooperated in testing seal coat applications. In 1975 Arizona Refining Company (ARCO) began experimental work with asphalt-rubber binder systems. The result of the experimental work conducted by McDonald, ADOT, Sahuaro, and ARCO has led to the use of asphalt-rubber in about 35 states.

National conferences have shown the need for additional information on performance, relationships between laboratory developed properties and performance, design techniques for specific applications, specifications and tests for compliance, and construction practices. Although recent work has helped to more clearly define some of those areas of concern, there is a continued need to more clearly define the circumstances in which these various treatments can best be used to solve the maintenance problems encountered.

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LABORATORY TESTING AND PRODUCTION OF ASPHALT-RUBBER BINDERS

Laboratory Testing of Asphalt-Rubber Binders

Concerted attempts have been made to evaluate asphalt-rubber binders by applying laboratory tests developed for specification testing and characterization of asphalt cements. Few attempts show much success. Repeatability depends on uniform consistency of the asphalt. Because asphalt-rubber is a blend of asphalt and fine rubber particles, the discrete nature of the rubber produces considerable variation in test results.

A variety of laboratory tests for characterizing asphalt-rubber materials has been evaluated by researchers such as Pavlovich et al. *(1)*, Shuler and Hamberg *(2)*, Jimenez *(3)*, Oliver *(4)*, and Chehovits et al. *(5)* (see Table 1). Although several of these test procedures offer promise for characterizing the behavior of asphalt-rubber or for detecting differences among various combinations of components, none appears to be suitable for use in specification testing of these materials. Indeed, little of the research to date has been directed toward defining the characteristics that an asphalt-rubber binder should have in order to meet prescribed performance requirements.

One of the most significant problems faced by asphalt technologists is that there are inadequate behavioral models to describe the function of the binder in an asphalt aggregate system. Therefore, technologists continue to use correlations between laboratory tests, such as the ring and ball softening point, and engineering properties, such as stiffness, developed by Shell researchers during the 1950s and 1960s.

These methods appear to work for asphalt cements and have been organized into well-developed, comprehensive design procedures. However, design methods such as the Shell Pavement Design Guide *(6)* cannot be applied to asphalt-rubber binders without extensive testing programs to develop the relationships among binder characteristics and mixture properties. In the current study, procedural or recipe methods were selected for preparation of the asphalt-rubber mixtures for both

TABLE 1 LABORATORY TESTS USED TO CHARACTERIZE ASPHALT-RUBBER

Laboratory Procedure	Pavlovich et al. *(1)*	Shuler and Hamberg *(2)*	Jimenez *(3)*	Oliver *(4)*	Chehovits et al. *(5)*
Ring and ball softening point	x				
Absolute viscosity at 140°F	x				
Ductility at 39.2°F and 77°F	x		x		
Double ball softening point (phase change temperature)			x		
Force ductility	x	x			
Constant stress (Schweyer) rheometer	x	x			
Sliding plate microviscometer/ rheometer				x	x
Falling coaxial cylinder viscometer			x		

laboratory studies and full-scale field projects until more fundamental relationships can be developed.

Factors That Affect Properties of Asphalt-Rubber Materials

Background

Rubber has been incorporated in asphalt roadways since the beginning of this century *(7)*. Early asphalt-rubber combinations used natural rubber, which is susceptible to oxidation and, when overheated, converts to an oil and loses its beneficial properties *(8)*. These deficiencies were overcome with synthetic rubber, which was compounded and vulcanized to resist heat and weathering. Although synthetic rubber lacked the solubility of natural rubber, it could be reacted with asphalt to produce similar characteristics, but much larger quantities were required.

In some cases the synthetic rubber appeared to absorb the oils out of the asphalt leaving blends that exhibited poor adhesive properties *(8)*. Researchers found that asphalts with low aromatic oil contents produced these dry blends. This problem was overcome when whole truck tire rubber, with about 18 percent natural rubber *(9)*, was used. With this high-natural-rubber scrap the blend exhibited the desired sticky elastic character of the early natural rubber blends but had greater heat stability.

On the basis of the knowledge gained from these trial investigations, formulations of asphalt, extender oil, and scrap rubber have been developed that produce a material with the desired characteristics. A discussion of the factors that affect asphalt-rubber properties follows.

Rubber Factors

The factors that most influence the formulation of asphalt-rubber are discussed in this subsection. Most of these factors have been investigated thoroughly. Although most are known to be important in asphalt-rubber production, their effect on specific performance-related factors is not well understood.

Rubber Type A wide assortment of scrap rubber is available for use in asphalt-rubber systems. The chemical composition of the rubber varies depending on the sources of the scrap such as automobile tires, and truck or bus tires, and whether the rubber is tread peel or whole carcass rubber. LaGrone *(10)* defined the terms related to processing of scrap rubber and provided typical composition of scrap rubbers available for production of asphalt-rubber binders. The type of rubber selected affects the elasticity of the resulting asphalt-rubber *(1, 2, 4, 5, 9)* and the stability of the reacted product *(8)*.

Rubber Processing Method The method of processing the scrap rubber significantly affects the digestion of the rubber and the properties of the asphalt-rubber. Oliver *(4)* found rubber morphology (structure) to be the most important factor affecting elastic properties. Shuler *(11)* reported differences in viscosity among rubbers of different morphology, but morphology was confounded with differences in particle size and natural rubber content.

Oliver *(4)* included electron micrographs of rubber particles to show the differences between the surface morphology of particles ground at ambient temperature and those cryogenically ground. These differences affect the surface area of the rubber and the rate at which the reaction occurs.

In addition to rubber morphology, the size of the rubber particles and whether the rubber has been processed after grinding (i.e., devulcanized) both affect the rate of reaction of the asphalt-rubber *(8, 9)*. These last two factors affect the type of asphalt selected for the digestion process *(12)* more than the engineering properties of the asphalt-rubber produced *(4, 11)*.

Rubber Concentration Asphalt-rubber, as currently used, includes between about 15 and 28 percent by total weight of dry rubber in an asphalt cement matrix. The rubber concentration is acknowledged by all researchers to significantly affect the properties of the reacted asphalt-rubber binder. Specifying agencies often use the general specifications of a supplier including the proportions of the asphalt-rubber components, the component specifications, and the blending conditions. Indeed, specifications from Texas *(13)*, New York *(14)*, and Arizona *(12)* are similar in style and content, which indicates

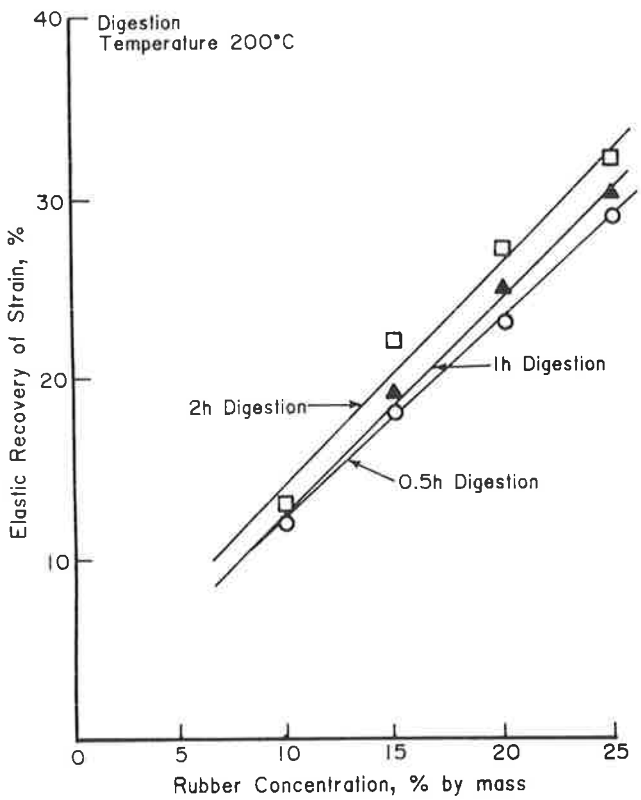

FIGURE 1 Effect of rubber concentration on elastic recovery *(4)*.

that the product is fairly well defined in terms of materials and processes.

Researchers all indicate that rubber concentration significantly affects the properties being measured. The effect of rubber concentration on elastic recovery found by Oliver is shown in Figure 1. Similar levels of strain recovery have been reported by Chehovits et al. *(5)*.

Reaction Temperature and Mixing Time Reaction temperature and mixing time combinations significantly affect asphalt-rubber properties *(1, 2, 4, 11)*. Figure 1 shows the effect of mixing time at a constant temperature on elastic recovery. Figure 1 data indicate that a prescribed elastic recovery could be achieved by reducing rubber concentration while increasing mixing time. However, Shuler *(11)* has shown that as the mixing time increases the amount of solid rubber in the mixture begins to be reduced. Shuler extracted the solid rubber from the asphalt-rubber mixture and performed gel permeation chromatography (GPC) tests on both the virgin asphalt and the asphalt-rubber. The GPC molecular weight distribution was shifted at both the high and the low ends indicating that, as digestion continues, some rubber may be lost to the asphalt. Huff and Vallerga *(8)* also discuss the reaction of natural rubber in asphalt cement and point out that when high-natural-rubber scrap is used the material exhibits the same characteristics as those with only natural rubber and asphalt. The major difference between these two types of mixtures is that synthetic rubber digestion is slower, but it is more heat stable than natural blends and more forgiving of field delays.

Shuler *(11)* has also shown that, even though various combinations of mixing temperature and time may be selected, the viscosity during digestion can be used to terminate the mixing process at a consistent viscosity level (Figure 2). Monitoring the viscosity in the field can also allow materials to be prepared in the laboratory at the same digestion level. That materials can be produced in the laboratory with properties similar to those produced in the field from the same ingredients has been verified by Shuler *(11)* and Shuler et al. *(15)*. However, Shuler et al. *(15)* indicated that low-level field digestion did not produce mixture properties corresponding to those produced by low-level digestion in the laboratory. Low-level field digestion was somewhere between low and moderate laboratory levels. The important thing is that laboratory-produced asphalt-rubbers are similar to those produced in the field. A suggested laboratory production procedure is described in the next section.

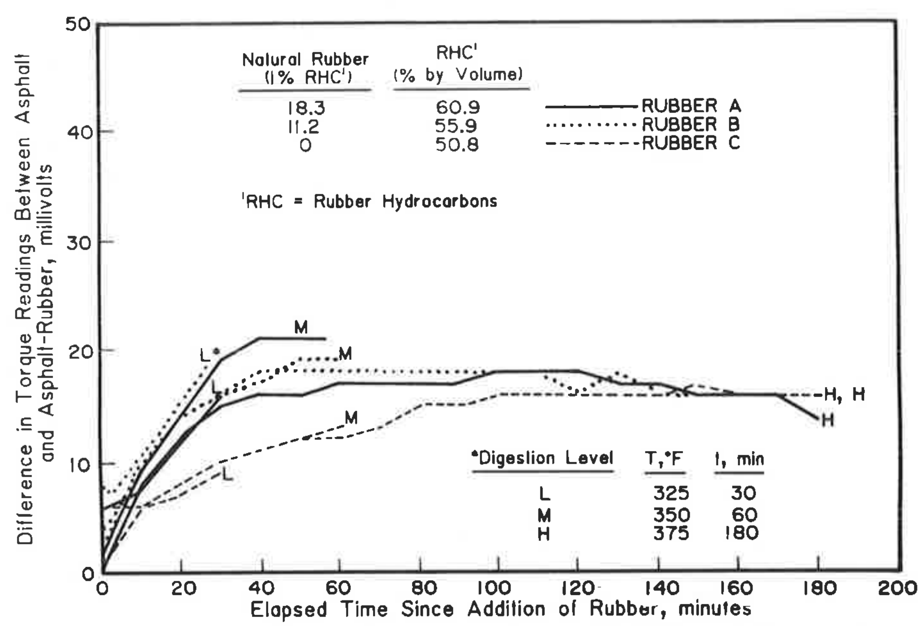

FIGURE 2 Torque-fork output for three rubbers used in El Paso at 22 percent rubber and three digestion levels *(11)*.

Laboratory Production of Asphalt-Rubber

Asphalt-rubber has been produced in the laboratory by using a variety of techniques that vary from open containers *(12)* to closed systems *(15)*. Apparently both systems produce suitable asphalt-rubber materials.

Reaction times in laboratory studies have varied from 0.5 to 2 hr at temperatures that typically range from 325°F to 450°F. The effect of reaction time has been evaluated using both viscosity and the properties of the reacted asphalt-rubber. Oliver *(4)* investigated the effect of both reaction time and temperature on elastic recovery strain of natural and synthetic rubbers. The plots in Figure 3 quite clearly show the interaction and that the properties can be significantly reduced by too long a reaction time. Notice in Figure 3 that the peak of elastic recovery shifts toward lower temperature as digestion time increases and that the elastic recovery drops off sharply for the 2-hr digestion time when temperature exceeds 425°F. Oliver showed that synthetic rubber is much more stable under higher digestion conditions than are natural rubbers.

Shuler et al. *(15)* conducted a study to evaluate the effect of rubber type, concentration, and digestion conditions on viscosity and properties of the resultant asphalt-rubber binders. A series of plots was included to show the influence of these factors. Figure 2 shows several of these effects. All three of these rubbers were vulcanized with Rubbers A and B ground under ambient conditions; Rubber C was cryogenically ground. A review of the plots shows that at least the medium level of digestion is required to achieve a stable viscosity within a reasonable time and that the high temperature (375°F) has the advantage that most mixtures reached a stable viscosity within 1 hr. Notice too that Rubber C is the slowest reacting mixture; it has no natural rubber and was cryogenically ground. However, Rubbers A and B both reach stable viscosities at 375°F after 1 hr of digestion.

On the basis of a survey of laboratory reaction conditions from selected literature and Shuler's *(11)* comparison of viscosities of rubber asphalts produced in both the field and the laboratory, it is recommended that laboratory mixing be performed at 375°F for 1 hr or until the viscosity-versus-time plot is relatively constant. A suggested procedure is described next.

Suggested Laboratory Procedure

Equipment

The following list of equipment is recommended for digestion of asphalt with rubber to produce binders for mixture design:

1. Induction motor stirrer—variable torque, constant speed motor capable of operating at 500 rpm to monitor viscosity and automatically adjust motor power to maintain selected speed.
2. Proportional temperature controller to maintain temperature in reaction kettle to within ±0.10°C for temperatures up to 250°C. Power available to heaters shall be approximately 750 watts.
3. Electric heating mantle for round bottom 2000-mL flash with thermocouple and power output of from 500 to 750 watts.
4. Three-neck reaction flask with 24/40 ground glass joints.
5. Teflon bearing for stirring rod used with Item 1 can be custom made or scavenged from a closed system stirrer for vacuum work such as Fischer 14-513-100 stirrer for vacuum work or Cole-Parmer K-4740-00 closed system stirrer with 24/40 glass joint.
6. Ring stand and supporting equipment.

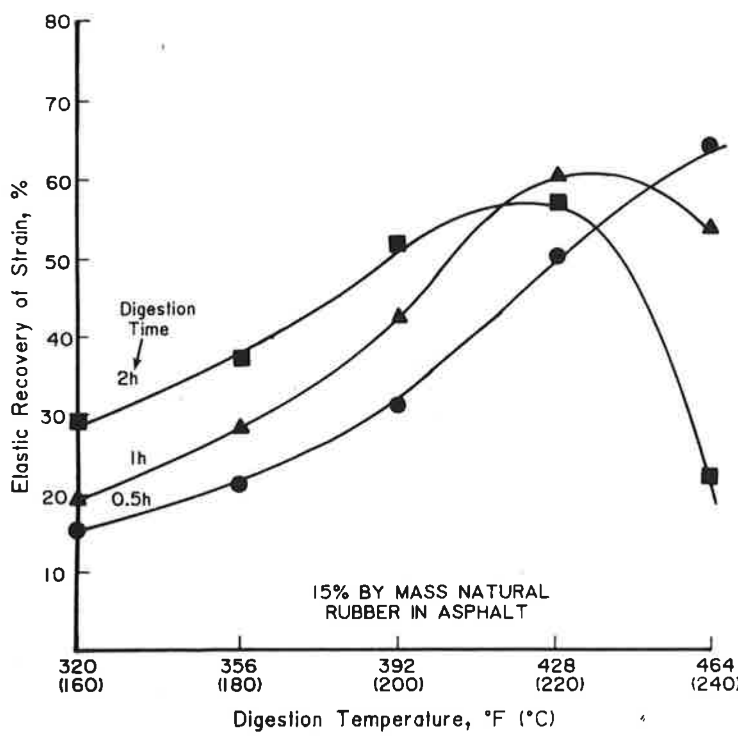

FIGURE 3 Effect of digestion time and temperature on elastic recovery for asphalt-rubber from national rubber tire buffings *(4)*.

7. Strip chart recorder for monitoring output of stirrer (optional).

Procedure

The suggested procedure is based largely on the experience of researchers from Arizona, New Mexico, and Texas *(1, 2, 11, 15, 16)*. The procedure is based on the assumption that reaction should continue until a stable viscosity (torque from the stirrer) is achieved. Even though a stable viscosity can be achieved using a variety of mixing times and temperatures, a particular combination is suggested in order to provide guidance in preparing suitable materials for mixture design. Figure 4 shows a typical equipment setup.

The proposed reaction system consists of a constant-speed motor with a propeller stirrer for constant agitation of the asphalt-rubber. Heat is supplied through an electric heating mantle and is monitored and adjusted by an electronic temperature controller. The stirrer acts as a rotational viscometer that can measure relative changes in fluid viscosity during digestion. Viscosity can be estimated by calibrating the stirrer output with viscosity measurements from a Haake portable rotational viscometer model VT-02 or a Brookfield viscometer. Shuler et al. *(15)* developed such a correlation with a Brookfield viscometer; the coefficient of determination was 99 percent. Such procedures appear to be quite satisfactory for controlling the mixing process and for correlating laboratory viscosity with viscosity during digestion in the field.

Samples of the materials to be used in the field should be secured using appropriate statistical sampling procedures to ensure that representative materials are obtained. Materials to be sampled include the asphalt cement, the rubbers, and the diluents.

- Step 1: Heat approximately 1000 mL of the asphalt slowly and stir to avoid local overheating. When the asphalt is fluid add the appropriate amount to the 2000-mL reaction flask; also add diluent if included in the mixture. Insert the mixer propeller, continue heating the asphalt, and increase the mixer speed to 500 rpm.
- Step 2: When the asphalt cement reaches 375°F, add the proper blend of rubber to the flask through the neck. Add the rubber as quickly as possible (approximately 10 sec). Begin digestion time as soon as the rubber has been added and the environment of the flask secured.
- Step 3: Continue reacting the asphalt-rubber for not less than 1 hr or until the output from the stirrer reaches a uniform level. Reaction time is a function of the type, morphology, concentration, and gradation of the rubber materials and can vary considerably (see Figure 2 and the attendant discussion).
- Step 4: On completion of blending, the asphalt-rubber is ready for mixing or storing.

ASPHALT-RUBBER CONCRETE MIXTURE DESIGN

Background

Although asphalt-rubber materials have been used extensively in seal coat and interlayer construction, only a limited amount of experimental work has been done using asphalt-rubber as a binder in asphalt concrete construction. Some of the earliest work reported in the literature was by Jimenez *(17)* in 1979 and later *(12)* in 1982. Jimenez prepared asphalt-rubber using the same techniques and formulations as those used for seal coats, membranes, and interlayers in Arizona. The aggregate was for a standard dense-graded surface with a top size of 3/8 in.

Jimenez used two different compaction methods:

1. The Triaxial Institute compactor, also known as the California kneading compactor, using test method ARIZ 803, and

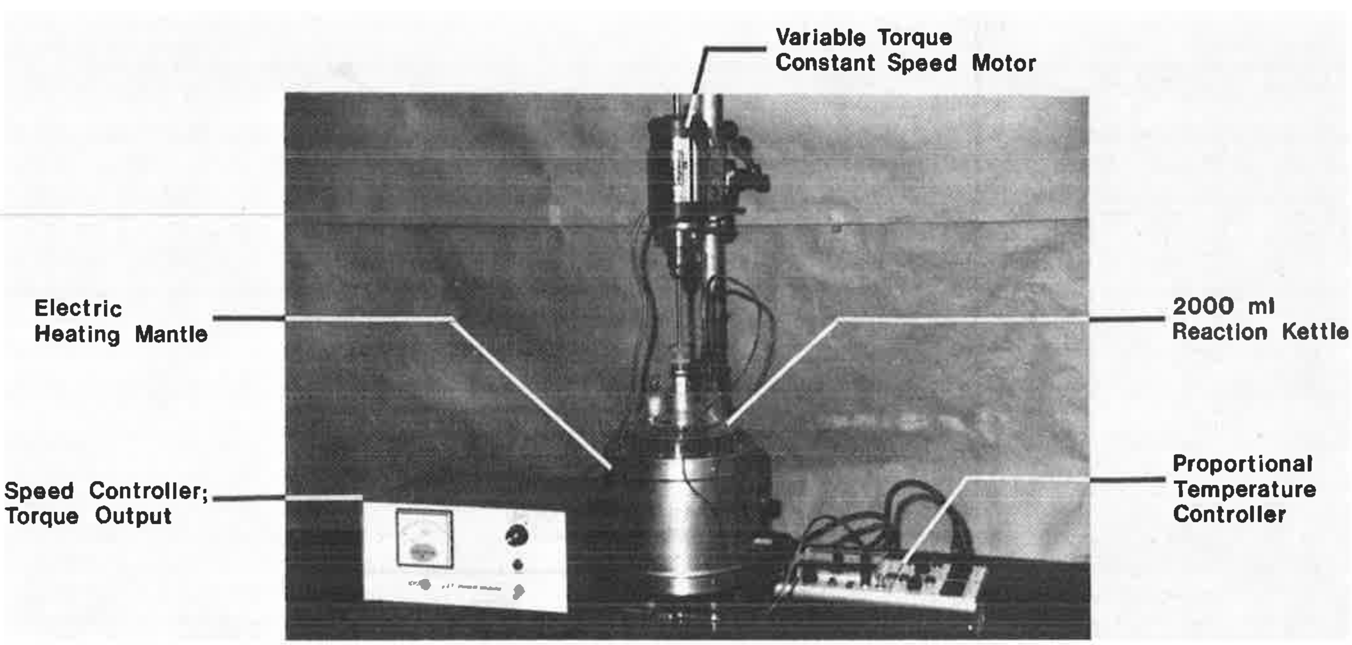

FIGURE 4 Suggested equipment setup for laboratory production of asphalt-rubber material *(15)*.

2. The vibratory kneading compactor described elsewhere *(18)*.

Both of these methods employ techniques for applying compactive energy that are considerably different from that of the standard Marshall test method used by the FAA *(19)*.

Jimenez observed a number of differences between the behavior of the asphalt-rubber concrete specimens and a standard asphalt concrete. He noted that, after compaction with the California kneading compactor, it was necessary to leave the asphalt-rubber concrete specimens in the mold for 3 days because, if extracted before then, the specimens would swell up to the point of cracking and the radial dimension would increase so much that the specimens would not fit into the Hveem stabilometer shell. During testing he found that the asphalt-rubber specimens would not hold the confining pressure without a preload applied before Hveem testing began.

Hveem specimens were also prepared using a modification of the vibratory kneading compaction procedure. The modification involved the application of a static load of 3,770 lb after vibratory compaction was completed. Apparently the swelling problems noted with the California kneading compactor did not occur with the vibratory kneading compactor *(12)*. None of the other researchers who have prepared asphalt-rubber concrete specimens have reported significant problems with swelling of specimens *(20, 21)*. Lalwani et al. *(20)* and Schuler et al. *(21)* used the Marshall hammer for specimen compaction. However, in this study, when specimens were extruded immediately after compaction, some swelling did occur. Therefore all specimens were allowed to cool to room temperature before extrusion from the mold.

Only a limited number of studies have been reported that used asphalt-rubber binders as defined in this paper. Other studies have included the use of scrap rubber in an asphalt concrete, but the rubber is treated as an aggregate and not reacted with the asphalt before mixing with the aggregates. One of the latest studies of this type was conducted by Takallou et al. *(22)*.

The combinations of mixing and compaction conditions for asphalt-rubber concrete included in the literature cited previously are given in Table 2. Notice that both the mixing and the compaction temperatures are considerably higher than those used for asphalt concrete. The primary reasons for the higher than normal temperatures are (a) the very high viscosity of the asphalt-rubber binder at typical mixing and compaction temperatures defined for the Marshall method *(19)* and (b) the difficulty of wetting the aggregate surface with the asphalt-rubber, which is more elastic than the untreated asphalt cement *(23)*. It should be noted, however, that in the laboratory no problems have been reported with coating aggregate particles using standard mixing equipment *(12, 17, 21)*.

Development of the Modified Mixture Design Method

Asphalt-rubber concrete was fabricated and tested in the laboratory. Two gradings of aggregate were evaluated using asphalt-rubber and conventional asphalt binders. Results of laboratory tests are compared with control asphalt concretes fabricated with identical types and gradations of aggregate. The control mixtures were fabricated using conventional techniques for asphalt cement binder. The experimental mixes were fabricated using slightly modified techniques and two asphalt-rubber binders.

Materials

Asphalt-rubber from two sources was used for the experiments reported in this section of the paper. Samples of the asphalt-rubber were obtained in the field from actual construction sites. Type A contained 25 percent rubber by weight and Type B contained 18 percent. The gradations of the rubber particles are shown in Figure 5.

Two standard laboratory aggregates used by the Texas Transportation Institute on numerous other research projects were used for the mix design. These aggregates are a subrounded river gravel obtained from a local Brazos River source and a limestone from near Brownwood, Texas. Gradations used for control asphalt concrete mixes are shown in Figure 5. Although these gradations follow the lower edge of the FAA specification band, it was reasoned that mixtures in this region would be most critical and that fabrication procedures suitable for them would function properly for coarser gradations. A slight modification was made in the gradations of these materials to allow room for rubber particles in the mix. A blending of the rubber grading and modified mineral aggregate grading resulted in a combined gradation that matched the control aggregate gradation without rubber.

Control asphalt concretes were prepared using AC-10 asphalt cement and subrounded river gravel and limestone at the gradations shown in Figure 5. Control asphalt concrete test

TABLE 2 ASPHALT-RUBBER CONCRETE SPECIMEN PREPARATION CONDITIONS REPORTED IN THE LITERATURE

	Compaction Type				Temperature Conditions (°F)		
					Mixing		
Investigator	California Kneading Compactor	Vibratory Kneading Compactor	75-Blow Marshall Hammer	Mixing Time (min)	Asphalt-Rubber	Aggregate	Compaction
Jimenez *(12, 17)*	x	x		2	375	300	250
Lalwani *(20)*			x	—[a]	—[b]	—[b]	—[b]
Dickson *(23)*			x	Until coated	375	375	375
Vallerga *(24)*			—[b]	—[b]	350	350	325

[a]Not included but no problem in mixing reported.
[b]Not included.

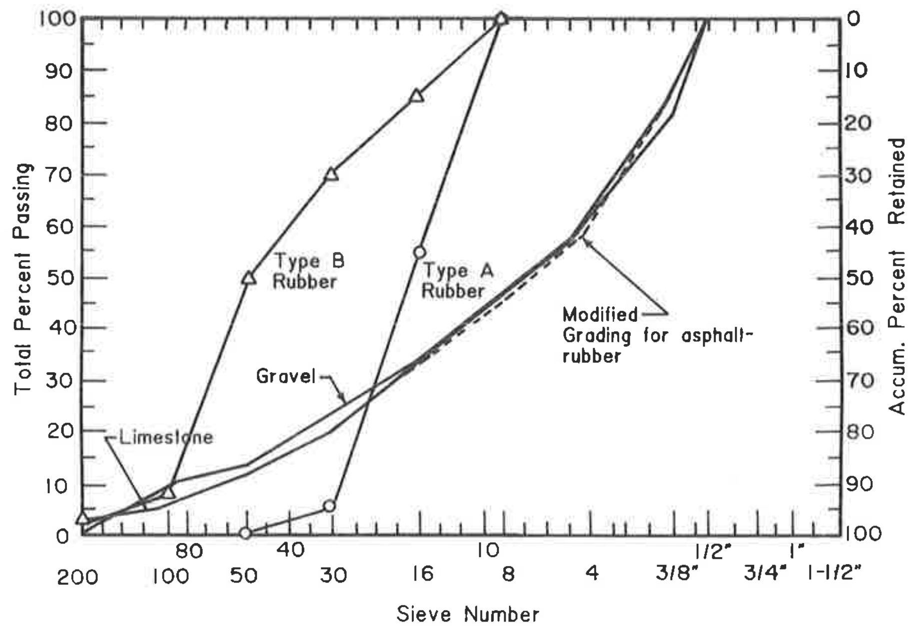

FIGURE 5 Gradations of aggregates and rubbers for asphalt-rubber concrete.

results for the gravel mix were obtained from a recent study by Button et al. *(25)*, and control asphalt concrete test results for the limestone mix were obtained during the course of this study.

Specimen Fabrication Experiment

To determine if the fabrication techniques for preparing laboratory specimens needed to be different from those of the standard Marshall mixture design method, an experiment was performed that included variations in compactive effort and mixing and compaction temperatures. These experiments were conducted using the subrounded river gravel because (a) the principal investigators thought that this material would be most sensitive to variations in the viscosity of the asphalt-rubber with temperature and (b) subrounded gravel is relatively easy to compact, so variations in response of the mixtures to compactive effort would primarily reflect the effect of the asphalt-rubber binder.

The fabrication experiment was conducted at a binder content of 5.5 percent by weight of the aggregate in order to yield an air void content between 6 and 8 percent. This range of air void content was selected to allow comparisons of the properties of the asphalt-rubber concrete and the control mixtures, which were prepared with air void content between 6 and 8 percent to allow running moisture susceptibility tests using the modified Lottman conditioning procedures.

Design of Experiment Asphalt-rubber concrete samples were fabricated at 5.5 percent binder by weight of aggregate using the Marshall method of compaction. Three different blow counts (25, 50, and 75 blows per face) and three different temperatures (275°F, 325°F, and 375°F) were used to determine an optimum fabrication technique. Because this portion of the study was a cooperative venture of the FHWA and the Texas State Department of Highways and Public Transportation (SDHPT), the following tests were performed on all specimens: Marshall stability (lb), Hveem stability (%), resilient modulus at 77°F (psi), and air voids (%).

Evaluation of Results Tests were performed on specimens fabricated at the various temperatures and compactive efforts and the test results are shown in Figure 6. Test results for Marshall stability show that both compactive effort and compaction temperature have a significant effect on Marshall stability. Even at the low compactive effort, a compaction temperature of 375°F reduces the viscosity of the asphalt-rubber at compaction sufficiently for the compacted specimen to show a stability much higher than that of the control asphalt concrete with an AC-10. The additional compactive effort of from 25 to 75 blows produces a mixture with an increase in stability at 375°F of about 50 percent. Hveem stability is fairly insensitive to temperature and number of blows of compaction. This is because Hveem stability is largely a measure of aggregate interlock and friction and is not particularly sensitive to binder viscosity. When the aggregates have achieved a fairly dense state, Hveem stability does not change much with changes in binder viscosity.

Air void content is fairly sensitive to both compaction effort and temperature. Air void content generally decreases either as mixture temperature increases or as compactive effort increases. Notice that only at 75 blows per face does the air void content approach the selected value of 7 percent. This perhaps reflects the difficulty of compacting fine dense-graded mixtures.

Resilient modulus is less sensitive to compactive effort than to compaction temperature. There is generally an increase in resilient modulus with an increase in both temperature and compactive effort. However, because the Marshall mixture

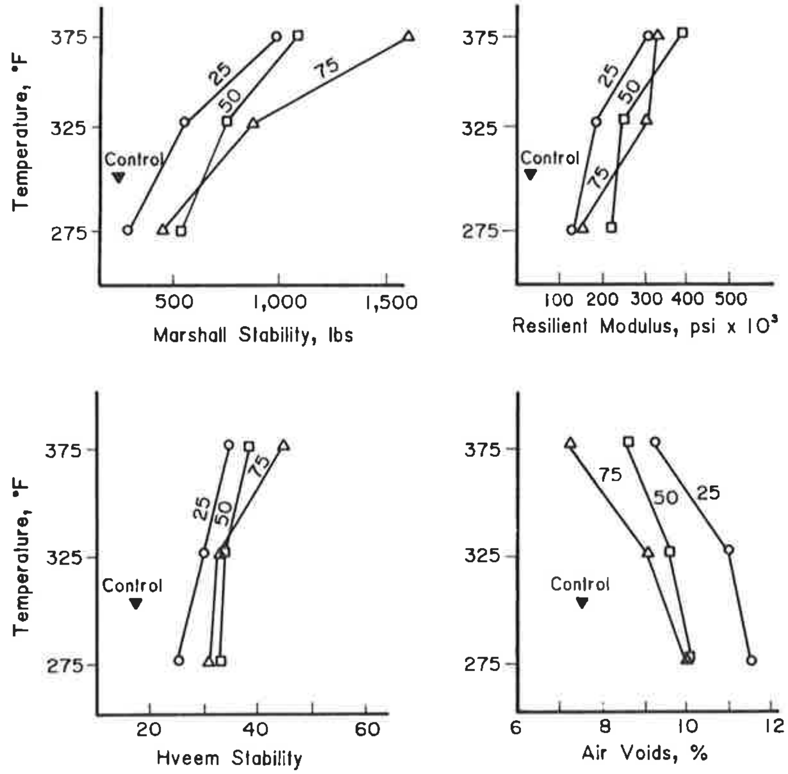

FIGURE 6 Asphalt-rubber concrete properties for fabrication experiment using gravel aggregate and Rubber B.

design method does not include resilient modulus, more emphasis was placed on the sensitivity of Marshall stability and air void content to fabrication conditions.

Because there is a clear effect of temperature and compactive effort on both Marshall stability and void content, it is not difficult to determine that both the highest temperature and compactive effort should be used in fabricating the asphalt-rubber concrete specimens for mixture design, and, indeed, the major modifications to the current MS-2 manual procedures include modifications of the mixing and compaction temperatures.

Sample Mixture Design

An example mixture design was performed in the laboratory to evaluate the modification to the design procedure and to verify that a satisfactory design could be developed using crushed materials and a different asphalt-rubber. A mixture design was developed using the crushed limestone with the gradation shown in Figure 5 and Type-A asphalt-rubber. The modifications to the standard MS-2 procedure included

1. Adjusting the aggregate grading to permit space for the rubber particles—in essence the rubber was treated as an additional aggregate;
2. Mixing and compaction temperatures were 375°F; therefore the aggregates and the asphalt-rubber were heated to 375°F before mixing;
3. Compaction effort was 75 blows per face without regard to gear load;
4. Mixing was performed using a high-energy mechanical mixer; and
5. Compacted specimens were allowed to cool to room temperature before being extruded from the mold.

Using these modifications, three specimens were prepared at each of the following asphalt-rubber contents: 4.5, 5.5, 6.0, 7.5, and 8.5 percent asphalt-rubber by weight of aggregate. The results of testing are shown in Figure 7 for the standard plots used in the Marshall mixture design procedure. These plots show behavior similar to that expected from any dense-graded aggregate, and the design laboratory asphalt content is 6.7 percent on the basis of the data in the following table.

Property	*Percentage Asphalt-Rubber*
Optimum for maximum stability	6.2
Optimum for bulk specific gravity	7.2
Median for air void content	6.7
Average	6.7

Summary

A set of modifications to the standard FAA mixture design procedure has been suggested. These modifications will permit the use of asphalt-rubber instead of asphalt in asphalt concrete.

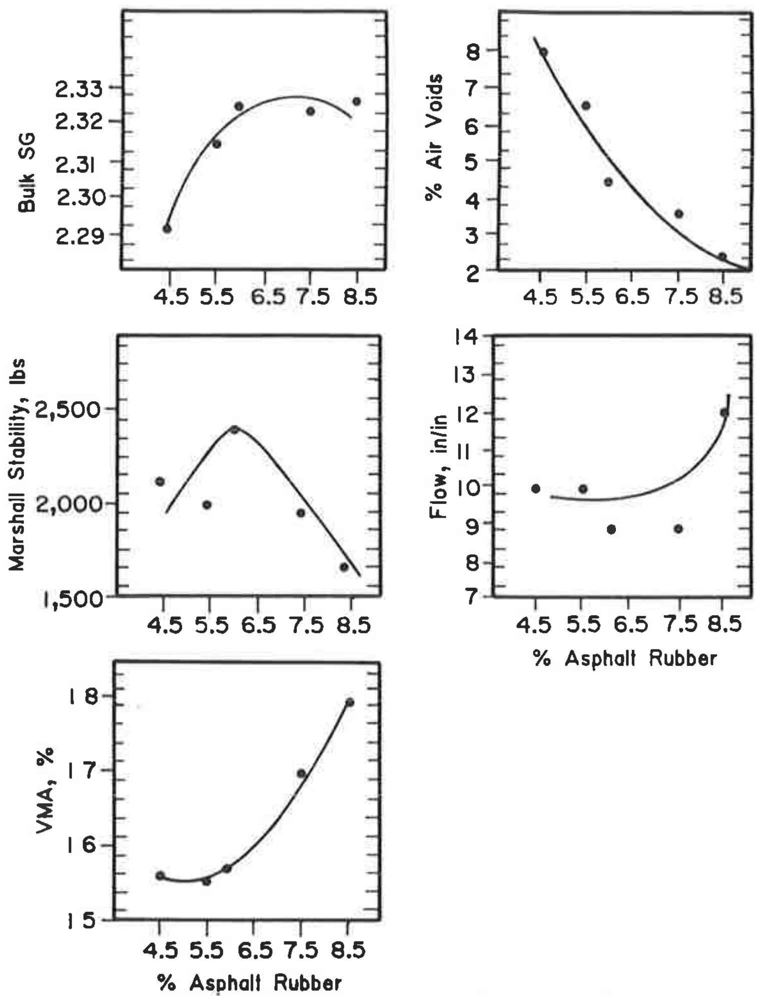

FIGURE 7 Asphalt-rubber concrete mixture design results for Type-A asphalt-rubber.

The only rubber included in this investigation was that produced by grinding scrap tires. The suggested modifications were developed on the basis of results of an experiment involving a range of mixing and compaction temperatures and compactive efforts.

A mixture design was performed on an asphalt-rubber and an aggregate that were different from those used to develop the modifications. No problems were encountered in the conduct of the mixture design nor in analysis of the test results.

CONCLUSIONS AND RECOMMENDATIONS

The primary objectives of this research were to define the laboratory conditions necessary for producing a mixture design of asphalt-rubber concrete. Because laboratory tests used to evaluate the properties of asphalt-rubber are not defined sufficiently for specification purposes, a procedural method has been included for laboratory production of asphalt-rubber.

Within the bounds of the experiments included in this overall research effort *(11, 15, 21, 26)*, the following conclusions and recommendations are deemed appropriate:

1. Laboratory-produced blends of asphalt-rubber binder using the same combination of asphalt and ground scrap tire rubber as is used in field installations have been shown to exhibit similar properties. Therefore, laboratory-prepared materials should exhibit characteristics similar to those of materials prepared in the field.
2. Reacted asphalt-rubber binders can be produced in the laboratory in quantities sufficient for use in asphalt-rubber concrete mixture design using a modification of the Marshall method of mixture design. These reacted materials can be prepared beforehand, cold-stored, and reheated for use in mixture design with no apparent effect on binder characteristics.
3. A laboratory procedure for producing asphalt-rubber binders has been presented.
4. Coating aggregates with hot asphalt-rubber is easily accomplished using a Hobart A200 mechanical laboratory mixer at temperatures well below those needed for compaction (375°F).
5. The aggregate gradation should be modified to allow space for the ground rubber. This is most easily accomplished by considering the rubber as an extra aggregate.
6. For Marshall hammer compaction, 75 blows per face at

375°F appear adequate. The specimens should be cooled to room temperature in the mold before extrusion.

7. Successful mixture designs can be accomplished in the laboratory using the procedures suggested in this paper. Mixtures prepared with asphalt-rubber binders exhibit higher stabilities than do similar mixtures made with asphalt cement.

8. Field trials should be conducted using dense-graded materials to ensure that these recommendations are applicable to a wider range of materials.

ACKNOWLEDGMENTS

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Chip Sealing in New Brunswick

JOHN E. MILLER

As an economic alternative to asphaltic concretes, the New Brunswick Department of Transportation has been placing surface treatments and chip seals on its highway system for more than 45 years. Surface treatments placed over granular or soil-stabilized bases on low-traffic-density roads have provided excellent dust-free riding surfaces for the province's rural population; chip seals, considered only for maintenance functions, have been successfully employed to extend pavement life, rejuvenate oxidized surfaces, seal cracks, and improve surface friction. A variety of cover aggregates from more than 100 sources as well as many grades of liquid asphalt are used to construct seal coats in New Brunswick. The particular combination of materials ultimately decided on for any seal project is dictated by field conditions and the type of riding surface required. The construction procedures and materials used by the department today for constructing this type of surface have evolved over nearly one-half century of trial and error. Some of that history is related in this paper, and present-day applications of surface treatments and reseals in the region are discussed.

This is not meant to be a technical paper but a brief outline and description of the practical applications of the chip seal and surface treatment processes in New Brunswick.

The province of New Brunswick is located on the East Coast of Canada adjacent to the state of Maine. New Brunswick has an area of 27,750 mi^2, approximately that of West Virginia, and a population of 750,000 people, 50 percent of whom live in rural areas.

As early as the 1930s government officials realized that, to provide a dust-free, relatively smooth, secondary road system for the widely dispersed rural population, a method that was more economical than placing expensive asphaltic concrete had to be developed. In 1939 this realization resulted in the New Brunswick Department of Transportation embarking on its first surface treatment program. This program consisted of contracting out 20 mi of provincial highway for surface treatment. That meager beginning has grown to the stage where the province now has a surface-treated road inventory of approximately 5,000 mi.

In 1957 senior management, thinking that it would be more economical and that better control could be exercised over the program, opted to field their own seal crews instead of contracting this work. This result of this decision was the establishment of five divisional crews, each responsible for carrying out the work in 3 of New Brunswick's 15 counties. That organizational format is still in existence today.

A typical crew consists of approximately 27 employees and the following equipment.

- One tank car heater,
- One power broom,
- One service truck,
- Six tandem gravel trucks,
- Two asphalt pressure distributors,
- One self-propelled aggregate spreader,
- One pneumatic-tired roller,
- One rubber-coated steel drum vibratory roller,
- One mobile field office,
- One dining trailer,
- One 20-man lodging trailer,
- Two equipment floats, and
- One minibus.

The province's chip seals and surface treatments are placed almost entirely by its five highly mobile crews.

No public tenders have been called by the department to place these types of surfaces since 1974, although contractors have been hired occasionally on a rental basis to assist in completing the programs when the work load has been too great for departmental crews.

The New Brunswick Department of Transportation has jurisdiction over a 10,600-mi highway network, 4,550 mi of which are chip sealed. (This term is used colloquially to describe two entirely different processes that will be defined later in this paper.) The following table gives a more detailed breakdown of the department's highway inventory.

Type	*Miles*
Dirt or gravel road	3,700
Asphaltic concrete highways	2,350
Chip-sealed highways	4,550
Total	10,600

Although cover aggregate gradations have varied, new asphalt binders have been developed, and equipment has become much more sophisticated, the basic principles of constructing surface treatments and chip seals have remained essentially the same, and so has their greatest adversary—the weather.

Because of climatic conditions, it has been found necessary to establish September 15 as the date when the placement of all seal coats in the province is to cease. Experience has shown that seals placed after this date run a high risk of failing. Even with the asphalt binders available today, chip seals and surface treatments cannot be placed with any degree of confidence after the established cutoff date.

Experience has proven that to construct economical and durable chip seals and surface treatments three factors must be considered:

- Materials,

New Brunswick Department of Transportation, P.O. Box 6000, Fredericton, New Brunswick E3B 5H1, Canada.

TABLE 1 COVER AGGREGATE SPECIFICATIONS

U.S. Standard Sieve Series	Percentage (by weight passing) of Cover Material				
	3/8 in.	1/2 in.	5/8 in. (A)	5/8 in. (B)	3/4 in.
3/4 in.	–	–	–	–	100
5/8 in.	–	–	100	100	–
1/2 in.	–	100	0–90	0–95	40–80
3/8 in.	100	40–90	0–60	0–80	20–62
No. 4	0–5	0–20	0–20	0–50	0–20
No. 8	–	0–8	0–8	0–45	0–10
No. 200	0–2	0–3	0–3	0–5	0–3

- Design, and
- Construction.

Each of these topics will be addressed individually.

MATERIALS

In New Brunswick a wide variety of cover aggregates from more than 100 different sources is used. Aggregates range from a basically single-sized 3/8-in. crushed quarried rock to a 3/4-in. crushed bank gravel that contains as much as 50 percent fines passing the No. 4 sieve; however, use of very sandy cover material is the exception rather than the rule and is limited to New Brunswick's Northeast where quality aggregate is nonexistent. Sources in that area contain very little crushable material so reducing the sand fraction of cover aggregates to 20 percent or less would mean wasting large quantities of sand and accelerating pit depletion.

By calling contracts for the supply of cover aggregates, the department maintains a province-wide inventory valued at $2 million. This inventory is supplemented by purchasing some material from approved commercial sources.

All cover aggregates must meet the department's specified grading limits (Table 1) and have a Los Angeles abrasion loss of less than 30 percent; however, consideration is being given to accepting aggregates with a higher abrasion loss for roads that have very low traffic volumes.

Members of a quality control field staff, equipped with mobile testing laboratories, are assigned to every crushing job. These inspectors exercise stringent control over aggregate producers to ensure that all materials fall within specification.

Various grades of emulsified and cutback asphalt binders are also used in New Brunswick. In general, emulsions are used for high-traffic-volume roads and cutbacks for roads with low traffic densities. One exception would be the use of high-float emulsions with sandy aggregates on roads with a traffic count of less than 500 vehicles per day (vpd).

TABLE 2 ASPHALT CONSUMPTION 1981 THROUGH 1984, INCLUSIVE

Year	Asphalt Usage (U.S. gal)		
	Cutback	Emulsion	Total
1984	2,932,781	3,149,782	6,082,563
1983	2,793,728	3,420,048	6,213,776
1982	2,813,293	4,788,209	7,601,502
1981	3,082,581	2,707,548	5,790,083

A typical construction season, lasting four months (May 15–September 15), would see 7 million U.S. gallons of asphalt binder sprayed. Approximately 50 percent of that total would be cutbacks although the department intends to use more emulsion products in the future because of increasing traffic volumes and the impact cutbacks have on the environment.

Table 2 gives the amount of asphalt binder used in New Brunswick's seal coat programs for the seasons of 1981–1984. Table 3 gives the various types of liquid asphalt binder most commonly employed in New Brunswick and their principal uses.

In 1985, 3.5 million U.S. gallons of PMC-3 were used. This was by far the largest quantity of any single type of binder used that year. For that reason and because this binder is peculiar to New Brunswick, its particular specifications are set out in Table 4.

Departmental specifications covering the manufacture and delivery of asphalt binders allow the placement of quality control technicians at the manufacturer's plant or refinery to monitor testing of materials slated for delivery to the chip seal crews. In addition, random field samples are taken and tested in the department's regional laboratories as a cross-check.

DESIGN

When chip seals are considered by the New Brunswick Department of Transportation for high-traffic-volume collector and arterial highways as an alternative to recapping with an asphaltic concrete seal mix, an aggregate average least dimension design is used to determine asphalt and cover aggregate application rates. This method was developed in New Zealand and introduced to New Brunswick in the early 1960s by Norman W. McLeod, a highly respected authority in asphalt circles around the world *(1)*.

Average least dimension design has been adopted by numerous highway boards and agencies in North America to help take the guesswork out of constructing chip seals and surface treatments. This method of design is also recognized by the Asphalt Institute as an acceptable practice *(2)*.

The department's central laboratory, located in Fredericton, the capital city, compiles information on the characteristics of cover aggregates and asphalt binders, such as

TABLE 3 ASPHALT TYPES AND USES

Type of Construction and Conditions	Type of Asphalt: Cutbacks N.B. Prime	Cutbacks PMC-3	Emulsions RS2K	Emulsions HF100s	Emulsions HF150s	Emulsions HF250s
Chip seal		X	X	X	X	X
Surface treatment		X			X	X
Prime	X					
Over open-graded aggregate	X					X
Over dense-graded aggregate		X			X	X
Over soil stabilization		X				
With sandy cover aggregate					X	X
With clean cover aggregate		X	X	X		
Low traffic volumes[a]		X			X	X
Moderate traffic volumes[b]		X		X	X	
High traffic volumes[c]			X			

[a]Fewer than 500 vpd.
[b]500 to 2,000 vpd.
[c]More than 2,000 vpd.

- Average least dimension,
- Aggregate bulk specific gravity,
- Loose unit weight, and
- Residual asphalt of binder.

Field crew technicians armed with this information plus the traffic count and surface condition of projects to be chip sealed can calculate quite accurate application rates of cover aggregate and asphalt binder.

A vast majority of chip seals placed in New Brunswick are on rural roads that have traffic volumes of less than 500 vpd. A formal design procedure for these roads has been found unnecessary; relying on experienced field technicians to establish application rates for binder and aggregate has produced excellent results.

CONSTRUCTION

Even though a great deal of consideration may be given to materials and design, acceptable chip seals and surface treatments can only be achieved by employing proven construction techniques, skilled workmen, and reliable equipment.

Before getting too deeply into construction practices, the terms used for the two basic types of seals placed on New Brunswick highways, surface treatments and chip seals, should be defined.

A surface treatment can be defined as the uniform application of an asphalt binder to a prepared gravel, crushed-stone, or soil-stabilized base followed by a uniform application of cover aggregate and compaction.

A chip seal is the uniform application of an asphalt binder to an existing paved surface followed by a uniform application of cover aggregate and compaction.

Typically, a road designated to be surface treated is first upgraded or constructed in accordance with departmental standards. In most cases a 6-in. layer of 1¼-in. crushed gravel or rock is placed as a top course. The top course is then shaped and compacted before the initial application of asphalt binder. The binder is applied at a predetermined rate (usually between 0.45 and 0.60 U.S. gallons per square yard) by a pressure asphalt distributor. Cover aggregate (usually ¾ in.) is then applied with a self-propelled aggregate spreader. The layer is then compacted. If rain is not threatening and the traffic volume

TABLE 4 SPECIFICATIONS FOR PMC-3 LIQUID ASPHALT, 1986

Characteristics	ASTM	Min	Max
Flash point (Cleveland open cup) (°F)	D 92-78	100	–
Viscosity (kinematic at 140°F)	D 3170-83	550	750
Distillation test	D 402-82		
Percentage of total distillate to 680°F			
At 374°F			30
At 437°F		25	50
At 500°F		40	68
At 600°F		70	
Distillation residue (% by volume)		78	
Penetration at 77°F	D 5-83	90	160
Ductility at 77°F	D 113-85	100	
Solubility in trichloroethylene	D 2042-81	99.5	
Water (%)	D 95-83		0.2
Pumping temperature (°F)		165	250
Spraying temperature (°F)		225	250

TABLE 5 COST DATA FOR 1985

Type of Work	Lane Miles Completed	Average Cost per Lane Mile ($ U.S.)	Average Cost per Square Yard ($ U.S.)
Reseal	804.4	4,780.00	0.68
Second seal	313.3	5,037.00	0.72
Surface treatment	444.2	10,495.00	1.49
Leveling ahead of reseals	804.4	3,110.00	0.44

is low, this type of surface is immediately opened to the motoring public.

In the case of multicoat surface treatments, which are usually the case in New Brunswick, best results are achieved when the first layer is given a week or more to cure and any imperfections are corrected before the next coat is applied.

A variation of this type of treatment is to spray the top course with N.B. primer, a penetration primer similar to R.C. 30, then compact with a steel-wheeled roller and allow a minimum of 72 hr curing time before applying a conventional surface treatment.

This type of application is not as common as it once was in the province for a number of reasons:

1. N.B. primer contains 50 percent naptha gas and in times of energy conservation fuel can be put to much better use.
2. Environmentalists frown on the naptha gas being allowed to evaporate off into the atmosphere.
3. Motorists will not tolerate the inconvenience of driving over such a surface while it cures when an alternate route is unavailable.

All surface treatments receive a chip seal the year after construction and this layer is termed "second seal" by local work crews. In essence, a surface treatment consists of three coats: a prime and seal or double seal the first year and a single application the second year.

A chip seal application is considered by the department to be a maintenance function and is intended to be applied to existing surface-treated roads every 5 years; however, budget restraints and rising costs have caused the resealing cycle to be extended to 10 or 11 years.

Before a chip seal is begun, it is most critical that the surface to be sealed is well patched and leveled with premix, for a chip seal application adds little or no strength and does not change surface contours. Chip seals do, however, extend pavement life, rejuvenate oxidized surfaces, seal cracks, and improve surface friction.

After the surface repairs are completed, the area to be sealed is thoroughly cleaned with a power broom to enhance the bond between the old surface and the new seal. The chosen asphalt binder is then applied to the surface at a predetermined rate. Cover aggregate is then placed with a self-propelled aggregate spreader. It has been observed that spreader hoppers equipped with aggregate placement screens improve embedment of graded aggregates. The mat is then compacted with a pneumatic-tired roller and a rubber-coated, steel drum, vibratory roller. On low-traffic-volume roads, the surface is opened to traffic immediately; however, on high-speed collector and arterial highways, traffic control is essential. It may even be necessary to convoy or reroute traffic for a few hours while these newly placed chip seals are in their most tender condition.

COST

In addition to the chip sealing and surface treatment of the highway network, New Brunswick transportation officials have found chip-sealed surfaces an economical alternative to hot-mix paving of parking lots, rural subdivisions, and shoulders on arterial highways. Table 5 gives the average cost incurred by the province for the various seal coat functions during the 1985 season.

The fastest escalating cost by far is that of preparing old surfaces for chip seal. This cost is directly attributable to the extended period a surface treatment is asked to perform without a reseal, from the recommended 5 years to the present 10 to 11 years.

OBSERVATIONS AND CONCLUSIONS

It has been observed that, when careful consideration is given to materials, design, and construction of chip seals and surface treatments, quality road surfaces will result 98 percent of the time in New Brunswick.

If chip-sealed surfaces, the subject of this paper, where not used in New Brunswick, undoubtedly many residents who now enjoy dust-free roadways would, for economic reasons, still live and drive on dirt and gravel roads.

One important aspect of chip seals and surface treatments must be understood: Seals in themselves have little or no strength and contribute very little to the bearing capacity of highways. This type of surface is only as good as the base on which it is placed.

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