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Asphalts, Aggregates, Mixes, and Stress-Absorbing Membranes

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Contents

Predictability Techniques for Asphalt Durability

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A procedure is needed for predicting the durability requirements of asphalts. Development of this procedure should result in specifications for improvement of asphalt quality. First, testing techniques must be identified that will be sensitive to changes in asphalt quality. Second, these changes must be related to in-service changes in pavements. Third, the quality specifications must be imposed to curb the inevitable time-dependent changes to the degree desired. This study identified testing techniques that are sensitive to changes in asphalt quality by comparing the results of tests of (a) chemical composition, (b) vanadium content, (c) weatherometer exposure, and (d) rolling thin film oven aging to the durability of environmentally aged specimens. Durability is considered to be measured by the combination of viscosity and asphaltene increases with time. Rankings of each of these testing techniques are compared to actual environmental rankings to illustrate the techniques that best identify the durability changes. The major finding of the study is the high degree of correlation between the vanadium content and environmental rankings. If compositional considerations are such that any asphalt imbalance or high volatility is a minimal factor, then the vanadium content is the best single parameter for predicting asphalt durability.

The weathering or durability of asphalt and resulting deterioration with time have resulted in distress of pavement structures and have necessitated the replacement of many surface treatments. The aging of asphalt can be caused by a number of factors such as oxidation, loss of oils, and structure changes. Considerable expenditure of funds annually is a direct result of the deterioration of asphalt. These funds are used for either the replacement or rejuvenation of existing surfaces.

The literature dealing with the mechanisms of asphalt durability spans a period of more than 60 years. Early works of Hubbard and Reeve (1), Speilman (2), and Streiter and Snoke (3) all indicated that oxidation was a prime cause of asphalt deterioration. The problem has been that little previous work has led directly to improvements through specifications for asphalt quality and the subsequent reduction in asphalt deterioration.

A testing technique that identifies or differentiates asphalt quality has not been developed. Once a sensitive technique is developed, then its relationship to in-service pavements and subsequent durability specifications can be developed. This study was initiated to identify a testing technique sensitive to asphalt changes in durability.

Several potential laboratory techniques for predicting asphalt durability are explored, including (a) rolling thin film oven, (b) weatherometer exposure, (c) chemical composition, and (4) vanadium analysis. These techniques are related to environmentally exposed specimens to produce predictable responses. The responses and combination possibilities for two or more procedures are presented.

BLENDING PROGRAM

For several years we have been investigating the possibility of improving the durability of asphalts. The asphalt blending phase of these investigations played such an important part in formulating the total program that it became an integral part of this paper.

There is little doubt that, in addition to construction problems (e.g., air voids), the most important factor in asphalt durability is the chemical composition. Although some construction problems can be eliminated by changes in construction practices, chemical composition is a factor that may be amenable to laboratory control. This was the basis for initiating the blending research program.

Phase A

Initial efforts in the blending program were confined to the large-scale separation and recombining of the various asphalt fractions. The standard method of asphalt analysis used in phase A was a modification of the Rostler-Sternberg procedure (5, 8). Because this method separates the various fractions by their reactivity, we felt that the procedure could be adopted for large-scale separations.

Phase A was soon found to be impractical from both operative and economic standpoints. The neutralization step in the separation procedure is extremely temperature sensitive and the reaction is exothermic. This imposes requirements that make the procedure uneconomical as a full-scale operation. The refinery processing

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equipment that is needed makes the cost prohibitive. After sufficient quantities of each of the fractions were produced to carry out this phase of the program, the method was abandoned.

However impractical phase A proved to be, it did yield some valuable data concerning the function of the various fractions. For example, the contributions of the asphaltene (A) and second acidaffins (A_2) fractions to asphalt performance were found to be the most important. Inasmuch as the primary function of asphaltenes is to act as a bodying agent, it follows that an increase in asphaltenes will result in a harder asphalt. However, if sufficient solubilizing oils such as second acidaffins are present, the hardening effect of increasing asphaltenes can be largely offset.

This offsetting effect was found early in the testing of phase A blends in the rolling thin film oven (RTFO). Minimal amounts of the A_2 fraction were found to exert a powerful influence on the blend viscosity because of their solubilizing function. However, the addition of A_2 in slight excess had effects beyond that of viscosity alteration. The A_2 fraction is one of the more volatile asphalt fractions. Because the criterion for evaluating RTFO performance is a viscosity ratio (aged viscosity/unaged viscosity) = aging index, some RTFO results can be misleading. With a film thickness of 1350 μm (1350 microns) at 163°C (325°F) and air blowing across the sample, the excess volatiles are lost. The result is a sharp increase in viscosity that is not entirely caused by the increase in asphaltenes. This can be especially misleading when the aging index (AI) based on the RTFO is used as the sole means for predicting long-term durability. When excess volatiles are present, the difference in their loss at 163°C (325°F) with air blowing versus long-term loss at pavement temperatures must be considered. As would be assumed, variations in the percentage of A_2 in the blend have a great effect on the chemical reactivity ratio (CRR). Because the asphaltene fraction is omitted from the CRR parameter, increasing the second acidaffin content causes a drop in the CRR value. Too great a drop is probably an indication of structural instability in the asphalt. This instability or structural alteration is the probable cause of the mediocre performance noted in some of the environmental samples. Further work is needed to determine the optimum A_2 addition that will maintain the structural integrity of the asphalt.

The effects of varying the percentage of asphaltene fraction are more apparent in the resulting viscosity than in other parameters. It was possible to use this property in the phase A blends as well as in the later work. By increasing the asphaltene content, it was possible to incorporate more second acidaffins into a blend and produce an asphalt with higher strength and greater durability. An example of the usefulness of this procedure can be seen in a blend made later in the program. An asphalt possessing excellent aging properties lacked sufficient viscosity to be of maximum usefulness in highway construction. The addition of 5 to 10 percent asphaltenes improved the viscosity without noticeably changing the viscosity ratio. It is probable that the primary change in asphaltenes that occurs in asphalt aging is one of quantity. Possibly there is a slight change in molecular weight as well, but this change, if any, is not significant. If this is true, then an asphalt with lower initial asphaltene content will probably have a longer useful life. This is of course true when the other factors involved in durability are equal. In the case of asphaltene addition, the effect on the CRR will be one of dilution inasmuch as it is not included in the equation.

In view of the confirmation of the importance of the asphaltene and second acidaffins fraction, the negative commercial aspects of phase A can be minimized. Fortunately both the asphaltene and second acidaffin fractions are available in commercial quantities and are economically attractive. Gilsonite is an excellent source of asphaltenes having a very high molecular weight compared to asphaltenes from petroleum. The numerous rubber extender oils, especially the aromatics, are high in second acidaffins. The availability of these fractions enabled the study to proceed to the next phase.

Phase B

Phase B involved the blending of selected asphalt components with asphalts from several sources. During this part of the study, deliberate imbalances were made by using excessive amounts of the individual fractions. These blends confirmed the functionality of the fractions as determined in phase A when asphalts from the several fractions were recombined. Because of their availability, extensive use was made of Gilsonite and extender oils as the source of asphaltene and second acidaffin fractions.

Phase C

The third and final phase of the blending program involved the blending of the two base asphalts, either alone or with one of the components (usually A or A_2) added. This procedure, because of its simplicity and economic advantages, was the method used in eventually moving the blending program from the laboratory to construction projects. The blends currently under field studies are composed of two asphalts, each of which contributes its desirable properties to the whole. Further information on the blending program is available (6).

SELECTION OF SAMPLES FOR ENVIRONMENTAL EXPOSURE

From the total blends prepared, samples were selected for an environmental exposure study. Several factors were considered in this selection process:

1. Availability of a particular asphalt to Arizona,
2. Feasibility of blended asphalt from a commercial viewpoint,
3. Effect of adding selected asphalt components to the base asphalt or asphalts,
4. Effect of blending two asphalts to secure the desired properties, and
5. Blended asphalts, even though deliberately imbalanced by adding extremely large quantities of one or more asphalt components.

All of these factors were weighed, and availability and feasibility were considered the most important. Assessing the commercial feasibility of a particular blend requires that the effect on the supplier be determined. If the added component requires additional refinery processing or equipment, it would probably be economically impractical.

Adhering to this selection process as closely as possible resulted in the selection of 53 samples.

COMPOSITION OF SELECTED SAMPLES

The asphalts selected were made from paving grade asphalts from three sources. Since this project continued over a considerable length of time, the supply of base asphalt changed somewhat in composition. Typical

Rostler-Sternberg (7) analyses of the various base asphalts and additives are given in Table 1.

ENVIRONMENTAL EXPOSURE

Specimens 102 mm (4 in) in diameter and 102 mm (4 in) in height were prepared according to Arizona Test Method 802B for natural environment exposure. The specimens were all prepared in a like manner by using the same aggregate, grading, and mixture parameters (voids in mineral aggregate, air voids, and so on). The specimens were then placed on 152.4 by 152.4 by 13-mm (6 by 6 by ½-in) plywood squares and placed on the laboratory roof. All the specimens were placed on a level surface in an area where they would receive uniform environmental exposure. Environmental exposure in Phoenix represents the most severe exposures to sunlight and oxidation possible within Arizona. The specimens were not protected except for the surface resting on the plywood squares. It was believed that the relationships developed during laboratory evaluation could subsequently be correlated with actual in-service pavement viscosity increases after a predictability technique was developed.

After 3 years of exposure, the specimens were returned to the laboratory for sample preparation, testing, and evaluation (Figure 1). Approximately one-third of each cylindrical specimen was used for test purposes in an effort to obtain a representative sample for evaluation. By cutting along a vertical axis, the section obtained would closely approximate the exposure to environmental conditions experienced in an actual roadway. This is an important consideration in climatological conditions found in Arizona.

The extraction procedure was performed in a Soxhlet extractor by using methylene chloride as the extraction solvent. The extraction was continued until the extract recycled with no trace of color. Usually five or six cycles were sufficient to reach this degree of extraction. The bulk of the methylene chloride was removed from the sample by warming on a hot plate. The last portion of solvent was then removed under a vacuum of 710 mm (28 in) of mercury while the sample was heated by two 250-W infrared heat lamps.

The viscosities of the recovered asphalts were obtained with a sliding plate microviscometer (8). Viscosities were determined at 25°C (77°F) and were calculated to a shear rate of 0.05 sec^{-1} by regression analysis. Calculations were based on viscometer runs of five different weight loadings. A 3-year viscosity ratio was calculated for each sample by dividing the viscosity of the recovered asphalt by the viscosity of the original asphalt before outdoor exposure. The viscosity ratio was then considered to be an indicator of the relative aging rate of the asphalts under study.

Comparison of viscosity ratios must take into consideration the actual viscosity. Although an asphalt of very low initial viscosity may show a high viscosity ratio, the aged viscosity may still be well within acceptable limits. However, this may not be true for an asphalt of high initial viscosity. As determined in the microviscometer at 25°C (77°F), the upper limit of viscosity measurement is probably about 100 MPa•s (10 megapoises). Beyond this point there is a possibility of film failure by cracking. Perhaps of greater importance in the evaluation of viscosity ratios is the difference in viscosity of the aged asphalt and this upper limit of viscosity measurement. It is probable that the greater the difference is, the more durable the asphalt will be, providing the viscosity is high enough for structural stability.

The 3-year viscosity ratios for the asphalts subjected to outdoor exposure for 3 years are given in Table 2. The relative ranking of these asphalts is also given in the table. Asphalts with the lowest ratios retained more of their original softness than those with the higher ratios. The latter are characterized by their brittleness as was observed during preparation of the microviscosity plates. Those asphalts with the lowest viscosity ratios were assigned the best ranking.

The increase in asphaltenes closely follows the increase in viscosity for most of the samples. The ranking of the samples by this parameter therefore fairly well agrees with the ranking by 3-year viscosity ratio. However, because of the wide range of initial asphaltene contents, a given change in asphaltenes had a more pronounced effect on an asphalt with low initial content. Disagreement in ranking occurs in those asphalts with very high or very low percentages of asphaltenes.

Determination of the relative ranking of the 53 samples aged in a natural environment took into consideration the combined ranking of both the 3-year viscosity ratio and asphaltene increase. The sum of the two ranking methods was used to determine an overall rank for the samples. This overall rank is given in Table 3 for the 53 samples.

Blend Designation	Viscosity Ratio Rank	Asphaltene Rank	Sum	New Rank
2C	1	1	2	1
22B	2	3	5	2
21B	4	2	6	3
15C	5	6	11	4

The development of asphaltenes and viscosity is used as a means to quantify durability.

FRACTIONAL ANALYSIS

The asphaltene content is one of several asphalt fractions determined by the Rostler-Sternberg method of analysis. A complete Rostler analysis was performed on all the asphalts examined in this study. Parameters derived from the Rostler analysis have been suggested as indicators for asphalt durability (9). The ratio of nitrogen bases to paraffins (N/P) was examined in the course of this study. There are some instances where the same ratio existed for several asphalts. In these cases, the same ranking was given to all the asphalts to denote equality. However, this parameter was found to correlate rather poorly with environmental aging. Regression analysis of the N/P data yielded a correlation coefficient of 0.38.

Rostler's CRR parameter was also examined and, as with N/P, some samples had the same value and were given equal ranking. The relationship of reactive to unreactive fractions (CRR) is

$$CRR = \frac{N + A_1}{A_2 + P}$$

where

N = nitrogen bases,
A_1 = first acidaffins,
A_2 = second acidaffins, and
P = paraffins.

Use of the CRR as the sole parameter for predicting asphalt durability is inadvisable. When asphalts are tested in the as-received condition or when the asphalt is only slightly modified, the CRR in itself has a reasonable value. However, its usefulness is very limited when it is applied to asphalts whose composition has been radically changed. This was shown quite graphically in the work carried out in phase A of the blending program.

Table 1. Chemical composition of blending fractions.

Material	A (%)	N (%)	A_1 (%)	A_2 (%)	P (%)
Base asphalt					
LA basin 40-50	19.0	37.1	8.0	24.4	11.5
LA basin 85-100	18.5	33.7	13.7	22.3	11.8
Ciniza 85-100	3.7	24.9	23.4	36.9	11.2
Idaho 120-150	20.3	21.4	8.5	38.3	11.4
Additive					
Gilsonite	75.2	20.6	1.0	2.3	0.9
Asphaltenes	95.3	2.5	1.5	0.5	0.2
Nitrogen bases	4.7	87.3	4.1	3.0	1.0
First acidaffins	8.6	17.4	53.6	12.3	8.1
Second acidaffins	0.2	0.3	8.8	77.0	13.7
Second acidaffins and paraffins	0.5	0.8	4.0	48.7	46.0
Paraffins	—	—	—	—	100.0
Reclamite resin	0.6	32.3	16.2	34.8	12.9
Dutrex resin	—	22.6	25.7	45.6	6.1
Antistrip agent	—	—	—	—	—

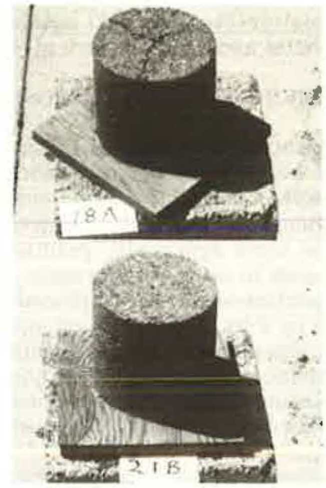


Figure 1. Environmental specimens after 3 years' exposure.

Table 2. Three-year viscosity ratio.

Blend Designation	Ratio[a]	Rank	Blend Designation	Ratio[a]	Rank
2C	7.6	1	10B	54.2	28
17C	9.1	2	19C	54.4	29
22B	9.1	2	18B-2	54.5	30
21B	9.3	4	19A	55.0	31
15C	9.4	5	15A	55.7	32
21A	10.0	6	22C	64.4	33
17D-1	11.3	7	23B	74.2	34
6C	11.3	7	9AX	77.6	35
16C	13.3	9	19B	78.8	36
16B	13.7	10	11SSY	82.9	37
16D	14.1	11	22E	83.2	38
7C	14.3	12	23A	90.0	39
16A	16.5	13	18C	93.0	40
8CY	18.0	14	19D	97.4	41
17D-2	20.2	15	14A	97.8	42
8CZ	20.8	16	6R	107	43
13SSB	23.6	17	24B	118	44
12SSA	26.0	18	10A	122	45
8CX	30.7	19	22F	124	46
11SS	35.6	20	18A	130	47
9A	37.6	21	5A	137	48
7C-1	39.0	22	24A	149	49
9C	40.8	23	18B-1	201	50
15D	46.3	24	2P	303	51
12SSB	46.7	25	18D	456	52
11SSN	48.9	26	3A	522	53
15B	51.5	27			

[a]Ratio = [microviscosity at 25°C (77°F) 0.05 s^{-1} (3 years)/microviscosity at 25°C (77°F) 0.05 s^{-1} (original)].

Table 3. Environment exposure ranking.

Blend Designation	Rank	Blend Designation	Rank
2C	1	19B	28
22B	2	18B-2	29
21B	3	5A	30
15C	4	22E	31
17C	5	6R	32
16B	6	11SSY	33
16C	7	10B	34
17D-1	8	23B	35
21A	9	15A	36
17D-2	10	7C-1	37
8CX	11	24B	38
7C	12	14A	39
16D	13	22C	40
8CY	14	19A	41
6C	15	23A	42
15D	16	2P	43
8CZ	17	19D	44
16A	18	9AX	45
11SS	19	24A	46
13SSB	20	3A	47
9A	21	18C	48
12SSA	22	18A	49
11SSN	23	10A	50
12SSB	24	22F	51
19C	25	18D	52
15B	26	18B-1	53
9C	27		

Figure 2. Rolling thin film oven test as an indicator of asphalt durability.

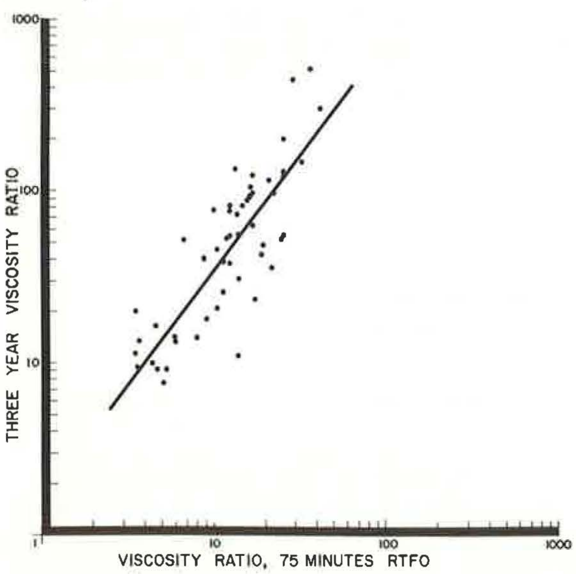
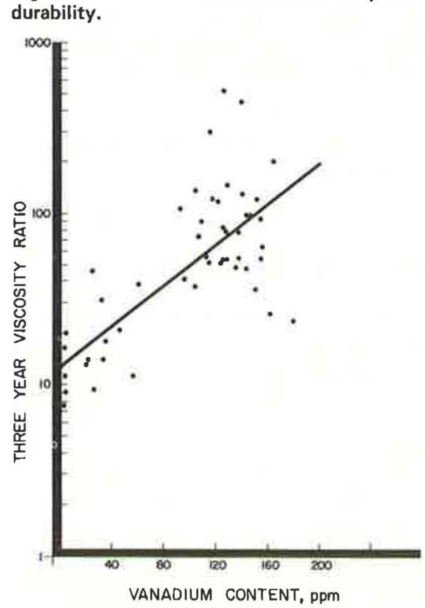


Figure 3. Vanadium as an indicator of asphalt durability.



All the asphalts in this part of the study were either recombined from various fractions or often deliberately imbalanced in composition. When the CRR of these phase A asphalts was added to the CRRs of the asphalts from phases B and C, the degree of correlation was radically altered. When only the straight or slightly modified asphalts from phases B and C were considered, the coefficient of correlation with the viscosity ratio was 0.60. When the CRR of the 16 asphalts from phase A were added and recalculated, the coefficient of correlation dropped to 0.28. The value of the CRR for predicting durability is most pronounced when used in combination with other parameters. This is discussed later.

ROLLING THIN FILM OVEN TEST

All the asphalts included for environmental exposure were tested by 75-min exposure in the rolling thin film oven in accordance with ASTM D 2872-70 (10). After the samples were removed from the oven, their microviscosities were determined.

A microviscosity ratio was determined by dividing the viscosity of the aged material from the RTFO by the viscosity of the original asphalt. The best correlation of the 75-min viscosity ratio with the 3-year viscosity ratio was found to occur as a log-log function (Figure 2). The regression analysis of the data yielded a correlation coefficient of 0.84.

Other parameters are also at work that prevent RTFO results from possibly more closely predicting the durability of an asphalt. A major drawback of the RTFO test is that it is conducted in darkness. Light is known to be an active weathering agent, especially in climates where there is a great deal of sunshine. This factor is of major importance where surface treatments are involved, such as in seal coats. Although the effects of ultraviolet radiation are confined to the surface, they cannot be disregarded. The weathering by rain and wind combined with abrasion from tires helps keep a new surface available for photooxidation. Tests such as weatherometer exposure, which directly or indirectly measure the effect of light, are useful in describing the aging characteristics of asphalt in the natural environment.

VANADIUM ANALYSIS

Previous work (11, 12) has suggested that the presence of vanadium (V) in asphalt affects the rate at which the asphalt ages. For the most part this work has been confined to the effects of vanadium on the rate of oxidation by actinic light. Weatherometer exposure has confirmed the catalytic action of vanadium in the photooxidation process. However, the effects of vanadium are not confined to this type of oxidation. Results of RTFO testing show a strong correlation with vanadium content. It is quite possible that this is the first time the role of vanadium in thermal oxidation of asphalts has been explored to this extent. Further work is being carried out to determine the distribution of vanadium in the various asphalt fractions. At this time it appears that the vanadium is almost entirely contained in the asphaltene and nitrogen base fractions, perhaps as much as 98 percent of the total.

The vanadium analysis was run on the environmentally exposed asphalts after their recovery from the cylindrical specimens by Soxhlet extraction. It was assumed that the vanadium content of the asphalt did not change during the 3-year exposure. The vanadium content was determined by atomic absorption analysis. Because of the small amount of recovered asphalt available, the average sample size was 7 g (0.24 oz). The instrument used for the analysis was a Perkin-Elmer model 306 atomic absorption spectrophotometer. The vanadium concentrations were typical of the asphalts normally used in Arizona.

Figure 3 shows the correlation between vanadium content and the 3-year viscosity ratio. The coefficient of correlation was 0.75. A similar effect of vanadium on the viscosity ratio of asphalt after 75 min in the RTFO has been noted. For these data, the coefficient of correlation was 0.81.

A good correlation exists between vanadium and the various aging tests used in this study. This suggests that vanadium content may have a significant influence on asphalt aging characteristics. The concentration of vanadium in asphalt is not a widely known parameter for estimating asphalt durability. The catalytic effect of vanadium and other metals may prove to be important in the choice of materials and design parameters for future specifications.

WEATHEROMETER ANALYSIS

Ten asphalts were selected as representative of the samples subjected to outdoor exposure. These asphalts were artificially aged in an Atlas 6000-W xenon arc weatherometer for periods of 25, 50, and 100 hours. The weatherometer was set to simulate sunlight only, and the wattage applied to the xenon lamp was adjusted to maintain a constant light intensity during the testing. The other variables were adjusted to maintain a specimen temperature of 38°C (100°F), a black panel temperature of 53°C (128°F), and a relative humidity of 50 percent.

Because the weatherometer was not acquired until well into this project, samples of the asphalt placed for environmental exposure were not available for testing. New samples were prepared to the same formulations as the specimens previously prepared for environmental exposure.

The sample mold was formed by attaching a 157-mm-diameter (6.25-in) by 6-mm (0.25-in) ring to a flat glass plate with epoxy cement. Special holders were prepared to hold the sample mold on the revolving specimen rack in the weatherometer. The sample molds could then be placed in the weatherometer in a vertical position facing the 6000-W xenon arc.

Asphalt films were formed in the molds by dissolving 1.000 ± 0.001 g (28 ± 0.003 oz) of asphalt in approximately 40 cm^3 (2.4 in^3) of reagent-grade benzene and pouring the solution into the sample mold. The evaporation of the benzene left a translucent asphalt film of approximately 50-μm thickness. If the film was not evenly cast in the mold, additional benzene was added to the mold to redissolve and recast the film (Figure 4).

After exposure in the weatherometer, the asphalt film was removed by scraping the mold with a square-ended spatula. Two scrapings, approximately 12 mm (0.5 in) wide, were made at 90 deg apart across the center of the mold to secure a representative sample of the exposed film for microviscosity determination. The remainder of the film was removed and analyzed by a modified Rostler analysis.

Figure 4 shows three of the specimens from sample 2C (number 2 in the rankings) and three from sample 10A (number 50 in the rankings). Exposure times are 25, 50, and 100 hours. The severity of the channeling is usually an indication of the aging taking place in the asphalt film. Photographs were made by transmitted light.

Microviscosities of the aged asphalts were determined and related to the microviscosity of the original asphalt by calculating a viscosity ratio (aged microviscosity/original microviscosity). The viscosity ratio after 100 hours' exposure was found to correlate best with the 3-year viscosity ratio. This relationship is shown in

Figure 4. Various exposures in weatherometer.

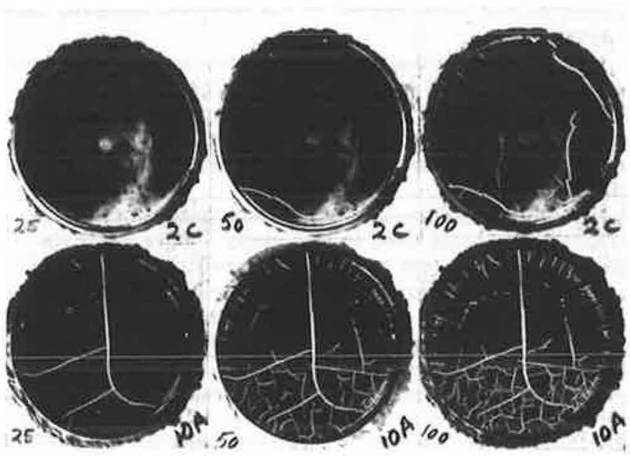


Figure 5. Weatherometer as an indicator of asphalt durability.

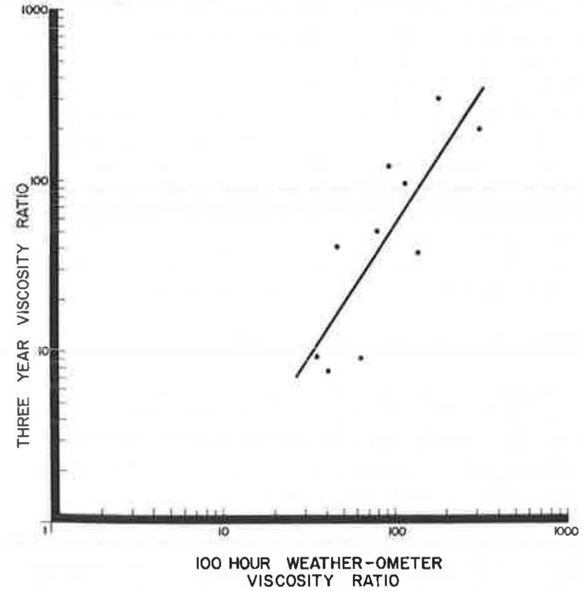


Table 4. Ranking and composition of blends.

Environment Exposure Rank	Sample	Composition	RTFO, 75 Min	Original N/P	Vanadium	Original CRR	Average Rank
1	2C	Cin 85-100	9	6	3	9	2
2	22B	Cin 85-100 + 5% A(G)	7	12	6	16	10
3	21B	Cin 85-100 + 5% A(G)	7	12	3	16	6
4	15C	Cin 85-100 + 5% A	3	7	13	9	3
5	17C	Cin 85-100 + 6.9% A(G)	10	18	3	16	13
6	16B	Cin 85-100 + 5% A(G) + 1% AS	4	12	1	16	4
7	16C	Cin 85-100 + 5% A	12	7	10	9	6
8	17D-1	Cin 85-100 + 6.9% A(G) + 1% AS	1	18	6	16	10
9	21A	Cin 85-100 + 5% A(G) + 1% AS	5	12	1	16	5
10	17D-2	Cin 85-100 + 6.9% A(G) + 1% AS	1	2	6	16	1
11	8CX	Cin 85-100 + 100% LA 40-50	30	25	14	27	21
12	7C	Cin 85-100 + 7% A	14	7	15	14	14
13	16D	Cin 85-100 + 5% A + 1% AS	11	7	11	9	6
14	8CY	Cin 85-100 + 33% LA 40-50	16	21	16	24	16
15	6C	Cin 85-100 + 12.5% A	31	26	18	48	30
16	15D	Cin 85-100 + 5% A + 1% AS	18	7	12	9	12
17	8CZ	Cin 85-100 + 50% LA 40-50	18	24	17	26	17
18	16A	Cin 85-100 + 5% A(G)	6	12	6	16	9
19	11SS	Idaho 120-150	43	3	45	1	19
20	13SSB	Idaho 120-150 + 6.7% A	39	4	53	2	22
21	9A	LA 40-50	24	45	22	42	37
22	12SSA	Idaho 120-150 + 6.4% A + 15% N	20	44	51	5	28
23	11SSN	Idaho 120-150 + 15% N	41	40	38	6	32
24	12SSB	Idaho 120-150 + 3.3% A + 15.5% N	40	41	43	4	33
25	19C	LA 85-100 + 1% AS	22	29	32	50	37
26	15B	LA 40-50 + 33% Cin 85-100 + 1% AS	13	37	27	33	26
27	9C	LA 40-50 + 50% Cin 85-100	15	32	21	38	23
28	19B	LA 85-100	24	29	35	50	41
29	18B-2	LA 40-50 + 10% D	46	20	39	28	37
30	5A	LA 85-100 + 20% A_2	27	17	23	7	15
31	22E	LA 85-100	24	29	32	50	40
32	6R	LA 85-100 + 30% R	35	26	20	48	35
33	11SSY	Idaho 120-150 + 23% N	32	53	31	14	36
34	10B	LA 40-50 + 6.2% A_2	45	33	36	38	42
35	23B	LA 40-50 + 43% Cin 85-100	28	34	24	28	27
36	15A	LA 40-50 + 33% Cin 85-100	28	37	26	33	31
37	7C-1	LA 85-100 + 100% Cin 85-100	21	23	19	28	18
38	24B	LA 40-50 + 12% A(G) + 23% R	42	51	30	40	45
39	14A	LA 85-100 + 6.5% A	44	28	44	50	46
40	22C	LA 40-50	36	45	50	42	52
41	19A	LA 40-50 + 1% AS	23	45	48	42	43
42	23A	LA 40-50 + 43% Cin 85-100 + 1% AS	34	34	25	28	29
43	2P	LA 85-100 + 10% P	53	1	28	25	24
44	19D	LA 40-50	36	45	47	42	51
45	9AX	LA 40-50 + 33% Cin 85-100	17	39	39	33	34
46	24A	LA 40-50 + 20% R	51	43	36	36	47
47	3A	LA 85-100 + 6.5% A + 35% A_2	52	5	34	3	20
48	18C	LA 40-50 + 1% AS	33	45	48	42	48
49	18A	LA 40-50 + 10% R	46	51	42	40	53
50	10A	LA 40-50 + 9.6% (A_2+P)	49	22	29	8	25
51	22F	LA 40-50 + 1% AS	36	45	46	42	49
52	18D	LA 40-50 + 10% R + 10% D	50	42	41	36	50
53	18B-1	LA 40-50 + 10% D	46	36	52	28	44

Figure 5. The 10 asphalts represented are not sufficient for a statistically satisfactory curve; however, a definite correlation can be seen.

Weatherometer testing of asphalts appears to provide an indication of how an asphalt will behave after exposure to the outdoor environment. The close matching of sunlight and temperature between the two environments could possibly provide a better correlation with outdoor exposure than is currently possible if sufficient data could be accumulated to determine correlation constants.

RANKING SUMMARY AND EQUATIONS

A summary of the ranking methods is given in Table 4. The asphalt blends are ranked in order of their overall 3-year outdoor exposure ranking. The other ranking methods discussed are given in this table for direct comparison of the various ranking parameters. The symbols used in this table are as follows:

Cin = Ciniza base asphalt (Four Corners),
LA = Los Angeles basin asphalt,
Idaho = Idaho base asphalt (American oil),
A(G) = asphaltenes from Gilsonite,
R = reclamite resin,
D = Dutrex resin, and
AS = antistripping agent.

Comparison of the various ranking methods shows that many times the rankings given an asphalt disagree. But it is interesting that, although the individual rankings may vary, the average of the rankings is quite close to the 3-year overall rank. Individual ranking methods may have inconsistencies inherent in them, or the asphalt properties may be too complex for such a simple ranking method, but the average of the individual methods seems quite consistent.

Table 5 gives the equations for the curves of the various parameters presented. The curves were prepared by calculating various forms of the available data by linear regression analysis to determine which form of the data yielded the best coefficient of correlation (linear, semilog, or log-log). The regression line was then calculated, and the data were plotted. Because of the nature of these parameters and the complex characteristics of asphalts, a great deal of precision cannot be expected from these equations. However, they may be helpful as guides when asphalts are compared to determine their approximate aging rates.

Just as the average of several ranking methods will often agree with the 3-year overall ranking more closely than any of the individual rankings, so also the final equation may help bring the calculated viscosity ratio into closer agreement with the 3-year viscosity ratio. This equation was derived by multiple linear regression analysis, and, when applied to the asphalts from phases B and C, it appears quite good. The equation involves two parameters, both of which are derived from chemical analysis. While the correlation lessens when phase A samples are included, the same two parameters are involved. The coefficient of correlation for vanadium content remains the best single parameter.

These relationships are derived from data pertaining to the asphalt sources used in this study. They may or may not apply to other asphalts from other sources.

CONCLUSIONS

1. The blending of asphalt or the addition of selected components can affect durability, as indicated by comparison of Tables 3 and 4.
2. Because the average rankings of the four parameters considered (Table 4) agree more favorably with the environmental exposure rankings than any single parameter, a combination of parameters should be used to predict asphalt durability.
3. The combination of parameters that produced the best durability prediction was vanadium content and chemical reactivity ratio (CRR). This relationship is expressed as

Log viscosity ratio = 0.48 + 0.0049 + 0.56 CRR

4. If compositional considerations are such that any asphalt imbalance or high volatility is a minimal factor, the vanadium content offers the best single parameter for predicting asphalt durability.

This study attempted to produce a laboratory technique for predicting durability or viscosity increases. It indicated that means are available for reasonably predicting these increases. The necessary work ahead requires the development of limiting values that are desired. The prediction of asphalt durability for 3 years of exposure or any other time element must be equated to actual in-service values and future design for asphalt quality based on a time-related acceptance value.

Although limited data were available regarding the weatherometer studies in this report, we are encouraged that it may also offer a satisfactory testing technique for durability prediction. Responses from the weatherometer closely parallel responses from the other techniques, but additional studies are warranted.

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Table 5. Statistical analysis of data.

Phase	Parameter	Equation	R-Value
B and C[a]	Vanadium	Log viscosity ratio = 0.0053 (V) + 1.07	0.85
	RTFO	Log viscosity ratio = 1.25 (log 75 min) + 0.26	0.78
	N/P	Log viscosity ratio = 0.71 (N/P) - 0.38	0.75
	CRR	Log viscosity ratio = 1.43 (CRR) - 0.13	0.60
	WO samples (7)	Log viscosity ratio = 0.0075 (WO) + 0.83	0.67
A, B, and C[b]	Vanadium	Log viscosity ratio = 0.00593 (V) + 1.106	0.75
	RTFO	Log viscosity ratio = 1.338 (log 75 min) + 0.207	0.84
	N/P	Log viscosity ratio = 0.372 (N/P) + 0.657	0.38
	CRR	Log viscosity ratio = 0.804 (CRR) + 0.760	0.28
	WO samples (10)	Log viscosity ratio = 1.549 (log WO) - 1.349	0.82

Note: The best relationship was log viscosity ratio = 0.48 + 0.0049 (V) + 0.56 (CRR) for R^2 = 0.87 and F = 98.5.
[a]37 samples. [b]53 samples.

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Evaluation of Viscosity-Graded Asphalt Cements in Utah

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The methods commonly used to grade asphalts (penetration, viscosity, and ductility) were investigated to determine their effectiveness as indicators of performance. Although these parameters were found to be of value in establishing asphalt quality and in controlling mix and lay-down properties, new methods are needed to better predict asphalt performance. A new procedure called force ductility has been correlated to performance. This modification in the standard ductility test measures the force in the asphalt sample versus elongation and is based on the theory that an asphalt must be able to relax as strain from traffic loading and temperature shrinkage is applied to a pavement. Asphalts with lower forces performed best in the field. Other tests found to be indicative of performance were temperature susceptibility, shear susceptibility, ductility versus temperature, and chemical fractionation.

Asphalts exhibit complex flow properties, and different asphalts perform differently. Asphalts from different sources have different shear and temperature susceptibilities at temperatures below 60°C (140°F), which can and do affect performance. Low-temperature rheology and its relationship to performance are discussed extensively in a number of reports (1-8). The penetration test is run at 25°C (77°F) but fails to reflect consistency at other temperatures or at more than one rate of shear. It therefore cannot detect shear or temperature susceptibility. Fromm and Phang (4) reported that the amount of paraffin wax in the asphalt has a marked effect on the penetration test results. Research has shown that field performance, as evidenced by the amount of low-temperature cracking, is associated with certain asphalt sources (4, 6, 11).

Pavement test sections were investigated to determine the effectiveness of grading asphalts by viscosity. The state of Utah used modified research specifications in experimental sections that were constructed in 1968. This report is based on the evaluation of those experimental sections. Additional studies done on 108 pavements and 35 asphalts support the general relationships in this paper.

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EVALUATION OF SURFACE DEFECTS

The results of the pavement rating effort are given in Table 1. Cracking was very extensive in sections 2 and 3, the Salt Lake City asphalt sections, and appeared to a much lesser degree in the Casper asphalt, sections 1 and 4. A chip seal was applied to the sections after the 57-month evaluation. The seal seemed to be more beneficial to section 1 than the others and reduced the cracking in that section.

The overall rating of the sections from best to poorest was as follows:

1. Section 1, Casper AC-6 (150 to 200 penetration);
2. Section 4, Casper AC-12 (85 to 100 penetration);
3. Section 3, Salt Lake City AC-12 (60 to 70 penetration); and
4. Section 2, Salt Lake City AC-6 (85 to 100 penetration).

Based on this ranking, one can conclude that the physical performance of the sections was affected more by the source of the crude oil than by the method by which it was graded.

FIELD AND LABORATORY RHEOLOGY

The difference in asphalt properties and the changes in these asphalt properties due to aging and temperature variations are of major concern in properly grading asphalts. Comparing the performance of the four sections with regard to these properties and changes in properties provides a better understanding of the causes of pavement distress and what constitutes good performance of an asphalt.

To supplement the field-aging analysis of the asphalt properties, samples taken from the storage tanks before mixing were subjected to the rolling thin film circulating (RTFC) oven, and laboratory tests were performed. Although application of the oven does not accurately simulate the field-aging of an asphalt, it is considered to be similar in effect to what happens during mixing in the pug mill.

Percentage of loss in weight after 75 min in the RTFC oven was measured, and a much greater loss of volatiles occurred in the Salt Lake City asphalts than in the Casper source (Table 2). This loss of volatiles is most likely a cause of the more rapid hardening observed in the changes in asphalt properties (viscosity, ductility, and so on) for the Salt Lake City asphalts. To gain a better understanding of this hardening effect, samples of the four asphalts were aged in the RTFC oven for various times: 75, 100 and 125 min (plus 175 and 225 min for section 1 because of the slower aging observed). Tests were performed on these aged samples, and the results are given in Table 2.

A general reduction in penetration and cycling effect is evident in the plot of penetration versus time in service (Figure 1). Figure 1 shows asphalt hardening with time, and the cyclic trend is most likely due to seasonal changes, indicating an environmental effect on penetration; the relationship is similar to that shown in Figure 1.

Penetration values for the field and laboratory samples proved to be poor as performance indicators; the best performer, section 1, was highest in penetration, but the second and third performers, sections 4 and 3, were lower in penetration than the worst performing asphalt, section 2.

Absolute Viscosity at 60°C (140°F) and Kinematic Viscosity at 135°C (275°F)

Throughout the 4 years of testing an increase in both the absolute viscosity at 60°C (140°F) and the kinematic viscosity at 135°C (275°F) was observed in the field samples. The aging index (AI), which is the ratio of the viscosity after aging and the viscosity before aging, was used as an indication of aging rate. The aging index shows that the Salt Lake source increased at a faster rate than did the Casper source asphalts (Figure 3). Plots of the viscosity readings versus time in service for the two temperatures are very similar in shape, and the seasonal effect is present but more noticeable at 135°C (275°F) than at 60°C (140°F). There were general increases at 3, 12, and 24 months and decreases at 6 and 18 months.

Samples taken from the storage tanks show a higher average viscosity for the Salt Lake City asphalts than for the Casper asphalts for both AC-6 and AC-12 grades and at both of the test temperatures (Table 2). This again indicates a softer asphalt from the Casper source.

Ductility

The ductility readings at 4°C (39.2°F) remained reasonably constant, except for seasonal variation, throughout the testing in sections 2, 3, and 4. Section 1, however, the best performing section, demonstrated a definite increase during the first year of service and then abruptly dropped to nearly the same level as the other three sections (Figure 4). This is inconsistent with theory because ductility should generally decrease with time in service except for slight fluctuations due to seasonal effects. The raw data show a wide variation in test results in section 1 for the first year, which most likely caused the average of the tests to differ from theory. The general comparison of magnitudes, however, shows a correlation between ductility and cracking, and the second best performing asphalt, section 4, demonstrated the second highest ductility. The Salt Lake City asphalts in sections two and three were lowest in ductility.

A dramatic difference is noted between the Salt Lake City and Casper asphalts in the plot of ductility versus time in the thin film oven (Figure 5). The ductilities of the Salt Lake City asphalts in sections 2 and 3 were less than 10 initially, and they steadily decreased with time in the thin film oven. The Casper asphalts in sections 1 and 4 were measured in the 100+ range initially and decreased at a decreasing rate with hardening. This difference in the ductility change with aging for the two asphalt origins seems to be indicative of the excessive cracking in sections 2 and 3, the less ductile sections.

Force Ductility

A modification in the standard ductility test greatly improves the usefulness of this procedure. A linear variable displacement transducer (LVDT) was mounted in a proving ring and placed on the drive bar of the ductility apparatus (Figure 6). This setup, when linked to a strip chart recorder, measures the tension in the sample at any time as it is elongated. This force ductility test is based on the theory that an asphalt must be able to relax as strain from the traffic loading and temperature shrinkage is applied to the pavement, but it must possess enough tenacity to maintain a proper matrix.

Shortcomings of the standard ductility test have stimulated much criticism of its use. These problems are nonrepeatability, variance in the cross-sectional area of the sample, imprecise data such as 100+ rather than a definite number, and the breaking of samples at the shoulder rather than at the sample center. For the most part these problems are not critical in the force ductility test. Because the data points are measured in the first 10 or 15 cm (4 or 6 in) of the test, the differences in cross-sectional area from one sample to another are insignificant. For this reason the repeatability of the test is much better. Indefinite data such as 100+ are eliminated, and few shoulder breakages seem to occur, most likely because of the slight give in the proving ring.

The results of the force ductility test are shown in Figure 7 for the original, residue, and 66-month samples. The poorly performing Salt Lake City asphalts reach a high maximum tension when stretched and have a steeper recovery slope. The Casper asphalts reach a much lower maximum tension and have a smaller recovery slope. This relationship is consistent for both original and residue samples. The maximum readings for the residue are higher, however, and the hardening effect increases the force necessary to pull the sample. The Salt Lake City asphalts increased in maximum tension more with aging in the RTFC oven than did the Casper asphalts, indicating a more rapid hardening effect for the Salt Lake City asphalts. For the field samples extracted from the sections after 66 months, however, the asphalt in section 4 reached a maximum tension equal to that in section 2 but peaked at a greater elongation. Sections 1 and 3 remained lowest and highest in maximum tension and recovery slope respectively.

In general the force ductility test measures the consistency of the asphalts and changes in consistency with aging. Perhaps an optimum area between the very viscous and the very brittle exists in which optimum performance occurs in a given climate. Based on extensive study, the force ductility test seems to be an indicator of temperature-associated cracking.

Cannon Cone Viscosity at 25°C (77°F) and 0.05 s^{-1}

All sections demonstrate a definite seasonal variation in the Cannon cone viscosity test and an overall increase in viscosity with time in service for the field samples. The Salt Lake City asphalts were higher within each

Table 1. Performance data.

Sample	Section	Transverse Cracking (m/100 m)	Longitudinal Cracking (m/100 m)	Load Cracking (m^2/100 m)	Opening Rating	ABR-ERO Rating	Multiple Rating	Ruts (IWP)	Ruts (OWP)
57-month	1	17	31	0	3.0	4.0	4.0	0.81	0.33
	2	94	22	6.1	3.0	3.5	3.0	0.61	0.36
	3	60	0	0	2.0	3.0	4.5	0.69	0.56
	4	14	8	0	3.0	4.0	4.0	0.58	0.38
66-month	1	15	20	–	–	–	–	0.81	0.28
	2	133	82	–	–	–	–	0.66	0.38
	3	107	25	–	–	–	–	0.41	0.38
	4	26	12	–	–	–	–	0.53	0.36

Note: 1 m = 3.28 ft.

Figure 1. Penetration versus field aging.

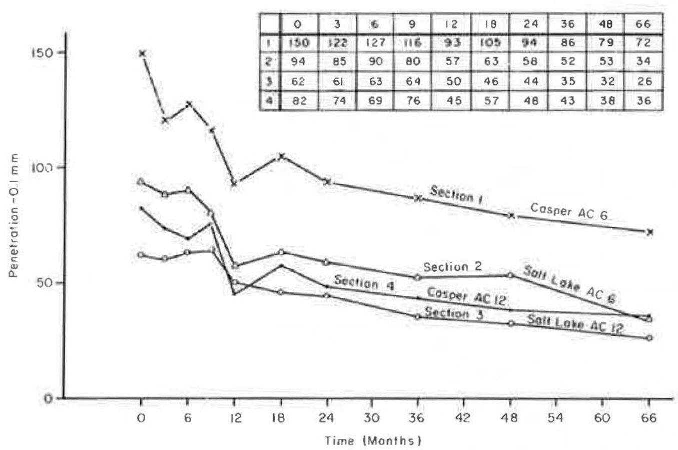


Figure 2. Penetration versus thin film oven time.

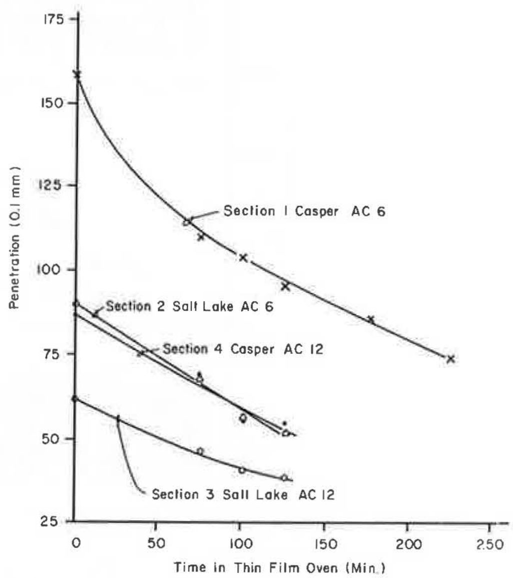


Table 2. Test results of storage tank samples of asphalt cements.

Characteristic	Time in Oven (min)	Section			
		1	2	3	4
Penetration at 25°C, mm/0.1 mm	0	159	90	62	87
	75	111	63	46	68
	100	103	56	45	55
	125	96	52	38	54
	175	86	–	–	–
	225	74	–	–	–
Ductility at 4°C, cm	0	100+	9.25	7.0	100+
	75	100+	5.0	4.5	32.0
	100	64	4.0	3.0	9.5
	125	42	3.5	2.5	7.5
	175	20	–	–	–
	225	11	–	–	–
Ductility at 25°C, cm	0	150+	150+	150+	150+
Cannon cone viscosity at 25°C and 0.05 s^{-1}, kPa·s	0	29.9	115.0	345.0	101.2
	75	64.4	276.0	552.0	184.0
	100	73.6	345.0	667.0	253.0
	125	92.0	464.0	1012.0	322.0
	175	110.4	–	–	–
	225	184.0	–	–	–
Cannon cone viscosity at 16°C and 0.05 s^{-1}, kPa·s	0	299.0	1380.0	2162.0	920.0
Absolute viscosity at 60°C, Pa·s	0	66.0	72.0	134.8	137.5
	75	110.0	127.9	233.8	225.5
Kinematic viscosity at 99°C, cm^2/s	0	14.38	15.15	20.90	–
Kinematic viscosity at 135°C, cm^2/s	0	2.08	2.12	2.71	2.91
	75	2.58	2.60	3.27	3.44
Flash point, °C	0	450	500+	500+	500+
Percentage loss in oven	75	0.085	0.223	0.176	0.009
Specific gravity	0	1.024	1.013	1.018	1.029
Solubility (CCl_4)	0	99.89	99.86	99.88	99.95

Note: 1 mm = 0.039 in, 1 cm = 0.4 in, 1 Pa·s = 10 poises, 1 cm^2/s = 100 centistokes, and 1°C = (1°F - 32)/1.8.

grade, in both initial values of viscosity and rate of increase with time, indicating a faster hardening effect in the Salt Lake City sections (Figure 8).

Cannon cone viscosity versus time in the thin film oven is shown in Figure 9; no significant relationship with respect to aging index is observed. Cracking correlated better with the laboratory-aged than with field-aged samples; the best performing asphalts generally were lower in Cannon cone viscosity.

Cannon Cone Viscosity Versus Shear Rate

The standard Cannon cone test is performed at a shear rate of 0.05 s^{-1}. The literature revealed that, by varying this shear rate, a relationship with viscosity value is obtained that can be related to performance (12). The Cannon cone test was performed at shear rates of 0.01, 0.05, 0.10, 0.50, and 1.00 s^{-1} on original samples as well as samples aged 75, 100, and 125 min in the RTFC oven. Viscosity was plotted versus shear rate on a semilog grid, and straight-line relationships were obtained as shown in Figure 10 for the four asphalts at the various oven aging times. A least square curve fit was performed on the data, and a multiple R of more than 0.86 was generated for each curve. The slope of each line was calculated and used in the correlation matrix, which substantiated that steeper slopes on the plot generally correspond with poor performance.

The correlation between the slopes of the Cannon cone viscosity-shear rate relationships and the force ductility readings is highly significant. This points to the similar mechanism of the tests, the Cannon cone measuring shear stress and the force ductility measuring tensile stress.

Temperature Susceptibility

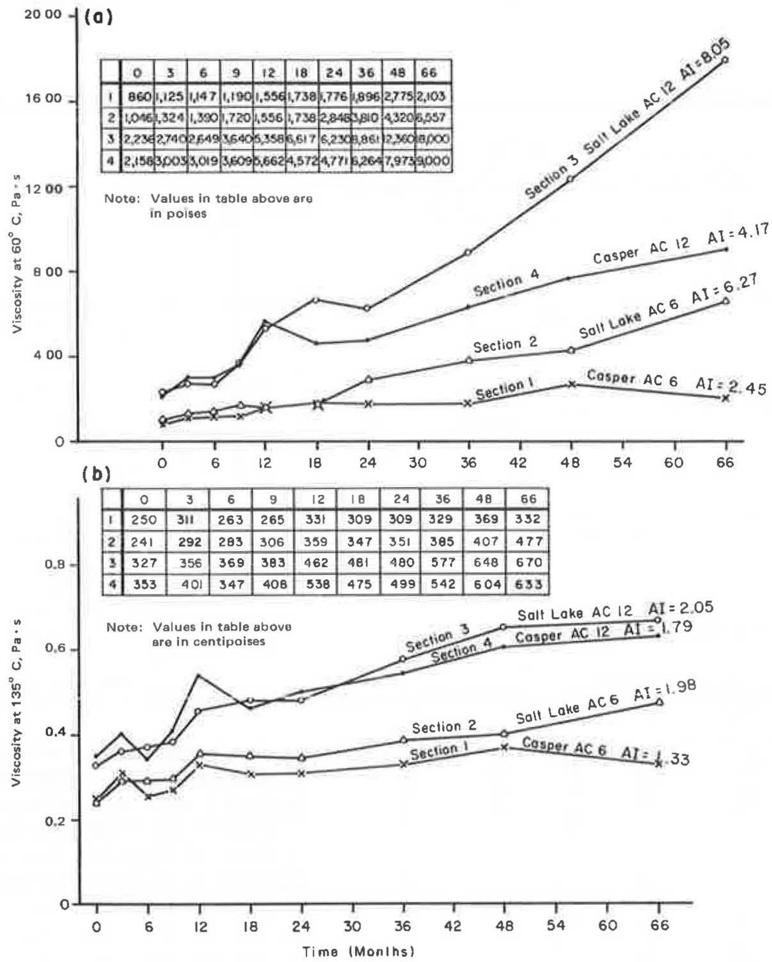
Temperature obviously has a substantial effect on the rheology of an asphalt. The extent of this effect is called temperature susceptibility (13) and is measured in this study by

$$\text{Temperature susceptibility} = \frac{\log\log \eta_2 - \log\log \eta_1}{\log T_2 - \log T_1}$$

where η_1 and η_2 (in square meters per second) are the viscosities at temperatures T_1 and T_2 (in absolute temperature).

Temperature susceptibility was calculated for the four

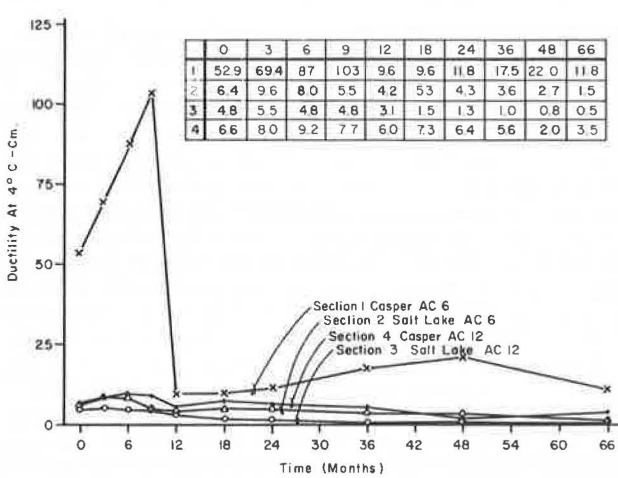
Figure 3. Viscosity at (a) 60°C (140°F) and (b) 135°C (275°F) versus time.



	0	3	6	9	12	18	24	36	48	66
1	860	1,125	1,147	1,190	1,556	1,738	1,776	1,896	2,775	2,103
2	1,046	1,324	1,390	1,720	1,556	1,738	2,848	3,810	4,320	6,557
3	2,236	2,740	2,649	3,640	5,358	6,617	6,230	8,861	12,360	18,000
4	2,158	3,003	3,019	3,609	5,662	4,572	4,771	6,264	7,973	9,000

	0	3	6	9	12	18	24	36	48	66
1	250	311	263	265	331	309	309	329	369	332
2	241	292	283	306	359	347	351	385	407	477
3	327	356	369	383	462	481	480	577	648	670
4	353	401	347	408	538	475	499	542	604	633

Figure 4. Ductility at 4°C (39.2°F) versus field aging



	0	3	6	9	12	18	24	36	48	66
1	52.9	69.4	87	103	9.6	9.6	11.8	17.5	22.0	11.8
2	6.4	9.6	8.0	5.5	4.2	5.3	4.3	3.6	2.7	1.5
3	4.8	5.5	4.8	4.8	3.1	1.5	1.3	1.0	0.8	0.5
4	6.6	8.0	9.2	7.7	6.0	7.3	6.4	5.6	2.0	3.5

Figure 5. Ductility versus thin film oven time.

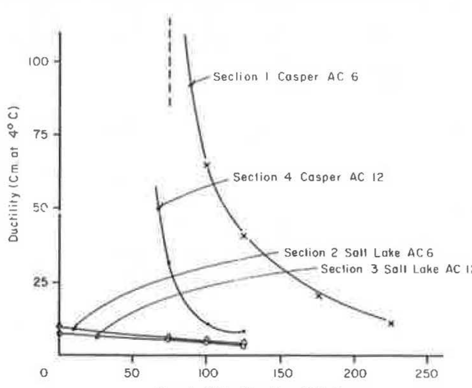


Figure 6. Force ductility apparatus.

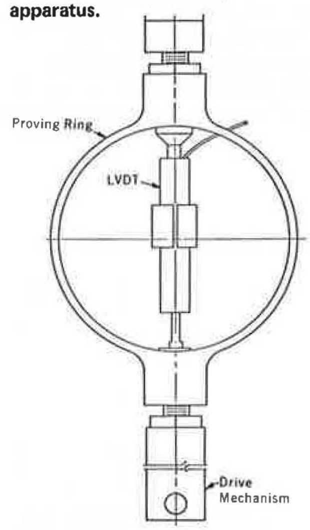
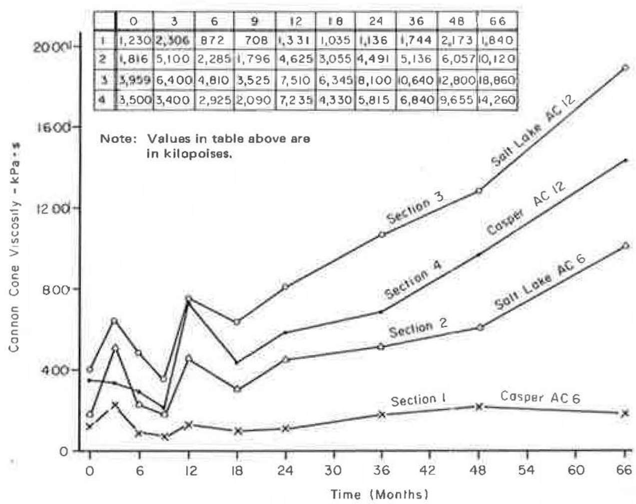


Figure 8. Cannon cone viscosity at 25°C (77°F) versus field aging.



	0	3	6	9	12	18	24	36	48	66
1	1,230	2,306	872	708	1,331	1,035	1,136	1,744	2,173	1,840
2	1,816	5,100	2,285	1,796	4,625	3,055	4,491	5,136	6,057	10,120
3	3,959	6,400	4,810	3,525	7,510	6,345	8,100	10,640	12,800	18,860
4	3,500	3,400	2,925	2,090	7,235	4,330	5,815	6,840	9,655	14,260

Figure 9. Cannon cone versus thin film oven time.

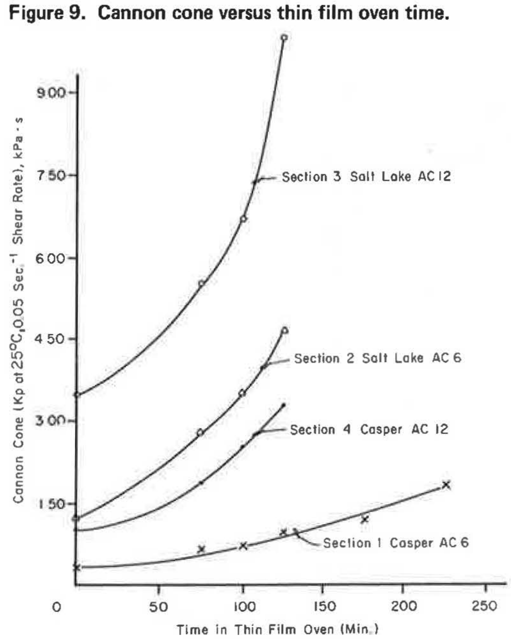
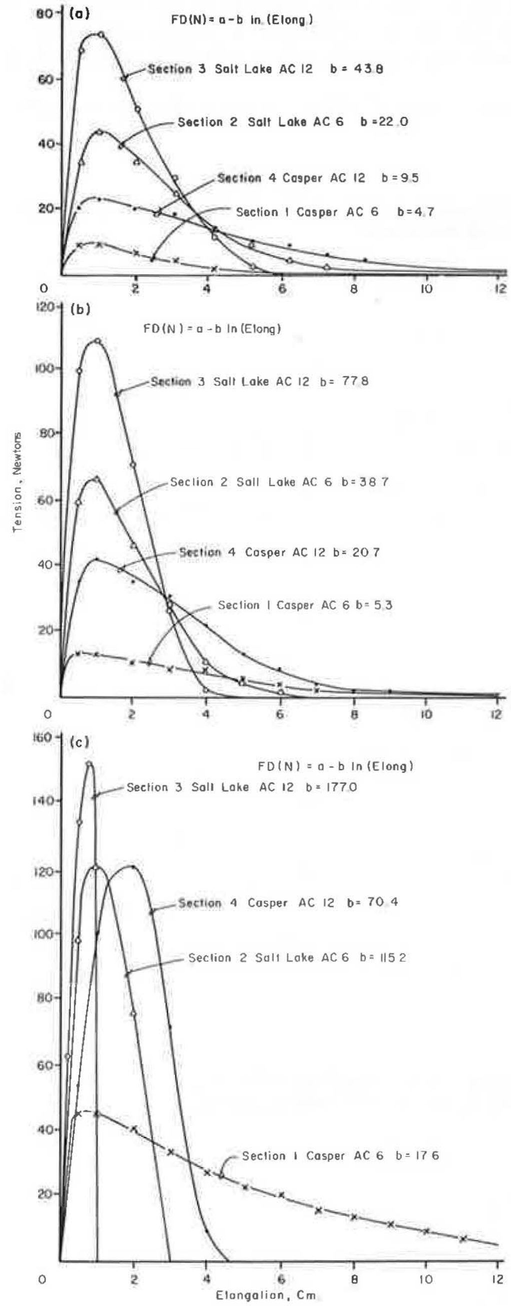


Figure 7. Force ductility of (a) original, (b) residue, and (c) 66-month samples.



asphalts for original, residue, and 66-month samples at 25 to 60°C (77 to 140°F), as well as 60 to 135°C (140 to 275°F). These values are shown in Figure 11.

Temperature susceptibility values for the original and residue samples are relatively the same for the four asphalts in the high temperature range. The temperature susceptibility of the original and residue samples in the low temperature range, closer to the actual pavement temperature range while in service, is greater for the Salt Lake City asphalts than for the Casper asphalts.

Temperature susceptibility values of the asphalts after 66 months in service show a somewhat different relationship from that of the original and residue samples.

A significant decrease is observed in temperature susceptibility; section 2 dropped to the same general range as the Casper asphalts, and section 3 decreased drastically to even a lower level than the Casper source. The AC-12 grades were higher in temperature susceptibility initially but decreased much more than the AC-6 grade to lower levels for the residue and 5½-year samples. Source is the more influential factor in this temperature range.

Temperature susceptibility values in the low temperature range correlate highly with the number of meters of transverse cracking present in each section. Sections 2 and 3, the more temperature-susceptible sections, had more cracking than the less temperature-susceptible sections, 1 and 4. Viscosity changes with temperature variation may be a major factor in predicting pavement performance.

Figure 10. Cannon cone viscosity versus shear rate of (a) original sample, (b) sample after 75 min in oven, (c) sample after 100 min in oven, and (d) sample after 125 min in oven.

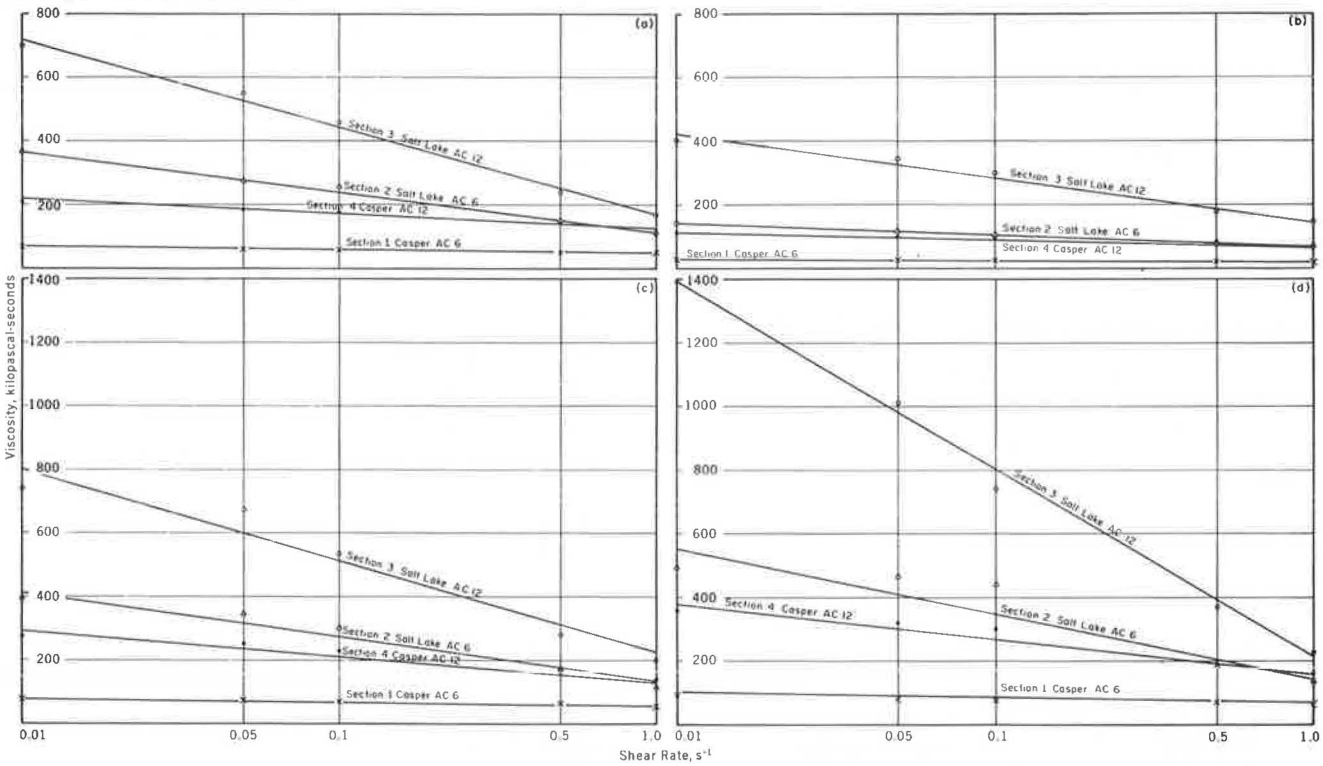


Figure 11. Temperature susceptibility at (a) 25 to 60°C (77 to 140°F) and (b) 60 to 135°C (140 to 275°F).

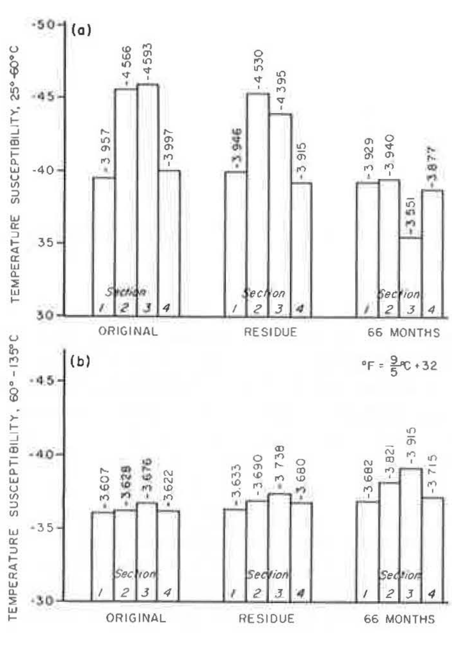
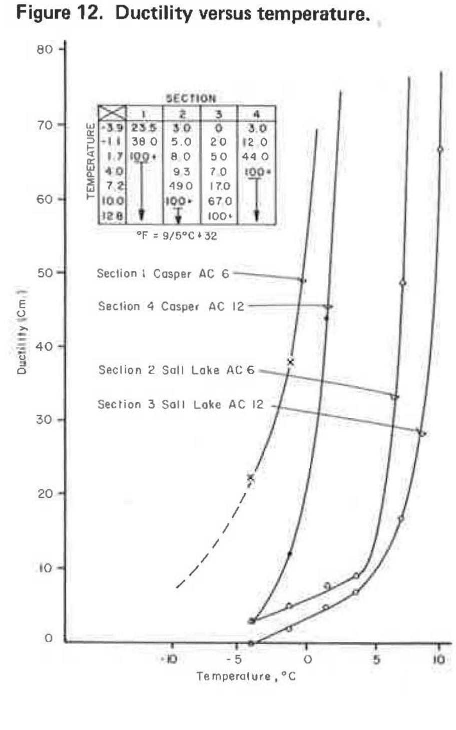
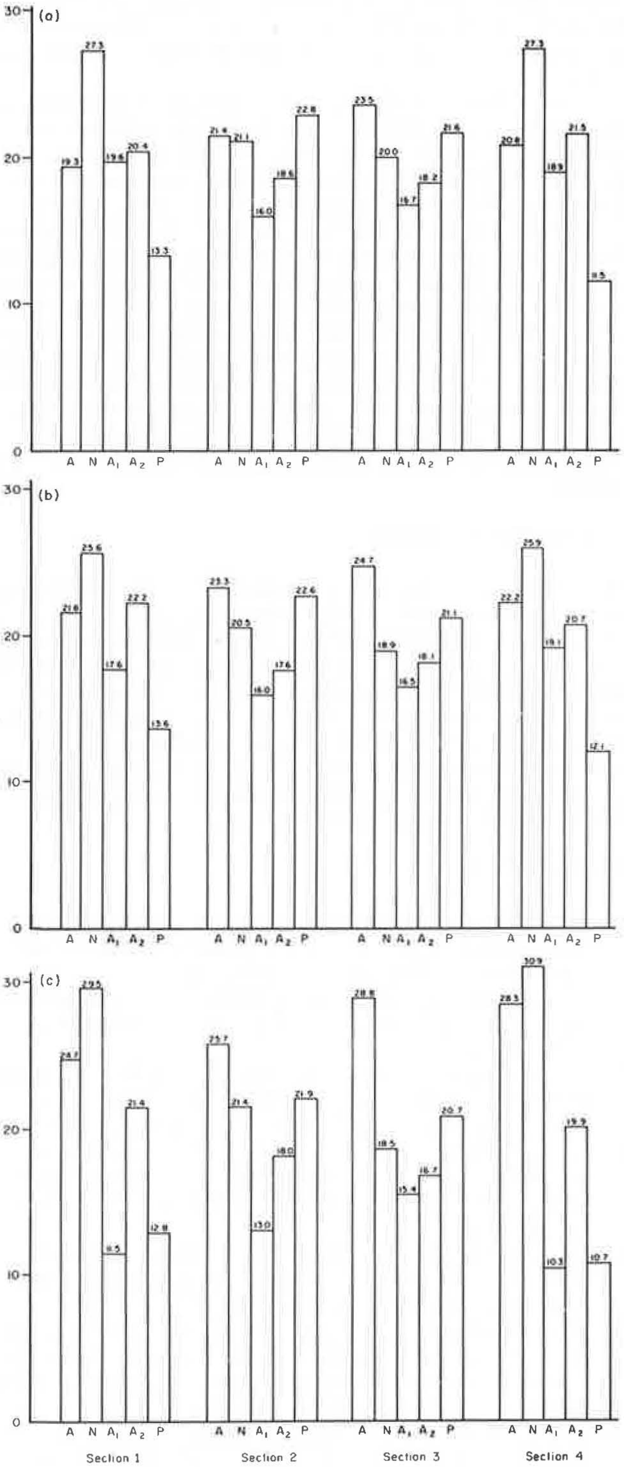


Figure 12. Ductility versus temperature.



To gain an indication of how temperature changes affect the ductility of the asphalts, ductility tests were run on storage tank samples at various water bath temperatures. This temperature-ductility analysis produced the curves shown in Figure 12. The Casper asphalts appear to remain ductile at a lower temperature than do the Salt Lake City asphalts. The AC-6 asphalts remained ductile at a lower temperature than the AC-12 asphalts within each source, but, as with temperature susceptibility, the source of the asphalts is a much more controlling factor.

Figure 13. Asphalt composition of (a) original, (b) residue, and (c) 66-month samples.



CHEMICAL ANALYSIS

The asphalts were broken down chemically to determine why the origin of an asphalt has such an effect on its properties. In the analysis, the following five constituents were determined: asphaltenes (A), nitrogen bases (N), first acidaffins (A_1), second acidaffins (A_2), and paraffins (P). The percentages of each constituent for the original, residue, and 66-month field samples are shown in Figure 13. No effort was made to determine what portion of the changes was due to volatility and how much was due to chemical reactions.

The transverse cracking of the sections correlates with the paraffin content and inversely with the nitrogen base content. High correlations were found between

Table 3. t-test for comparison of group mean.

Item	D.F.	Student t Comparison of Sources	Student t Comparison of Viscosity Grading
Transverse cracking	2	-9.37[a]	1.48
Longitudinal cracking	2	-0.75	1.46
Rut depth	32	0.03	0.19
Profilograph	23	0.51	1.37
A			
Original, residue	16	-35.12[b]	-0.52
Original, field	16	-29.32[b]	-47.52[b]
Original, residue, field	24	-28.85[b]	-33.96[b]
N			
Original, residue	16	192.70[b]	18.84[b]
Original, field	16	293.85[b]	23.67[b]
Original, residue, field	24	238.48[b]	21.05[b]
A_1			
Original, residue	16	61.64[b]	-10.01[b]
Original, field	16	-4.12[b]	-6.47[b]
Original, residue, field	24	15.38[b]	-13.35[b]
A_2			
Original, residue	16	6.78[b]	1.65
Original, field	16	6.29[b]	1.02
Original, residue, field	24	67.16[b]	10.84[b]
P			
Original, residue	16	-64.89[b]	18.54[b]
Original, field	16	-212.35[b]	35.24[b]
Original, residue, field	24	-172.71[b]	-24.36[b]
Penetration at 25°C			
Original, residue	2	13.08[c]	12.76[c]
Original, field	2	12.62[c]	12.80[c]
Original, residue, field	5	9.91[b]	9.53[b]
Viscosity at 60°C			
Original, residue	2	-1.85	-22.28[c]
Original, field	2	-8.57[a]	-12.50[c]
Original, residue, field	5	-4.37[c]	-6.96[b]
Viscosity at 135°C			
Original, residue	2	4.53[a]	-43.08[b]
Original, field	2	-1.61	-6.18[a]
Original, residue, field	5	-0.97	-5.06[c]
Cannon cone at 25°C			
0, 75, 100, 125 min	8	-12.55[b]	-9.34[b]
Original, field	2	-3.74	-5.57[a]
Original, residue, field	5	-2.09	-2.31
Slope			
0, 75, 100, 125 min	8	8.70[b]	5.87[b]
Original, field	2	9.92[c]	11.97[c]
Original, residue, field	5	5.49[c]	5.92[c]
Force ductility at 4°C			
Original, residue	2	-32.76[b]	-18.36[a]
Original, field	2	-2.98	-2.35
Original, residue, field	5	-3.78[a]	-2.70[a]
Temperature susceptibility at 25 to 60°C			
Original, residue	2	8.82[a]	-0.38
Original, field	2	0.70	-0.29
Original, residue, field	5	1.81	-0.50
Temperature susceptibility at 60 to 135°C			
Original, residue	2	4.86[a]	4.04
Original, field	2	2.60	1.09
Original, residue, field	5	4.07[c]	1.63

Note: 1°C = (1°F - 32)/1.8.

[a]At 5% significance. [b]At 0.1% significance. [c]At 1% significance.

transverse cracking and N/P. Holstead, Rostler, and White (14) found that the higher the value of $(N + A_1)/(A_2 + P)$ is, the more abrasion occurs in an asphalt concrete sample. The data here reveal that the higher $(N + A_1)/(A_2 + P)$ is, the less cracking present in the pavement. A definite relationship exists among compositional components, force ductility, temperature susceptibility, and transverse cracking of the pavements. It seems that the change in physical properties with changes in temperature, which is controlled by the chemical configuration of the asphalt, is critical in pavement cracking. This relationship may be one reason why the Salt Lake City source cracked more readily in Utah's severe high and low temperatures and many freeze-thaw cycles.

SOURCE VERSUS VISCOSITY

The data from sections 1 and 4 were grouped and analyzed against those from sections 2 and 3; then the sources were compared by using the Student t-test. Grouping sections 1 and 2 against sections 3 and 4 produced a viscosity comparison. The results of the t-test are given in Table 3 for selected performance variables, the compositional fractions, and the asphalt properties.

Based on the t-test, transverse cracking is significantly affected by the source of the asphalt and not by the viscosity grade. A significant difference between sources is observed for all of the compositional components; the same difference occurred in most cases when viscosity grades were compared. The fractions that appear to vary more with respect to source than viscosity grading are the nitrogen bases, second acidaffins, paraffins. This is also obvious upon inspection of Figure 13. These three components are correlated highly with respect to transverse cracking.

The test results from this study imply that the source of asphalt affects transverse cracking, temperature susceptibility, and compositional makeup more than viscosity grading does. This emphasizes the fact that viscosity grading cannot be relied on too heavily and further that new and better methods of grading asphalts are needed.

SUMMARY

It is widely accepted that new methods for grading asphalts are needed and would greatly improve highway performance, pavement design, and failure prediction. Penetration, viscosity, and ductility tests currently used to grade asphalts have been shown to be of value as indicators of asphalt quality but are limited in accuracy and repeatability.

The results of this study indicate that force ductility gives a good indication of pavement performance. The data obtained from the force ductility test are in a much more workable form than the standard ductility test. Results of the ductility test are somewhat vague and indefinite. Also, more accurate readings are obtained in the early stages of the force ductility test, where the cross-sectional areas are uniform for all samples. In theory the force ductility test is based on measuring the stress of a sample due to deformation. This stress in a pavement is caused by thermal expansion, traffic loading, or both.

The relationship between the Cannon cone viscosity and shear rate has been shown in recent publications to be of value in evaluating asphalts. This is generally true in this study; the better performing asphalts have a smaller slope on the viscosity-shear rate semilog plot. The slope of the viscosity-shear rate relationship increases with age, as expected.

Temperature susceptbibility appears to be a major factor in predicting pavement cracking in Utah. It is insufficient to determine an asphalt property at a single temperature and then to predict pavement cracking. By comparing trends in viscosity change with changes in temperature, a much more realistic view of pavement consistency at extreme temperatures is acquired.

Certainly chemical makeup of an asphalt, as suggested by Rostler and adopted here, is a useful tool in understanding the behavior of asphalt cements. Correlations exist among various chemical fractions, asphalt properties, and performance criteria.

Chemical makeup is related to the transverse cracking observed in the test sections. Higher paraffin content and lower nitrogen base and first acidaffin contents indicate more transverse cracking. Similarly, the higher the factors N/P and $(N + A_1)/(A_2 + P)$ are, the fewer are the number of transverse cracks that develop in the pavement.

In this study, penetration was not a good indicator of performance, but generally the higher penetration asphalts performed best. In Utah, chemical makeup, temperature susceptibility, and pavement cracking are influenced more by the source than by viscosity grading. A general comparison of the asphalts in this study shows that the softer the asphalt is, the better it will perform. The asphalts higher in penetration and ductility and lower in viscosity have less tendency to crack. Rutting, however, occurs if a pavement is too ductile, but, in the environment of the Green River test sections, rutting was not a problem for the grades and sources of asphalt used. Similar penetration and viscosity grades from other sources may perform differently in the same climate. Methods to better define these limits of performance are needed.

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Use of Power Plant Aggregate in Bituminous Construction

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Coal-fired power plant aggregates are the portion of the ash rejected by the stack and collected at the base as a waste product. Two aggregates are produced: boiler slag, a glasslike material, and bottom ash, more commonly called cinders. Approximately 16 percent of the annual ash production is used, and the remainder is disposed of as a waste product. This paper discusses engineering properties of power plant aggregates. Although they behave in many ways as conventional aggregates, they also differ in many ways from conventional aggregates. Consequently, new or modified test methods and specifications are needed before power plant aggregates can be used routinely in highway construction. Both field and laboratory data are given for bituminous mixtures using power plant aggregates. Based on these data and on limited service records, power plant aggregates can be used successfully in bituminous mixtures. Boiler slags are best used as partial replacements in conventional mixtures. Bottom ash is best used "as is" in stabilized base or shoulder construction.

Shortages of natural aggregates, increasing stockpiles of power plant ash, and the popularization of recycling have led to renewed interest in the use of power plant ash in highway construction.

ORIGIN OF POWER PLANT AGGREGATE

Coal-fired power plant aggregate is the portion of the ash rejected by the stack and collected at the base as a waste product. Two types of power plant aggregate are produced: dry bottom ash and wet bottom boiler slag (1). The term power plant aggregate includes both bottom ash and boiler slag.

Dry bottom ash, often referred to as cinders, is produced by burning pulverized coal over open grates. The ash that does not go up the stack falls as a solid to the ash hopper at the bottom of the furnace. The word dry refers to the solid state of the ash as it falls to the hopper. In a typical dry bottom furnace, 75 to 80 percent of the ash is fly ash and 20 to 25 percent is bottom ash. The newer and larger power plants are generally of the dry bottom type, and, therefore, production in the future will increasingly be of the dry bottom type.

Wet bottom boiler slag is produced by burning crushed or pulverized coal in a furnace where the bottom ash is kept molten and tapped off as a liquid. The molten slag is periodically drawn from the furnace and dropped into water where it is quenched and fractured to an angular glasslike material. The word wet is used to describe the molten state of the slag as it is drawn from the furnace. Depending on the type of furnace, from 50 to 85 percent of the total ash is boiler slag and the remainder is fly ash.

ANNUAL PRODUCTION AND USE OF POWER PLANT ASH

Annual ash production has increased from 23 Tg (25 million tons) in 1966 to 45 Tg (49 million tons) in 1973 (2). Annual ash production could easily increase to five times the 1973 figure by the year 2000 (3). The use of high-ash western coals and a switch to coal from other energy sources could further increase this figure. Although the percentage of ash being used has increased during the last few years (12 percent in 1967 versus 16 percent in 1973), in view of the increased production, the net amount of stockpiled ash is increasing each year. There is no reliable estimate of the quantity of ash that has been accumulated over the years in old stockpiles.

Of the 13.3 Tg (14.7 million tons) of bottom ash and boiler slag produced in 1973, 9.7 Tg (10.7 million tons) were dry bottom ash and 3.6 Tg (4.0 million tons) were boiler slag (2). Only 17 percent of the dry bottom ash was used whereas 45 percent of the boiler slag was used, and only 0.1 percent and 1.2 percent of the bottom ash and boiler slag were used in asphaltic concrete.

A large 1000-MW power plant may burn 2.7 Tg (3 million tons) or more of coal per year. With an ash content of 14 percent, approximately 0.38 Tg (420 000 tons) of ash, 0.08 Tg (80 000 tons) of dry bottom ash, and 0.30 Tg (340 000 tons) of fly ash would be produced each year. On the other hand, many of the smaller plants around the country produce only a few thousand megagrams of ash per year. Although some of this ash is of questionable economic value because of its limited quantity, most of the ash is produced at large plants. In fact, 80 percent

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of the coal is burned in plants that burn more than 0.9 Tg (1 million tons) of coal per year.

The production figures given above can be misleading, however. First, the production of ash may be interrupted when plants are occasionally shut down for unscheduled repairs, and, second, the quality and uniformity of the ash can vary. Ash quality and uniformity are controlled by a number of factors including coal source, degree of pulverization, load on plant, and burning temperature. These parameters are controlled to optimize power production, not ash production. Therefore, periodic and uncontrolled changes in ash quality and quantity may be expected, especially in plants that burn blended or multisource coals as necessitated by recent pollution regulations. These variations may or may not be significant in bituminous construction depending on the particular application of the ash.

STOCKPILING AND DISPOSAL

Because the production rate at a given plant is less than the rate at which the ash is used in a typical construction operation, the ash usually must be stockpiled. Although many plants do follow good stockpiling practice, stockpiling operations are generally designed not to maintain the quality of the ash but to facilitate handling and disposal. Often, the primary purpose of the stockpiling is to dispose of the ash, and only later is there an attempt to reclaim it.

Practices such as recombining pyrite with the ash and mixing fly ash or coal wash water with the ash are not uncommon. Because the stockpiled material is often transported hydraulically or end dumped from a large pile, it may be badly segregated. Old stockpiles are often very heterogeneous because of stockpiling practice and variations due to the operation of the power plant. Considerable care should be exercised before an existing stockpile of ash is accepted for potential use.

ENGINEERING PROPERTIES OF POWER PLANT AGGREGATE

Power plant aggregate is composed principally of silicon, aluminum, and iron and small percentages of calcium, magnesium, sodium, and other elements (4, 5). The composition of the ash is primarily controlled by the source of the coal and not by the type of the furnace. Chemical composition itself is of little practical importance in evaluating the engineering properties of power plant aggregates in bituminous mixtures.

Wet Bottom Steam Boiler Slags

Data from standard tests for several West Virginia wet bottom and dry bottom ashes are given in Table 1. Most boiler slags are predominately one size, and the bulk of the material occurs in the minus No. 4 to plus No. 16 range. This size range is typical of ash sampled from different sources in Kansas, Florida, West Virginia, Ohio, and Indiana. Traditional boiler slag is a hard, dense, black, glassy, angular material with a smooth surface texture, much like crushed glass.

Boiler slag can also be vesicular in nature, as if the molten slag was frothy prior to solidification. For example, the slag produced within the last few years at the Kammer, West Virginia, power station has been somewhat vesicular, but previously it was dense and nonvesicular. This change in the ash is apparently due to changing sources of coal.

As an extreme example, boiler slag from the Willow Island, West Virginia, power station was dramatically changed as the result of an experiment in which limestone was injected into the furnace to control sulfur emissions. The result was a frothy looking, greenish ash with little crushing resistance. Whereas the LA abrasion resistance (ASTM C 131-69) of the Willow Island ash had been consistently below 40, the limestone-injected slag had an LA value in excess of 50. The properties traditionally associated with wet bottom boiler slags—hard, angular, nonvesicular—may well be changed as the burning of western coal and the adoption of less traditional plant procedures become more prevalent.

Wet bottom boiler slags are fractured to size as a result of the thermal stresses created in the slag as it is quenched in water, and many of the particles are highly stressed internally. This fact is recognized in Germany, where wet bottom boiler slag is crushed before it is used in portland cement concrete (6). Wet bottom boiler slag is generally lacking in the coarser sizes (plus No. 4), and, except for the oversized material, it is not customary to crush boiler slag used for highway construction in the United States.

The presence of high residual stresses may account for the unexpectedly high soundness values (ASTM C 88-73) recorded for some of the dense, nonvesicular boiler slags (Table 1). The soundness losses may be partially due to thermal cycling during drying and not to the expansive forces of the sodium or magnesium sulfate. Boiler slag will often crackle and snap as it is suddenly heated or cooled.

LA abrasion values for wet bottom boiler slags are customarily in the 30 to 40 range. The data are indicative not necessarily of the hardness or wear resistance of the slag but of the fracture resistance. This is particularly true of the more vesicular or porous boiler slags that lack toughness. The more porous the slag is, the higher the percentage loss will be. The coarser fractions of wet bottom boiler slag tend to be more porous than the finer sand-size fractions. Consequently, LA abrasion data for the coarser fractions, on which the LA abrasion test is usually performed, may not be representative of the finer, sand-size fraction, which predominates in most boiler slags. The fines produced during the LA abrasion test are intermediate in size and are nonplastic.

Dry Bottom Ash

The physical and chemical properties of dry bottom ash are more variable than those of wet bottom boiler slag. This is true in terms of both plant to plant variation and daily or yearly variation. This is to be expected because dry bottom ash is the direct result of the burning process whereas wet bottom slag is solidified from the molten slag. Typical aggregate properties are given in Table 1. Gradation curves for two West Virginia dry bottom ashes are shown in Figure 1. Similar curves have been reported for other bottom ashes (4, 7) in which the ash was well graded from coarse to fine. The minus No. 200 sieve material is essentially coarse fly ash and is nonplastic.

Dry bottom ash contains hard boiler slaglike particles as well as popcornlike particles. The popcorn particles are essentially poorly sintered agglomerates of coarse fly ash. The softer of these agglomerates can be broken with the fingers to individual coarse fly ash particles. Dry bottom ash also contains hard boiler slaglike particles that accumulate and solidify on the super heaters or fall into the ash hopper in a molten state. These particles are hard, glassy, and vesicular. The popcorn and slaglike particles are found in both the coarse and fine fractions of the ash.

The specific gravity of dry bottom ash depends on the mineralogical composition of the ash as well as the porosity of the particles. A dense dry bottom ash may have a bulk specific gravity as high as 2.6 while a poor ash, with a large percentage of both porous and popcorn particles, may range as low as or even lower than 1.6. To some degree, specific gravity is an indicator of quality. The smaller the percentage is of popcornlike particles, the higher the specific gravity will be.

Water absorption data for dry bottom ash (ASTM C 127-73) are quite variable and depend on the porosity (surface texture) of the ash and the percentage of popcorn particles. The popcorn particles will invariably absorb water but not asphalt. Water absorption data are, therefore, not always a reliable indicator of how the ash will behave with asphalt.

Soundness data for dry bottom ashes tend to be on the high side but often meet specification limits for natural aggregates. Many of the pores in dry bottom ash are so large that the ash has no opportunity to build up stresses during the drying cycle. As a consequence, the soundness test does not discriminate ash quality, particularly with respect to the presence of popcorn particles.

A potentially serious problem can occur if the iron pyrite from the coal cleaning operation is recombined with the bottom ash. The pyritic particles are subject to degradation in the pavement and should be eliminated in ash that is to be used in bituminous construction. Further experience with other bottom ashes indicates that abnormally high sulfate contents can be caused by other than pyrite-contaminated ash. It appears that the sulfate can precipitate in the furnace as complex soluble sulfate salts. In the case of Hoot Lake bottom ash (Table 2), the water-soluble residue from the evaporation of ash leachate contained 28 800 mg/liter (216 oz/gal) of soluble sulfate, equivalent to 28.8 g of $SO_4^=$ ion per 1000 g (28.8 oz/1000 oz) of ash. Compacted samples of this ash stabilized with penetration grade asphalt were allowed to set in the laboratory. The samples subsequently expanded and produced cracking much like that produced by reactive aggregates in portland cement concrete. It is hypothesized that moisture absorbed by the salts caused volume changes sufficient to crack the specimens.

Leachate data for a number of other power plant ashes are given in Table 2. The data show that the pH of the leachate does not always correlate with sulfate content, nor is the pH of the fly ash necessarily that of the bottom ash. A high iron content may or may not be associated with a high sulfate content, depending on the source of the sulfate ion. The effect of the soluble salts on the durability of the bituminous binder is of questionable concern, but abnormally high sulfate contents are of concern to adjacent structures, particularly portland cement concrete (8).

BITUMINOUS MIXTURES CONTAINING WET BOTTOM BOILER SLAGS

The technical literature contains very little information on the use of power plant ash in bituminous mixtures, even though boiler slag has been used successfully at various locations in the United States. Much of the early work centered around the use of boiler slag as skid-resistant aggregate in finite graded mixtures, ostensibly because of the hardness and angularity of the aggregate.

Rockdale Slag Aggregate

Jimenez and Galloway (9, 10) reported on the design of mixtures using wet bottom boiler slag from lignite coal. The boiler slag was black, somewhat porous, and essentially of one size, ranging between the No. 30 and No. 8 sieves. The slag was combined with the fly ash in the disposal operation, and, therefore, the ash used in the mixtures contained about 5 percent fly ash.

Acceptable dense-graded wearing course mixtures with a top size of 9.5 mm (0.4 in) were obtained by blending 25 to 50 percent limestone screenings (50 percent minus No. 16) with the slag. The addition of limestone screenings was necessary to achieve adequate stabilities (Hveem stabilities of 35 to 50). When an immersion compression strength test was used, 90 percent of the strength was retained after 24 hours. Field results were favorable in terms of mixture and lay-down properties and resistance to weather and traffic. Acceptable skid resistance was reported.

Florida Wet Bottom Boiler Slag

The state of Florida evaluated wet bottom boiler slag as a surfacing aggregate (11). Adequate Marshall stabilities were obtained by blending the boiler slag with a stable, fine-graded sand. LA abrasion loss for the slag was reported as 43, exceeding the specification limit of 40. Approximately 95 percent of the bituminous coating was retained in a stripping test.

Wet bottom boiler slag has been used extensively in the past in the immediate Tampa area and by the city of Tampa. It was used extensively in paving the parking lot at Disney World. Since the conversion from coal to oil, boiler slag is no longer produced at the Tampa power station.

West Virginia Experience

Boiler slag has been used regularly in the northern panhandle of West Virginia as an aggregate in type 3 wearing course mixture (4, 7). The mixture is approximately 50 percent boiler slag, 39 percent river sand, 3 percent fly ash, and 8 percent asphalt cement. A great deal of this material has been placed during the last 10 years with a good record of service. In some cases it has been used under heavy traffic, such as on US-250 through Wheeling, West Virginia, which carries considerable heavy truck traffic. The mixes are not promoted as anti-skid mixtures; instead, boiler slag is used to upgrade a sand that is deficient in coarse fractions.

A similar mixture was used as a deslicking mixture on a hazardous stretch of road on Easton Hill in Morgantown, West Virginia. This mixture, placed in November 1969, was composed of 52 percent boiler slag, 23 percent limestone sand, 21 percent river sand, and 4 percent fly ash. Field skid data are not available, but the overlay did significantly reduce the accident frequency at the site (7). A problem in aggregate retention was encountered in the wheel tracks, and, by 1974, especially on the curves, the overlay had worn through to the old pavement. Some of the aggregate loss may be attributed to poor lay-down conditions (wet weather and the lateness of the season).

Special Considerations in Designing Boiler Slag Mixtures

The biggest difficulty in using wet bottom boiler slag is its smooth surface texture and tendency to be one size. To achieve acceptable stabilities and gradations requires that the slag be blended with other aggregates. The poor stability is due to the smooth glassy surface texture and lack of interparticle friction. The high angles of internal friction obtained in direct shear tests on boiler slags (Table 1) are due to particle interlock and are obtained only when the slag is confined. Data given in Table 3 show the effect on Marshall stability of replacing sand with constant gradation boiler slag and asphalt content. The stability is reduced as the percentage of boiler slag is increased.

Based on field experience and laboratory data, mixtures with more than 50 percent boiler slag will generally lead to unacceptable stabilities. This rule of thumb must vary according to the properties and gradation of the slag and the other aggregates in the blend. The more stable the other aggregates are and the more widely graded the slag is, the greater the allowable percentage of slag will be. Perhaps some of the more vesicular boiler slags may be used in greater percentages, but if excessively vesicular these slags tend to be weak and to lack crushing resistance. Boiler slag can be used without any special consideration in conventional mixtures if the percentage of boiler slag is limited to less than approximately 50 percent of the aggregate. The best use of boiler slag is as a partial replacement for the sand fraction in base and surface mixtures. Mixtures with acceptable skid resistance that use boiler slag as the top size aggregate can be designed by limiting the percentage of boiler slag in the mix and by avoiding open-graded mixtures with low filler content.

BITUMINOUS MIXTURES CONTAINING DRY BOTTOM ASH

Very little use has been made of dry bottom ash in bituminous construction. Ash from the Fort Martin and Mitchell power stations was used in cold mixes for maintenance work in West Virginia and in shoulder construction at the Fort Martin power station. The ash was used without beneficiation except for scalping on the 19-mm (¾-in) sieve.

Mixture Properties

Gradation curves for the Fort Martin and Mitchell ash (Figure 1) show considerable variation with sampling in 1972, 1973, and 1975. A set of Marshall design curves using the 1972 ash are shown in Figure 2.

As with conventional mixtures, the effect of kneading compaction instead of Marshall compaction is to increase the stability and density and decrease the optimum asphalt content. The effect of compaction method varies with different ashes and is more important with the more friable ashes, which are more easily degraded during compaction. For the Fort Martin ash, the density produced by kneading compaction was 160 Kg/m^3 (10 lb/ft^3) greater than that produced from the 50-blow Marshall compaction (Figure 2).

Kneading compaction more closely approximates field compaction. The rough surface texture and high angle of internal friction of the bottom ash apparently limit the amount of shear deformation in the compaction mold under drop hammer compaction, but kneading compaction breaks down the internal structure more the way field rolling does. Gyratory compaction may be more suitable and should be investigated relative to the compaction of dry bottom ash mixtures.

The low flow values shown in Figure 2 are typical of many dry bottom ash mixtures. This indicates a tendency toward brittle mixtures, and this has been observed in the field. With a loss in subgrade support, these mixtures appear to crack more readily and extensively than bituminous mixtures made with conventional aggregates.

Dry bottom ashes often exhibit high air void contents, even when well graded and compacted with the kneading compactor. The high air voids, relative to mixtures with similarly graded conventional aggregates, are due to the high angle of internal friction and the rough surface texture of the ash particles.

Special Considerations

A serious consideration with some dry bottom ash is the presence of popcorn particles. The popcorn particles invariably do not get coated throughout with asphalt. During field or laboratory compaction, the particles may be crushed and a void may be left that is essentially filled with crumbled, coarse fly ash particles. Because of these particles, using many of the dry bottom ashes as a surface aggregate is questionable. Instead, they can be more effectively used without beneficiation as stabilized base where aggregate quality and gradation requirements are less severe.

Immersion Marshall data for Mitchell and Fort Martin dry bottom ash show a 95 percent retention in stability after immersion. Retention of asphalt in the presence of water does not appear to be a problem with dry bottom ashes based on immersion data and field experience. As discussed previously, however, some pyritic or high sulfate ashes degrade when exposed to water over an extended period of time. The standard immersion tests are too short in duration to detect the sulfate problem. A test method, such as a water leachate test or a modified immersion test, should be developed to identify this problem. An upper limit on the soluble sulfates (leachate test) may provide an adequate criterion for identifying a potential sulfate problem.

Field Experience

Each summer since 1972, several hundred thousand megagrams of Fort Martin and Mitchell bottom ash have been used to upgrade secondary rural roads in northwestern West Virginia. Most of the ash was mixed with 6 to 7 percent residual asphalt by using both modified MS-2 and CMS-2 emulsions. Cost of the mix (at the plant) was \$10 to \$11 per megagram in 1975 (slightly more than half of the cost was attributed to the emulsion). The cost of equivalent hot mixes using conventional aggregate was \$17 to \$25 per megagram.

Field lay-down experience with this material was excellent. Although the material was a bit fluffy in the spreader, little or no difficulty was encountered whether the material was placed with a paver or spreader box or merely end dumped and leveled with a grader. The mix was very stable under a pneumatic-tired roller in depths up to 15 to 20 cm (6 to 8 in), although in the deeper lifts it had to be tracked with a grader before it could be successfully rolled. In general, the more satisfactory rolling was done with pneumatic-tired rollers. Compaction was essentially complete after 4 or 5 passes with the pneumatic-tired roller, and the mix was sufficiently stable to carry loaded dump trucks immediately after rolling.

Field densities for the Fort Martin ash were 1760 to 1810 kg/m^3 (110 to 113 lb/ft^3) at 10 to 12 percent air voids. These densities more closely approximate the densities achieved with kneading compaction than with the drop hammer or Marshall compaction. Although the data are not yet complete, preliminary results from research now in progress show that modified kneading compaction can be used to approximate the degradation that occurs during compaction. Sieve analyses of material entering the mixing plant may vary significantly from the material in the pavement after compaction; for example, compare the edge and conveyor graphs shown in Figure 3. Although the degradation is greater than that for conventional aggregates, it apparently has not adversely affected the performance of these pavements except in the one case noted below.

Table 1. Engineering properties of power plant aggregates.

Source of Ash	Type of Ash	Bulk Specific Gravity[a]	Percentage of Water Absorption[a]	LA Abrasion[b]	$MgSO_4$ Soundness[c]	Friable Particles[d]	Florida Bearing Value	Angle of Internal Friction (deg)
Fort Martin	Dry	2.31	2.0	30 to 45	15	Yes	196	40
Mitchell	Dry	2.68	0.3	30 to 40	10	No	—	43
Kammer	Wet	2.76	0.3	37	10	No	72	41
Willow Island	Wet	2.72	0.3	33	15	No	46	42

[a]ASTM C 127. [b]ASTM C 131. [c]ASTM C 88. [d]ASTM C 142.

Table 2. Chemical analyses of ash leachate.

Source	Type	pH	Ca^{++}	Fe^{++}	Mg^{++}	Na^{+}	$SO_4^{=}$
Fort Martin	Pyritic fly ash	8.2	1200	<20	30	42	3 500
Fort Martin	Pyritic bottom ash	3.0	960	7000	266	600	23 500
Fort Martin	Nonpyritic fly ash	10.3	700	<20	<2	156	2 460
Fort Martin	Nonpyritic bottom ash	8.4	100	<20	14	38	450
Harrison	Fly ash	12.1	1260	<20	<2	242	1 880
Harrison	Bottom ash	8.1	120	<20	14	20	520
Mitchell	Bottom ash	2.5	720	120	16	28	3 760
Big Sandy	Bottom ash	8.2	60	<20	4	60	250
Willow Island	Lime-injected boiler slag	12.5	1060	<20	<2	40	8
Hoot Lake, Minnesota	Bottom ash	10.8	800	<20	<2	9900	28 800
Montana	Bottom ash	9.6	520	<20	2	46	1 000
Ohio	Bottom ash	2.1	1260	2000	12	112	8 260

Note: Ion concentrations are reported as mg/liter (134 oz/gal) of ion soluble in water: 1:1 by weight, aggregate:water, agitated for 24 hours.

Figure 1. Gradation of dry bottom boiler ashes.

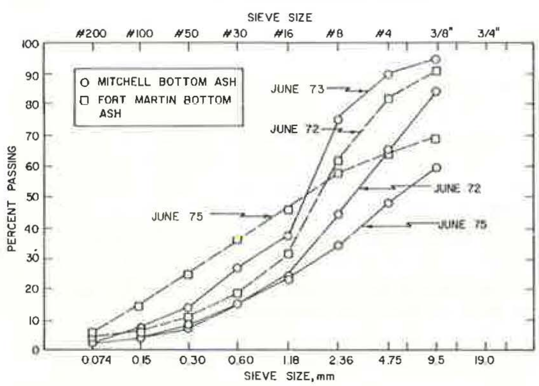


Table 3. Effect of percentage of boiler slag-sand on mixture stability.

Mixture	Marshall Stability (kg)	Flow (mm)
60% Kammer, 40% limestone sand 1	125	1.9
50% Kammer, 50% limestone sand 1	152	1.8
40% Kammer, 60% limestone sand 1	172	1.8
35% Kammer, 65% limestone sand 2	659	3.4
48% Kammer, 52% limestone sand 2	488	3.8

Note: 1 kg = 2.2 lb; 1 mm = 0.039 in.

Figure 2. Mixture design of Fort Martin and Mitchell, West Virginia, bottom ash.

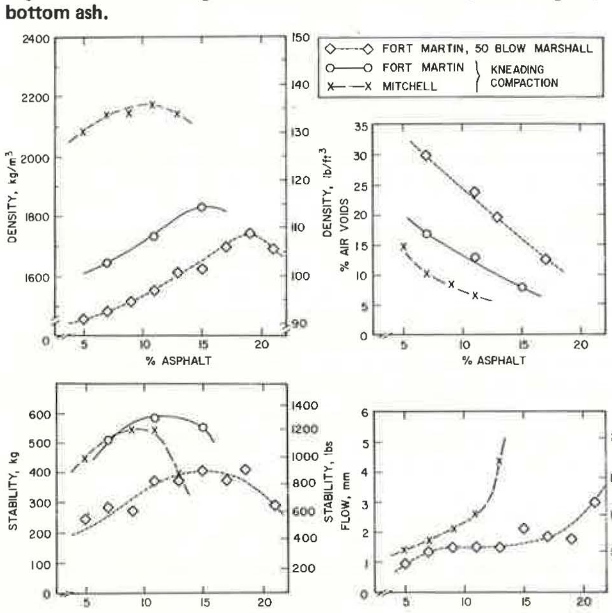
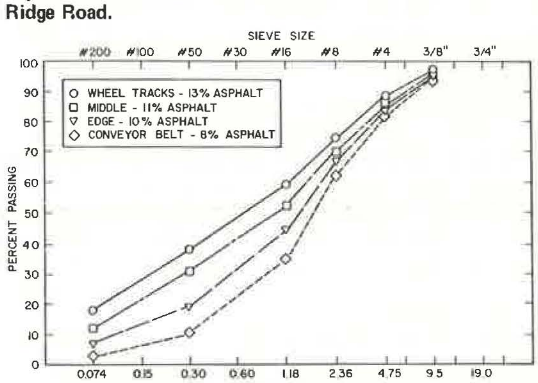


Figure 3. Gradation of Fort Martin ash for Baker's Ridge Road.



Service Behavior

The Mitchell bottom ash is representative of a good quality ash, has had an excellent service record with no reported problems, and demonstrates that bottom ash can be used successfully as an aggregate. The pyritic Fort Martin ash, placed in 1972, is representative of a poor quality ash and has brought out several potential problems as described in the following paragraphs.

Excessive pyrite and soluble sulfates present a durability problem. In spite of the asphalt coating, with time, the pyrite weathers throughout the depth of the pavement and forms weak pockets in the pavement. The weathered pyrite also tends to expand, causing popouts near the surface. This was especially severe on the shoulders of the access road to the Fort Martin power station (June 1975). In addition, the weathering of the pyrite also stained the pavement surface a deep, permanent, ferric oxide red color.

Disintegration of the popcorn particles at the pavement surface caused by compaction and traffic has left a pockmarked appearance. Below the surface these particles are crumbled and appear as voids filled with uncoated, loose, coarse fly ash. These voids could become a source of failure if they fill with water and freeze, but there is no evidence of this to date (June 1975).

Another problem was identified near Morgantown on Baker's Ridge Road, a rural road also paved in 1972 with Fort Martin ash. In the curves, the pavement in the wheel tracks has flushed badly with lateral shoving of 100 mm (39 in) or more. Gradation data and asphalt contents for samples taken from the wheel paths and the untrafficked part of the pavement (June 1975) are shown in Figure 3. The cornering forces of the traffic, some of which consists of loaded coal trucks, have caused severe degradation of the ash. The asphalt contents are higher than designed but still not sufficient to cause the flushing without the degradation.

With traffic, the surfaces paved with Fort Martin and Mitchell ash tend to skin over, giving a gritty but fine texture much like No. 80 grit sandpaper. This gritty surface is apparently maintained over a period of time. After 3 years of traffic, BPN skid numbers (ASTM E 303-74) have leveled at 60 to 80, depending on the level of traffic.

Except for the curves on Baker's Ridge Road, little if any additional densification of Fort Martin ash has occurred since it was placed. Density data from a variety of projects are relatively unchanged after 2 to 3 years, remaining at 1760 to 1810 kg/m^3 (110 to 113 lb/ft^3).

SUMMARY AND CONCLUSIONS

Based on prior usage and laboratory data, it is apparent that power plant aggregates can be successfully used in bituminous mixtures. Before this can be done on a routine basis, however, additional work must be done to develop test methods and specifications that are appropriate to power plant aggregate.

Need for New or Modified Test Methods and Specifications

In some cases the current test methods and specifications are too restrictive and exclude acceptable material. In other cases they are not sufficiently discriminating and allow material that is unacceptable. For example, the standard Los Angeles abrasion test does not sufficiently identify popcorn particles in dry bottom ash, nor is the test indicative of degradation that might occur under field compaction.

Still other questions can be raised: Are the high air voids associated with some dry bottom ashes acceptable? How significant is the soundness test for boiler slags, and what are acceptable test limits? Is specific gravity an adequate indicator of popcorn particles in dry bottom ashes, and, if so, what lower limit is acceptable? Clearly a test method is needed to identify potential sulfate problems in dry bottom ashes.

Potential Use

For power plant aggregates to be used successfully, they must be used properly. They should not, in general, be approached as conventional aggregates with the stock-in-trade question, "Do they meet specifications?"

Boiler slag can be used without special consideration in conventional mixtures if the percentage of boiler slag is limited to less than approximately 50 percent of the aggregate. The most favorable use of boiler slag is in base and surfacing mixtures through blending with other aggregates. Mixtures with acceptable skid resistance that use boiler slag as the top size aggregate are possible if careful attention is paid to mixture design.

Dry bottom ash is best used as is in base mixtures or for shoulder construction where gradation and toughness requirements are not so critical. Many bottom ashes are not acceptable in wearing courses. Although it may be feasible to blend bottom ash with other aggregates, extensive blending for beneficiation may be unacceptable economically.

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Repeated-Load Indirect Tensile Fatigue Characteristics of Asphalt Mixtures

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The determination of the fatigue characteristics of pavement materials is necessary for the design and evaluation of highway and airport pavements. This paper summarizes the findings of a study in which the controlled-stress, repeated-load indirect tensile test was used to investigate the fatigue characteristics of asphalt mixtures. The logarithmic relationships between fatigue life and both applied stress and initial mixture strain were evaluated and found to be linear. In addition, linear relationships were found between n_1 and the logarithm of K_1 for the strain-fatigue life relationships and between n_2 and the logarithm of K_2 for the stress-fatigue life relationships. The effects on fatigue life produced by load, asphalt content, aggregate type, and testing temperature are discussed. Fatigue life could not be estimated from only applied stress or stress difference; however, equations relating fatigue life to both the initial mixture strain and the stress-strength ratio were developed. From this study it was concluded that the repeated-load indirect tensile test is suitable for evaluating the fatigue characteristics of asphalt mixtures.

A knowledge of the fatigue behavior of asphalt materials is important in the design and evaluation of highway and airport pavements. Many of the design procedures currently used are empirical and deterministic in nature. However, in view of the developments in pavement system design and the recognition that fatigue cracking is a basic distress mode, the effects of repeated loads, or fatigue, should be considered in design. A necessary step is the determination of the fatigue characteristics of pavement materials.

Many test methods have been used to study the fatigue behavior of asphalt materials. These test methods generally have produced excellent results that have contributed to the knowledge of the fatigue behavior of asphalt materials. Nevertheless, these tests are costly and in most cases difficult to conduct. Thus, it is highly desirable to use a test method that is economical to apply and easily implemented, and an operating agency rather than a research agency should conduct the test.

Previous work (1, 2, 3, 4) suggests that the repeated-load indirect tensile test represents such a test. Thus, a laboratory study using the repeated-load indirect tensile test was conducted to investigate the fatigue behavior of asphalt materials and to demonstrate the use of the test.

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EXPERIMENTAL PROGRAM

The primary objectives of the experimental phase of the study were to

1. Use the repeated-load indirect tensile test to study the fatigue characteristics of asphalt mixtures,
2. Determine the effect of selected factors on fatigue characteristics, and
3. Investigate the relationships between fatigue life and other load and mixture characteristics.

Experiment Design

The basic experiment design is given in Table 1. Factors investigated were aggregate type, asphalt content, testing temperature, and stress level. The aggregates were a crushed limestone and a rounded river gravel, which were combined to produce a gradation approximating the Texas specification for a class A, type B fine-graded base or level-up coarse or a type C coarse-graded surface course (11). These aggregates were combined with an AC-10 asphalt cement at five asphalt contents (4, 5, 6, 7, and 8 percent by total weight of the mixture). The resulting mixtures were tested at 10, 23.9, and 37.8°C (50, 75, and 100°F) at stress levels ranging between 55 and 827 kPa (8 and 120 psi).

Specimen Preparation

The materials were mixed at 135°C (275°F), and the mixtures were compacted at 121.1°C (250°F) by using the Texas gyratory shear compactor and by following the procedure specified in test method Tex-206-F, part 2 (5). After compaction, the specimens to be tested at 23.9°C (75°F) were cured for 2 days at room temperature, 23.9 ± 1.1°C (75 ± 2°F), before testing. For tests conducted at 10 and 37.8°C (50 and 100°F), specimens were cured at room temperature for 24 hours and then transferred to temperature-controlled rooms and kept at 10 ± 1.1°C (50 ± 2°F) or 37.8 ± 1.1°C (100 ± 2°F) for another 24 hours before testing.

Test Equipment and Procedures

The basic equipment for the repeated-load indirect tensile test included a loading system and a means of monitoring the applied loads and the horizontal and vertical deformations of the specimen. A closed-loop electrohydraulic system was used to apply the loads, and a special loading device was used to ensure that the loading platens and strips remained parallel during the test. The 12.7-mm-wide (0.5-in) fitted steel loading strips were mounted on the upper and lower platens.

Loads were monitored with a load cell and were recorded continuously. Vertical and horizontal deformations for any particular load application were measured by using linear variable displacement transformers. Permanent horizontal deformations were measured by using a cantilevered arm device that had been used in previous studies (6, 7).

A typical load pulse is shown in Figure 1, and the resulting load-deformation-time relationships are shown in Figures 1 and 2. Fatigue life was defined as the number of load applications required to completely fracture the specimen as shown in Figure 2.

ANALYSIS AND DISCUSSION

Relationships With Applied Stress

As in previous studies, a linear relationship was found to exist between the logarithm of the applied stress and the logarithm of fatigue life (Figures 3 and 4), which could be expressed as

$$N_f = K_2(1/\sigma_T)^{n_2} \qquad (1)$$

where

N_f = fatigue life,
σ_T = applied tensile stress, and
n_2, K_2 = constants.

Values of K_2 varied between 11.9 Pa and 270 MPa (3.26×10^5 and 1.90×10^{13} lbf/in^2) while values of n_2 varied between 2.66 and 5.19. These values of n_2 compare favorably with those reported by other investigators using other test methods (2, 8, 9, 10); however, the values of K_2 are significantly smaller, resulting in much lower fatigue lives.

In previous work Porter and Kennedy (3) analyzed the results obtained from other test methods and compared the characteristics of these tests and concluded that the results obtained from the repeated-load indirect tensile test were compatible if stress is expressed in terms of stress difference. Expressing fatigue life in terms of stress difference changes the value of K_2 but does not affect the value of n_2; thus the relationship can be expressed in the form

$$N_f = K_2'(1/\Delta\sigma)^{n_2} \qquad (2)$$

where

N_f = fatigue life,
$\Delta\sigma$ = stress difference, and
K_2', n_2 = constants.

Values of K_2' ranged from 513 Pa to 368 CPa (1.41×10^7 to 2.53×10^{16} lbf/in^2), as shown in Figures 5 and 6.

Approximate linear relationships were found to exist between n_2 and the logarithm of K_2 and between n_2 and the logarithm of K_2'. The relationships developed from a regression (by using K_2 and K_2' for stress in lbf/in^2) were

$$n_2 = 0.919 + 0.312 \log K_2 \quad (r = 0.95, S_e = 0.23) \qquad (3)$$

$$n_2 = 0.734 + 0.266 \log K_2' \quad (r = 0.96, S_e = 0.19) \qquad (4)$$

The 95 percent confidence interval for the correlation coefficient r was between 0.85 and 0.98 for equation 3 and between 0.88 and 0.99 for equation 4. By making n_2 the independent variable, the following relationships were obtained:

$$\log K_2 = -1.867 + 2.886\, n_2 \quad (r = 0.95, S_e = 0.70) \qquad (5)$$

$$\log K_2' = -1.859 + 3.485\, n_2 \quad (r = 0.96, S_e = 0.70) \qquad (6)$$

Because of the above relationships, a regression analysis was conducted on the combined data from previous studies (2, 8, 9, 10). Values of K_2 for the indirect tensile test were assumed to be the values of K_2'. This analysis yields the following average relationships:

1. When n_2 is the dependent variable,

$$n_2 = 0.069 + 0.322 \log K_2 \quad (r = 0.96, S_e = 0.32) \qquad (7)$$

2. When n_2 is the independent variable,

$$\log K_2 = 0.860 + 2.869\, n_2 \quad (r = 0.96, S_e = 0.97) \qquad (8)$$

The 95 percent confidence interval for r was between 0.94 and 0.97. Figure 7 shows the positions of the above relationships relative to all data points. Thus, it appears that there is a relationship between K_2 and n_2, irrespective of mixture properties or test procedure.

Relationships With Initial Strain

The initial strains were estimated by dividing the applied dynamic stress by the average static modulus of elasticity (4). These estimated strain values were used to develop relationships between the logarithm of strain and the logarithm of fatigue life (Figures 8 and 9). A regression analysis was used to establish these relationships and to obtain values of the constants K_1 and n_1 for the general equation

$$N_f = K_1(1/\epsilon_{mix})^{n_1} \qquad (9)$$

where

ϵ_{mix} = initial strain and
K_1, n_1 = constants.

Values of K_1 ranged from 5.65×10^{-17} to 5.01×10^{-7} and n_1 ranged from 2.66 to 5.19. These values were compared to those obtained from previous flexural and axial load tests on other mixtures (2, 8, 9, 10). Reported data indicate that K_1 generally ranges from 5.0×10^{-20} to about 5.0×10^{-5}, and n_1 ranges from about 2.5 to 6.3, depending on asphalt content, testing temperature, and testing procedure. The values of K_1 and n_1 obtained by using the repeated-load indirect tensile test compare favorably with previously reported values, and values from this study were essentially the same as those obtained for similar mixtures with the same asphalt contents and tested at the same temperature.

Thus, it appears that comparable values of K_1 and n_1

can be obtained for essentially the same mixtures, irrespective of test procedures. Because the mechanism of failure is probably strain dependent, consideration of strain at least partially accounts for the states of stress, temperature, and load pulse.

Relationship Between K_1 and n_1 Values

A linear relationship was also found to exist between n_1 and log K_1:

1. When n_1 is the dependent variable,

$$n_1 = 0.938 - 0.261 \log K_1 \quad (r = 0.98, S_e = 0.13) \tag{10}$$

2. When log K_1 is the dependent variable,

$$\log K_1 = 3.211 - 3.719\, n_1 \quad (r = 0.98, S_e = 0.47) \tag{11}$$

The 95 percent confidence interval for r was between 0.94 and 0.99. Recently, Pell and Cooper (10) reported an expression that was similar to equation 10:

$$n_1 = 0.5 - 0.313 \log K_1 \tag{12}$$

Because of this similarity, an evaluation of the relationship between n_1 and log K_1 was also conducted for the combined values obtained from various other test methods and for different kinds of mixtures (2, 8, 9, 10), and the following linear relationships were obtained:

1. When n_1 is the dependent variable,

$$n_1 = 1.350 - 0.252 \log K_1 \quad (r = 0.95, S_e = 0.29) \tag{13}$$

2. When n_1 is the independent variable,

$$\log K_1 = 3.977 - 3.609\, n_1 \quad (r = 0.95, S_e = 1.09) \tag{14}$$

The 95 percent confidence interval for r was between 0.92 and 0.97. Figure 10 shows the positions of the above relationships relative to all data points.

The high correlation coefficients associated with these expressions show that a linear relationship exists between n_1 and log K_1, irrespective of mixture properties and test procedures.

Table 1. Factorial experiment design for repeated-load tests.

Test Temperature (° C)	Stress Level (N/cm²)	Limestone					Gravel				
		4	5	6	7	8	4	5	6	7	8
10	49.6			3					3		
	66.2			3					3		
	82.7			3					3		
23.9	5.5				5					5	
	11.0	3	5	6	8	3	3	5	7	9	3
	16.5	5	5	7	5	5	5	5	7	9	3
	22.1	3	5	7	6	3	3	6	7	9	3
	27.6	4	5	7	8	3	3	5	7	8	3
37.8	5.5			3					3		
	11.0			3					3		
	16.5			3					3		

Note: 1 N/cm² = 1.45 psi; 1° C = (1° F - 32)/1.8.

Figure 1. Typical relationships of load and deformation versus time for repeated-load indirect tensile test.

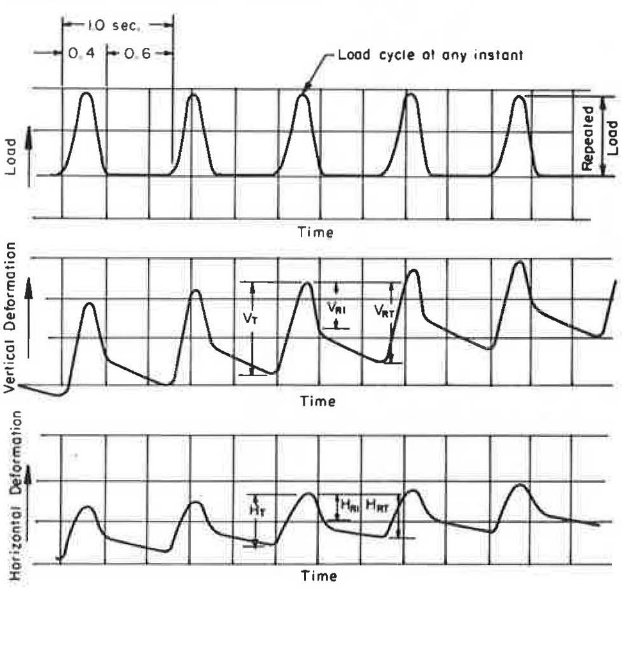


Figure 2. Relationships between number of load applications and vertical and horizontal deformation for repeated-load indirect tensile test.

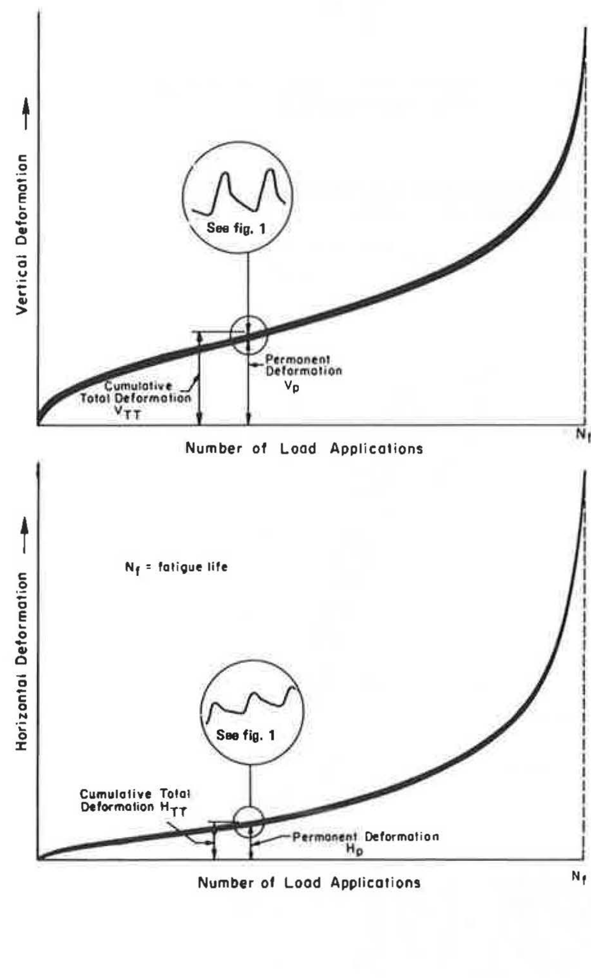
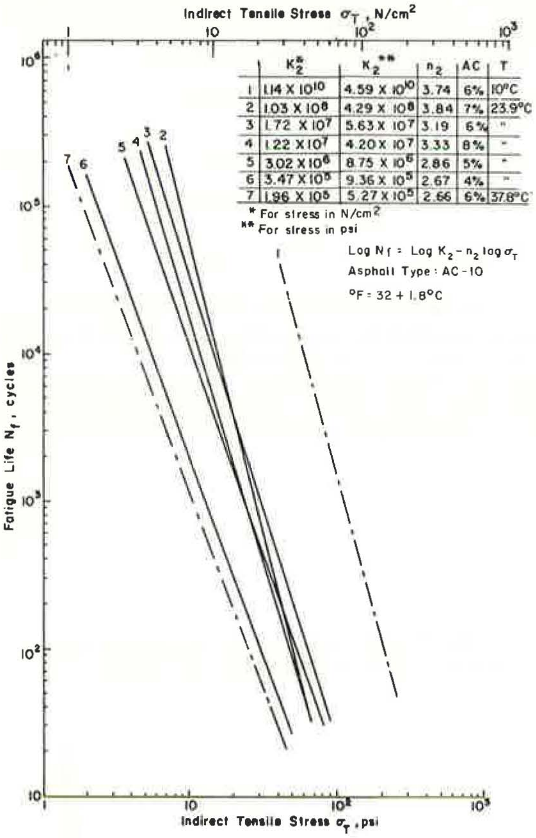


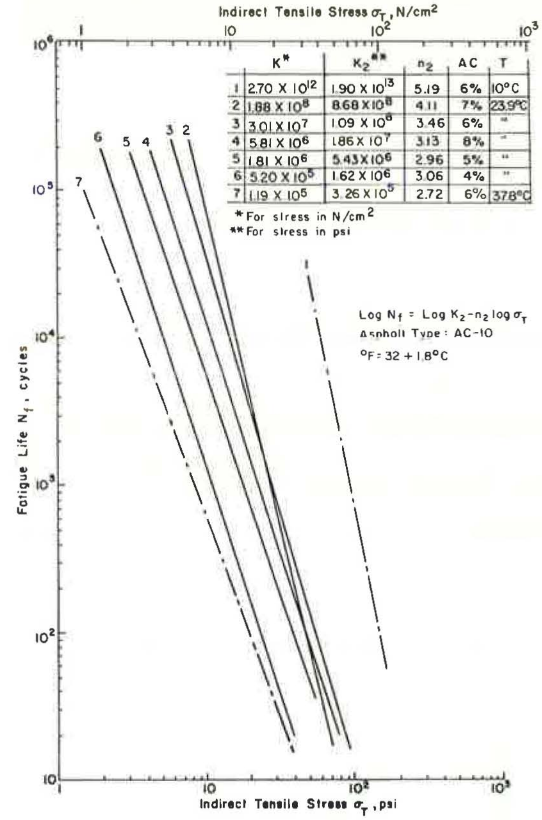
Figure 3. Relationship between fatigue life and applied tensile stress for limestone mixtures.



	K_2^{*}	K_2^{**}	n_2	AC	T
1	1.14×10^{10}	4.59×10^{10}	3.74	6%	10°C
2	1.03×10^{8}	4.29×10^{8}	3.84	7%	23.9°C
3	1.72×10^{7}	5.63×10^{7}	3.19	6%	"
4	1.22×10^{7}	4.20×10^{7}	3.33	8%	"
5	3.02×10^{6}	8.75×10^{6}	2.86	5%	"
6	3.47×10^{5}	9.36×10^{5}	2.67	4%	"
7	1.96×10^{5}	5.27×10^{5}	2.66	6%	37.8°C

* For stress in N/cm^2
** For stress in psi

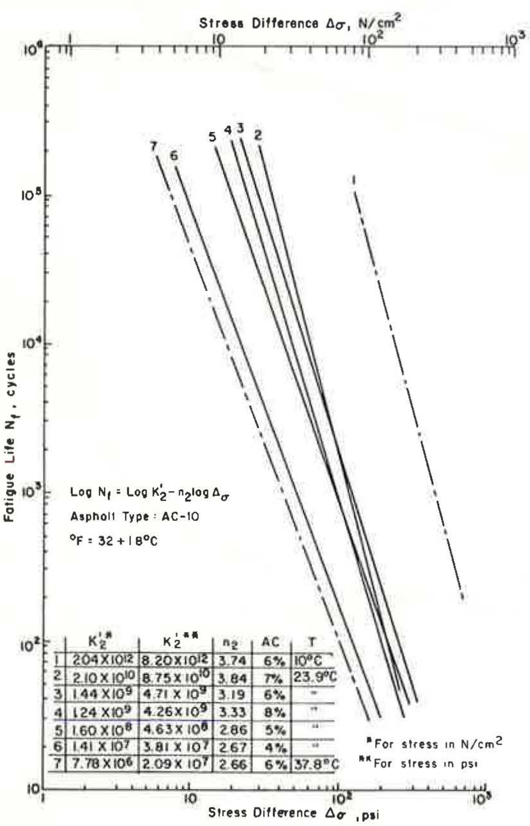
Figure 4. Relationship between fatigue life and applied tensile stress for gravel mixtures.



	K^{*}	K_2^{**}	n_2	AC	T
1	2.70×10^{12}	1.90×10^{13}	5.19	6%	10°C
2	1.88×10^{8}	8.68×10^{8}	4.11	7%	23.9°C
3	3.01×10^{7}	1.09×10^{8}	3.46	6%	"
4	5.81×10^{6}	1.86×10^{7}	3.13	8%	"
5	1.81×10^{6}	5.43×10^{6}	2.96	5%	"
6	5.20×10^{5}	1.62×10^{6}	3.06	4%	"
7	1.19×10^{5}	3.26×10^{5}	2.72	6%	37.8°C

* For stress in N/cm^2
** For stress in psi

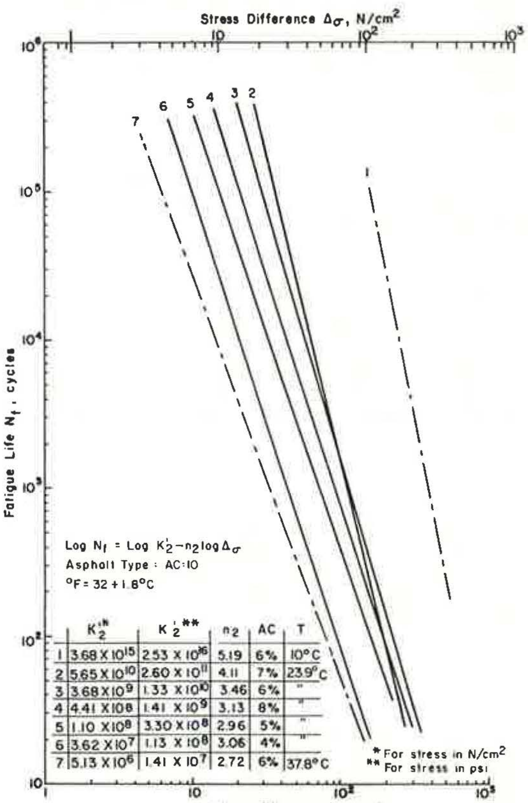
Figure 5. Relationships between fatigue life and stress difference for limestone mixtures.



	$K_2'^{*}$	$K_2'^{**}$	n_2	AC	T
1	2.04×10^{12}	8.20×10^{12}	3.74	6%	10°C
2	2.10×10^{10}	8.75×10^{10}	3.84	7%	23.9°C
3	1.44×10^{9}	4.71×10^{9}	3.19	6%	"
4	1.24×10^{9}	4.26×10^{9}	3.33	8%	"
5	1.60×10^{8}	4.63×10^{8}	2.86	5%	"
6	1.41×10^{7}	3.81×10^{7}	2.67	4%	"
7	7.78×10^{6}	2.09×10^{7}	2.66	6%	37.8°C

* For stress in N/cm^2
** For stress in psi

Figure 6. Relationships between fatigue life and stress difference for gravel mixtures.



	$K_2'^{*}$	$K_2'^{**}$	n_2	AC	T
1	3.68×10^{15}	2.53×10^{16}	5.19	6%	10°C
2	5.65×10^{10}	2.60×10^{11}	4.11	7%	23.9°C
3	3.68×10^{9}	1.33×10^{10}	3.46	6%	"
4	4.41×10^{8}	1.41×10^{9}	3.13	8%	"
5	1.10×10^{8}	3.30×10^{8}	2.96	5%	"
6	3.62×10^{7}	1.13×10^{8}	3.06	4%	"
7	5.13×10^{6}	1.41×10^{7}	2.72	6%	37.8°C

* For stress in N/cm^2
** For stress in psi

Focal Point of Strain-Fatigue Life Relationships

Pell and Cooper obtained a focal point of intersection, i.e., a point at which all strain-fatigue life relationships intersect, which occurred at a strain of 0.000 63 and a fatigue life of 40.

In this study, different focal points occurred and were found to depend on whether equation 10 or 11 was used, i.e., whether n_1 or log K_1 was the dependent variable in the regression analysis. When equation 10 was used, the focal point occurred at a strain of 0.000 147 and fatigue life of 3926. On the other hand, the use of equation 11 resulted in a focal point at a strain of 0.000 191 and fatigue life of 1626.

Figure 7. Relationships between n_2 and K_2.

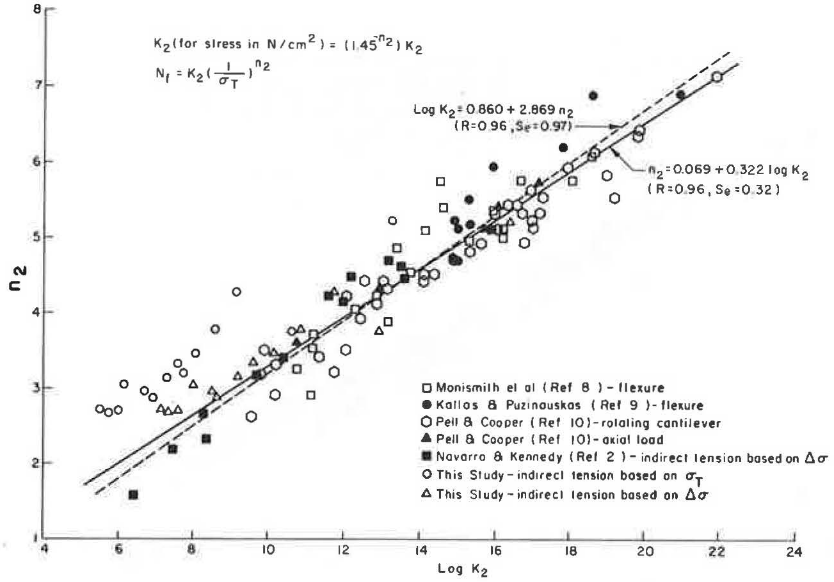


Figure 8. Relationships between fatigue life and initial strain for limestone mixtures.

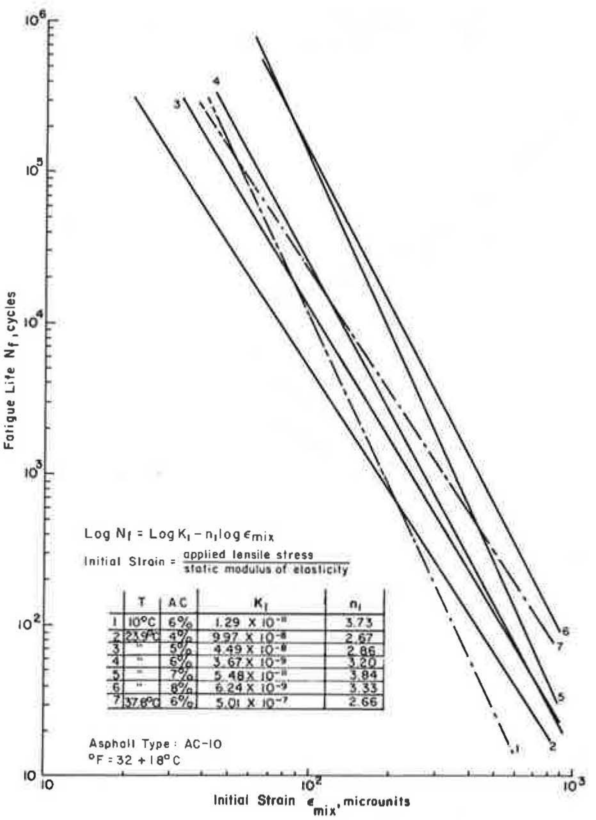
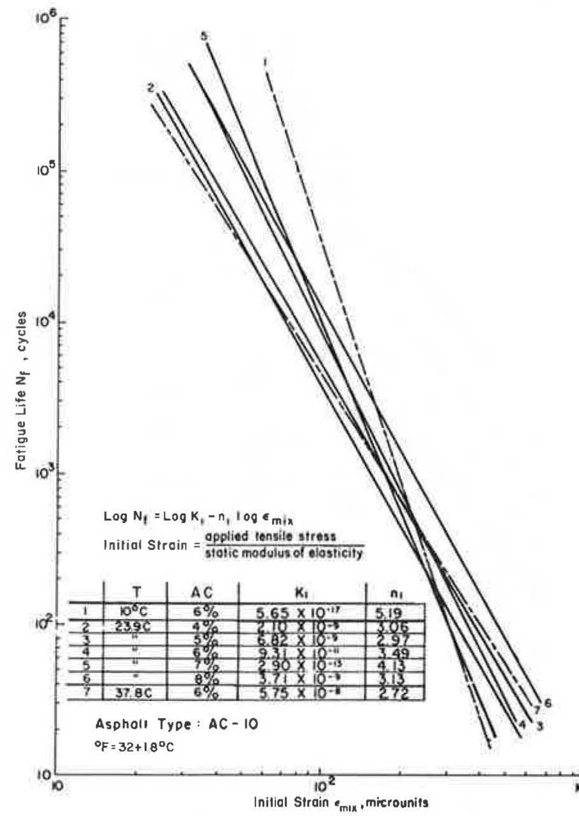


Figure 9. Relationship between fatigue life and initial strain for gravel mixtures.



Factors Affecting Fatigue Life

Asphalt Content

The relationships between fatigue life and asphalt content are shown in Figure 11 for the limestone and gravel mixtures tested at 23.9°C (75°F). The geometric means were used to obtain each point since fatigue life apparently has a logarithmic normal distribution.

It is evident that there was an optimum asphalt content for maximum fatigue life. For the mixtures and stress levels used in this study, stress level had a negligible effect on the optimum asphalt content; however, for the limestone mixtures the optimum appeared to decrease slightly with increased stress level.

Figure 10. Combined relationships between n_1 and K_1.

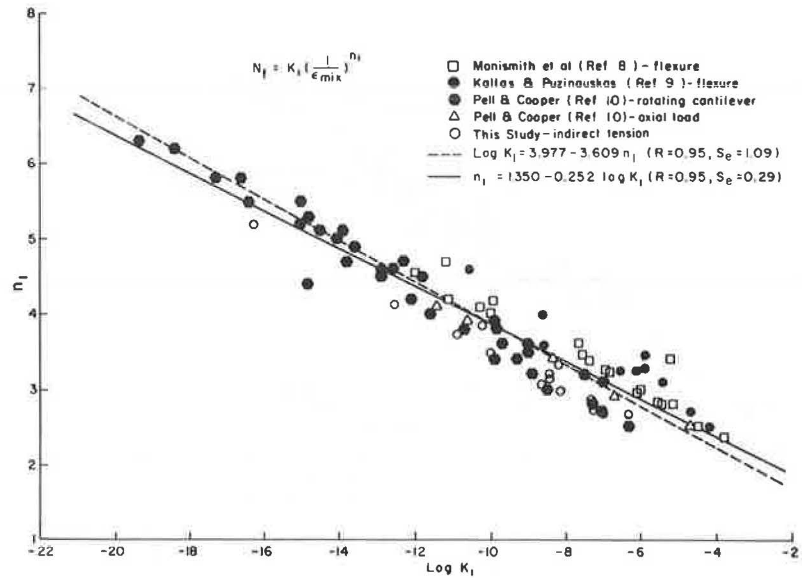


Figure 11. Effects of aggregate and asphalt content on fatigue life.

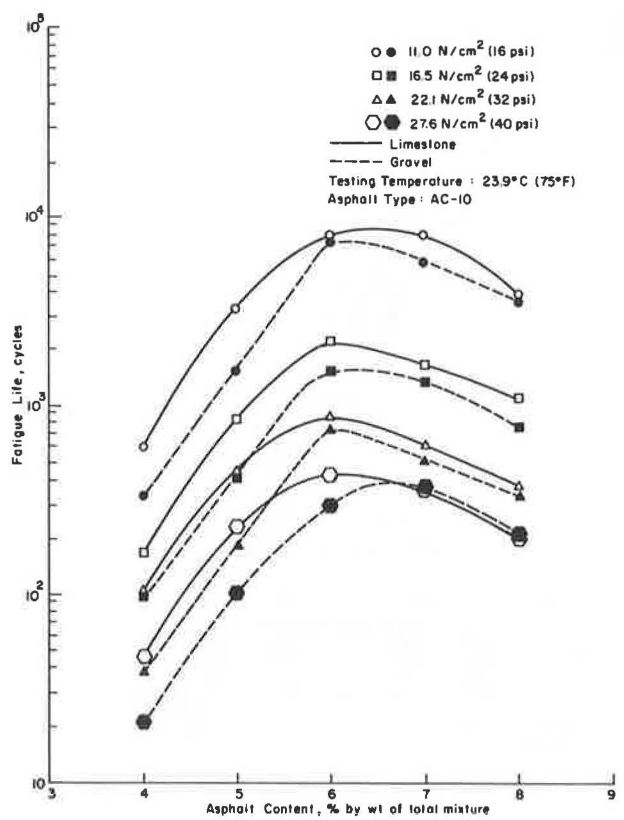
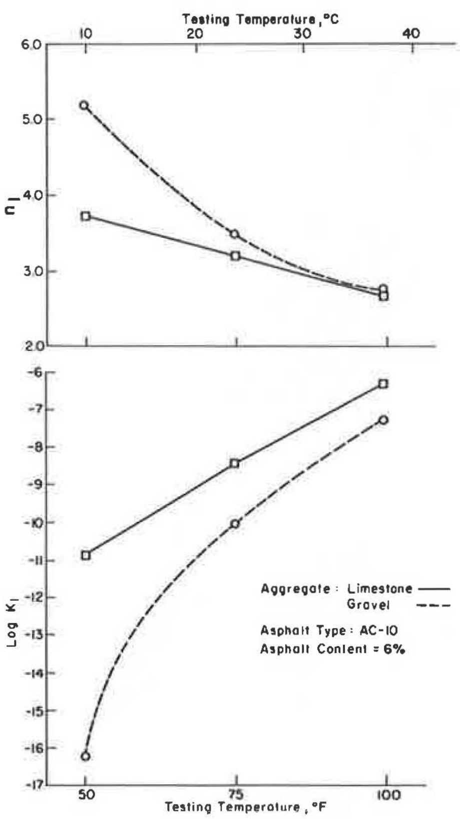


Figure 12. Effects of testing temperature on the values of K_1 and n_1.



The optimum asphalt content for maximum fatigue life was generally in the range of 6 to 6.5 percent, which is slightly less than the optimum asphalt content for maximum bulk density (4).

Maximum values of n_1 and minimum values of K_1 occurred at an asphalt content of about 7 percent, which is slightly higher than the optimum asphalt content for maximum fatigue life. In addition, maximum values of K_2, K_2', and n_2 occurred at an asphalt content of approximately 7 percent.

Aggregate Type

The limestone mixtures tended to have slightly longer fatigue lives than the gravel mixtures (Figure 11). The difference decreased with decreased stress and increased asphalt content, and, in some cases, the gravel mixtures had a longer fatigue life. In nearly all cases, however, the differences in fatigue lives for limestone and gravel mixtures were very small and it was felt that both mixtures exhibited approximately the same fatigue lives. In an earlier study on the same mixtures, Moore and Kennedy (1) reported that the gravel mixtures had longer fatigue lives than the limestone mixtures; however, this effect was not a significant behavioral difference. A significant interaction effect, however, was detected between aggregate and stress: Gravel exhibited significantly longer fatigue lives at low stress levels.

Thus, it would appear that, for the conditions of this study, aggregate was not important but that, for low stress levels such as those experienced in the field, aggregate type could be a significant variable.

The gravel mixtures exhibited higher values of n_1 and lower values of K_1 than the limestone mixtures. In terms of the stress relationships, the gravel mixtures exhibited higher values of n_2, K_2, and K_2' than the limestone mixtures.

Temperature

The effect of temperature (Figures 3 and 4) was found to agree with the findings of previous studies based on stress-controlled tests. In general, fatigue life increased with decreasing temperatures for the three temperatures considered.

The effect of temperature on the regression coefficients K_1 and n_1 is shown in Figure 12. An increase in testing temperature produced an increase in K_1 but a decrease in n_1. Figure 13 shows the effect of temperature on the regression coefficients K_2, K_2', and n_2; an increase in temperature produced a decrease in both K_2 and K_2' and also in n_2.

Fatigue Life Prediction

Efforts were made to develop mathematical relationships to predict fatigue life on the basis of load variables or mixture properties so that time-consuming and costly fatigue tests would not need to be conducted.

Initial Strain

A single expression was developed for estimating fatigue life in terms of initial strain ϵ_{mix}, irrespective of asphalt content, testing temperature, and aggregate type:

$$N_f = 9.38 \times 10^{-8}(1/\epsilon_{mix})^{2.76} \tag{15}$$

Equation 15 was obtained with a coefficient of determination of 0.70 and a standard error of estimate (a

Figure 13. Effects of testing temperature on the values of K_2, K_2', and n_2.

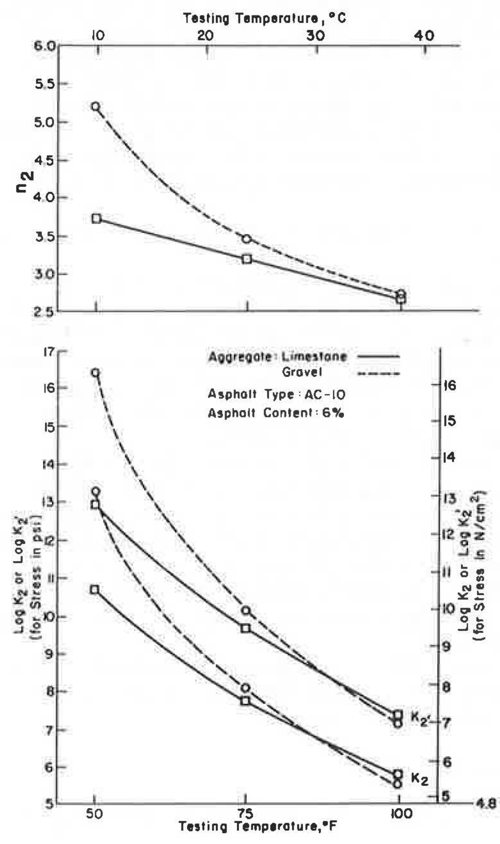
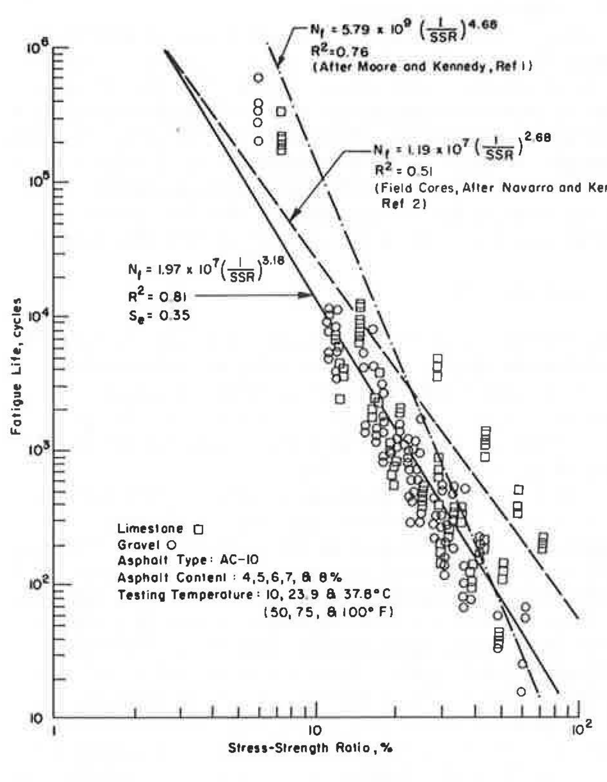


Figure 14. Relationships between fatigue life and stress-strength ratio.



measure of the variation of the logarithm of fatigue life about the regression line) of 0.44.

In view of the inherent variability associated with fatigue behavior, equation 15 would be relatively accurate; however, errors should be expected.

Stress or Stress Difference

The values of K_2, K_2', and n_2 for each mixture and testing temperature can be used with the general stress-fatigue life or stress difference-fatigue life equations to predict fatigue life.

Further regression analyses were conducted to obtain predictive equations independent of temperature, aggregate type, and asphalt content, but these produced very low coefficients of determination. Thus, it appears that there is no universally applicable method of predicting fatigue life on the basis of applied stress or stress difference.

Stress-Strength Ratio

To account for mixture and testing differences, the relationship between fatigue life and the stress-strength ratios, which previously has been shown to be a reasonable prediction (1), was evaluated. The stress-strength ratio was defined as the ratio of the applied repeated stress to the estimated indirect tensile strength (4).

There were linear relationships between the logarithm of fatigue life and the logarithm of stress-strength ratio SSR. From regression analysis, the following prediction equation, which is independent of asphalt content, temperature, and aggregate type, was obtained:

$$N_f = 1.97 \times 10^7 (1/SSR)^{3.18} \quad (r^2 = 0.81, S_e = 0.35) \quad (16)$$

The average fatigue life predictive equation from this study was compared with those obtained from earlier studies (1, 2); Figure 14 shows that all lines fall fairly well within the data points.

Results from this study indicated that prediction of fatigue life in terms of SSR is as good as that in terms of initial strain. Like initial strain, stress-strength ratio is easy to estimate and does not involve long-term repeated-load studies. Inasmuch as estimation of tensile strength requires only the measurement of failure load and does not require deformation measurements, this relationship probably has more application in laboratories, which are not equipped to run repeated-load tests.

Air Void Content

Many studies have indicated that fatigue life of asphalt mixtures increases with decreasing air void content. However, using the indirect tensile test method in a controlled-stress test, Moore and Kennedy (1) reported that air void content was not directly related to laboratory tensile fatigue life.

In this study, the optimum air void content for maximum fatigue life was about 3 percent. However, the compactive effort as well as the gradation was held constant so that changes in air void content were dependent on the changes in asphalt content, and thus the apparent relationship with air void content was judged to be due to the effect of asphalt content since the approximate optimum asphalt content for maximum fatigue life was about 6 percent, which corresponds to an optimum air void content of about 3 percent (4).

It is, therefore, felt that any apparent relationship between fatigue life and air void content is probably due to the fact that both air void content and fatigue life are related to other factors.

Density

Density and air void content are mixture properties that are inversely related to each other. Many researchers have correlated fatigue life with air void content rather than with density. Nevertheless, because it is much easier to estimate density than air void, an attempt was made to correlate fatigue life with density.

In this study, an approximately linear relationship was found to exist between bulk density and the logarithm of fatigue life for each stress level for tests conducted at 23.9°C (75°F). However, no general ralationship was found to exist that was independent of stress.

CONCLUSIONS

Within the limits of load, mixture, and temperature variables considered in this study, the following conclusions were made.

1. The repeated-load indirect tensile test is suitable for the study of fatigue characteristics of asphalt mixtures. This is important in view of its simplicity and the ease of conducting it.
2. A linear relationship exists between the logarithm of fatigue life and (a) the logarithm of tensile stress (equation 1), (b) the logarithm of stress difference (equation 2), and (c) the logarithm of initial strain, defined as the applied repeated tensile stress divided by the average static modulus of elasticity (equation 9).
3. For the constants K_1, n_1, K_2, n_2, and K_2' of equations 1, 2, and 9, K_1 ranged between 5.65×10^{-17}, and n_1 ranged between 2.66 and 5.19. For stress, K_2 ranged between 11.9 Pa and 276 MPa (3.26×10^5 and 1.90×10^{13} lbf/in^2), K_2' ranged between 513 Pa and 368 MPa (1.41×10^4 to 2.53×10^{16} lbf/in^2), and n_2 ranged between 2.66 and 5.19. The relationships between n_1 and the logarithm of K_1 and between n_2 and the logarithm of K_2 (or K_2') were linear.
4. The optimum asphalt content for maximum fatigue life was slightly less than the optimum for density. The optimum for fatigue life decreased slightly with increasing stress level for the limestone mixtures; however, stress level did not affect the optimum asphalt content.
5. The effect of aggregate type on fatigue life was not important, but, for low stress levels such as those experienced in the field, aggregate type could be a significant variable.
6. Fatigue life increased with decreasing testing temperature.
7. Although the methods of estimating fatigue life on the basis of applied stress or stress difference were not universally applicable, the fatigue life could be estimated on the basis of its relationship with initial strain ϵ_{mix} and stress-strength ratio, which is the ratio of the applied repeated tensile stress to the indirect tensile strength from equations 15 and 16 respectively.
8. Based on the results of this study and previous studies, it was concluded that there is no general relationship between laboratory fatigue life and air void content. Although a relationship may exist for a given mixture, other mixture and construction variables are probably more important, and the effect of air void content on fatigue life will depend on the process by which the air void content is varied.
9. There was a linear relationship between the logarithm of fatigue life and bulk density, but the relationship was dependent on stress.

ACKNOWLEDGMENTS

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The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Dynamic Analysis of Asphaltic-Aggregate Mixtures

Richard W. Stephenson, University of Missouri—Rolla

Ultrasonic probes have been constructed for use in dynamic nondestructive testing of asphaltic concrete specimens. Use of the probe systems was superior to use of their main components alone, the piezoelectric crystals. The probes have the advantages of reducing ringing time, concentrating ultrasound in one direction, elimination of noise due to unshielded connections, and increased durability. The probes were used in a limited testing program. Compression and transverse wave velocities in the specimens tested were measured simultaneously. Sample conditions suitable for maximum wave velocity corresponded to conditions producing maximum strength and minimum voids in the mineral aggregate. A relationship of increasing wave velocity with increasing strength was noted. Wave velocity was observed to increase as the percentage of voids in mineral aggregate decreased. Various elastic constants were calculated from the wave velocities measured. Maximum values of E and G moduli as well as Poisson's ratio corresponded to minimum voids of the test specimens. Relative attenuation measurements were conducted by using water as a standard material. An evaluation of relative attenuation with changes in asphalt content indicated maximum damping at asphalt contents associated with high stability values.

Consideration of the dynamic behavior of a pavement system is becoming a very important part of pavement design practice. It is obvious that the thickness and mix design techniques currently used in the laboratory are not always applicable in the field because the techniques are based on static, destructive test methods. These methods do not accurately model the actual loading conditions occurring in the field.

Testing methods have been sought that evaluate the strength parameters of pavement components as they undergo the actual strain levels that will occur in the field. The parameters of Young's modulus E, shear modulus G, and Poisson's ratio μ are required in certain mathematical models to calculate design thicknesses for pavement layers.

Elastic theory has been used in the above-mentioned models. Because the asphaltic concrete layer of the pavement system conforms reasonably well to elastic theory for the strain levels in question, the use of elastic moduli for this particular layer may be justified. As dynamic loading conditions are reached, the behavior displayed by the asphaltic concrete layer tends to agree more reasonably with elastic theory. The behavior of other layers of the pavement system, such as the base and subgrade, deviates substantially from elastic theory.

The increasing interest in full-depth asphaltic pavements has added to the need for an even more rational design theory. One of the theories being considered involves modeling the pavement as a viscoelastic system. The theory behind this model uses the strength parameters previously mentioned plus a measure of the damping capacity (δ = logarithmic decrement) of the material. Thus a single testing method that evaluates these material characteristics dynamically would be very useful in pavement design.

Numerous testing methods have been developed that attempt to measure the needed design parameters. The test methods most desirable are those that approximate actual field conditions most closely. The ideal test should also be nondestructive so that a single specimen can be tested under a variety of loading conditions.

In this study a testing technique involving the generation and detection of ultrasonic waves in asphaltic concrete samples is used to measure the parameters in question. These values were determined by nondestructively measuring the propagation velocity of ultrasonic waves through the test material. The equipment required in this study was developed at the University of Missouri—Rolla. The main purpose of this study is to further the knowledge of this testing technique as applied to flexible pavement materials. In particular, an ultrasonic probe more suitable for attenuation measurements has been developed.

It was a goal of this study to measure the strength and damping parameters of the surface course of a flexible pavement system. The parameters E, G, μ, and δ depend on the properties of the asphalt-aggregate mixture. The effects of physical properties such as asphalt content, aggregate gradation, voids ratio, and density have, in the past, been evaluated by empirical testing procedures. An attempt has been made to correlate the results of one of these procedures, the Marshall method of mix design.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Paving Mixtures to Meet Structural Requirements.

THEORY OF ULTRASOUND

The theory of ultrasound is very similar to that of audible sound. Sound is the result of mechanical disturbance of a material, i.e., a vibration. In general, three types of waves are generated by a source vibration: compression, shear, and Rayleigh waves.

By using elastic theory, a relationship between the speed of propagation and amplitude of these waves and certain properties of the media through which they are traveling can be determined:

$$E = V_c^2 \rho [(1 + \mu)(1 - 2\mu)]/(1 - \mu) \tag{1}$$

$$G = V_s^2 \rho \tag{2}$$

$$\mu = \frac{1 - 1/2(V_c/V_s)^2}{1 - (V_c/V_s)^2} \tag{3}$$

and

$$\delta = (2.302/n) \log_{10}(A_o/A_n) \tag{4}$$

where

V_c = velocity of compression wave,
V_s = velocity of shear wave,
μ = Poisson's ratio,
E = Young's modulus,
G = shear modulus,
ρ = mass density = γ/g,
δ = logarithmic decrement (attenuation per cycle),
A_o = initial value of the amplitude, and
A_n = amplitude after n oscillations.

In this study, the method of direct transmission of ultrasonic waves was used to evaluate V_c, V_s, and A. The elastic constants were calculated from these measurements.

INSTRUMENTATION AND MATERIALS

Electronic Equipment

The electronic equipment used in this study was previously used, and partially developed at the University of Missouri—Rolla (1). The equipment necessary for conducting the tests includes a pulse generator, an oscilloscope, and two ultrasonic probes (transmitter and receiver).

The design of an ultrasonic probe suitable for dynamic evaluation of asphaltic concrete mixtures is hindered by various complications. The development of a probe that effectively overcomes these complications thus became a major part of this investigation.

Crystal Description

The crystals initially used were thickness expanders manufactured by Gulton Industries. The resonant frequency of these crystals is approximately 308 kHz/s in the thickness mode. They were 2.5 cm (1.0 in) in diameter and were made of lead piezoelectric zirconate titanate (PZT).

The second set of crystals used was the same size, and they were primarily radial expanders (shear mode) manufactured by Clevite. The crystal material was PZT and is identified as PZT-4 by the manufacturer. These crystals had a resonant frequency of 90 kHz in the radial mode and 640 kHz in the thickness mode.

Probe Construction

Previous work on the design of ultrasonic probes for flaw detection in metals has been published (2, 3). The basic characteristics of the probes used in flaw detection coincide with the properties of the probes used for ultrasonic testing in asphaltic concrete. Three of the main features considered in the probe design were (a) crystal selection, (b) mechanical damping of the crystals, and (c) concentration of ultrasonic waves in one direction.

The backing material suggested by Washington and used in this study is a mixture of tungsten powder and casting resin or Araldite. The tungsten and Araldite ratio suggested was 2:1 by weight. Figure 1 shows a section of the probe as built.

The ultrasonic probe has two additional advantages. First, the probe design offers complete electronic shielding. This eliminates noise that often hinders ultrasonic measurements. Second and more practical, the durability of the probe system has been increased.

Sample Preparation

The asphaltic concrete samples used in this study were prepared according to Marshall test procedures. These procedures are given by ASTM Designation D 1559.

The aggregates used in preparation of these samples were obtained from aggregate stockpiles at the University of Missouri—Rolla, which stocks the aggregates (crushed stone and Meramec River sand) for laboratory and research purposes. The sand and crushed stone were sieved into various sizes and recombined to conform to the Missouri State Highway Department gradation specifications for type D asphaltic concrete.

The asphaltic cement used in this study was 85 to 100 penetration grade and was obtained for research from the Shell Oil Company. The specific gravity of the asphalt is 1.010 (4).

Three Marshall specimens at each asphalt content were prepared. Identical combinations of aggregates were used for each specimen. The specimens were compacted by 50 blows of a standard Marshall hammer on each face of the specimen. The compacted specimens had a diameter of 10.2 cm (4 in) and a height of approximately 6.4 cm (2.5 in).

TESTING TECHNIQUES

The ultrasonic testing technique used in this study is known as the direct transmission method. The use of this method was necessary because of the significant amount of scatter associated with the multiple reflections and longer path lengths typical of other methods.

Sample Supporting Devices and Acoustic Coupling

The sample supporting device initially used was a testing frame normally used for unconfined compressive strength soil tests. The proving ring normally used for load measurement was replaced by a much lighter ring constructed from plexiglass. This smaller ring was used to obtain constant pressure between the probes and the test sample.

The main advantage of this sample holding device is its simplicity. Samples can be mounted, tested, and removed quickly and with little effort. The constant pressure obtained is also advantageous in that it minimizes any deviations due to changes in coupling pressures.

As in other studies (4, 5), pressure coupling of the probes to the samples did not always prove adequate. Better quality results were obtained by using an intermediate coupling material. Silicone grease as a coupling medium proved adequate for measurements involving velocity determinations.

Grease coupling was not adequate for tests measuring wave amplitude. Therefore, a second coupling material and sample supporting device were selected. The device consisted of a plexiglass tank constructed such that acoustic coupling could be obtained by using water as the coupling medium. The two ultrasonic probes were positioned and sealed into holes on opposite faces of the tank. Dimensions of the tank were such that the Marshall samples could be placed between the probes without any probe-sample contact. The distance between the receiving and transmitting probes was then kept constant during the tests.

Compression wave velocities measured by using this technique were identical to values from the pressure coupling method. Disadvantages of the method for velocity measurement were apparent. More calculations were involved because the time of wave travel through the water had to be taken into consideration. Transverse waves are not transmitted through water; thus, with this method, measurement of transverse wave velocities is impossible.

The water coupling method did result in more accurate measurements of the amplitude of the transmitted wave. This was expected and resulted partially from the fact that pressure coupling effects were eliminated completely. Better quality signals also resulted from using water as a coupling medium. As opposed to grease, the water quickly and effectively filled all air voids between the probe and sample and diminished the effects of sample roughness on the quality of acoustic coupling.

Velocity Measurements

All major velocity measurements were obtained from the pressure coupling method using silicone grease as a coupling medium. A major advantage was that the initial compression wave and shear wave arrivals could be detected on the same trace.

Velocities of the compression wave and shear wave were easily obtained by dividing the length of the sample by the respective arrival times. This exposed the least accurate measurement in the velocity determinations, the measurement of sample thickness. Thickness measurements were made with a micrometer and were taken a number of times to ensure the highest accuracy possible.

Damping Measurements

Tests involving amplitude measurements were initially conducted by using pressure coupling with grease as a coupling medium. The object of these measurements was to determine the decrease in energy of the wave per length of travel through the specimen, i.e., the attenuation per length.

A sample submersion tank was used to measure relative damping in samples using water as the coupling agent. The ultrasonic probes were sealed into holes in the sides of the tank with a flexible, easily removed sealant. The dimensions of the tank were such that, when the Marshall specimens were placed inside, the sample was centered between the probes. At this point, the tank was filled with water at a constant temperature. A constant pulse width, pulse interval, and pulse voltage were chosen, and testing was begun. The distance between the probes was held constant throughout the testing program.

Amplitude measurements on the sample using water coupling were reproducible. There was, however, a small increase in amplitude of the compression wave as the specimen became saturated. Increased energy transmission with increased saturation of the sample resulted from the decrease of air voids with saturation. This increase in amplitude was easily controlled by either testing the samples as soon as possible after submersion or allowing them to become saturated before amplitude was measured. The results with the least scatter came from testing the samples just after submersion.

Another advantage of this testing technique was using the tank filled with water as a standard for relative attenuation measurements. The distance between the probes was varied, and the amplitude of the compression wave through the water was measured at each setting. From these data, the damping capacity of the water was determined and expressed in volts per centimeter. This value is essentially the slope of a line representing amplitude versus distance between probes. The assumption of linearity of this relationship may be questioned but was of no consequence since the same assumption was made when the specimens were in place. Any nonlinearity would be constant between sample and standard, thus cancelling when relative measurements were considered.

The asphaltic concrete samples were tested, and a value of compression wave amplitude was determined. This amplitude was always much less than the amplitude through water alone. The amplitude loss due to the sample was evaluated and expressed in volts per centimeter of sample length. Relative attenuation of the specimens was then expressed as the dimensionless ratio of amplitude loss per unit length of sample to amplitude loss per unit length of water.

Destructive Testing of Asphaltic Concrete Specimens

Marshall specimens are normally tested destructively according to specifications outlined by ASTM D 1559-73. These testing specifications include a test temperature of 60°C (140°F) and a constant strain rate of 5.1 cm/min (2 in/min). The testing program used in this study conformed to those specifications with the following exceptions.

Because of the large variations in behavior of asphaltic concrete with changes in temperature, it was desirable to hold temperature constant through all phases of this study. Testing was thus conducted on Marshall specimens at room temperature, 23°C (73°F). At that temperature, the Marshall stability values exceeded 44.5 kN (10 000 lbf). These values were above the load limit for any constant strain rate testing machine available. Therefore, a testing machine was used that applied a constant rate of load application.

The testing machine used for the destructive tests was a Tinius Olsen model 200 D. A constant loading rate was chosen that resulted in a strain rate close to 5.1 cm/min (2 in/min). This strain rate was estimated by timing revolutions of a dial gauge attached to the loading head. This constant strain rate was then used for each test.

Limitations in the accuracy of this method of obtaining constant strain rate were considered. The slope of the load-deformation curve and the failure load were known to be highly dependent on strain rate. Because of this, the accuracy of estimating constant strain rate with the constant loading rate mentioned was evaluated.

Five identical asphaltic concrete samples were compacted. One of these samples was tested at room temperature in a normal Marshall testing device. This test was terminated at a 44.5-kN (10 000-lbf) load without completely failing the sample, 44.5 kN (10 000 lbf) being the limit of the machine. Three of the five samples were tested in the Tinius Olsen testing machine at the loading

Figure 1. Section of the ultrasonic probe.

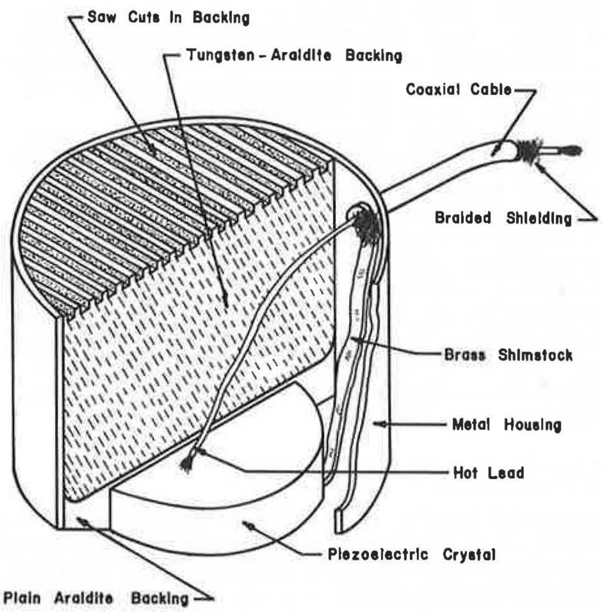


Figure 2. Modified stability versus asphalt content.

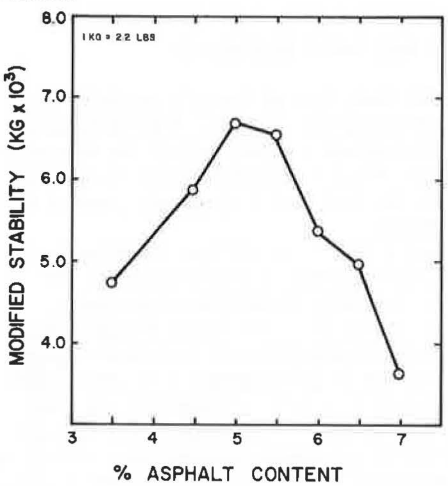
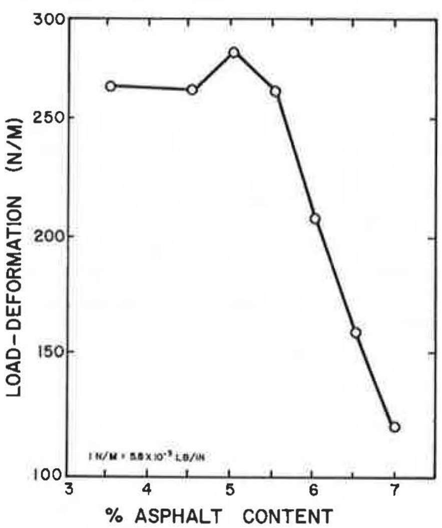


Figure 3. Load-deformation values versus asphalt content.



rate previously mentioned. The initial tangent slopes of these three load-deformation curves ranged from 35.0 to 40.3 MN/m (200 000 to 230 000 lbf/in). The maximum load or failure load of these three tests ranged from 48.9 to 53.4 kN (11 000 to 12 000 lbf). The deviations of these samples seemed insignificant when compared to the test results of the final sample. The fifth specimen was failed at a slower load rate than the previous three specimens. The slope of its load-deformation curve was 28.0 MN/m (160 000 lbf/in) and the failure load was 30.7 kN (6900 lbf). This analysis led to the conclusion that the constant loading rate applied resulted in a reasonably constant strain rate. Any significant deviations in slope were due to variations in characteristics of the specimens rather than strain rate differences.

TEST RESULTS AND DISCUSSION

Ultrasonic velocity tests were performed on the asphaltic concrete samples. Twenty 10.2-cm-diameter (4-in), 6.4-cm-high ($2^1/_2$-in) samples were prepared at seven asphalt contents and identical aggregate gradations.

Density-Voids Analysis

The percentage of air voids (percentage of total volume) decreased from a maximum of 9.68 at 3.5 percent asphalt content to a minimum of 2.21 at an asphalt content of 7.0 percent. The maximum unit weight of 234 g/cm^3 (146.1 lb/ft^3) occurred at 6.5 percent asphalt content. At 5.0 percent asphalt content, the minimum percentage of air voids was measured at 14.8.

Static Strength and Deformation Tests

The results of the modified Marshall stability tests discussed in the previous chapter are shown in Figures 2 and 3. The tests were conducted at a specimen temperature of 23°C (73°F) and at a constant loading rate that resulted in a strain rate of about 5.1 cm/min (2 in/min).

Figure 2 shows the modified stability (Pm) values versus asphalt content. The modified stability value is the failure load (strength) resulting from the testing procedure mentioned above. The maximum stability of 51.2 kN (11 500 lbf) occurred at 5.0 percent asphalt content.

Because of the empirical nature of the Marshall test, the state of stress and strain in the specimen is difficult to evaluate. This difficulty results from the mode of sample failure and the nonuniform stress distribution that occurs on a partially loaded disk. Krokosky and Chen (4), in their viscoelastic analysis of the Marshall test, encountered the same problem. In their study, the above researchers developed a shape factor, Φ, which related the Marshall load-deformation ratio $P(t)/f$ to the static modulus $E(t)$ by the equation

$$E(t) = \Phi[P(t)/f] \tag{5}$$

where

Φ = shape factor,
$P(t)$ = Marshall stability,
f = flow value or deformation corresponding to $P(t)$, and
$E(t)$ = modulus obtained from unconfined tests.

It should be noted that the Marshall stability P and Young's modulus E are given as a function of loading time. According to Krokosky and Chen, Φ seemed to be independent of asphalt content. The variation of this value with changes in aggregate gradation was not determined. This statement and the differences in aggregate gradations used in the two studies prohibited the use of the actual factor used by these researchers. However, the fact that the shape factor did not seem to be affected by changes in asphalt content was useful in this study. From the previous equation it can be seen that, if Φ is constant, any decrease or increase in modulus due to change in asphalt content should result in a relative change in the load-deformation ratio, Pm/f. From this statement it can be concluded that any change in the load-deformation values shown in Figure 3 would be an indication of changes in static modulus.

Wave Velocity Tests

Ultrasonic wave velocities were measured by using the ultrasonic testing technique that specifies pressure coupling of probes to the sample. Both compression wave and transverse wave transmission times were measured simultaneously for each specimen.

Compression Wave Velocity

Figure 4 shows compression wave velocity versus asphalt content. Asphalt content is expressed in percentage by weight of aggregate. Moderate amounts of data scatter occurred. Thus, for purposes of clarity, the average velocities at each asphalt content have been plotted. One possible reason for this scatter could have been differences in particle orientation between samples of equal asphalt content. The values of compression wave velocity obtained in these tests agreed with those obtained by other investigators (5) using similar methods. An optimum asphalt content for compression wave transmission occurred at approximately 5 percent asphalt content. The compression wave velocity at this optimum condition was approximately 3.96 km/s (13 000 ft/s).

Transverse Wave Velocity

A plot of the transverse wave velocities versus asphalt content is shown in Figure 5. Again, average wave velocities for each asphalt content have been plotted. The magnitude of transverse wave velocities also agreed with the values obtained in other investigations. Optimum wave transmission was observed at an asphalt content of approximately 5 percent. The transverse wave velocity occurring at this optimum condition was 1.94 km/s (6370 ft/s).

Wave Velocity Results

Figures 4 and 5 show that optimum wave transmission of both compression and transverse waves existed at an asphalt content of 5.0 percent. Consideration of what conditions are favorable for wave transmission was based on observations of certain density-voids data. The data used were the percentage of air voids and the percentage of voids in the mineral aggregate (VMA). Percentage of VMA is defined as the percentage of voids in the compacted specimen inclusive of air voids and asphalt binder.

Percentage of VMA did undergo a significant change at this optimum asphalt content. A trend of decreasing wave velocities with increasing VMA was apparent. So much scatter occurred that a unique straight-line relationship could not be shown. However, a range of values was indicated that include the major portion of the data points.

In general, optimum conditions for wave transmission seemed to correlate with minimum VMA. Minimum VMA logically indicated a highly dense aggregate network. Minimum VMA thus suggested a maximum ratio of aggregate volume to volume of voids inclusive of asphalt binder. This ratio indicated high acoustic impedance of the sample because air voids and asphalt binder were lumped together as low-impedance materials and the aggregate was a high-impedance material.

Maximum stability corresponded to maximum wave velocities, and a trend of increasing pulse velocity with increasing strength was noted. The dense aggregate network associated with minimum VMA and maximum wave velocity suggested large amounts of grain contact. High amounts of grain contact are in turn indicative of high internal friction and, logically, greater strength.

In summary, optimum conditions for wave transmission correspond with minimum VMA and maximum stability. These relationships are best explained by the resulting dense aggregate network, high grain contact, and high internal friction.

Dynamic Moduli and Poisson's Ratio

As discussed earlier, certain elastic constants (Young's modulus, shear modulus, and Poisson's ratio) can be determined from measurements of compression wave and transverse wave velocities in an elastic material. The assumption of elasticity in asphaltic concrete is obviously questionable. Deviations from this assumption, however, are minimized when low temperature and dynamic loads are approached.

Young's Modulus E and Shear Modulus G

Figures 6 and 7 show the plots of Young's modulus and shear modulus versus asphalt content. The variations of these moduli with asphalt content reflect the behavior of the wave velocities from which they were calculated. This is indicated by the fact that both curves peak at 5 percent asphalt content.

Through the assumption of a constant shape factor as discussed earlier, the dynamic Young's modulus was compared to the measured load-deformation values. Comparing the two curves (Figures 6 and 3) shows that both plots possess a breaking point at 5 percent asphalt content. Because of the large amounts of scatter a definite relationship between E and P_m/f (dynamic E and static E) was difficult to infer. By considering the constant differences in loading times between the two testing procedures a general relationship between static modulus (load-deformation ratio) and dynamic modulus was observed. The dynamic Young's modulus increased with increasing load-deformation ratio. However, this trend did not seem to hold true for the lowest asphalt content.

Poisson's Ratio μ

Poisson's ratio, also calculated from the wave velocities versus asphalt content, is shown in Figure 8. Poisson's ratio increased up to an asphalt content of 5.5 percent. At this point the values began to decrease with increasing asphalt content. This behavior seemed to be the result of the sensitivity of the values of Poisson's ratio to changes in the wave velocities from which they were calculated. A unit change in wave velocity can result in a tenfold change in Poisson's ratio.

Attenuation Measurements

Relative damping or attenuation measurements were made using water as the coupling medium. Water at 23°C (73°F) was used as a standard, and relative damping was expressed as the dimensionless ratio of amplitude loss per unit length of sample to amplitude loss per unit length of

Figure 4. Compression wave velocity versus asphalt content.

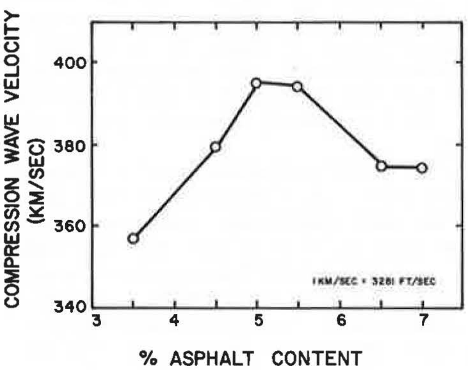


Figure 5. Transverse wave velocity versus asphalt content.

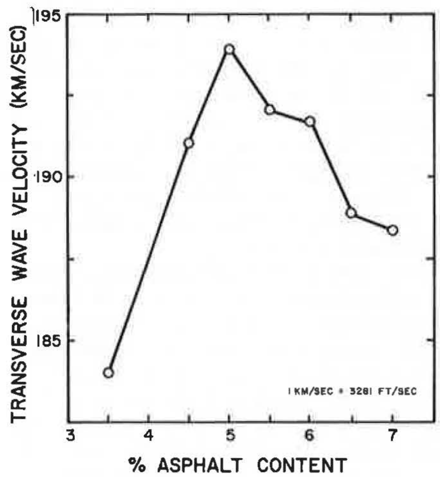


Figure 6. Young's modulus E versus asphalt content.

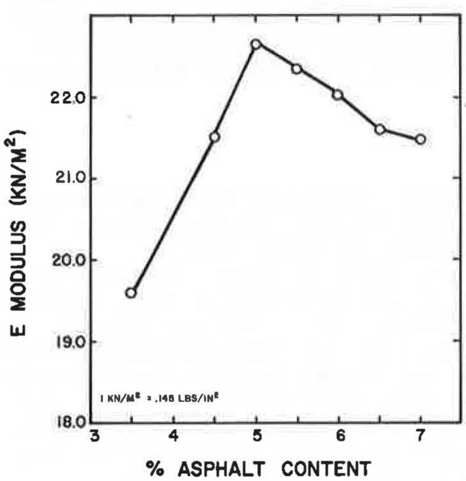


Figure 7. Shear modulus G versus aphalt content.

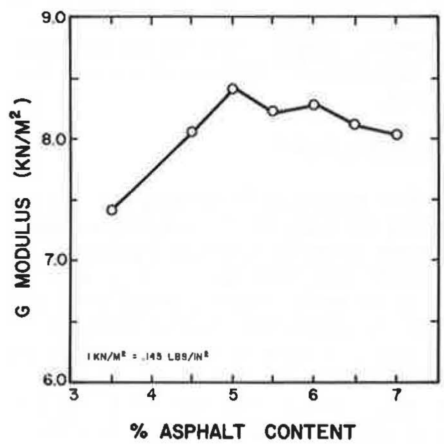


Figure 8. Poisson's ratio μ versus asphalt content.

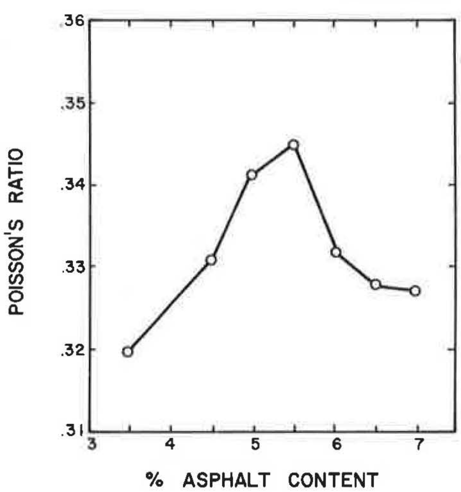
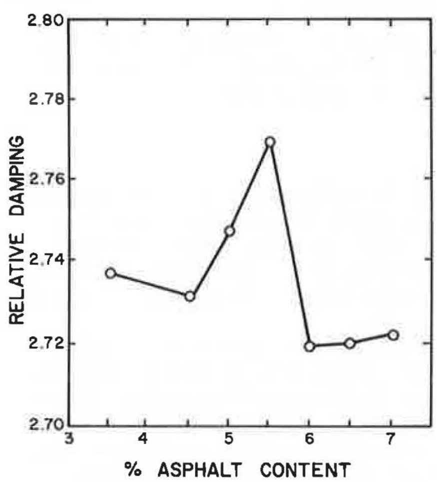


Figure 9. Relative damping versus asphalt content.



water. Significant amounts of scatter occurred, and Figure 9 shows average attenuation versus asphalt content. The damping values shown in Figure 9 indicate that systematic deviations (e.g., linear decrease or increase) of damping with changes in asphalt content did not occur. However, there was a peak in damping at an asphalt content of approximately 5.5 percent. This peak corresponded roughly to the optimum conditions resulting in the peak velocity and peak stability discussed previously. Again, if we ignore the extreme lower and higher asphalt content points, an increase in stability corresponded to increased damping.

The damping values measured resulted from the decrease in intensity of the compression wave as it traveled through the sample being tested. This decrease in energy was due to three major sources: (a) scattering of the wave by reflections at aggregate-particle interfaces, (b) divergences of the wave with distance, and (c) absorption of the wave energy due to heat losses. Inasmuch as this study used samples of uniform size and aggregate gradation, the effects of the first two sources should remain constant. Any major deviations in damping should thus be the result of a change in the magnitude of energy loss due to absorption.

Peak damping of the ultrasonic wave seemed to occur at an optimum condition of high grain-to-grain contact (associated with minimum VMA). Definite conclusions regarding damping were difficult because of the numerous influencing factors.

SUMMARY AND CONCLUSIONS

The key purpose of this study was to develop and construct a probe suitable for velocity and amplitude measurements of ultrasonic acoustic waves in asphaltic concrete specimens. From these values, E, G, μ, and damping properties of the material were evaluated. It was concluded that these properties may be of importance in the dynamic design of asphaltic concrete systems.

The following are the results and conclusions observed from the design, construction, and use of the ultrasonic probes in question. It should be stressed that the test results came from a limited testing program. A larger and more complete testing program should be conducted before definite conclusions can be drawn.

Probe Design

An ultrasonic probe having the following properties was designed and constructed: (a) short ringing time, (b) high electronic shielding, and (c) durability.

Test Results

1. The results of the testing program indicated that values of longitudinal and transverse wave velocities could be measured simultaneously. This concurrent measurement of both velocities on the same electronic trace was a result of the better quality signal produced by the constructed probes.

2. Wave velocities were evaluated versus asphalt content. An optimum condition for wave transmission was noticed at an asphalt content of 5 percent. The relationships of wave velocity with ultimate strength and wave velocity with percentage of VMA were observed. Wave velocity appeared to increase with increasing strength and to decrease with increasing percentage of VMA. Maximum wave velocity, maximum strength, and minimum percentage of VMA occurred at the 5 percent optimum.

3. Various parameters were calculated from the wave velocities according to elastic theory. The curves of dynamic moduli (E and G) versus asphalt content indicated peak values at 5 percent asphalt content.

4. Poisson's ratio increased with asphalt content until a value of 5.5 percent was reached. Above 5.5 percent, Poisson's ratio decreased. The behavior was felt to be the result of material deviations from the assumptions of elasticity and homogeneity necessary for calculations of the value.

5. The values of E mentioned above were compared to load-deformation values (P_s/f) from destructive tests conducted on the Marshall specimens. Changes in load-deformation values were considered indicative of changes in engineering modulus through the assumption of a shape factor (Φ). By this assumption similarities of engineering modulus with the dynamic modulus calculated from the ultrasonic method were observed. A general trend of increasing dynamic Young's modulus with increasing load-deformation values was observed.

6. Relative damping measurements were made by using water as a standard material. Large and systematic deviations of damping with changes in asphalt content did not occur. However, there was an indication of peak damping conditions at an asphalt content of 5.5 percent. The closeness of this peak to the peak corresponding to that of maximum stability suggested a correlation between high damping capacity and high strength. Possible theoretical explanations of this relationship were proposed, but statistical proof from the testing program was limited.

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Field and Laboratory Evaluation of Asphalt-Treated Base and Full-Depth Pavements in Ohio

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This paper reports the results of laboratory and field evaluation of three full-depth asphaltic pavement in Ohio. The performance of these flexible pavements was investigated in the field by using nondestructive dynamic deflection measurements, and the in situ pavement moduli were calculated. The data presented show that the in situ pavement moduli and other deflection parameters can accurately characterize pavement conditions. The field specimens were evaluated in the laboratory under controlled conditions, and the dynamic moduli, tensile strength, and fatigue life were determined. Based on the results of laboratory data and fatigue life simulation, the structural equivalency of asphaltic concrete surface and base course layers was also determined.

During their service lives, bituminous pavements are exposed to environmental and traffic loading conditions that can alter the characteristics of paving materials and reduce anticipated performance. Exposure of bituminous mixtures to environmental elements results in a progressive hardening of the asphaltic binder, which, in turn, effects rigidity of the pavement structure at the expense of the flexibility needed to withstand repeated loading conditions. Such changes in the material characteristics can result in pavement cracking and, subsequently, in reduction of anticipated pavement life.

Various forms of pavement disintegration have been attributed to the durability of asphaltic materials. However, the rate of structural deterioration in the field is doubtless affected by other properties, such as asphalt percentage and type, aggregate type and gradation, loading, and environmental conditions. The structural deterioration due to the combined influence of repeated load applications and adverse environmental effects generally takes two distinct forms: cracking and rutting.

The problem of damage accumulation and the resulting pavement deterioration due to cracking has been studied extensively in the last 30 years, and considerable research has been conducted on fatigue behavior of asphalt mixes. In this paper, the results of an investigation of the durability of flexible pavements relative to crack resistance of bituminous mixtures under actual traffic and environmental conditions are reported. This study, concerned with the improvement of material selection specifications and revision of design and construction practices, was divided into three phases:

1. A laboratory investigation to determine mixture characteristics affecting the crack resistance of bituminous paving mixtures;
2. A field study to determine the variability of material parameters affecting crack resistance and to determine the significance of various material characteristics; and
3. Establishment of guidelines to improve material selection specifications, prediction of field performance based on results of laboratory and field studies, and examination of the applicability of those techniques developed to predict material response.

The results of phase 1 of this study were reported previously (1). The results of phase 2, which involved analysis of crack resistance parameters, evaluation of in situ support conditions, evaluation of paving component materials, and prediction of pavement performance, are presented here.

FIELD INVESTIGATION AND VERIFICATION

Site Selection and Evaluation

Three flexible pavements were selected for detailed laboratory and field evaluation. The following project sites were chosen in consultation with the Ohio Department of Transportation: project 673-70, Ohio-161 in Union County; project 4-71, Ohio-316 in Franklin County; and project 432-70, Ohio-124 in Pike County.

1. Project 673-70. Project 673-70 is a flexible, full-depth pavement constructed on Ohio-161, section 11.24. The total project length is slightly more than 1.6 km (1 mile) and consists of 23 cm (9 in) of asphalt-aggregate base (Ohio mix item 301), 3.2 cm (1.25 in) of asphaltic concrete (Ohio mix item 402), and 3.2 cm (1.25

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in) of asphaltic concrete (Ohio mix item 404).

2. Project 4-71. Project 4-71 is a 4.3-km-long (2.7-mile) flexible pavement in the southeastern section of Columbus and consists of 10.2 cm (4 in) of granular base (Ohio mix 304), 15.2 cm (6 in) of asphalt-aggregate base course (mix 301), and asphaltic concrete layers of 3.2 cm (1.25 in) of mix 402 and 3.2 cm (1.25 in) of surface course mix 404.

3. Project 432-70. Project 432-70 is a full-depth bituminous concrete pavement structure located in Pike County on Ohio-124. Although the total project length is more than 6.4 km (4 miles), the test section for this study was located between stations 970 and 1080. The pavement structure consists of 23 cm (9 in) of bituminous-aggregate (tar) base course (mix 301), 3.2 cm (1.25 in) of asphaltic concrete (mix 402), and 1.9 cm (0.75 in) of asphaltic concrete surface course (mix 404).

Selection of test sites was based on subgrade support values and their variability within each project. A preliminary investigation of the subgrade of potential test locations within each site was carried out by using Dynaflect deflection. The dynamic deflection measurements were then translated into load spreadability characteristics and modulus response of the subgrade soil. Based on results of this preliminary evaluation, sections with good, intermediate, and poor support conditions were selected.

In addition to the preliminary subgrade evaluation, dynamic deflection measurements were carried out on the entire length of the projects during construction. Deflection testing preceded final proof rolling of the subgrade.

The pavement layers were also investigated by using Dynaflect instruments. Upon construction of each intermediate layer, dynamic deflection measurements were carried out at 15.3 and 30.5-m (50 and 100-ft) intervals. Areas showing high deflection or questionable displacement under dynamic loads were tested at closer intervals to approximate zones of potential problems. The results of deflection measurements were then analyzed to obtain an estimate of the moduli of the component layers.

To evaluate the engineering properties of the pavement component layers, field specimens were taken from each pavement layer. During construction, the layers and locations to be sampled were identified and covered with a double layer of plastic sheet. After compaction but before placement of the upper layers, the plastic-covered sections were sawed, and field specimens were retrieved.

Specimen retrieval involved sawing a section 1.2 by 1.2 m (4 by 4 ft) and dividing it into smaller sections. For the asphalt-aggregate base course (mix 301), field specimens were 10.2 by 10.2 by 122 cm (4 by 4 by 48 in). The asphalt concrete binder and surface course (mixes 402 and 404) were sampled together as one layer because the 404 layer was not thick enough to tolerate the sawing and sampling operation. These field specimens were sawed to dimensions of 5.1 by 5.1 by 61 cm (2 by 2 by 24 in). To ensure that specimens were representative of subgrade support characteristics at selected test sections required that undisturbed specimens be obtained from locations identified in the preliminary study.

Moisture-density and temperature variations within the subgrade were accurately evaluated at each site by using in situ instrumentation. The instrument sites were selected during the preliminary site evaluation. Deep-probe nuclear gauge pipes were installed at these sites and properly protected by special covers. Temperature measurements were performed by using thermocouples installed at different depths.

Field Evaluation

The field investigation and site evaluation programs used in this study were initiated during construction and continued after completion of the paving operation. The program included routine daily visits to each project site during construction and seasonal visits afterward as needed.

The structural condition of the pavement was evaluated in the field by using nondestructive dynamic deflection measurements. The dynamic deflection equipment (Dynaflect) induces a constant dynamic load of constant frequency and, by using geophones or sensors, detects displacements in the layered system. From the slope and magnitude of the measured displacement profile, the characteristics and support capacity of each test layer can be determined. The results of dynamic deflection measurements can then be analyzed to obtain an estimate of the pavement layer moduli. These analytic procedures are reported elsewhere (2).

The theoretical concepts and mathematical relationships involved in the use of dynamic deflections have been discussed in more detail (2) and will not be repeated here; but, briefly, theoretical and experimental data have indicated that each test layer can be evaluated by using the following performance parameters:

w_1, the maximum deflection or first sensor reading, is a measure of the pavement's structural characteristics and support conditions;

$SCI = w_1 - w_2$, the surface curvature index, is predominantly an indicator of the structural conditions of the surface layer;

$BCI = w_4 - w_5$, the base curvature index, measures the base support conditions;

$SP = \left(\sum_{i=1}^{i=5} w_i \Big/ 5w_1\right) 100$, spreadability or average deflection as a percentage of maximum deflection, measures the load-carrying capacity and stiffness ratio of the pavement structure; and

w_5, the fifth sensor reading, has been shown to be an indirect measure of subgrade support moduli.

Dynamic deflection measurements were carried out on the subgrade and pavement structures. The subgrade support moduli were calculated by using either the Ohio elastic modulus programs or suitable graphic procedures (Figure 1).

The measured deflection profiles were found to be dependent on the structural and material variables as well as prevailing environmental conditions. The construction of successive layers of asphaltic pavement components increases the support characteristics of the pavement component layers and results in a reduction of maximum deflection and surface curvature index. The spreadability or percentage of SP, as a function of the moduli ratio E_1/E_2, is affected by the relative stiffness of the pavement structure as well as its support characteristics E_2.

Figure 2 shows typical variations of the pavement deflection profile for the component layers of project 673-70. As noted, the construction of successive layers of 301, 402, and 404 resulted in a reduction in the maximum deflection and surface curvature index. The spreadability of all three layers also increased, indicating a stiffening or increase in slab action.

Figure 3 shows the maximum deflection measurements for project 432-70. This project exhibited the greatest magnitude of variability in deflection when compared to the other two pavement systems.

The results of dynamic deflection measurements were

Figure 1. Subgrade modulus values for project 4-71.

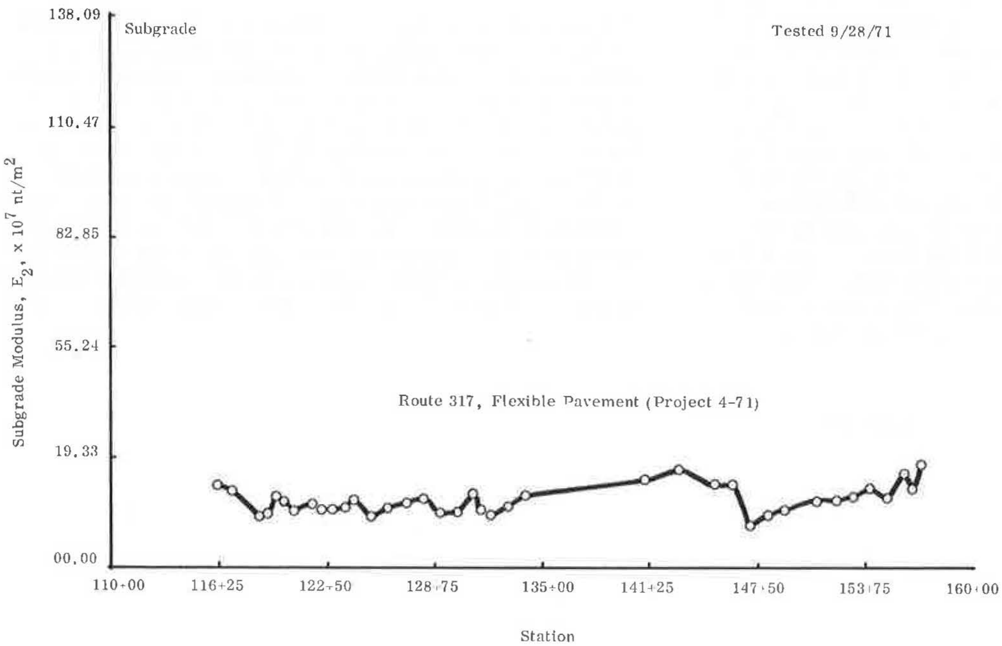
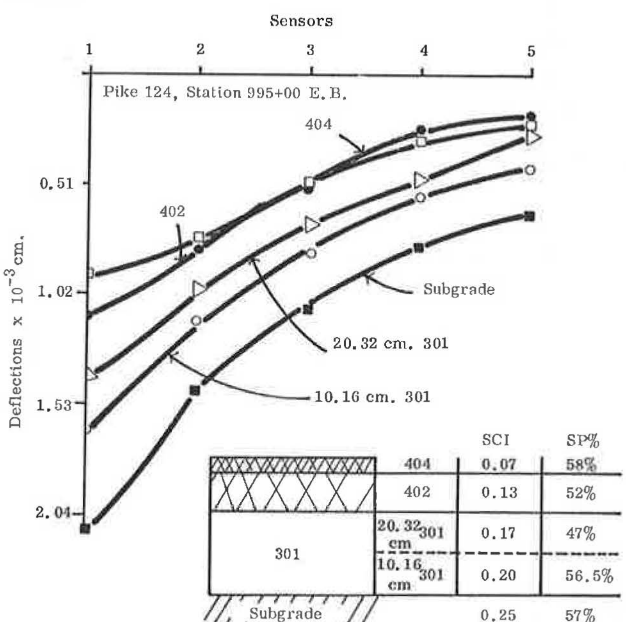


Figure 2. Variation of pavement deflection with layered structure stiffness.



translated into effective moduli of pavement component layers, corresponding to the average pavement temperature at which tests were performed. Figure 4 shows the average or effective moduli of the asphalt layer E_1 of project 432-70. The time and temperature of testing appear to have significant influence on the calculated modulus of the structure; the pavement modulus E_1 varies with the station site, time of measurement, and climatic conditions prevailing during the test. This temperature dependency of pavement response is also reflected in the measurements of deflection, SCI, and spreadability. Figure 5 shows the variation of deflection profile with air temperature. It was found that lower pavement deflections undoubtedly correspond to lower temperatures.

Figures 6 and 7 show variations of spreadability and SCI with air temperature. The data in these figures represent measurements conducted on the three projects at different times of the year. Figure 8 shows the variation of average pavement modulus, as calculated by the Ohio elastic moduli program, in terms of the average pavement temperature. That figure also shows the modulus-temperature relationship of asphaltic concrete surface course (404), as reported by other researchers. The lower values of in situ moduli are due to the average moduli effects of mixtures 404, 402, and 301. The variability of these measured moduli (Figure 8) not only shows the temperature dependency of the material but also reflects the inherent variations in the engineering properties of asphaltic mixtures and the pavement structure response at different stations.

LABORATORY EVALUATION OF FIELD SPECIMENS

Material Preparation

The laboratory study was initiated to evaluate the material characteristics of the field specimen. This investigation was primarily aimed at determining those material characteristics that affect the performance of flexible pavement structures. The laboratory study was also aimed at verifying the applicability to field-compacted specimens of fracture and crack resistance concepts developed previously.

Field specimens procured at selected sites of each project included both core and beam specimens. The cores were taken at the full depth of the pavement so that all layers of 301, 402, and 404 mixtures would be included. The beam specimens, on the other hand, were sawed from the lower and upper layers of 301 and from layers 402 and 404 together.

These field samples were sawed to prepare specimens of uniform dimensions, as needed for laboratory testing. The cylindrical specimens were first sawed at each end and then capped with capping compound.

The beam specimens were prepared for both fatigue and dynamic modulus testing. The specimens used for dynamic modulus testing were obtained from a 402-404 composite mix layer. The asphaltic beams were then sawed into beams of separate 404 and 402 layers, each 5.1 cm (2 in) wide and 30.5 cm (12 in) long, with a variable thickness (approximately 1.8 to 2.5 cm or 7 to 10 in). Before these 402 and 404 beams were sawed and separated, each field specimen was X-rayed to determine the boundary between the 404 and 402 layers.

The beam specimens used for fatigue testing required less preparation because the beams were cut to almost the required dimensions in the field. However, most specimens had to be trimmed to obtain smooth testing surfaces for proper seating of the beams.

Testing Procedures

The evaluation of material characteristics was carried out by using the loading function, the pattern and frequency of load-time history, and testing procedures discussed previously. Dynamic moduli of the 301 cylindrical specimens were determined by using half-sine dynamic loads. A static load of 89.3 N (20.1 lbf) was used for seating, and a maximum dynamic load of 1340 N (300 lbf) or 172 kPa (25 psi) was used. Vertical deflections were measured by using a Roving LVDT where readings are taken at four obliquely opposite points of the loading head.

The elastic dynamic modulus of the asphaltic concrete beams of layers 402 and 404 was determined by using a

Figure 3. Maximum deflection at stations on project 432-70.

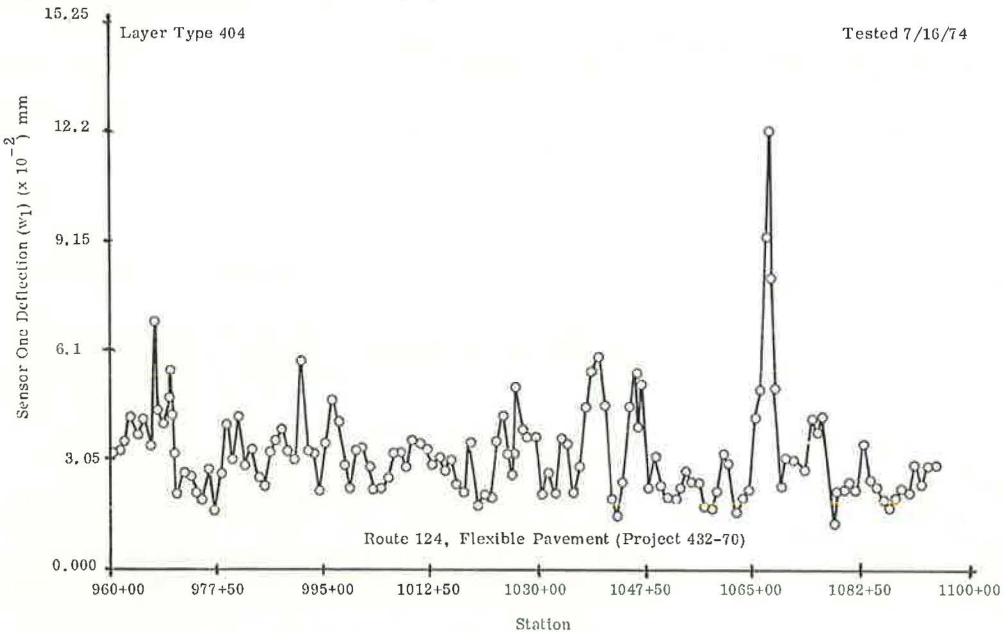
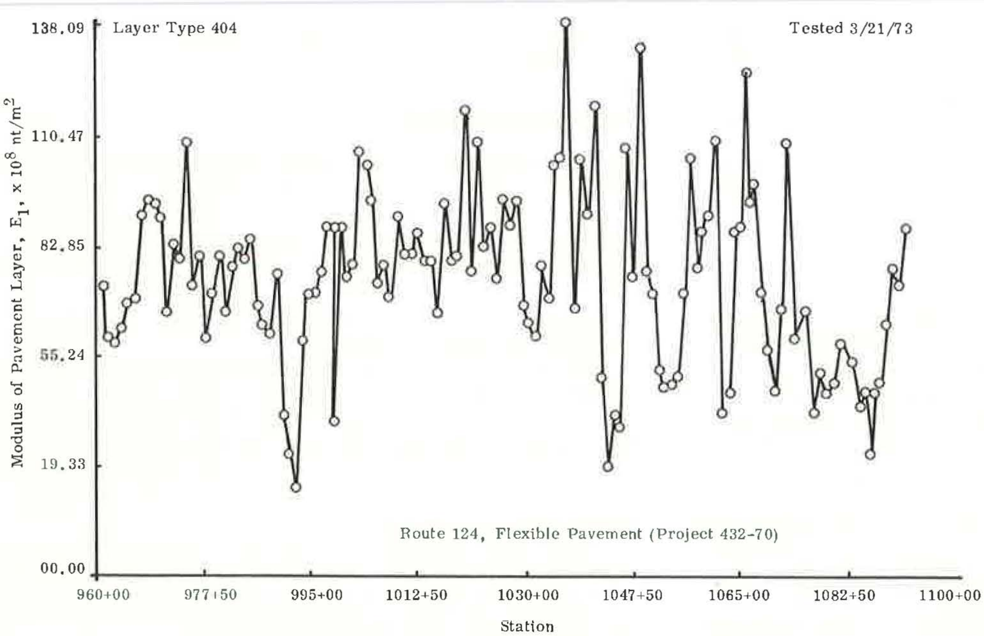
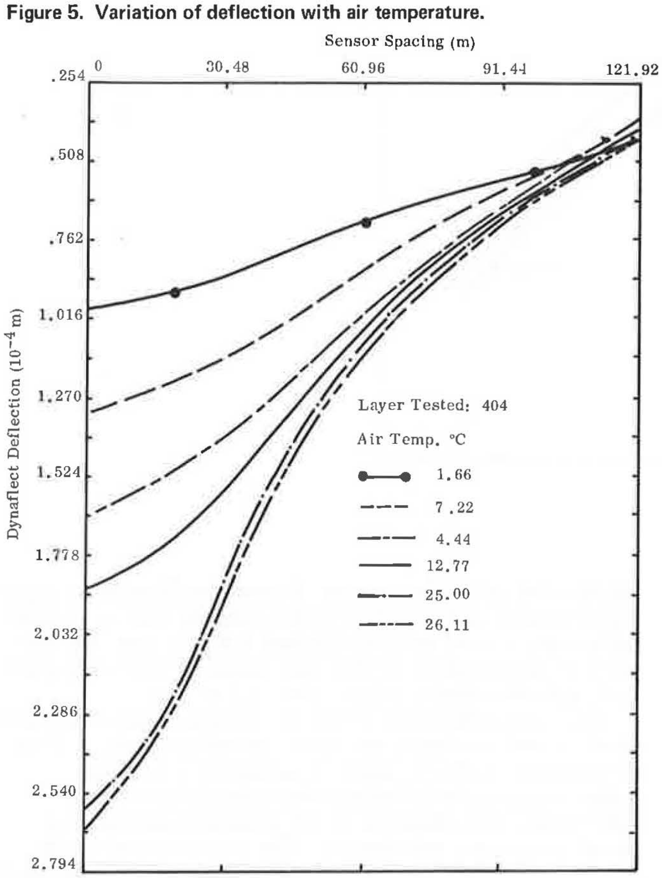


Figure 4. Equivalent pavement layer modulus.



simply supported beam setup in which the specimen was subjected to a half-sine dynamic load superimposed on a small static load of about 4.5 N (1 lbf). The load was applied at the midspan position, and the resultant deflection was measured with an LVDT placed under the midspan of the beam.

Figure 5. Variation of deflection with air temperature.



Fatigue testing of the field specimens was performed in a manner similar to the procedure discussed earlier. All field specimens of layers 402-404 and 301 were supported on an elastic foundation having a modulus value of 1034 kPa (150 lbf/in^2. The 301 beams, because of their length, were tested on an elastic foundation 13 by 10.2 cm (5 by 4 in) in cross section and 1.2 m (4 ft) long. For these specimens, the load was applied with a 10.2 by 10.2-cm (4 by 4-in) steel loading head resting on a cushion of hard rubber.

In addition to the modulus and fatigue life determinations, measurement of indirect tensile strength σ_y was also included in the experimental program.

Analysis of Results

The experimental procedures followed in this study included determining the dynamic modulus of asphaltic layers, indirect tensile strength, and expected fatigue life. The detailed experimental and analytical procedures for determining these parameters have been presented in the literature and will not be repeated here.

The results of moduli determinations of mixtures 402, 404, and 301 at different test temperatures indicate that the variations of these mixture moduli with test temperatures follow previously recorded trends. These laboratory results also reflect the variability of pavement layer moduli with project, site, and specimen station. These moduli vary at different stations as well as within each test location (Table 1).

The results of indirect tensile strength σ_y tests conducted on the field specimens and at various temperatures are shown in Figure 9.

Analysis of fatigue of field specimens was carried out in accordance with test procedures and specifications reported previously (1). As has been reported, the fatigue crack growth law for asphaltic systems can be simplified by a power law equation given by

$$dc/dN = A K^n \quad (1)$$

The results of fatigue experiments and evaluation of fatigue life parameters A and n are discussed by

Figure 6. Variation of SCI with air temperature.

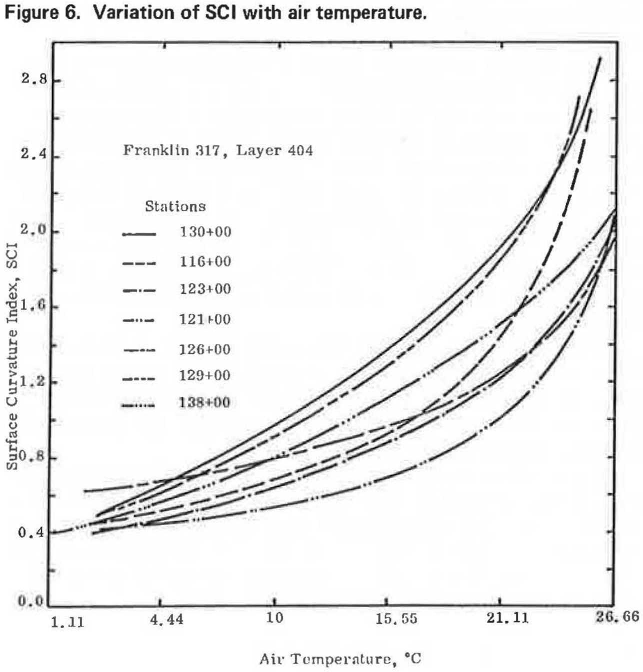


Figure 7. Variation of spreadability with air temperature.

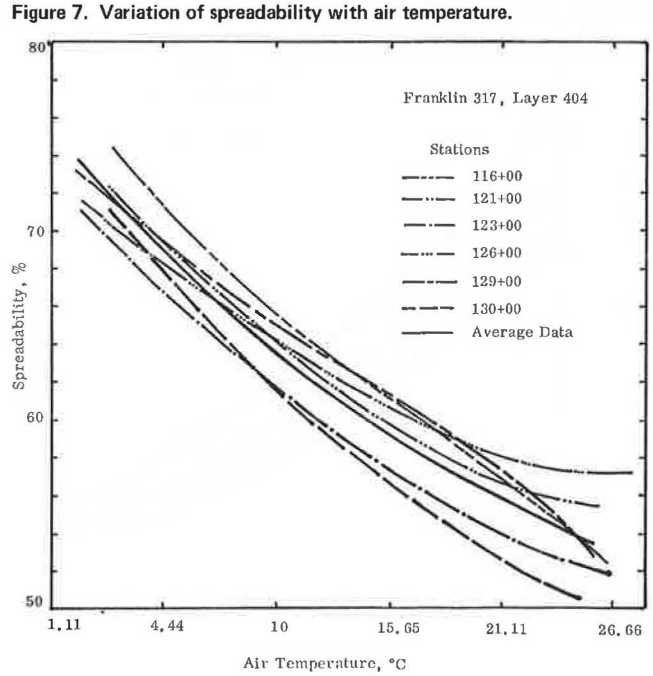


Figure 8. Modulus-temperature relation.

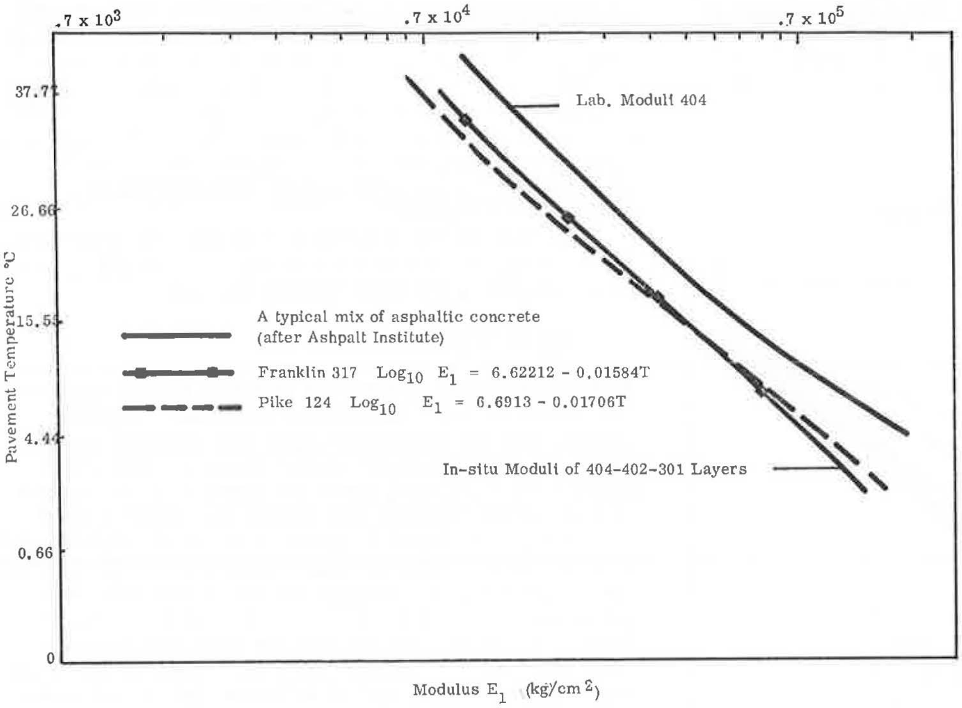
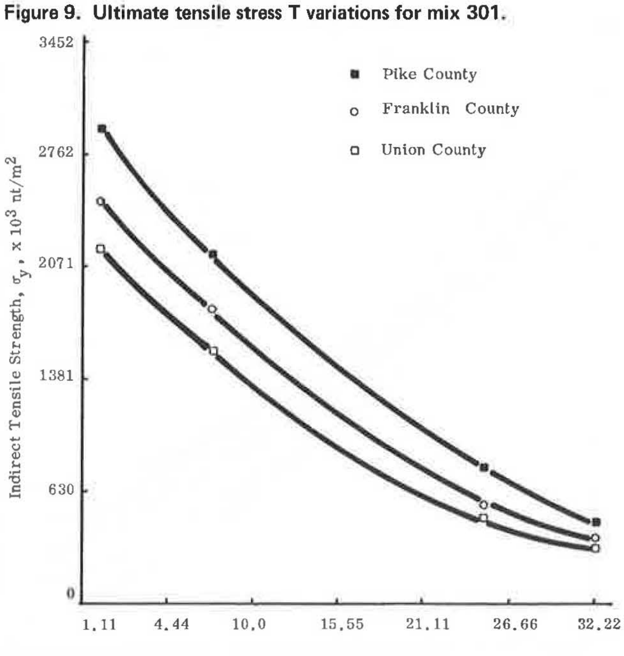


Table 1. Summary of fatigue parameters A and E*.

Project	Type of Asphaltic Sample	A Mean	A σ	E* Mean	E* σ
Franklin 317	402-404	0.743	0.352	3473	206
Union	402-404	3.131	1.885		
Pike	402-404	12.419	5.84	349	357
Union	301	5.189	2.914	204	101
Pike	301	3.129	2.383	250	249
Franklin 317	301			201	197

Note: A-values are given in billionths; E*-values are given in thousandths.

Figure 9. Ultimate tensile stress T variations for mix 301.



Majidzadeh (3). The results of experimentation on composite beams of 402-404 and 301 indicate that n, in most instances, varies between 2.0 and 4.0. In fact, the results of experiments on 301 specimens at room temperature indicated that n ranges from 2.1 to 3.0.

The observed lower values in n might imply that there exists a retardation of the crack growth process. This retardation of crack growth is probably due to the asphalt-aggregate interface characteristics of field specimens, as compared to the idealized and homogeneous laboratory specimens. The reduction in crack growth rate might also be attributed to the fact that cracks do not always propagate in a straight line and the crack surfaces change their direction. It has been reported that the parameter n is affected by the magnitude of the local stress field ahead of the crack tip.

The fatigue constants A and n exhibit variations that are due to a number of factors, such as differences in testing procedures by different people, differences in inherent material characteristics between test samples, and temperature dependence. Because of the variability of A and n, fatigue life prediction is a tedious task and often leads to cumbersome evaluation procedures. The simplification of the power law, assuming that n is a constant and independent of testing procedures, could result in significant improvement in the evaluation of fatigue life.

Therefore, an attempt was made in this investigation to develop a unified method of analysis of crack growth data where error is minimized and to carefully evaluate the simplified as well as the generalized versions of the fatigue crack growth law.

The review of various theories and analytical procedures indicated that, depending on the degree of mathematical and statistical complexity desired, various procedures could be used for determining the parameter n. For example, the crack growth resistance of asphaltic mixtures could be represented by

$$\text{Rate of crack growth} = A_1K^{n_1} + A_2K^{n_2} + A_3K^{n_3} \quad (2)$$

where A_1, A_2, and A_3 and n_1, n_2, and n_3 can be obtained by statistical analyses of data.

Although the use of equation 2 might provide a higher correlation coefficient on the other hand, it might also hinder the practical application and usefulness of the fracture mechanics approach as a design and material selection tool.

However, because it is desirable from a practical viewpoint for the crack propagation law to be as simple as possible, the use of a single power law with constant n given by

$$\text{Crack growth rate} = A\,K^n \qquad (3)$$

has been investigated. In equation 3, A is a material constant and can be represented by an average $\bar{A}$ and a standard derivation σ. The parameter n is also a material constant. The results of analysis indicate that for mixtures 402 and 301, having relatively large aggregates, n could be selected as equal to 2.0. Table 1 gives the fatigue constants A and E^*.

ANALYSIS OF PAVEMENT LAYER EQUIVALENCY

The structural layer equivalency of asphaltic mixtures was investigated by conducting stress analysis by using elastic layer programs. The analysis consisted of a pavement design simulation using varying thicknesses of 404-402 and 301 mixtures. The tensile strain at the bottom of the pavement layer was calculated and the equivalencies were determined by using equal strain concepts. Then the equivalencies were determined by using equal fatigue life concepts.

These analyses indicated that equivalency concepts also depend on the pavement structural arrangement. When the pavement design is implemented by placement of layer 404 or 301 directly on the subgrade, the coefficients are different. In such a case, it has been shown that 10.7 cm (4.2 in) of 404 mix, when constructed on a subgrade, is equivalent to 14.5 cm (5.7 in) of 301 mix. That is, 2.5 cm (1 in) of 404 mix is equivalent to 3.4 cm (1.35 in) of 301. In terms of AASHTO structure coefficients, this corresponds to $A_1 = 0.45$ for mix 404 and $A_2 = 0.33$ for mix 301.

However, when a pavement is constructed with a sequence of 301, 402, and 404 layers, the structural coefficients differ from the above case. The analysis of equivalency values for various structural arrangements indicated that, when 2.5 cm (1 in) of 404 and varying thicknesses of 301 are used, the structural equivalency value is found to be 2.5 cm (1 in) of 404 equivalent to 2.8 cm (1.1 in) of mix 301.

Similarly, when the fatigue life was used as a criterion for structural equivalency, 23.4 cm (9.2 in) of 404 was equivalent to 26.7 cm (10.5 in) of 301, or 2.5 cm (1 in) of 404 was equivalent to 2.8 cm (1.1 in) of mix 301.

As a result, it might be concluded that, for mixtures studied in this investigation, 2.5 cm (1 in) of mix 404 may be taken as equivalent to 2.8 cm (1.1 in) of mix 301. Therefore, if it is assumed that the AASHTO structural coefficient of asphaltic concrete surface base is $A_1 = 0.45$, then the coefficient of asphalt-aggregate base is 0.40, a ratio of 1.0/1.13.

SUMMARY AND CONCLUSIONS

In this paper the structural response and performance characteristics of three selected flexible pavements are investigated. The study included a detailed field evaluation of pavement deflection response under Dynaflect dynamic loadings and a laboratory evaluation of field core and beam specimens. The following conclusions are drawn:

1. Dynamic deflection measurements and analysis of deflection profile can provide an accurate in situ characterization procedure for flexible pavements.
2. The in situ dynamic modulus of pavement layer E_1, as determined by using deflection profile, agrees reasonably well with the laboratory values of moduli of asphaltic concrete.
3. Laboratory data on dynamic modulus, indirect tensile strength, and fatigue life exhibit variabilities associated with material type, test sites and stations, and environmental factors.
4. A multilayer elastic analysis of structural layer equivalency of pavement layers indicates that 2.5 cm (1 in) of asphaltic concrete surface course is equivalent to 2.8 cm (1.1 in) of asphaltic base course layer 301.
5. The structural equivalency analysis using the fatigue failure mode indicates that 2.5 cm (1 in) of asphaltic concrete surface course 404 is equivalent to 2.8 cm (1.1 in) of asphaltic concrete base course 301.

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Virginia's Experience With Open-Graded Surface Mix

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The Virginia Department of Highways and Transportation has used the bituminous open-graded, porous friction course (PFC) since 1972 to resurface hazardous locations and roadways needing routine resurfacing. A before and after accident study of one location revealed a significant decrease in wet pavement accidents after it was paved with the PFC. Skid tests have revealed good to excellent skid resistance on all sections of the PFC. Some problems have been encountered with flushed spots in the pavement, minor raveling, deformation under severe traffic, damage from spilled fuel, and dirt in air voids. Most of these problems have been resolved, and future use will be determined by the long-term durability of the mix.

In Virginia, constant attention is directed toward the development, construction, and maintenance of pavement surfaces that provide good skid resistance. Most of the bituminous pavement surfaces on the state's highly traveled highways are dense-graded, although the surface texture may vary because of differences in aggregate gradations among the mix types used. When properly constructed, pavements having a dense-graded mix in the surface course usually give good skid resistance; however, to prevent wet weather skidding accidents requires a mix that will provide excellent drainage of surface water. For example, good surface drainage is a paramount consideration where thick surface water films are prevalent because of geometrics and where speed and traffic maneuvers create uncontrollable hydroplaning skids.

Because of the apparent need for an antihydroplaning mix, an open-graded, porous friction course (PFC) that had been used by several states was investigated. The PFC is designed as an open mix with interconnecting voids that provide drainage for heavy rainfall. The rainwater drains vertically through the PFC to an impermeable underlying layer and then laterally to the edge of the pavement.

In addition to providing high skid resistance during rainfall, the PFC, as compared to other mix types, reportedly (1)

1. Reduces spray and splash,
2. Enhances the visibility of pavement markings,
3. Reduces nighttime surface glare in wet weather,
4. Reduces tire-pavement noise, and
5. Allows use of thin layers and a minimum of material.

Virginia's experience has verified these advantages; however, noise measurements have not proved that porous friction course reduces tire-pavement noise. There was no appreciable difference in measured noise levels for a dense-graded mix and the PFC in a series of tests performed on a limited number of Virginia pavements.

FIELD INSTALLATIONS

In 1972, PFC was used to resurface two short test sections. A 0.72-km (0.45-mile), two-lane section was placed on US-60 in central Virginia by using a lightweight aggregate in one lane and a crushed marble aggregate in the other. Some problems were encountered in controlling the aggregate gradation for the test section, which is often the case with small quantities of materials, particularly when a new gradation is used. The variability in the gradation caused some areas to be overly dense and to lack the desired permeability.

A second section, 0.6 km (0.4 mile) long on a four-lane roadway, was placed on US-23 north of the Virginia-Tennessee state line. Several wet pavement accidents had occurred at this location, and a before and after accident study was made to determine the safety benefits of the mix.

These two sections represented the initial experience with the PFC for both state and contractor personnel, although at least one Virginia contractor had worked with an open-graded mix in North Carolina. Initially, contractor personnel were rather dubious about the PFC and the required construction techniques; however, experience has tended to dispel their doubts.

In 1973 the PFC was placed on at least one section in each of the state's eight highway districts. As experience with PFC increased, problems were eliminated and workmanship was improved. Approximately 8.2 Gg (9000 tons) were placed in 1973, some of it on a section

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of Interstate highway.

In 1974, approximately 22.7 Gg (25 000 tons) were placed, including 16.3 Gg (18 000 tons) on Interstate highways. Most of the Interstate sections on which the PFC was placed carry traffic volumes of about 13 000 vehicles per day on four lanes; however, a section on the Richmond-Petersburg Turnpike carries approximately 46 000 vehicles per day on six lanes.

MIX DESIGN

The first of the two sections placed on US-60 in 1972 had the following design gradation:

Sieve	Percentage Passing	Sieve	Percentage Passing
12.5 mm	100	No. 8	10 to 30
9.5 mm	90 to 100	No. 16	0 to 20
No. 4	30 to 50	No. 200	0 to 4

The asphalt contents were 7 and 11 percent for the crushed marble aggregate and lightweight aggregate mixes respectively. The optimum asphalt content was selected by visual observation of trial mixes in the laboratory. The mixing temperature was specified to be between 104 and 132°C (220 and 270°F).

An 85 to 100 penetration asphalt cement was used for both the mix and the tack coat. A heavy tack coat [0.32 dm^3/m^2 (0.07 gal/yd^2)] was used to ensure an impermeable underlying layer. When applying the tack coat, the contractor had to raise the temperature of the asphalt very high before it would flow uniformly, which the contractor considered objectionable. A cationic emulsion tack coat was used on the US-23 job. Approximately 0.5 dm^3/m^2 (0.1 gal/yd^2) of residual CAE-2 (CRS-2) asphalt was applied on US-23, and, because of the ease with which it was applied and the apparent success achieved, a decision was made to allow either 85 to 100 penetration asphalt cement or cationic emulsion to be used on future jobs. Although the US-23 job was successful, during the long haul from the plant to the job the asphalt drained to the bottom of the truck, and the result was several fat spots in the pavement.

The mix gradation specified in 1973 was very similar to size No. 8 stone (ASTM D 692), which is usually available at all quarries. As before, the optimum asphalt content was selected through visual inspection of laboratory samples, including some observations of asphalt drainage from mixes placed on glass plates. The 1973 and current specifications are given elsewhere (4). The mixing temperature was specified to be 93 to 124°C (200 to 255°F); a temperature of approximately 107°C (225°F) was found preferable.

Very few changes were made in the specifications for 1974; however, the Federal Highway Administration (FHWA) design procedure (2) was examined with the idea of possibly improving the mix and refining Virginia's design procedure. The FHWA procedure consists basically of a gradation design and an asphalt content design. This procedure was used to develop designs for six mixes that had previously been used in pavement resurfacing, and the designs were compared to those actually used (3). Pavement performance was used to evaluate whether the correct design had been used. The design asphalt contents from the FHWA procedure were consistently lower than those used in the field, and only one aggregate type was found to require an asphalt content significantly different from that selected by the sample observation method. However, the design procedure was useful for certain aggregates. The method was adopted and is now being used to estimate optimum asphalt content; however, because Virginia's PFC gradation is rather fixed, i.e., No. 8 stone, the gradation design was not considered useful.

PROBLEMS

Construction

The CAE-2 (CRS-2) tack coat, on occasion, tended to stick to truck tires, then drop off, and puddle, causing flushed spots. However, when the emulsion has been allowed to cure properly, it has proved to be satisfactory.

Problems have been experienced when the mixing temperature has not been properly controlled. An unsatisfactory pavement was obtained when the temperature of the mix was too high. Excess asphalt migrated to the bottom of the trucks and, when deposited in one area, formed flushed spots in the pavement.

Long hauls tended to magnify the problems of asphalt drainage in truck bodies and excessive cooling and lumping of the mix on the exterior surface. It is advisable to limit the plant-to-paver time lapse.

Maintenance

No major maintenance problems have been experienced, although some minor ones have arisen. A section on I-81 consisting of crushed gravel appeared to be raveling approximately 6 to 12 months after construction. To determine future maintenance possibilities, a diluted cationic emulsion was applied to a short length of the section to arrest the raveling. The emulsion penetrated the PFC and will, it is hoped, provide an asphalt coating to prevent further raveling. The treated and untreated sections will be observed and compared.

At a toll booth on the Richmond-Petersburg Turnpike, on the lanes carrying heavy truck traffic, some deformation and densification are beginning to show approximately 1 year after construction, which indicates that possibly this mix should not be used under very severe traffic conditions.

The PFC is more susceptible to damage from spilled fuels than is a dense-graded mix, because spillage penetrates the mix and the underlying surface. Fuel spilled from an overturned truck on the Richmond-Petersburg Turnpike severely damaged a short section of pavement. This situation requires removal of the damaged mix, usually with a heater planer, and replacement with fresh mix.

Voids of cores from nine projects ranged from 21 to 32 percent, and these projects have maintained good drainage characteristics. However, examination of cores from the Richmond-Petersburg Turnpike revealed reduced voids in wheel paths and between wheel paths, apparently from the accumulation of dirt. The effectiveness of the PFC may be reduced if the environment is very dirty and dirt penetrates the voids.

RESULTS

Durability

One of the initial primary concerns with the PFC was the service life and general durability because the open-grading would expose the asphalt cement and thus make it subject to early oxidation and weathering. To prevent early deterioration due to oxidation and weathering, the mix is designed with thick asphalt films and should contain as much asphalt as possible.

Inasmuch as the oldest PFC in Virginia has been in service for only 3 years, it is not possible to reach any conclusions on its long-term durability at the present time.

Stability

Rut depths were measured on 11 sections of open-graded surface mix and two sections of regular dense-graded surface mix. The open-graded mix had a rut depth range of 1 to 3 mm (0.04 to 0.12 in) and averaged 2 mm (0.07 in).

The dense-graded surface mix, which showed no visible rutting, had a rut depth of 3 mm (0.10 in), and the dense-graded section with some visible rutting had a measured rut depth of 7 mm (0.26 in). Therefore the rut depth and stability of the open-graded mix are equivalent to those of a good dense-graded mix.

Skid Resistance

Table 1 gives the average skid numbers for some of the PFC pavements. Although not shown in Table 1, some of the sections have been tested at regular intervals and have not shown any appreciable change in skid resistance.

All of the sections have good skid resistance, and the first section, the one with lightweight aggregate on US-60, has a skid number of 72. This section has low traffic volume, but it is expected that similar results would be obtained under high traffic volumes.

Most of the sections have been placed for routine maintenance purposes; however, some were designed to reduce wet pavement accidents by enhancing the road's skid resistance. The section on US-23 had a high incidence of wet pavement accidents, which are believed to be caused by excessive surface water films. The excessive films were believed to be a result of the pavement geometry and surface type, so it was thought that a porous surface would alleviate the problem.

A survey of accidents 1 year before and 1 year after installation of the PFC revealed a significant reduction in wet pavement accidents. In the year before installation, 39 percent (7 of 18) of the accidents occurred during wet weather, but during the year after installation only 17 percent (2 of 12) of the accidents occurred during wet weather, which is considered normal. In this instance the benefits of the PFC are evident.

Other sections have been placed in locations that had experienced skidding accidents, but it is too early to evaluate their effectiveness.

FUTURE PLANS

Evaluation of the PFC sections will be continued. Such courses probably will be used for some routine maintenance overlays and at locations where wet pavement accidents are a problem. The durability of the mix will probably determine the long-range use.

Table 1. Average skid numbers of typical PFC pavements, spring 1975.

Route	Year Constructed	Vehicles per Day per Lane	Skid Number[a]
US-23	1972	900	66
US-23	1973	900	66
US-460	1973	2650	64
US-460	1973	2200	60
US-60[b]	1972	450	57
US-60[c]	1972	450	72
US-60	1973	2000	54
US-301	1974	3700	54
US-17	1973	2950	58
US-301	1973	1050	60
US-29	1973	4300	60
I-81	1973	3400	68
I-81	1974	3250	60
Va-42	1973	1800	66
US-50	1973	1300	51

[a]Predicted car values from ASTM skid test trailer.
[b]Marble aggregate.
[c]Lightweight aggregate.

ACKNOWLEDGMENTS

When Virginia began using the PFC in 1972, North Carolina highway engineers who had considerable experience with the mix were consulted and provided invaluable assistance. The original specifications and construction procedures were patterned after those of North Carolina.

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Discussion

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The author has pointed out asphalt drainage problems during transit from the mixing plant to the job site. This can possibly be avoided if the mix temperature is established on asphalt viscosity considerations. FHWA has recommended that the target mixing temperature be in the range that will correspond to asphalt viscosities of 7 to 9 cm^2/s (700 to 900 centistokes) (5). The Pennsylvania Department of Transportation has designed and placed seven experimental sections of open-graded asphalt friction course from 1974 to 1975. According to our experience, the viscosity range should be 11 to 15 cm^2/s (1100 to 1500 centistokes). We did not encounter any drainage or crusting problem within this range, even though the mix was hauled more than 32 km (20 miles) in some cases.

The mix gradation specified by Virginia in 1973 consists essentially of coarse aggregate only. However, FHWA has recommended that at least some fine aggregate be used to provide a choking action for the stabilization of the coarse aggregate fraction. Use of coarse aggregate alone is not only economical, but also should result in improved permeability. It is hoped that the continual evaluation of these experimental sections will indicate whether the fine aggregate is needed from the standpoint of stability and durability.

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Author's Closure

The temperature-viscosity curves of AC-20 asphalt cements obtained in the past under viscosity specifications were examined. They showed little variation, so the required mixing temperature was rather constant. We felt that it was preferable to specify the mixing temperature, which has been established by experience. Some of the drainage problems may be attributed to the fact that the mix is designed to have thick asphalt films and that sufficient fines to soak up excess asphalt were absent.

We do not use the fine aggregate for the choking action, and there have been no apparent problems with stability under normal use. As I indicated, rut depths for the open-graded mix are comparable to those for a well-designed dense-graded mix.

Asphalt-Rubber Stress-Absorbing Membranes: Field Performance and State of the Art

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Pavement cracking and subsequent reflection through overlays are directly associated with pavement deterioration and maintenance problems. During the early 1960s a process was developed using a composition of 25 percent ground tire rubber reacted with 75 percent hot asphalt. This process has been used in full-scale field projects in the form of stress-absorbing seal coats and interlayers and as waterproof membranes. Results from approximately 3200 lane-km (2000 lane-miles) of construction clearly show that this basic elastomeric material performs as a waterproof membrane and has a high capacity to absorb direct tensile, flexural, and shearing stresses. The paper reviews the performance of the asphalt-rubber seal coats placed since 1967 and the present state of the art of design and construction. It further reviews the potentials of stress-absorbing membranes placed to (a) prevent reflective cracking of overlays placed over both flexible and rigid pavements; (b) provide bridge deck protection; (c) control differential movement of existing pavements constructed over expansive clays; (d) provide economical construction for low-volume roadways; and (e) provide improved elastomeric sealing of cracks and joints.

A pavement is no sooner built than deterioration begins, and most pavements usually require several major corrective measures and possibly complete reconstruction in a lifetime. The combined actions of traffic, sun, rain, frost, and soil moisture create multiple problems. These problems are all associated with stresses and related strains within the pavement structures, which are manifestations of forces created by the major factors noted.

The problems associated with these strains in pavements may be generally divided into the following three general categories:

1. Cracking of the pavement surface as a result of repeated flexural stresses,
2. Cracking of pavement surface by direct tensile stresses, and
3. Differential vertical movement between adjacent sections.

To state the problem basically, either the elastic properties or the tensile strength of the pavement structure is insufficient in responding without fracture to the forces acting on the pavement.

The most common and costly problem of pavement distress is fatigue cracking of asphalt pavements (1, 3, 9, 12). This type of cracking is due to repeated deflection of the pavement of as little as 0.25 mm (0.01 in) under wheel loads (1). In the advanced stage, it is manifested by an alligator or chicken-wire cracking pattern accompanied by the release of small pieces, which later creates potholes, particularly during wet weather.

Obviously, the problem here is due to a lack of flexibility (6, 10) or elasticity in the asphalt pavement component required to respond, without cracking, to the resilient nature of the substructure under load. This resilience in the substructure is due to the entrapment of air within soil pores and held there by the capillary pressure of moisture between the soil grains. The effect is most pronounced in fine-grained soils and results in a pneumatic or air mattress effect under load.

The second type of pavement distress, direct tensile stresses, occurs immediately after construction of rigid pavements and may be accentuated at later dates. In flexible pavements, direct tensile stress occurs usually after some period of service although it may occur in as few as 3 months; in rare cases, it does not occur during the normal design life of the pavement. The compromise solution to this specific problem is to provide weakened planes in rigid pavement so that the cracks occur at designated intervals, in a straight line; this eliminates potential spalling and provides a reservoir for sealant to prevent entry of moisture and incompressibles into the joint. Although this solution delays the problem, it is common knowledge that it does not eliminate it.

The direct tensile failure in a flexible pavement system is of a different nature and is generally caused by one of the following conditions:

1. The viscoelastic properties of the asphaltic concrete are such that at low temperatures its elastic limit is exceeded;
2. Oxidation and other chemical actions decrease the elastic limit of the asphaltic concrete, and the pavement cracks as a result of repeated temperature re-

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versals in a fatigue mode of failure; or

3. Subgrade and subbase volume changes due to fluctuating moisture and temperature reflect stresses on the asphaltic concrete in excess of its tensile strength.

The third category of pavement distress, differential vertical movement, occurs after the pavement has cracked because of flexural or tensile failure modes. This type of distress is important in that it directly causes poor riding quality. It is also probably the critical factor in the initial reflection in overlays of existing cracks in underlying old pavements.

Efforts to solve these types of pavement deterioration with the limited use of elastomers date back for a number of years. The direction of these efforts has been to modify characteristics of the asphalt, or the asphaltic concrete, to reduce the effects of temperature change on the stiffness and elasticity of the structure (2). Unfortunately, costs have usually limited the percentage of elastomer to an amount inadequate to ensure sufficient elastic protection, particularly in cold weather when the pavement is most brittle.

SOLUTION TO THE PROBLEM

This paper discusses the development of a system that can be used to correct the problems of a failed pavement and that can also be applied as a preventive measure in the design phase.

In the early 1960s, a concept was developed for overcoming the problem of fatigue cracking. Field trials were initiated in the winter of 1964-65, and results were reported to the Highway Research Board in January 1966 (8). The concept was based on a composition consisting of 25 percent ground recycled tire rubber reacted with asphalt at a high temperature to form a thick, jellied material with good elastomeric properties. To keep costs down and still get the benefit of a high rubber content required that this material be spread in a thin membrane over the cracked surface, covered with chips to provide a wearing surface.

The first full-scale field trial of this material took place in January 1967 on the main taxiway of the Phoenix Sky Harbor Airport (7). This pavement had been designed for DC-3 aircraft and was developing severe alligator fatigue cracking under Boeing 707 and similar aircraft. This application, though crude, served so well that by spring of 1968 the equivalent of some 24 lane-km (15 lane-miles) had been placed at the airport and on the streets of Phoenix.

The Arizona Department of Transportation became interested in this concept for preventing reflection of alligator cracking and, in summer of 1968, constructed an asphalt-rubber stress-absorbing seal over about 4 km (2½ miles) of severely fatigue cracked pavements on frontage and access roads of the Black Canyon Freeway, I-17 (4).

The general appearance of the application was poor because proper equipment and application techniques for the viscous asphalt-rubber composition had not yet been developed. However, with the passage of time and traffic, the unevenness of the application smoothed out until the appearance became reasonably acceptable. Most important, and in spite of construction difficulties, it proved the efficacy of the asphalt-rubber material in preventing reflection of alligator cracking. Today the surface is in excellent condition and shows only minor crack reflection after 8 years of hard service. Thermal or shrinkage cracking did reflect through the asphalt-rubber seal coat, but the cracks were narrow and have not spalled. Reflection of the extensive alligator cracking has not occurred (Figure 1).

The Arizona DOT and other public agencies placed several other projects between 1968 and 1971 by using the asphalt-rubber system and had success in controlling the fatigue cracking problem and variable results in overcoming construction problems, as application techniques were gradually perfected. One of the most notable projects treated with asphalt-rubber during this period was the main street of Tolleson, Arizona (US-80), in the summer of 1969. The street was severely deteriorated with extensive fatigue cracking and innumerable potholes. It actually appeared as though reconstruction would be imperative inasmuch as a conventional overlay of sufficient thickness to control the cracking could not be used because of curb height restrictions and drainage conditions.

The town authorities decided to try an asphalt-rubber treatment. First, the numerous potholes and wider cracks had to be filled. After 6 years of service, this project has required no maintenance and shows only a few minor reflective cracks and no evidence of spalling. Figure 2 shows the condition of the pavement before the treatment and now.

STATE OF THE ART

A major improvement in the construction process, which improved the reliability of obtaining good workmanship and appearance, was introduced in spring of 1971. This was the solvent dilution process whereby a small quantity of kerosene was introduced into the viscous, reacted asphalt-rubber composition. This dilution temporarily reduces the viscosity of the composition and improves application uniformity and initial wetting of the chips. After not more than 2 hours, the viscosity of the composition increases to approximately its original high viscosity (2). This effect of the solvent dilution is apparently the reaction of the mixture to selective absorption of solvent by the rubber particles. Figure 3 shows laboratory tests confirming this behavior in the viscosity of the material.

As a result of this development and the encouraging performance of the Tolleson and Black Canyon Freeway projects, the Arizona Department of Transportation decided to more fully evaluate the concept. Other public agencies also increased their use of the asphalt-rubber system.

Three projects placed by the Arizona DOT have played an important role in the development of the current specifications, procedures, and evaluation of the capabilities of the system. These projects are commonly known as the Aguila, the Flagstaff, and the Minnetonka.

Aguila Project

The Aguila project consisted of 9.6 km (6 miles) on US-60 and 9.6 km (6 miles) on Ariz-71 immediately east and north respectively of Aguila.

The pavements on these highways were in an advanced stage of fatigue, and plans called for a 150-mm (6-in) overlay to restore the structural integrity. The typical condition of the pavement is shown in the foreground of Figure 4. Insufficient funds were available for an overlay, so the asphalt-rubber seal coat was placed as an interim treatment. Because of the heavy traffic volume, especially on US-60, which temporarily carried I-10 traffic, synthetic lightweight cover material was used to reduce windshield breakage (13).

The seal coat was placed during July 1972, under extreme climatic conditions with ambient temperatures of approximately 45°C (113°F). In an effort to prevent aggregate rollover and pickup under the heavy traffic and high temperature, the asphalt-rubber was modified to use a harder asphalt (85 to 100 penetration) and 5 percent kerosene diluent. Traffic was held off the newly placed portions of the seal for as long as possible (at least 2 hours) and then directed through at low speeds. In spite of these precautions, some pickup occurred. The problem was finally corrected by applying a light layer of sand [1 to 1.5 kg/m^2 (2 to 3 lb/yd^2)] before the final rolling. This procedure is included in the present specifications.

On this project the use of abutted longitudinal joints (as opposed to lapped joints) was also specified, as is the case in normal seal coat procedures. The lateral flow of the binder materials in the asphalt-rubber system is insufficient, so longitudinal joints must be lapped. These laps iron out under traffic and pose no permanent problem. This procedure is also part of the current specification. This asphalt-rubber seal coat is serving extremely well, although the cracking in the pavement was so pronounced that the cracking pattern can be observed in the uncracked seal, as shown in the background of Figure 4 and in Figure 5.

Flagstaff Project

Arizona is basically separated into two climatic zones: The high and cold Colorado plateau runs somewhat diagonally from northwest to southeast across the northern part of the state, and the largely semitropical Sonoran zone is in the warm southern part. The major asphalt-rubber projects that have been discussed were placed in the warmer south although small test installations had been placed in the cold northern area and in other states as early as 1966 (11).

In August 1973, the Arizona DOT placed its first major asphalt-rubber treatment in the northern part of the state (5). This 16-km (10-mile) project is located on US-89 approximately 8 km (5 miles) northeast of Flagstaff and is at an elevation of more than 2200 m (7200 ft). The winters are cold with minimum temperatures as low as -40°C (-40°F) and frost depths of 1 m (3 ft) in shady areas.

The existing surface was severely "alligatored" with fatigue cracking aggravated by frost susceptible base course that caused severe breakup during thawing periods. It was very rough and virtually impassable in the spring of 1973, and it was necessary to place a thin cold-mixed patching course on most of the project to fill the many potholes. In August the asphalt-rubber treatment was placed by using volcanic cinders as cover aggregate.

Some pickup was experienced on this project for a short time as the chip size was small and the asphalt-rubber application rate less than optimum. Normally, a 10-mm (3/8-in) nominal size is used, but these chips had a nominal size of approximately 6 mm (1/4 in). This project has performed excellently without reflection cracking and with zero maintenance to date. The present contrast between the treated and untreated surfaces at the north end of the project is shown in Figure 6.

Minnetonka Project

In 1971 the Arizona DOT participated in the National Experiment and Evaluation Program on Prevention of Reflective Cracking in Overlays. A 21-km (13-mile) section of I-40 extending east from Winslow to Minnetonka was chosen for the studies. The project included 26 experimental sections, three of which used asphalt-rubber—one placed as a stress-absorbing seal coat and the other two placed between the overlay and the asphaltic concrete friction course as a stress-absorbing membrane interlayer (SAMI).

The final inspection of the project was performed in the spring of 1975, and the report is in preparation. Conclusions are that the asphalt-rubber SAMI was highly effective in preventing reflection of all types of cracks, including fatigue, shrinkage, and differential vertical strain, and the asphalt-rubber seal coat was effective primarily in controlling fatigue cracking (Figures 7, 8, and 9). As a result of this project and other evidence, in 1975 the Arizona DOT implemented the use of the SAMI as standard procedure for all overlays less than 100 mm (4 in) thick that are placed over pavements where cracking is a problem. The cost of this inclusion is absorbed by reducing overlay thickness.

LABORATORY RESEARCH

To date virtually all of the knowledge of the asphalt-rubber systems has been developed by trial and error on numerous small-scale experiments and full-scale field installations. Although there has been only a limited amount of laboratory work, this work has provided valuable insight on how and why the asphalt-rubber systems have performed in such an excellent fashion.

In most previous work the term rubberized asphalt has been used, but it may be more accurate to describe this process as rubber reacted by asphalt. Laboratory testing has shown that the minus No. 25 to plus No. 40 crumb rubber, when mixed with asphalt and held at a temperature of 190°C (375°F) for approximately 20 min, swells to approximately twice its original volume.

In addition to swelling, the rubber particles become much softer and more elastic. This phenomenon is the result of chemical and physical reactions between the resins (aliphatic oils) in the asphalt and the rubber. The extent of this reaction can be modified by manipulating the composition of the asphalt (Figure 10) and is also obviously subject to change by variations in the gradations and the amount of rubber crumbs. Figures 11 and 12 show that gradations of rubber finer or coarser than No. 10 to No. 40 do not react to produce desired characteristics.

Low-temperature fracture characteristics of various mixtures have been evaluated by extensive laboratory testing. When fracture results are plotted on a graph of temperature versus percentage of rubber, the resulting curve shows a sharp change in slope of a rubber-asphalt ratio of 1:5. Field experience has indicated that a ratio of 1:3 is required to ensure the desired long-term elastic qualities.

The individual rubber particles appear to coalesce with time and react in strain as continuous fibers. To date, no attempt has been made to duplicate this long-term behavior in the laboratory.

Given these phenomena and the high percentage of rubber used in the system, it is postulated that the asphalt is modifying the elastic properties of the rubber rather than the rubber modifying the characteristics of the asphalt. This difference from previous research into the concept of asphalt and asphaltic concrete using low percentages of rubber is basic and must be recognized in concept if the behavior of the asphalt-rubber system is to be understood.

It was previously noted that a major improvement in quality of construction was achieved by adding kerosene (or a high boiling point diluent) to the reacted asphalt-rubber mixture. The addition of kerosene caused a sizable reduction in viscosity, which resulted in improved application and wetting of the cover material. Most important is that the decrease in viscosity is temporary and that in 1 to 2 hours the mixture regained its initial viscosity (Figure 3). Inasmuch as this increase in viscosity occurs long before evaporation of the diluent occurs, this reaction is puzzling and of interest. It is

Figure 1. Asphalt-rubber seal coat on right, after 8 years.

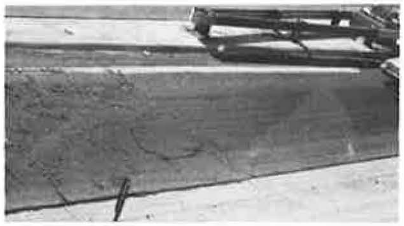

Figure 2. Main street of Tolleson before application of asphalt-rubber (July 1969) and currently (1976).

Figure 3. Solvent dilution phenomenon.

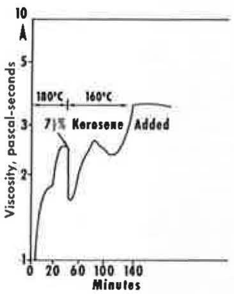

Figure 4. Asphalt-rubber seal coat on Aguila project in background.

Figure 5. Crack pattern under asphalt-rubber seal.

Figure 6. Treated and untreated sections on Flagstaff project.

Figure 7. Minnetonka project in March 1971 before treatment.

Figure 8. Minnetonka project after asphalt-rubber seal coat.

Figure 9. Minnetonka project after asphalt-rubber membrane interlayer.

Figure 10. Increase in volume of rubber by extender oils.

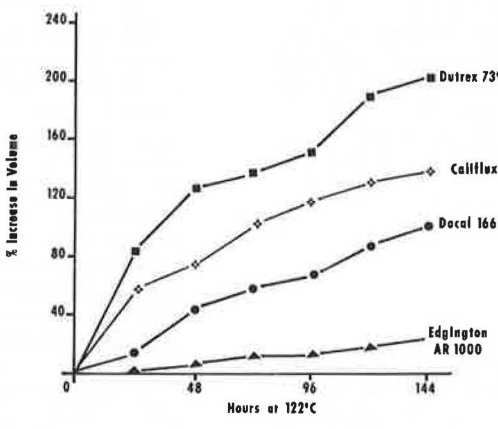

Figure 11. Solvent dilution phenomenon with fine rubber.

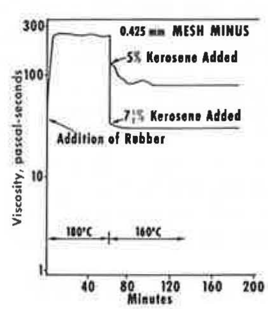

Figure 12. Solvent dilution phenomenon with coarse rubber.

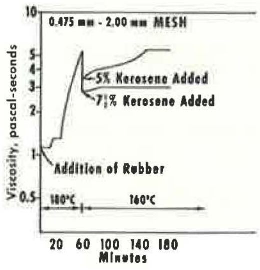

Figure 13. Temperature susceptibility versus viscosity.

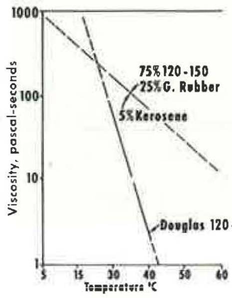

Figure 14. Asphalt-rubber seal coat placed directly on clay loam.

Figure 15. Madison Avenue in Phoenix before and 3 years after treatment.

Figure 16. I-17 partially treated with asphalt-rubber.

theorized, but not confirmed, that the increase in viscosity is caused by the diluent slowly penetrating the asphalt-rubber interface and that the subsequent process of selective absorption of the diluent by the rubber results in the increase in viscosity. Moreover, it should be noted that the viscosity characteristics of the initial mixture and the final mixture appear to be unchanged. It is emphasized that the final mixture is much less temperature susceptible than the original asphalt (Figure 13), and it is also noted that the temperature-susceptibility curve is flatter for the kerosene-diluted mixture than for the asphalt-rubber alone.

The need for and potential benefit of formal research of the asphalt-rubber systems are apparent. Procedures that provide major benefits have been developed, but we know little of the basic character of chemical and physical reactions occurring and even less of rational engineering procedures to optimally utilize the asphalt-rubber system. Arizona DOT, in cooperation with the FHWA, has initiated a research program to explore the basic chemical and physical processes and to develop engineering criteria in relation to pavement deflection, elastic modulus, overlay thickness, and other physical characteristics.

OTHER APPLICATIONS

Although the discussion so far has been limited to placement of the asphalt-rubber system to control reflective cracking, the potential of asphalt-rubber systems extends far beyond that of reflective crack control.

During the course of developing the design and construction procedures for asphalt-rubber for seal coat applications, the city of Phoenix and the Arizona DOT constructed experimental projects containing more than 200 experimental sections. The performance of these projects indicated that the asphalt-rubber not only serves as an elastomer but also provides an impervious membrane.

An interesting application was made directly on the subgrade of a street in Phoenix in April of 1971. The street (55th Avenue north of Clarendon Street) had previously been paved half the width on the west side with a standard asphalt pavement. An irrigated alfalfa field, at a slightly higher elevation than the street, occupied the east half. The alfalfa field was bladed back, and the clay loam soil (plasticity index of 18 with 80 percent passing the No. 200 sieve) was spongy, cloddy, and difficult to compact. Approximately 25 mm (1 in) of disintegrated granite was spread to smooth the surface. To control the dust the surface was lightly primed and allowed to cure for 1 week. Asphalt-rubber and chips were then applied to the surface of the roadway and ditch slope next to the alfalfa field. Although the street carries an ADT of 6000 vehicles (predominately cars), it is serving well to this time and has had only minor maintenance (Figure 14). This application is, of course, a modification of the membrane-encapsulated pavement structure as developed by the Corps of Engineers. The performance of this project and discussions with Corps of Engineers research staff have indicated that the lower membrane is not necessary where adequate drainage and a low water table exist. A 16-km (10-mile) experimental project to develop reliable data on moisture contents, density, and strength parameters of this concept is now in the planning stage by the Arizona DOT. The potential savings on roadways with low traffic volumes is obvious.

Asphalt-rubber has been used and tested as a waterproof membrane on several bridge decks to date. The Arizona DOT performed standard wet condition resistivity tests and found substantially infinite resistivity through the surface of these decks even though they all consisted of a single application. A double application is recommended for this use, however.

The realization that the system was also serving as a moisture barrier led to other considerations. A major problem exists with high-volume-change clays and shales in northern Arizona. The montmorillonite clays derived from the Chinle formation shales are the principal offenders, although there are some limited areas in the northeast portion of the Mancos formation that has also been troublesome to the states north and east of Arizona. If subgrade moisture can be maintained in a uniform condition, volume change will not occur.

In these semiarid ranges, the primary sources of moisture are surface runoff and moisture generated through the process of hydrogenesis in open-graded base course materials. Brakey and Carroll showed that, for new construction, a membrane placed over the subgrade effectively controls the moisture content and subsequent expansion of the clay (14). Effective procedures have not been developed, however, for existing highways constructed over expansive clays. In 1975, major overlays were scheduled for 38.6 km (24 miles) of I-40 where expansive clays had caused serious heaving and cracking of the pavements. A SAMI was planned to control reflective cracking of the overlay, and it was decided to extend the membrane to cover the shoulders in an effort to reduce the problem of subgrade expansion. There is little question that this membrane will prevent entrance of surface moisture. It is not known, however, whether the membrane will prevent development of water through hydrogenesis or whether redistribution of existing moisture will result in excessive differential swelling of the clay. This section of pavement has not been in service for a sufficient time to arrive at any conclusions.

The use of subgrade materials to provide structural base course led to a review of problems connected with stabilized subgrades. Everyone in highway and street engineering is familiar with the normal shrinkage cracking inherent in soil cement and similar treated bases, which reflects through thin bituminous surfaces. An asphalt-rubber treatment appears to offer an answer to this crack reflection. In the city of Phoenix, many kilometers of soil cement pavement were placed with a 38-mm (1½-in) bituminous surfacing and an asphalt-rubber treatment. The earliest of these projects was placed in 1972, and reflective cracking from the soil cement has not occurred to date.

Before the interlayer concept was tested and developed, one weakness of the asphalt-rubber seal coat application was that rideability of the pavement was not significantly improved. Many distressed pavements are rough, as well as cracked, and in need of a leveling course. In 1971, the city of Phoenix began to combine the asphalt-rubber in some form with a leveling course, and several experimental sections were placed on the main taxiways at Sky Harbor International Airport.

After a few months it was apparent that the only test section controlling reflective cracking was an asphalt-rubber composition flushed into a lean open-graded asphalt mix. This result and the experience with soil cement interested the Arizona DOT in the possibility of using the process as an overlay for leveling rough concrete paving. A cooperative project was arranged with the city of Phoenix, and a test section was placed in summer of 1973 over a very old, rough, cracked pavement subject to heavy industrial truck traffic. The basic elements of this test were essentially the same as the successful one at Sky Harbor International Airport. After 3 winters the experimental section has exhibited only minor hairline reflective cracking in isolated spots (Figure 15).

The Arizona Department of Transportation extended the experiment to the concrete pavement of the Black Canyon Freeway (I-17), which runs through the city of Phoenix and which over the years had developed considerable roughness and an undesirable level of skid resistance. A 1.8-lane-km (1.14-lane-mile) test section was placed in the spring of 1974 and contained most of the elements involved in the 1971 Sky Harbor International Airport tests plus a control section consisting of standard open-graded asphalt concrete finishing course. All of the overlays averaged 19 mm (0.75 in) thick, so that reflection cracks from the joints in the underlying concrete pavement, if they were going to occur, would occur quickly. Further, the section carries 35 000 ADT and was located at the end of the concrete pavement where maximum movement occurs.

An unplanned development occurred in connection with the application of the flushed asphalt-rubber composition. Two parallel applications were made, one at 2.0 dm^3/m^2 (0.5 gal/yd^2) and one at 3.3 dm^3/m^2 (0.85 gal/yd^2). The work was done at night under difficult visual conditions, and, when the distributor started the second application, it inadvertently lapped the preceding application for approximately 100 m (33 ft) before a correction was made. This resulted in a total application 5.2 dm^3/m^2 (1.38 gal/yd^2) on the lap and 0.0 dm^3/m^2 on the outside edge. With ordinary asphalt this would have resulted in severe bleeding on the lap, but with the asphalt-rubber only blackening of the surface occurred. This confirms previous observations that overapplications of this material are not critical. On the edge that was skipped because of the lap, reflection cracks occurred within 9 months at every joint but stopped where the asphalt-rubber flush began. The contrast is quite dramatic.

The end results of these experimental projects are similar to those on the 1971 airport project in that the underasphalted open-graded mix flushed with the asphalt-rubber composition is the only one that shows no reflective cracking to date (Figure 16).

The elastomeric properties of this material make it a natural for crack sealing, and experimental sections that have been in service for more than 2 years are showing excellent service. The development of procedures that permit modification of mixture to achieve optimal characteristics for climatic and crack width conditions has led to a large experimental program to further develop materials and procedures. Two districts representing extremes of climatic conditions have been supplied equipment and materials and are testing the various compositions and application techniques. From this work it is anticipated that prepackaged mixtures will be developed for specific applications.

SUMMARY

The beneficial use of recycled tires in an asphalt-rubber system has been demonstrated and tested. When placed as a seal coat (SAM), the system controls reflection of fatigue cracks and is an effective alternate to a major overlay or reconstruction. When placed as an interlayer (SAMI), the system effectively controls reflection of all cracks.

The performance of the system as a water barrier has been demonstrated on bridge decks. Potential applications and experimental projects have been described to evaluate its potential use

1. For membrane-encapsulated subgrades in lieu of

base courses,

2. For control of expansive clay subgrades under existing highways,

3. As a thin overlay for renewing without crack reflection and skid resistance on portland cement concrete pavement, and

4. As an effective crack sealer for maintenance.

In an era of tight money and energy conservation, the use of asphalt-rubber membranes offers a very attractive incentive. In addition, the use of recycled tires in highways provides a solution to a major problem of waste material disposal. The magnitude of this problem is rather overpowering when it is realized that there are more than 2 billion scrap tires currently in storage or littering the landscape and that each year this total increases by an estimated 200 million.

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