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Bituminous Materials, Mixtures, and Performance

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Contents

ASPHALT CEMENT CONSISTENCY
S. C. Shah 1

EVALUATION OF PROPERTIES OF ASPHALT-EMULSION-TREATED MIXTURES BY USE OF MARSHALL CONCEPTS
Ahmed Atef Gadallah, Leonard E. Wood, and Eldon J. Yoder 7

PRACTICAL METHOD FOR EVALUATING FATIGUE AND FRACTURE TOUGHNESS OF PAVEMENT MATERIALS (Abridgment)
Kamran Majidzadeh and Moses Karakouzian 15

PETROGRAPHIC INSIGHTS INTO THE SUSCEPTIBILITY OF AGGREGATES TO WEAR AND POLISHING
S. H. Dahir 20

STUDY OF TRANSVERSE CRACKING IN FLEXIBLE HIGHWAY PAVEMENTS IN OKLAHOMA
Magdy S. Noureldin and Phillip G. Manke 28

MINNESOTA HEAT-TRANSFER METHOD FOR RECYCLING BITUMINOUS PAVEMENT
Richard C. Ingberg, Richard M. Morchinek, and Ronald H. Cassellius 33

DETERMINATION OF MOISTURE CONTENTS IN BITUMINOUS MIXTURES BY A NUCLEAR METHOD
Prithvi S. Kandhal and Robert C. Klotz 41

RECYCLING ASPHALTIC CONCRETE: ARIZONA'S FIRST PROJECT (Abridgment)
James A. McGee and A. James Judd 47

CONSTRUCTION OF OPEN-GRADED ASPHALT FRICTION COURSES BY USING ASPHALT CEMENT AND ASPHALT EMULSION (Abridgment)
James A. Scherocman and Frank W. Parsons 50

Asphalt Cement Consistency

S.C. Shah, Louisiana Department of Transportation and Development

Comparative data on viscosity-graded and penetration-graded asphalt cements with respect to their rheological and aging characteristics are discussed. Data on the durability of American Association of State Highway and Transportation Officials AC-40 and AC-20 asphalt cements are further discussed in light of the 60-month performance of sections constructed by using these asphalts from four suppliers. The principal findings are that (a) the hardening of asphalt cements is a hyperbolic function of time; (b) asphalt cements with original high viscosity tend to harden more and at a rapid rate; (c) there was no significant difference in durability between the two types of asphalts and, for a given source, there was no recognizable difference in their performance in pavements; (d) by all durability criteria, the AC-20 grade asphalts are more durable and sections constructed with these softer grade asphalts are performing better than any AC-40 grade asphalts after 60 months of service; and (e) there was no association between high-temperature-susceptible, viscosity-graded asphalts and pavement performance or between voids in the pavement and the rate of asphalt hardening.

For years the grading of asphalt cements was done on the basis of the empirical penetration test at 25° C (77°F). A considerable amount of data has been accumulated regarding asphalts and their behavior in terms of this test. However, some earlier studies (1, 2) had indicated a significant association between viscosity and strength parameters of asphaltic concrete mixtures. Such studies supported the concept of grading of asphalts on the basis of viscosity at 60°C (140°F) rather than penetration at 25°C. One of the arguments in opposition to the grading of asphalts by penetration alone is the fact that such a grading system does not represent the temperature conditions generally associated with maximum pavement temperatures or the temperatures used in some of the mixture design methods. Furthermore, the arbitrary number specified by penetration does not represent the fundamental flow or rheological property of the material as does viscosity. Also, variation in the crude sources and refining processes of suppliers has sometimes resulted in marked differences in the viscosities of asphalt cements of a given penetration grade at 60°C.

The opposing argument for viscosity grading arises because of differences in viscosity-temperature susceptibilities and the problem of pavement cracking caused by harder asphalts. These controversies were resolved, however, and the current specification of the American Association of State Highway and Transportation Officials (AASHTO) based on viscosity grading was adopted. Although a wealth of information has been published on the aging characteristics of such asphalts and their effect on pavement performance (3), comparative data are lacking on the rheological properties of penetration and viscosity-graded asphalts and their performance in asphaltic concrete pavement. This paper is an attempt to provide such comparisons with respect to the aging characteristics of these asphalts and associated relations to pavement durability. The information presented here necessarily limits application to the material, construction procedures, and traffic and environmental conditions prevalent in Louisiana.

STUDY DESIGN

Test Sections

The investigation was conducted toward the end of 1970 on LA-1 approximately 65 km (40 miles) from Baton Rouge. Test sections were constructed over an 8-km (5-mile) stretch of existing 15.2-cm (6-in) portland cement concrete pavement that carried 3100 vehicles/d. The construction contract required widening and an overlay of 5.1 cm (2.0 in) of binder course and 3.8 cm (1.5 in) of wearing course.

Figure 1 shows the layout of the various test sections. These test sections were constructed by using four different asphalt sources (suppliers). Each source was requested to supply a penetration-graded asphalt cement and a viscosity-graded asphalt cement. The penetration-graded asphalt was the typical asphalt cement used in Louisiana at the time, and its consistency was controlled by penetration criteria [60 to 70 penetration range at 25°C (77°F)]. The viscosity-graded asphalt cements were controlled for consistency by absolute viscosity at 60°C (140° F) [360 to 700 Pa·s (3600 to 7000 poises)]. These latter grades were specifically prepared by the suppliers for this study. Detailed listing of the specifications and physical properties of the original asphalts can be found elsewhere (4). Asphalts represented by sections 9 and 10 are both viscosity-graded asphalts, one grade softer than those represented by sections 2, 4, 6, and 8. This softer grade was included primarily to seek information on its performance compared with that of harder asphalts.

Construction Control

Good control was maintained throughout construction of the test section. Care was exercised to maintain uniformity in all material and construction variables except type and source of asphalt. The mixture consisted of gravel, sand, and filler that met the standard requirements for type 1 asphaltic concrete of the Louisiana Department of Transportation and Development.

Field Sampling Procedures

The sampling frequencies for evaluation of the aging characteristics of asphalts and durability of asphaltic concrete since construction were formulated in advance as follows: 1 d, 36 d, and 110 d; 1 year, 3 years, and 5 years.

Four 31-cm (12-in) samples were obtained from the outside wheel path from a randomly selected single sample site approximately 30 m (99 ft) long for each test section. The same sampling site and pattern were used in all sampling periods. A typical location of samples for a section is shown in Figure 1. To control additional hardening of asphalt cements in the mixture (after sampling), the samples were stored in a deep freeze until ready for extraction testing. Extraction, recovery, and testing of the recovered asphalts were performed by the department and the Asphalt Institute laboratory in Maryland. The major thrust in this duplication of effort was to investigate the variability associated with the extraction and recovery process.

Tests

Standard testing procedures of AASHTO and the American Society for Testing and Materials (ASTM) were used for penetration-viscosity-ductility determinations. The asphalt cement samples were obtained from the wearing course specimens by the reflux method of extraction and the Abson method of recovery (ASTM D 1856). Trichloroethylene rather than benzene was used as the solvent. The Asphalt Institute data reflect the centrifuge method of extraction with benzene as solvent.

At each periodic evaluation, gradation, density, stability, and air voids were determined by using standard testing procedures. The air voids were computed by using percentage of maximum theoretical design specific gravity. Performance or durability of asphaltic concrete was also evaluated in terms of riding quality by using the Mays road meter, longitudinal wheel-path depressions (rutting), and the magnitude of block and alligator cracking. The physical performance measurements were supplemented by visual rating of sections by a team of 14 engineers from the transportation department, the Asphalt Institute, and the suppliers of asphalt cement. The rating was done during November of 1975 after 60 months of service. The evaluation consisted of rating the different test sections for ride quality, raveling, cracking (block and alligator only), and loss of matrix. A 244-m (800-ft) section was randomly selected in each test section for this evaluation.

Table 1 gives a listing of the performance ratings of the sections (with the exception of rutting, these results are given in U.S. customary units of measurement). The subjective rating was converted to a numerical rating by using a scale of zero to three—zero indicating poor performance with respect to the condition evaluated and three signifying absence of the defects defined by these conditions. There has been no significant change in traffic volume since reconstruction; it has remained fairly stable at 3100 vehicles/d.

Based on the data given in Table 1, evaluation of surface condition indicates that section 1 seems to be performing poorly and sections 9 and 10 superiorly. These two extreme ratings were also found to be statistically significant. However, the magnitude of ratings within each source of asphalt is negligible and statistically insignificant. Performance is related to some rheological properties elsewhere in this paper.

DISCUSSION OF RESULTS

Comprehensive field and laboratory data on material characteristics for each test section can be found elsewhere (4). The hardening characteristics of each asphalt are depicted through graphical presentations. The rheological properties of penetration, viscosity, and ductility for each pair of asphalts are shown in Figures 2 through 4. In these figures, the curves represent Asphalt Institute data. The numerals signify the section number.

Relation Between Hardening of Asphalt and Time

Except for some ductility data, the changes in the rheological properties (Figures 2 through 4) seem to fit the hyperbolic function (5, 6, 7) of the following form:

$$\Delta Y = T/(a + bT) \quad (1)$$

where

ΔY = difference between the zero life value (immediately after compaction) and any subsequent time T for a given test property,
T = time in months, and
a and b = constants of the equation.

In this equation, the asymptotes are defined by the reciprocal of the slope b and represent the ultimate change of the test variable at infinite aging time. It has been suggested (5, 6) that this limiting value of the change in any given property can be used as a measure of the durability of the asphalt. Specifically, larger ultimate change (1/b) would be considered a property of less durable asphalt.

Data from the present study for penetration at 25°C (77°F) and viscosity at 60°C (140°F) were fitted to this equation to determine the constants a and b. Using the 1-d aged data as zero life value and the last three periods as each subsequent time T, the constants b were determined and are given in Table 2. The values of the reciprocal of the slope b as the asymptotes of limiting values of the changes with time are also given in Table 2. The negative value of the slope for section 1 must necessarily invalidate the equation since at some finite time T the change in viscosity would be infinity. Such discrepancy for section 1 may be associated with the 1-d data, which had showed a threefold increase in the viscosity at 60°C. This initial threefold increase in viscosity may be significant since such an abrupt increase in hardness enhances the subsequent hardening process, as shown by the exponential trend fixed by the last data point.

Based on this limiting change criterion for penetration, all viscosity-graded asphalt cements seem more durable than penetration-graded asphalt cements because of lower ultimate change. However, the fact that all viscosity-graded asphalts had lower penetration values to start with should not be overlooked. To compensate for this, the ultimate limiting penetration values were computed (Table 2). Based on these values, asphalt 1 is the least durable of all and asphalts 9, 10, and 5—in that order—are the most durable. Asphalts 9 and 10, it will be recalled, are the softer asphalts [viscosity at 60°C (140°F) of 200 ± 40 Pa·s (2000 ± 400 poises) and penetration at 25°C (77°F) of 65+]. These limiting values do not provide any consistent trend as to the superiority, with respect to durability, of one type of asphalt over the other.

Application of this limiting value concept to viscosity data [60°C (140°F)] provides some correlation with penetration data. Asphalts 9, 10, and 5 indicate lower values of limiting viscosity, and asphalts 1, 4, and 3—in that order—indicate the largest values. Once again, no trend is discernible; no one group of asphalts (penetration versus viscosity) seems more durable than another.

If durability and performance are synonymous, there should be some correlation between the observed performance of these asphalts in the pavement and the above durability ranking. The data in Table 1 bear this out, at least for extreme (good and poor) conditions of performance as reflected by overall subjective rating. Sections 9 and 10, which have the more durable asphalt, are performing much better than section 1, which has the least durable asphalt.

The analysis and discussion above indicate that soft asphalts initially exhibit more desirable hardening characteristics than asphalts with higher original viscosities. Such low-viscosity sections, notably sections 9 and 10, have likewise shown less pavement distress than some of the other sections. However, an argument against the use of softer asphalts in the state of Louisiana is the early manifestation of wheel-path rutting. This may be true since the magnitude of ruts, which is not of any great concern, is nevertheless higher for sections 9 and 10.

Changes in Rheological Properties

Penetration and Viscosity

The data for penetration in Figure 2 show that, for both types of asphalts, there is a rather rapid rate of hardening during the first 12 months and a decreasing rate thereafter. This rate of hardening with time, for viscosity at 60°C (140°F) (Figure 3), is not consistent although the rate is slower for viscosity-graded asphalts than for the corresponding penetration-graded sections. This is indicated by the Asphalt Institute data in Figure 5, which shows a plot of viscosity index at 60°C versus time of service on a logarithmic scale. The viscosity index, frequently called the aging index, is simply the ratio of the viscosity of the aged asphalt at a temperature to the viscosity of the asphalt before aging. Use of this term tends to eliminate the variability caused by differences in the viscosities of the original raw asphalt. Section 3, the penetration-graded asphalt section, shows a thirteenfold increase in this viscosity measurement after 60 months of service. Likewise, this section had the highest original and thin-film residue viscosity.

The slopes indicated by the curves for aging index versus time in Figure 5 can be used as indicators of the relative durability of various asphalts. Specifically, a flat

Figure 1. Layout and identification of pavement sections.

TEST CORES

SECTION NO.	1	2	3	4	5	6	7	8	9	10
ASPHALT SOURCE	A	A	B	B	C	C	D	D	A	B
ASPHALT CRUDE	HAWK	HAWK	MEX	MEX	LIGHT ARK	SMACK OVER	HAWK, ARAB	HAWK, ARAB	HAWK	MEX
ASPHALT GRADE	PEN	VISC	PEN	VISC	PEN	VISC	PEN	VISC	VISC	VISC

Table 1. Pavement condition rating.

Criterion	Section 1	2	3	4	5	6	7	8	9	10
Riding	1.31	1.53	1.63	1.67	1.80	1.78	2.01	1.93	1.89	2.06
Raveling	2.08	1.52	1.79	1.81	2.21	1.62	2.03	2.21	2.28	2.51
Loss of matrix	1.95	1.41	1.70	1.76	2.09	1.64	2.12	2.34	2.37	2.47
Cracking										
Block and alligator	1.52	2.36	2.30	2.03	2.27	2.16	2.10	2.06	2.48	2.52
Transverse and longitudinal	1.51	1.98	1.93	1.67	1.91	1.95	1.95	2.04	2.18	2.45
Overall subjective rating	1.67	1.76	1.87	1.79	2.06	1.83	2.04	2.12	2.24	2.40
Rutting, mm	7.1	4.4	4.1	3.4	4.6	3.8	4.6	4.6	5.3	6.6
Mays roughness, in/mile	144	131	123	123	111	108	96	85	125	85
Dynaflect deflection, $\frac{1}{1000}$ in	1.64	1.40	1.35	1.27	1.33	0.95	1.03	0.89	0.91	1.32
Block and alligator cracking, $ft^2/1000\ ft^2$	42.76	9.22	0.36	3.33	20.26	13.91	3.70	6.98	0.68	0.00

Note: 1 mm = 0.039 in.

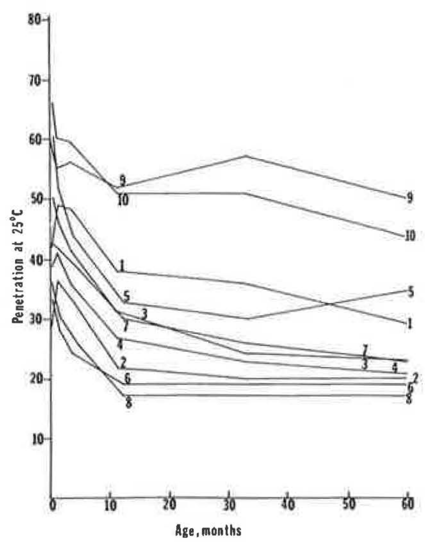


Figure 2. Penetration at 25°C (77°F) versus time of aging.

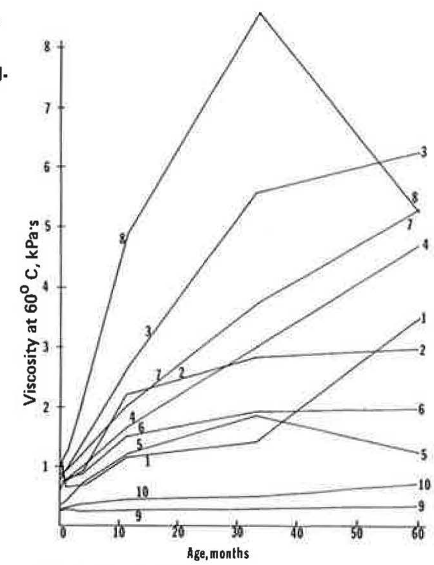


Figure 3. Viscosity at 60°C (140°F) versus time of aging.

Note: 1 kPa·s = 10 kilopoises.

slope implies a more durable asphalt. Accordingly, based on 60-month data, all viscosity-graded asphalts would be classified as more durable than the corresponding penetration-graded asphalts. Likewise, sections 9 and 10, which have the smallest slopes, have the most durable asphalts, and section 3 has the least durable asphalt.

Ductility

The importance of ductility requirements in specifications has long been a subject of debate mainly because of the empirical nature of the test. However, it is recognized that the ductility values provide some measure of asphalt quality related to flexibility. In Figure 4, the ductility values for various asphalts indicate inconsistency in their rate of hardening. Sections 3, 4, 7, and 8, however, have hardened at a more rapid rate than the other sections. A similar trend was indicated by some of these same sections in Figure 5.

Shear Index

The change in the characteristics of the shear index with time is shown in Figure 6. Once again, the relative position of each curve is directly associated with its aging characteristics (as depicted by its rheological characteristics). Specifically, sections 3, 4, 7, and 8 are more susceptible to shear, and sections 9 and 10 are less so. These sections had also exhibited an increased rate of hardening during the 5-year period. Source (crude) may be an influential factor for such aging characteristics.

A close association between ductility values and shear index was also observed from the data. In general, asphalts with shear indexes of less than 0.40 had ductilities greater than 100 cm (39 in) (4).

Temperature Susceptibility

Asphalt consistency is greatly affected by changes in temperature. The extent of this effect is expressed as temperature susceptibility, which can be measured by using the Walther relation. The value of this property for each asphalt was evaluated for original and 60-month samples by using the service temperature range of 25°C to 60°C (77°F to 140°F) and also the mixing and compaction range of 60°C to 135°C (282°F).

These values are shown in the form of a bar chart in Figure 7, where it can be seen that the viscosity-graded asphalts are more susceptible to temperature changes than the corresponding penetration-graded asphalts. The figure also shows that, for both groups of asphalts, the 60-month values are higher than the corresponding original values in the 60°C to 135°C temperature range. However, in the service temperature range of 25°C to 60°C, the 60-month data show a significant decrease in slope from the original.

High-ductility asphalts are more susceptible to temperature as shown by sections 2, 5, 6, 9, and 10 in Figure 4. These sections had indicated high values of retained ductility after 60 months of service. Correspondingly, their temperature susceptibility is also higher in the service temperature range. Furthermore, sections 9 and 10, which show no loss of ductility values after 60 months, also exhibit the least change in their temperature susceptibility in the service temperature range.

Values of temperature susceptibility have been shown to be highly correlated with the transverse cracking in the pavement (8). The performance data given in Table 1 do not show any correlation between the two variables. In fact, sections 9 and 10, the most susceptible sections, are the best performing sections with respect to any form of cracking distress.

Air Voids in Pavement and Asphalt Hardening

Figure 8 shows changes in void content with time. Almost all sections had reached the void content of 5.0 percent within 3 years of traffic service. During the past 2 years,

Figure 4. Ductility at 25°C (77°F) versus time of aging.

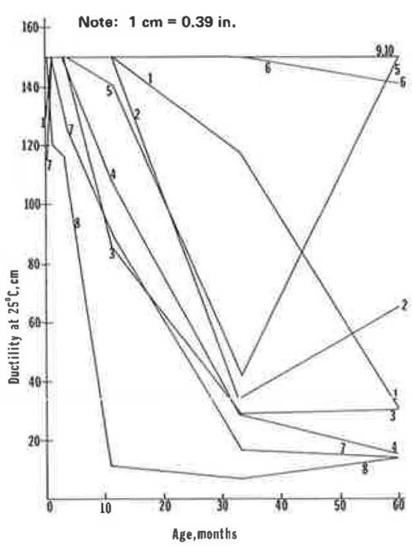


Table 2. Limiting value of property changes with age.

Section	Slope (b) Penetration	Slope (b) Viscosity	Limiting Change Penetration	Limiting Change Viscosity (kPa·s)	Ultimate Change Penetration	Ultimate Change Viscosity (kPa·s)
1	0.033	-0.17	30.3	-[a]	12	-[a]
2	0.105	0.043	9.5	2.33	19	3.35
3	0.033	0.010	30.3	10.00	20	10.81
4	0.049	0.002	20.4	50.00	19	50.90
5	0.041	0.118	24.4	0.85	35	1.25
6	0.072	0.063	13.9	1.59	19	2.19
7	0.044	0.007	22.7	14.29	20	15.18
8	0.051	0.023	19.6	4.35	16	5.39
9	0.083	1.016	12.1	0.10	48	0.39
10	0.044	0.117	25.0	0.86	41	1.16

Note: 1 kPa·s = 10 kilopoises.

[a]Infinity at some finite time T.

Figure 5. Viscosity index at 60°C (140°F) versus time of aging.

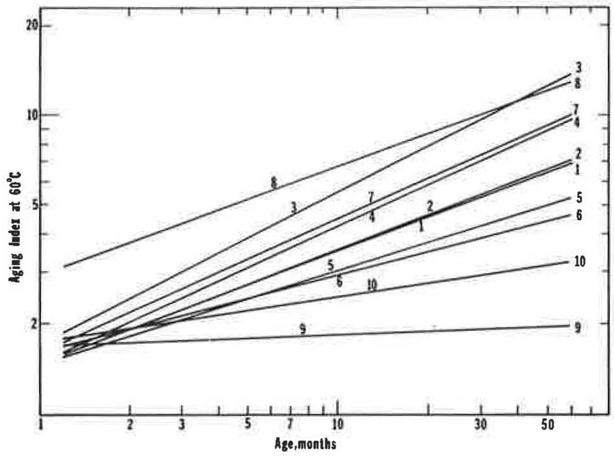


Figure 6. Shear index versus time of aging.

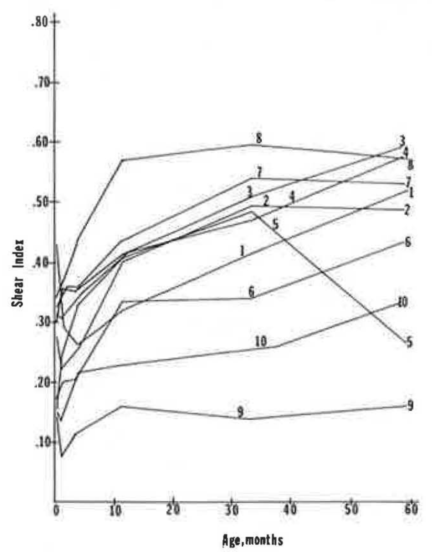


Figure 8. Void content in pavement versus time.

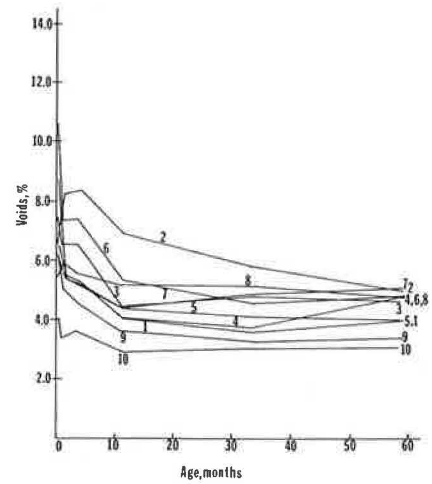
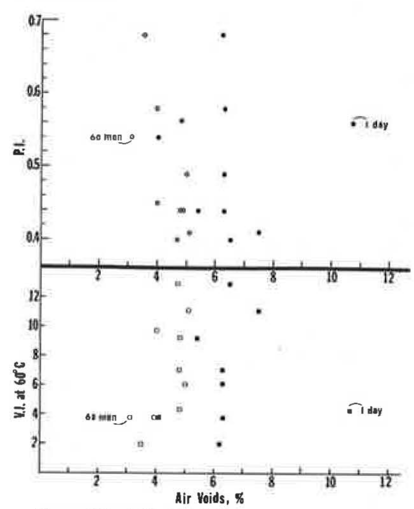
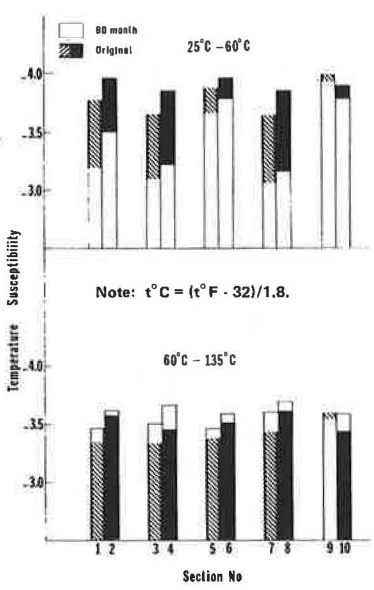


Figure 9. Effect of air voids on hardening of asphalts.



Note: t°C = (t°F - 32)/1.8.

Figure 7. Temperature susceptibilities of test sections.



Note: t°C = (t°F - 32)/1.8.

there has been practically no change in void content. The resistance to compaction may be associated with the consistency of asphalts. The harder the original viscosity is, the greater its resistance to compaction may be. Sections 9 and 10—the softer grade asphalt sections—have shown the least resistance to traffic compaction, and asphalt section 8—the high-viscosity asphalt—has shown the most resistance.

It is generally recognized that the degree of initial and final compaction or void content or both in the pavement has an effect on the rate of hardening of asphalts. More specifically, the higher the initial void content is, the greater the rate of hardening of asphalt in the pavement will be. Figure 9 was prepared to investigate this relation. The plots show initial and final void content and viscosity index (VI) at 60°C (140°F) and penetration index (PI) at 25°C (77°F). The data are too scattered to indicate any association of hardening rate with air void content in pavement. This disassociation should not be construed to mean that the magnitude of air void content does not affect the hardening rate of asphalt binder. What it

Table 3. Durability ranking by use of various criteria.

Durability Criterion	Section or Asphalt Type									
	1	2	3	4	5	6	7	8	9	10
Field performance[a]	10	9	6	8	4	7	5	3	2	1
Viscosity index										
25°C	6	2	10	5	4	7	8	2	1	3
60°C	6	7	10	9	4	3	8	5	1	2
Penetration index	5	8	10	9	4	3	6	7	1	2
Limiting penetration	10	6	5	8	3	7	4	9	1	2
Limiting viscosity (60°C)	10	5	7	9	3	4	8	6	1	2

Note: t°C = (t°F - 32)/1.8.
[a]See Table 1.

does indicate is the fact that the air void variability in pavement is so pronounced that it overshadows the resulting effect on the hardening process. In general, the binder viscosity of the original asphalt may be more critical since the data show that softer asphalt offers less resistance to compaction (sections 9 and 10) and is thus able to satisfy lower void content requirements.

Gradation and Stability

After 60 months of traffic, no aggregate degradation was evident in these sections. As would be expected, the strength values have increased because of the increase in binder viscosity. The effect of asphalt content on the hardening characteristics of various binders could not be isolated since all sections started with the same percentage of binder.

FIELD PERFORMANCE VERSUS RHEOLOGICAL PROPERTIES

It has been shown above that asphalts exhibit a wide variation in their hardening characteristics. Specifications for asphalts generally relate to their durability, which in turn relates to their useful life in pavement. However, the most complex problem is the establishment of a single criterion that would define durability and early prediction of performance in pavements.

The relative durability of penetration- and viscosity-graded asphalts was studied by using criteria of limiting value and various aging indexes. Based on these criteria, the 10 asphalts were ranked from the most durable to the least durable on a scale of 1 to 10 respectively. The rankings based on such criteria were then compared with the rankings of various asphalt sections with respect to their overall field performance (Table 1). These comparative rankings are given in Table 3. As was discussed before, the rankings according to various index criteria are based on the magnitude of the slopes of the index-time curve relation, and flatter slopes indicate more durable asphalts. The limiting value rankings were determined from the hyperbolic relation discussed previously (Equation 1).

Data given in Table 3 show that practically all durability criteria seem to be consistent with respect to the ranking of the most durable asphalt, sections 9 and 10. Rankings according to these criteria are also consistent with actual field performance rankings. However, there is no consistency for the least durable scale. Furthermore, prediction of durability by using limiting value criteria correlates better with the observed field performance of various asphalts.

The data in the table do not indicate any recognizable consistency in the performance of the two types of asphalts investigated in this study. Likewise, the various durability criteria also fail to provide any recognizable trend with respect to one type of asphalt being more durable than another. What is most indicative, by all criteria, is the fact that softer grade asphalts are more durable than harder asphalts. The poor performance of section 1 cannot with certainty be attributed to any single criterion. The instability of the underlying layers may have contributed to the distress observed since deflection measurements made by using the Dynaflect had indicated maximum value of this parameter for the section. However, because of a lack of original data on deflection, this association may not be definitive. If major maintenance is not done on these sections, they may be evaluated in the future to establish definite cause-effect trends.

SUMMARY AND CONCLUSIONS

The primary intent of the study reported here was to make a comparative evaluation of the durability and performance of penetration- and viscosity-graded asphalt cements by means of field installation in asphaltic concrete mixtures. The principal findings summarized below are applicable within the constraints of the environment, materials, construction, and traffic that existed at the test site:

1. Hardening of asphalt cements, regardless of how they are graded, is a hyperbolic function of time but at different rates.
2. Asphalts that have a high original viscosity tend to harden more and at a rapid rate. This rate of hardening at 60°C (140°F) is slightly lower for viscosity-graded asphalt than for corresponding penetration-graded asphalts.
3. For a given asphalt source, the difference in durability between the two types of asphalts was not significant. Likewise, no significant difference was evident in their performance in the field.
4. By all durability criteria, asphalts one grade softer (AC-20) than the harder viscosity-graded asphalts (AC-40) show desirable characteristics of durability. After 60 months of service, sections constructed with these asphalts are performing better than sections in which any other harder grade asphalt has been used.
5. Viscosity-graded asphalts are more susceptible to temperature than corresponding penetration-graded asphalts, but there was no correlation between this characteristic and pavement distress.
6. There was no association between voids in the pavement and rate of hardening.

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First and foremost, I wish to acknowledge the cooperation and effort of the Asphalt Institute in conducting the various tests on asphalt cements and that of the asphalt producers in providing the specified experimental asphalts. The study was conducted under the Louisiana highway planning and research program in cooperation with the Federal Highway Administration. I am solely responsible for the opinions, findings, and conclusions expressed.

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Evaluation of Properties of Asphalt-Emulsion-Treated Mixtures by Use of Marshall Concepts

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Findings are reported of a laboratory investigation concerning the effect of asphalt emulsion content, added moisture content, and aggregate gradation on the design parameters and properties of asphalt-emulsion-treated mixtures (AETMs) by use of Marshall equipment. The evaluation was conducted at different curing stages of the mix. One type of aggregate (sand and gravel), one type and grade of asphalt emulsion, a modified Marshall method for preparing and testing AETM specimens, and autographic Marshall equipment that produced a continuous record of load deformation were used in the study. The evaluation resulted in a number of significant results. AETM properties are an outcome of a complex array of factors. Evaluating the mix properties in relation to only a single factor is not sufficient; the interaction of these factors influences the behavioral properties of AETMs and must be considered in the evaluation. The study also showed that water-sensitivity tests must be an integral part of the Marshall design procedure for AETMs.

This study reports the findings of a laboratory investigation that evaluated the effect of asphalt emulsion content, added moisture content, and aggregate gradation on the design parameters and properties of asphalt-emulsion-treated mixtures (AETMs) by using Marshall equipment. The evaluation was conducted at different curing stages of the mix. One aggregate type (sand and gravel) and one type and grade of asphalt emulsion were used along with a modified Marshall method developed by Gadallah, Wood, and Yoder (5) for preparing and testing AETM specimens and autographic Marshall equipment that produced a continuous record of load deformation.

The evaluation of AETM properties resulted in a number of significant results. AETM properties are an outcome of a complex array of factors. Evaluating the mix properties in relation to only a single factor is not sufficient; the interaction of these factors influences the behavioral properties of AETMs and must be considered in the evaluation.

EQUIPMENT AND MATERIALS

Marshall Equipment

The Marshall equipment used in the study consisted mainly of a mechanical compaction hammer and an autographic stability apparatus. The mechanical compaction hammer was used for compacting the standard Marshall specimens at 50 blows/side.

The stability apparatus used in this investigation is essentially the same as the standard Marshall equipment, but it provides a continuous recording chart for load (lb) versus deformation (0.01-in units) throughout the testing range from which stability and flow values can be obtained (since this equipment is calibrated in U.S. customary units of measurement, no SI equivalents are given). Figure 1 shows a typical trace for the Marshall test.

Mineral Aggregate

One type of aggregate, which consisted mainly of terrace sand and gravel, was used in this investigation. Three aggregate gradations that lie within an Indiana State Highway Commission (ISHC) gradation size 73B with a maximum size of 19 mm (0.75 in) were used (Figure 2): The first gradation (MG) follows the midspecification of the ISHC

Figure 1. Typical trace for the Marshall test.

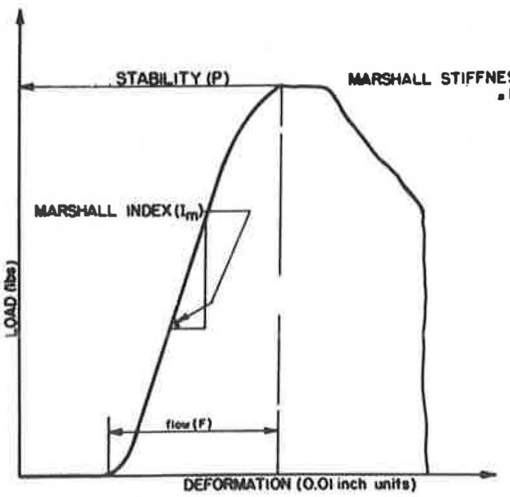
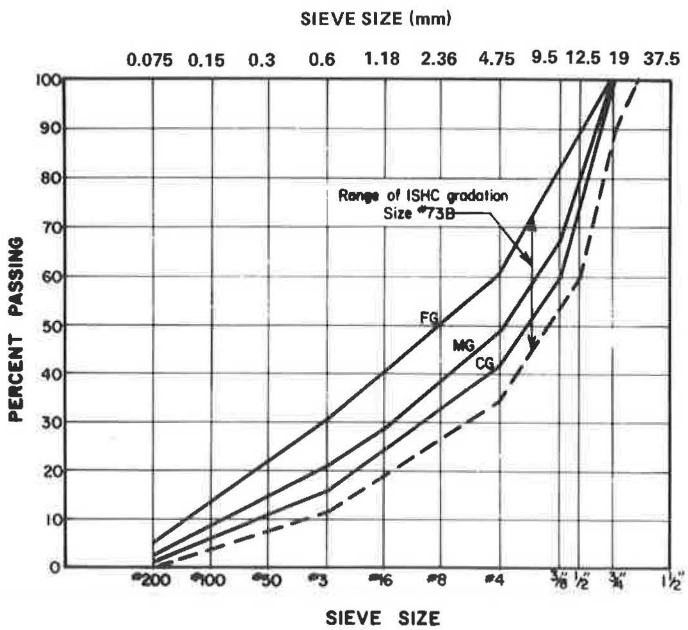


Figure 2. Aggregate gradations.



73B gradation band, the second gradation (FG) follows the upper limit of the gradation band, and the third gradation (CG) was selected between the midpoint and the lower limit of the gradation band to provide better handling and control of the mix. The test properties of the aggregate used are given below [0.075-mm (no. 200) nonplastic filler]:

Property	Gradation FG	MG	CG
Apparent specific gravity	2.699	2.707	2.710
Bulk specific gravity, SSD	2.603	2.607	2.608
Absorption, %	1.13	1.20	1.24

Asphalt Emulsion

ISHC designation AE-150 mixing grade emulsified asphalt was used in this study. The physical properties of the asphalt emulsion, which was formulated and provided by the K. E. McConnaughay Laboratory in Lafayette, Indiana, are as follows [$t^\circ C = (t^\circ F - 32)/1.8$]:

Property	Value
Residue by distillation, %	70.0
Penetration of residue after distillation, at 25°C, 5 s, 100 g	188.0
Specific gravity of residue after distillation, at 25°C	0.986

TESTING PROCEDURE

The following is a summary of the procedure that was used for preparing and testing AETM specimens (5):

1. The aggregate was prepared in 1200-g batches based on the aggregate gradation required.
2. The required amount of initial moisture (distilled water) was added to the cold aggregate and mixed thoroughly.
3. The aggregate-water mixture was left for 10 to 15 min before the asphalt emulsion was added.
4. The amount of asphalt emulsion that is needed to provide a certain asphalt emulsion residue in the mix was added cold to the wet aggregate and mixed by using a mechanical mixer for about 2 min and a 30-s hand mix with a spoon during the mixing period.
5. The mix was cured for 1 h in a forced-draft oven at 60° C (140° F) and then remixed for 30 s.
6. The mix was compacted by using 50 blows on each side of the specimen with the Marshall compaction mechanical hammer.
7. The compacted specimens were left in the mold for about 30 min before they were extruded.
8. The samples, which measured 10.2 cm (4 in) in diameter by 0.75 m (2.5 ft) in height, were then left to cure at room temperature [~21°C (~72° F)] for the required curing time before testing. Whenever the design called for the "ultimate" curing condition, the AETM specimens were cured for 3 d in a forced-draft oven at 48° C (120° F). The specimens were then permitted to adjust to room temperature before testing. Generally, 4 h were enough for the samples to adjust.
9. The cured AETM specimens were tested at room temperature by using Marshall equipment to determine Marshall indexes. Before testing, analyses of density and air voids were performed.
10. Whenever the design called for conducting water sensitivity analysis, the method reported by the Asphalt Institute laboratory (2) was used. In this method, the specimens were subjected to 1 h of vacuum at 30 mm Hg. After the 1-h period, water at 21° C (72° F) was drawn into the vacuum chamber, submerging and vacuum saturating the specimens. The vacuum was released, and the specimens were then transferred to a 21° C water bath where they remained for 24 h. Before testing for Marshall indexes, the saturated surface-dry weight of the specimens was determined. The percentage of water absorption was then obtained.

EXPERIMENTAL DESIGN

The effect of aggregate gradation, asphalt emulsion residue, and added moisture contents on the properties of AETMs was evaluated. Three aggregate gradations were used in the study. The gradations were selected within ISHC gradation size 73B and identified as FG, MG, and CG (Figure 2).

Figure 3. Factorial design for study of the effect of aggregate gradation on AETM properties.

Curing time • condition	%W \ %AE (residual) \ Agg. Gradation	F.G. 2.5	F.G. 3.25	F.G. 4.0	M.G. 2.5	M.G. 3.25	M.G. 4.0	C.G. 2.5	C.G. 3.25	C.G. 4.0
1 day	1.5%	X	X	X	X	X	X	X	X	X
	3%	X	⊗	X	⊗	⊗	⊗	X	⊗	X
3	1.5									
	3	X	⊗	X	⊗	⊗	⊗	X	⊗	X
7	1.5	X	X	X	X	X	X	X	X	X
	3	X	X	X	X	X	X	X	X	X
ult. cond.	1.5									
	3	X	⊗	X	⊗	⊗	⊗	X	⊗	X

Note:
1- X dry test
2- O water sensitivity test
3- The ANOVA was conducted for mix combinations within the two blocks (1 and 7 days air-dry curing)

Two replications of the experiment (blocks) were used to provide more inference on the analysis and evaluation of the effect of aggregate gradation together with percentage asphalt emulsion (AE) and percentage added moisture (W). Curing time at 1 and 7 d represented the two blocks (Figure 3). Using the two levels of curing provided the necessary information about the main effects: aggregate gradation, asphalt emulsion content, and added moisture content and their interactions. In addition, all interaction effects were evaluated in relation to the curing factor. However, no testing for the effect of curing time (1 versus 7 d) was available in this study mainly because of the restriction on randomization caused by the blocking effect.

Figure 3 shows the factorial design for the study. The two blocks are shown within the heavy lines for 1 and 7 d of curing. Three levels of aggregate gradation, three levels of asphalt emulsion content (throughout this paper, asphalt emulsion content refers to the asphalt emulsion residue content in the AETM), and two levels of added moisture content were incorporated in the design. In addition to these two blocks, several mix combinations were tested at 3 d of curing and the ultimate curing condition, as shown in Figure 3. The later mixes were not used in the analysis of variance. However, reference is made to these test results whenever it is necessary to provide a general trend for the curing effect and for the water sensitivity analysis.

The AETM properties were analyzed within the framework of a fixed-effect, randomized, complete block design—RCBD (1). The curing time (1 and 7 d) corresponded to the blocks of RCBD. The detailed analysis is presented elsewhere (4).

Response Variables

The response (dependent) variables that were used to evaluate AETM properties by using the modified Marshall method were as follows:

1. Wet density (γ_w), which refers to the density of the mix, including the moisture portion of it, at the time of testing, and dry density (γ_d), which was determined by excluding the moisture portion in the specimen;
2. The percentage of moisture retained in the specimen (WC_o), at the time of testing expressed as percentage by weight of the dry aggregate;
3. The percentage of total liquid at the time of testing (TL) (percentage AE residue plus percentage WC_o);
4. The percentage of voids in the mix, including (a) percentage of air voids (V_A), which represents the percentage of air voids available in the mix but excludes the voids that are filled with moisture, and (b) percentage of total voids (V_T), which represents the total amount of voids available in the mix and includes the air voids (V_A) and the voids filled with moisture (V_W);
5. Marshall stability P, measured at a room temperature that was maintained at approximately 21°C (72°F);
6. Marshall flow F, the maximum deformation that occurs as the specimen reaches failure, expressed in units of 0.25 mm (0.01 in);
7. Marshall stiffness S_m, determined as the ratio of Marshall stability and flow ($S_m = P/F$); and
8. Marshall index I_m, which is represented by the slope of the linear portion of the load-deformation trace obtained from the autographic Marshall equipment.

Two new parameters, S_m and I_m, provide measures for the mix characteristics at the failure condition and throughout the loading process respectively (Figure 1). It is believed that the use of these two parameters in conjunction with the traditional Marshall design parameters would provide better control and evaluation of AETMs.

ANALYSIS OF RESULTS

Percentage of Moisture Retained in the Sample

The percentage of moisture retained in the AETM samples was significantly affected by all factors and their interactions except the interaction between curing, gradation, and added moisture content, which was not significant.

Figure 4 shows the percentage of moisture retained at the time of testing for the specimens cured for 1 and 7 d as a function of aggregate gradation, percentage AE, and percentage W. The percentage of moisture retained (WC_o) ranged between 0.5 and 1.6 percent (by weight of the dry aggregate). At 7 d of curing, the difference in percentage WC_o attributable to varying aggregate gradation, percentage AE, or percentage W was relatively less than that observed for specimens cured for 1 d. In addition, for 1-d specimens (Figure 4a), the effect of aggregate gradation and percentage of added moisture on the percentage WC_o values was more pronounced at the low asphalt content. This effect was reduced as asphalt content was increased. For 7-d specimens (Figure 4b), the range of percentage WC_o was small for FG and MG aggregate mixes at different percentages of AE and W. However, there was relatively large variation in percentage WC_o for CG aggregate mixes.

In view of these results, it can be concluded that at early curing conditions (e.g., 1 d) the effect of aggregate gradation and added moisture content on percentage WC_o depends on the asphalt content in the mix. The higher the percentage of AE, the less is the variation in percentage of WC_o that results from changing aggregate gradation or adding moisture content or both. However, after relatively longer periods of curing (e.g., 7 d), the effect of the interaction between aggregate gradation and percentage W is not significantly apparent.

Dry and Wet Unit Weights

The main factors—aggregate gradation, percentage AE, and percentage W—significantly affected the dry and wet unit weights of the AETM specimens. In addition, aggregate gradation and percentage AE had a greater effect on dry and wet unit weights than added moisture content (percentage W) (this is based on the mean square value that is attributed to each factor in the analysis of variance). It was also noted that all two-factor interactions were significant except the interaction between aggregate gradation and added moisture content, which was not significant. The most significant two-factor interaction was the interaction between curing time and added moisture content.

The relation between dry unit weight (γ_d) and aggregate gradation, percentage AE, and percentage W for the two curing periods is shown in Figure 5. It is apparent that, the higher the asphalt content in the mix is, the higher the γ_d values are. Besides, the CG gradation samples resulted in higher dry unit weights than the MG gradation samples. The FG gradation samples resulted in the least dry unit weights.

Marshall Stability

Marshall stability (P) was significantly affected by all the main factors as well as by most of the two-factor and three-factor interactions. In comparison with the effects of asphalt emulsion and added moisture, aggregate gradation showed the most significant effect on the stability values. Figure 6 shows the stability values in relation to aggregate gradation, asphalt emulsion content, and added moisture content for the two curing periods. Presenting the data in this form aids in providing a better understanding of the effect of the main factors and their interactions.

For specimens cured for 1 and 7 d, FG gradation provided the highest stability at all levels of percentage AE and percentage W. The lowest stability values were obtained for mixes with CG gradation. The MG-gradation mixes resulted in a higher stability than the CG-gradation mixes. It should be noted that the difference between FG and MG grain-size distributions was twice the difference between MG and CG grain-size distributions (Figure 2). This affected the characteristics of the stability results. The Marshall stabilities provided by the MG mixes were

Figure 4. Effect of interaction among aggregate gradation, percentage AE, and percentage W on percentage WC_o for two curing periods.

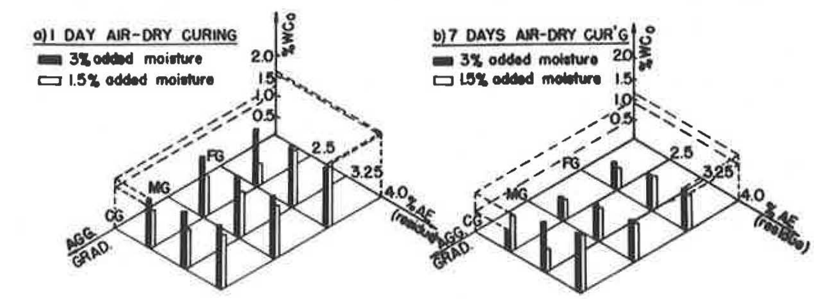


Figure 5. Effect of aggregate gradation, AE, and W on γ_d.

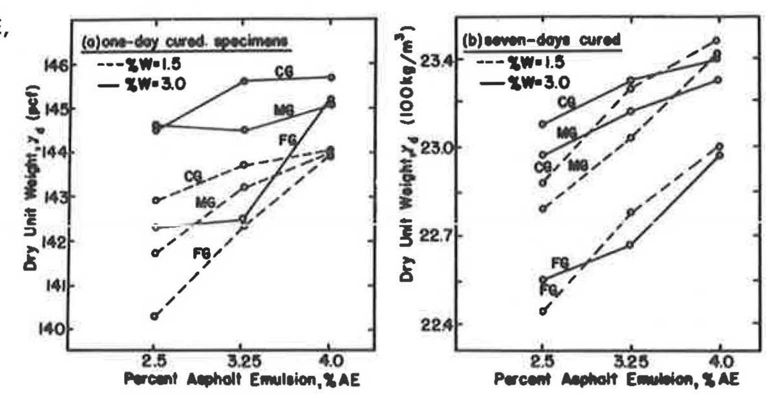


Figure 6. Effect of aggregate gradation, percentage AE, and percentage W on P.

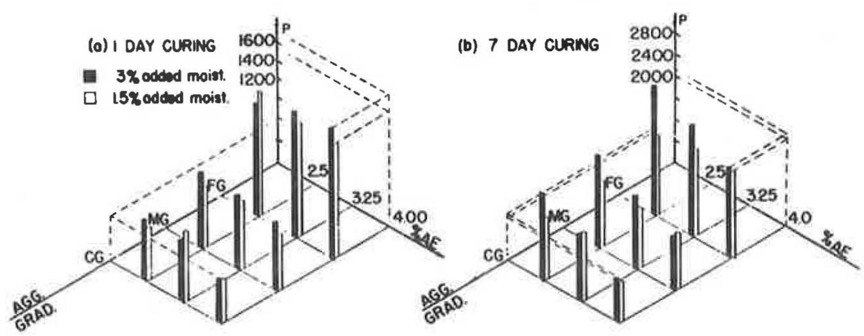
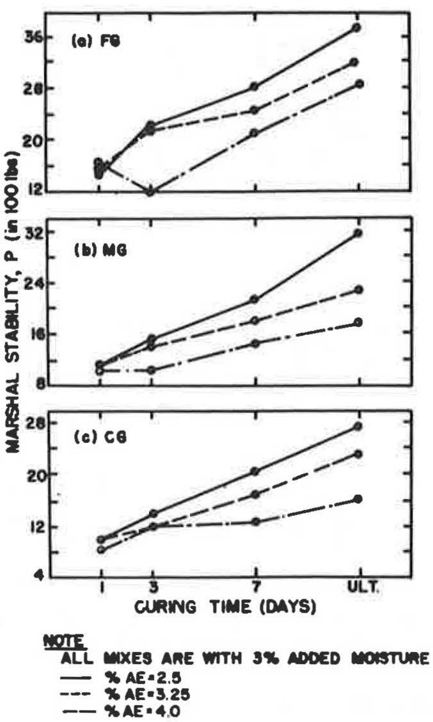


Figure 7. Effect of aggregate gradation and percentage AE on P as a function of curing time.



closer to those of the CG mixes than to those of the FG mixes.

The effect of asphalt emulsion content on AETM stability is not significantly apparent at early stages of curing (1-d air-dry curing), mainly because of the nature of the asphalt emulsion present in the mix at that time. However, the significant effect of percentage AE becomes increasingly important during the curing process, when the asphalt emulsion residue gradually starts to affect the mix properties (Figure 6).

The AETM with FG and MG gradations produced, in general, higher stability values for mixes with 3 percent W than for mixes with 1.5 percent W. However, the effect of added moisture content for CG mixes was very small or was reversed when the CG gradation was compared with the remaining two gradations. This is caused by the fact that FG and MG gradations have more surface area than the CG gradation and thus require relatively larger amounts of liquid for adequate coating and strength. This is more apparent when one considers the results of 7-d curing, which show that, if the samples were evaluated after 1-d curing, the effect of the added moisture on the strength of the AETM (P in this case) could have been underestimated. Evaluating the AETM properties after relatively long periods of curing would provide more understanding of the role of each of the liquid components in the mix, especially the added moisture content.

The increase or gain in stability values through the curing process for the different aggregate gradations and asphalt emulsion contents is shown in Figure 7. The more the aggregate gradation shifts toward the fine limit of the gradation band, the steeper will be the curing trend and consequently the more rapidly the AETM will develop its strength (represented here as the Marshall stability). This can be demonstrated by a comparison of the stability trends for the different gradations at any specific percentage AE (Figure 7). In addition, the gain in stability through the curing process depends on the asphalt emulsion content. The lower the asphalt emulsion content is in the mix, the greater the gain in stability that will be attained through the curing process.

Marshall Flow

Values of the Marshall flow ranged between 6 and 8.5 for all mix combinations after 1 d of curing. The flow range for specimens cured for 7 d was from 6.0 to 11.5. In general, the curing time and its interaction with the added moisture content had the most significant effect on the flow values.

Larger amounts of asphalt emulsion resulted in higher flow values. Increasing values of F occurred as a result of extending the curing time before testing the specimens. This is a direct result of the fact that during the curing process the mix loses a portion of the available moisture and this in turn makes the role of the emulsion residue in the mix more apparent in terms of an increase in flow values.

In addition, aggregate gradation significantly affected flow values. The FG aggregate provided the mix with relatively lower flow characteristics; this was more apparent in mixes with low asphalt emulsion content.

Marshall Stiffness and Marshall Index

Asphalt emulsion content, added moisture content, and their interaction significantly affected the Marshall stiffness (S_m) and Marshall index (I_m) values (Figure 8). But asphalt emulsion content showed a greater influence on S_m values than added moisture content. Generally, by decreasing the percentage AE, both S_m and I_m will increase as the mix becomes less plastic and the slope of the load-deformation curve will be steeper. The same trend holds for the effect of percentage W. I_m trends are about the same as S_m trends but have higher values because of the nature of the parameters themselves. The index values represent the slope of the linear portion of the load-deformation curve, whereas the stiffness values represent the slope of the line that connects the initial or starting loading point with the failure point.

Aggregate gradation was the most significant factor that affected both I_m and S_m. The more the aggregate gradation shifts toward the fine limit of the gradation band, the higher will be the resulting stiffness parameters (I_m and S_m). Curing time and percentage AE also significantly affected these two parameters. The interaction effect of percentage AE and percentage W is important in influencing the I_m and S_m values. However, this effect also depends on the curing factor (especially in the case of I_m).

It is important to note the effect of interaction between aggregate gradation, percentage AE, and percentage W at the two curing periods (Figure 8). I_m and S_m values for FG aggregate mixes were higher for samples with a high percentage of added moisture, but the trend was reversed by increasing percentage AE to 4 percent. It should be noted that the difference was reduced through the curing process (1 versus 7 d). For the MG aggregate mixes, the trend depends on percentage AE residue. For FG mixes, relatively low added moisture content (1.5 percent) provided the mix with higher I_m and S_m at the two curing periods than did 3 percent added moisture. In general, the higher the amount of initial moisture that is used in FG mixes, the higher the measured strength parameters will be.

Figure 8. Effect of aggregate gradation, percentage AE, and percentage W on I_m.

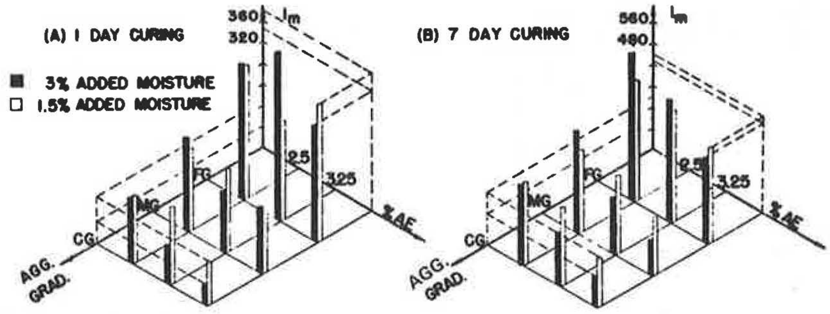


Figure 9. Relation between S_m and percentage TL for different aggregate gradations and percentages AE.

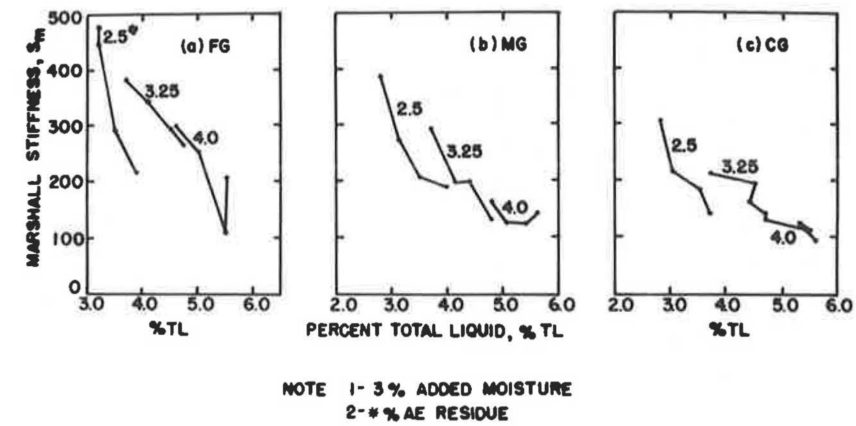
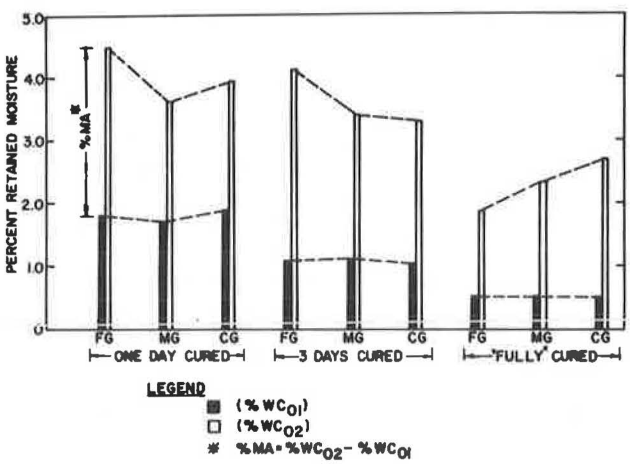


Figure 10. Effect of aggregate gradation on percentage MA as a function of dry curing time (3.2 percent AE and 3 percent W).



However, for CG mixes, low initial added moisture contents resulted in higher strength parameters than did high initial added moisture contents.

Figure 9 shows the S_m results as a function of the percentage of total liquid at the time of testing (TL) for samples with 3 percent added moisture. The change in percentage TL was obtained through the curing process. The I_m and S_m results generally show the same trends, the I_m values being relatively higher. Note the significant effect of aggregate gradation in controlling the position of the relation between S_m and percentage TL in the S_m scale (the same is true for I_m). In addition, the interaction between aggregate gradation and asphalt emulsion content is more pronounced in the graph.

RESULTS OF WATER-SENSITIVITY TESTS

AETMs containing 3.25 percent asphalt emulsion residue content and 3 percent added moisture content were used to study the effect of aggregate gradation on the results of water-sensitivity tests. The comparison study was conducted for the three aggregate gradations at three different curing periods: 1 and 3 d of air-dry curing and the ultimate curing condition (Figure 3). Therefore, it must be emphasized that the discussion that follows pertains to specific asphalt emulsion and added moisture contents.

The effect of asphalt emulsion content on the results of water-sensitivity tests was also studied but for a specific aggregate gradation (MG) and 3 percent added moisture content (Figure 3).

Percentage of Moisture Absorption

At early curing periods (1 and 3 d of air-dry curing), the percentage of moisture absorption (MA) was higher for FG mixes than for MG or CG mixes (Figure 10). However, after the mixes were cured to the ultimate condition, this relation was reversed. An increased amount of MA was obtained for the coarser gradations (given that the other mix components were the same). The percentages of moisture retained in the specimens for the different aggregate gradations before the water-sensitivity test were about the same for each curing period, and the variation in percentage WC_o before the water-sensitivity tests as a result of varying the gradation was reduced through curing (Figure 10).

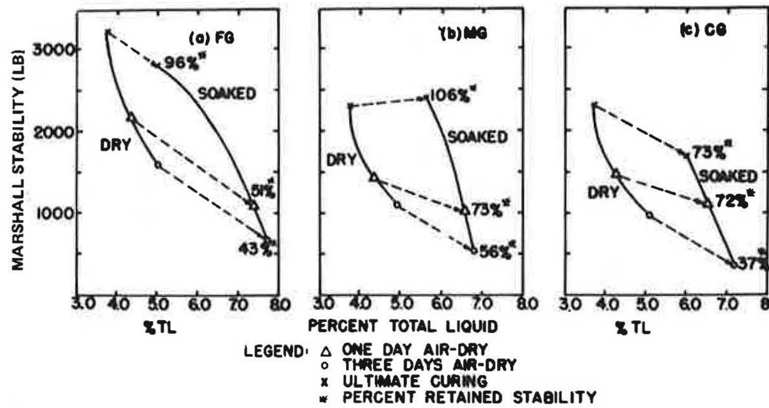


Figure 11. P-values as a function of percentage TL for air-dried and soaked specimens (3.25 percent AE and 3 percent W).

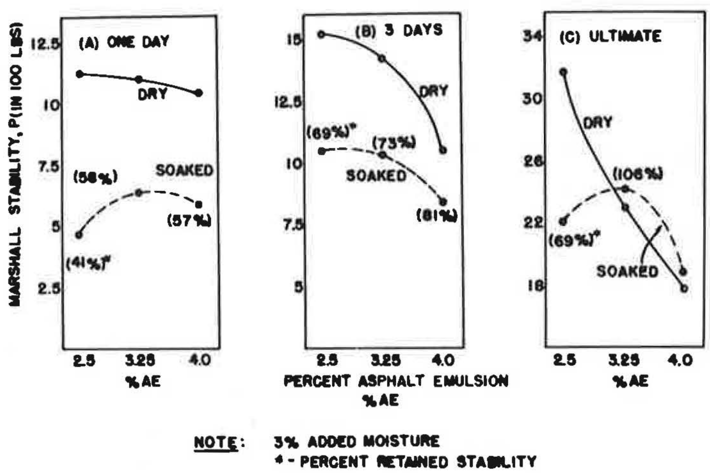


Figure 12. P for dry and soaked specimens after different curing periods (MG aggregate).

Percentage of Retained Stability

Figure 11 shows Marshall stability values as a function of percentage TL at the time of the test for both the dry and soaked specimens. The relations are shown for each aggregate gradation. The percentage of retained stability is shown between brackets on the soaked condition trends.

The dry Marshall stability increases with decreasing percentage TL through the curing process, and stability values for FG mixes are higher. The percentage of retained stability for MG mixes was higher than that for FG or CG mixes at all the curing periods. The MG gradation is closer to the maximum density gradation curve (Fuller's maximum density curve) than the other two gradations, which could be the main factor affecting the performance of the AETM.

In addition, for the same percentage TL that is available in the AETM, the stability values are dependent on the nature or the mechanism of the presence of moisture in the sample (losing moisture through air-dry curing versus gaining moisture through soaking). This is more apparent for the ultimate cured specimens subjected to the water-sensitivity test (see the data points identified by asterisk in Figure 11). The soaked stability values are much higher than the dry stability values for specimens cured for 1 d in spite of the fact that the two conditions correspond to about the same percentage TL.

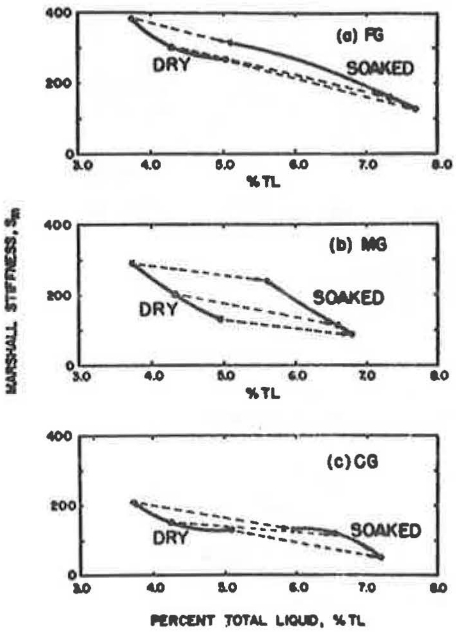
To illustrate the effect of percentage AE and curing time on AETM resistance to water, the Marshall stability values for dry and soaked conditions at the three curing periods for midgradation aggregate mixes are shown in Figure 12. A significant result of this test shows that at any curing level the percentage of retained stability increases with increasing asphalt emulsion content in the mix. In addition, relations between stability and asphalt emulsion content for the soaked samples follow a curvilinear pattern with an optimum percentage AE value that corresponds to a maximum stability value. In contrast, results of the dry test followed a decreasing trend with increasing percentage AE. Longer curing periods for the dry specimens resulted in steeper stability trends (Figure 12). However, after the samples were subjected to the water-sensitivity tests, a significant drop in stability values occurred for samples with a low asphalt residue content.

Percentage of Retained Stiffness

Marshall stiffness (S_m) responded to the water-sensitivity test in the same manner as did the stability values (Figure 13). The MG mixes provided the highest percentage of retained stiffness[(S_m soaked/S_m dry) x 100] of the three mixes. The CG mixes showed the least resistance to water damage. In addition, the trend for S_m versus percentage TL was also dependent on the method by which the moisture was present in the AETM system components.

It has been shown that the dry test results are not enough to provide adequate control and design of the AETM. Water-sensitivity results provide a more important indication of the performance of the mix and must therefore be an integral part of the AETM design procedure.

Figure 13. S_m as a function of percentage TL for air-dried and soaked specimens (3.25 percent AE and 3 percent W).



SUMMARY OF RESULTS

The analysis and evaluation of the test data in the study revealed a number of significant results that pertain to the effect on the mix properties of aggregate gradation, asphalt emulsion content, and added moisture content. A summary of the main results follows.

1. Aggregate gradation significantly affected all AETM properties. It should be noted that the three aggregate gradations fall within certain specified gradation limits. This draws attention to the importance of controlling the aggregate gradation in the mix. Designing the AETM by using a specific aggregate gradation curve (e.g., midpoint of the specification) does not ensure the same performance and properties of the AETM in the field because of the wide bandwidth within the specified aggregate gradation.
2. MG and CG aggregate gradations were close to the "theoretical maximum density gradation," provided an adequate range of particle sizes, and resulted in mixes with higher densities and fewer air voids than the FG mixes.
3. The percentage of total voids for a specific mix was about the same throughout the curing process. The increase in percentage V_A through the curing is accompanied by a decrease in percentage V_W of about the same magnitude.
4. FG mixes provided the highest stability values throughout the curing process. This was generally accompanied by low flow values in comparison with MG or CG mixes.
5. The effect of percentage AE on Marshall stability was not significantly apparent at the early curing condition. However, the asphalt emulsion content significantly affected the Marshall stability when the samples were allowed to cure for longer periods of time.
6. Marshall stiffness (S_m) and index (I_m) parameters show a unique trend that depends on percentage of total liquid, asphalt emulsion content, and amount of added moisture (for a specific aggregate type and gradation). S_m and I_m values decreased with increasing percentage of total liquid at the time of testing.
7. High stiffness indexes (I_m and S_m) were obtained for FG mixes in comparison with MG or CG mixes. In addition, the trends for I_m or S_m versus percentage TL were significantly dependent on aggregate gradation.
8. Marshall stiffness or index or both could be used, in addition to the conventional design parameters for the Marshall method of mix design, to better control the mix properties by setting minimum values for these two parameters.
9. The effect of aggregate gradation on percentage MA depends on the curing state. For the air-dry curing, FG mixes resulted in higher percentage MA than MG and CG mixes. However, at the ultimate condition, the FG mixes absorbed the least amount of moisture, the MG mixes the next least, and the CG mixes the most of the three.
10. The percentage of retained stability was higher for MG mixes than for FG or CG mixes at all curing levels. The percentage of retained stability for any mix combination increased through the curing process.
11. The nature of the water present in the mix (drying through curing versus soaking) affects the response parameters of the AETM.
12. The test results for the unsoaked (dry) specimens showed that Marshall stability increases with decreasing percentage AE in the mix. However, mixes with low percentage AE showed the least resistance to water damage. The shape of the relation between stability and asphalt emulsion content for soaked specimens was different from that obtained for dry samples. This difference was more pronounced when the samples were allowed to cure for extended periods of time. The percentage of retained stability for any mix combination increased through the curing process.

ACKNOWLEDGMENT

The contents of this report reflect our views, and we are responsible for the facts and the 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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Abridgment

Practical Method for Evaluating Fatigue and Fracture Toughness of Pavement Materials

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One of the most frequently occurring modes of distress in asphalt highway pavements is the fatigue cracking that results from traffic loads. Few theories have been suggested by researchers for approaching the problem of fatigue in materials in general and in pavement materials in particular. Linear fracture mechanics is one of the methods that has been used to analyze fatigue. Recent studies such as that by Majidzadeh and Kauffmann (1) show that the method is applicable to asphaltic pavement materials.

Crack behavior can be classified into three distinct modes: (a) opening, (b) in-plane sliding, and (c) tearing (Figure 1). Each crack-mode movement is associated with a stress field in the vicinity of the crack tip. The distribution of stress in the vicinity of the crack tip is a problem related to the mathematical theory of elasticity, in which it can be shown that all stress fields at the crack tip exhibit inverse square root singularities. Thus, the stress field can be expressed for the opening mode as follows:

$$\begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{xy} \end{Bmatrix} = (K_1/\sqrt{2\pi r})\cos(\theta/2) \times \begin{Bmatrix} 1 - \sin(\theta/2)\sin(3\theta/2) \\ 1 + \sin(\theta/2)\sin(3\theta/2) \\ \sin(\theta/2)\cos(3\theta/2) \end{Bmatrix} \qquad (1)$$

The parameter K_1, called the stress-intensity factor, governs the magnitude of the local stresses in the vicinity of the crack tip.

Paris, Gomez, and Anderson (2) first introduced the idea of relating the stress-intensity factor to rates of fatigue crack propagation. Experimental data show that the rate of crack propagation (dc/dN) is proportional to some power of the stress intensity factor; i.e., $dc/dN = AK^n$, where A and n can be characterized as material constants.

The purpose of this paper is to introduce a new method of testing for fatigue and fracture in asphaltic pavement material, a method that is believed to be superior to current procedures. Current methods are discussed and compared with the proposed one.

COMPARISON OF CURRENT AND PROPOSED TEST METHODS

Simplicity of specimen preparation, testing, and analysis of results is the basic advantage of the new method over current procedures. Currently, the determination of fatigue parameters A and n in the fatigue model $dc/dN = AK^n$ is done experimentally by testing beams placed on an elastic foundation and loaded with a cyclic load. Fracture toughness is determined by testing beams that have larger dimensions than the fatigue beams under three-point loading conditions. In the proposed new method, both fatigue testing and fracture testing use an identical experimental setup and identical specimens of discs cut from cylindrical specimens with a triangular notch (Figure 2).

Current Method

To test for fatigue, parameters A and n in the equation $dc/dN = AK^n$ are determined experimentally by testing beam specimens placed on an elastic foundation. The progress of crack growth and the corresponding number of load applications are visually observed and recorded. The steps are

1. Obtain experimental crack minus cycled (c - N) data;
2. Use either a polynomial or nonlinear exponential equation, obtain the best fit to the available experimental c - N data, and develop an analytical expression;
3. Differentiate the analytical (predicted) c - N equation to obtain the dc/dN - N relation;
4. Use the analytical expression available between the stress-intensity factor K and crack length c to develop a set of stress-intensity factor values and their corresponding dc/dN values; and
5. Find the fatigue parameter by using the information in step 4.

For the determination of fracture toughness of asphaltic materials, beam specimens 10.16 by 7.62 by 40.64 cm (4 by 3 by 16 in) are tested. The beams are notched to a depth of 1.524 cm (0.6 in) to produce a c/d ratio of 0.15. They are tested under three-point loading conditions—i.e., a simply supported beam with a central load. Fracture toughness is determined by using the maximum load that the beam can withstand before fracture. This load is used to determine the moment that is used in the calculation of K_{Ic}. A number of investigators have used the K/P-c relations developed for simply supported beams. The best recognized form of such a relation is the Winne and Wundt equation (4).

Proposed Method

Specimens

The specimens are prepared for cylindrical samples. The samples can be manufactured in the laboratory or obtained from existing pavements by coring. Marshall size

Figure 1. Modes of crack deformation.

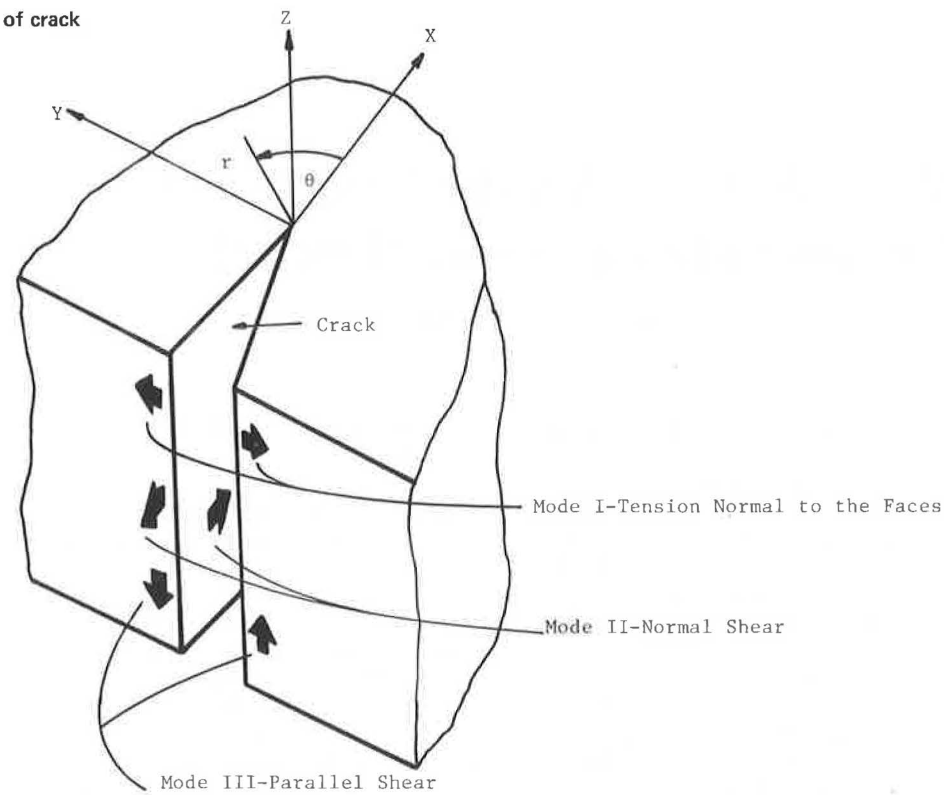
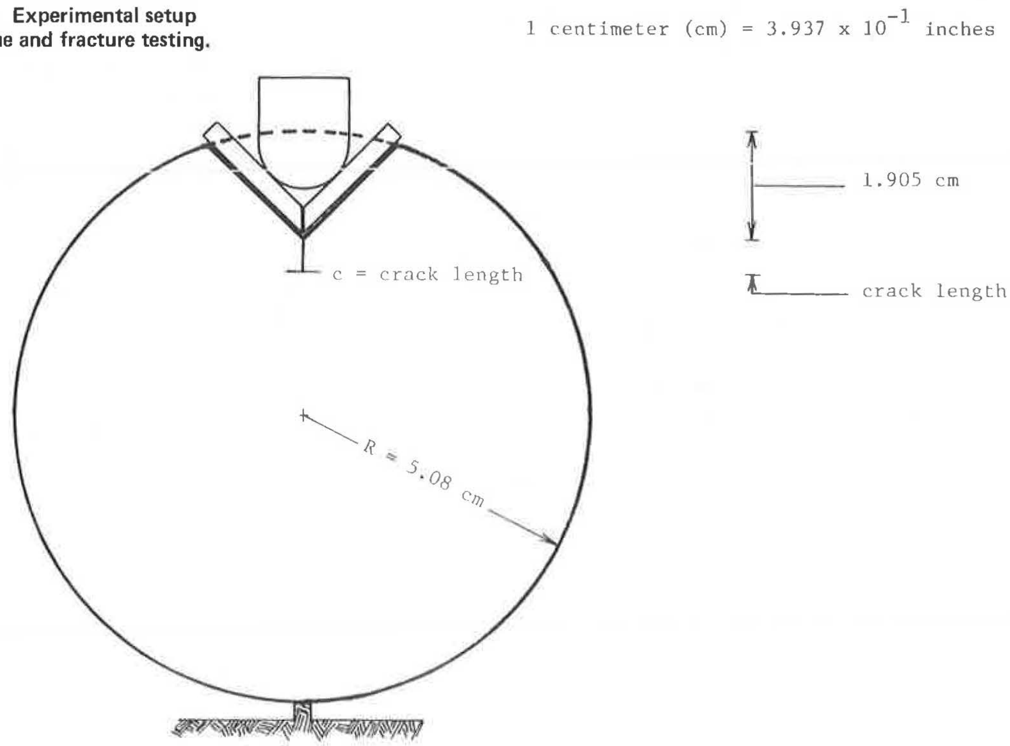


Figure 2. Experimental setup for fatigue and fracture testing.



specimens could also be used. The diameter of the cylinder can vary, but in this study cylinders 10.16 cm (4 in) in diameter were used. Discs of predetermined thickness are cut from the cylinders. The required thickness of the discs is such that the plane strain condition is achieved, a condition assumed in the theoretical modeling of the experiment. A right-angled wedge is cut into the disc specimen to accommodate the loading device. The dimensions of the wedge for the 10.16-cm (4-in) diameter specimen are shown in Figure 2.

Extreme care should be taken in cutting the sample to specifications to ensure symmetry about the vertical axis and smoothness of the loaded surface to avoid stress concentrations not accounted for in the model. Cracks of various lengths can be introduced by extending them from the tip of the wedge by mechanical means such as sawing. Cyclic loading can also induce similar cracks and lead to the eventual fatigue failure of the specimen, which is the main objective of the fatigue and fracture study of pavement materials.

Experimental Setup

The specimen is set on a base so that the wedge points vertically upwards. A three-piece device is used to transmit the vertical force applied by a Material Testing System loading machine. A semicircular piece of rod of sufficient length and rigidity is used to transmit the vertical load to two plates placed on the wedge of the specimen. The plates are rigid enough to transmit the load uniformly to the two surfaces of the wedge. The dimensions of the wedge, the plates, and the semicircular rod are such that symmetry of loading is maintained (Figure 2).

Linear Fracture Mechanics Analysis

Dimensional analysis of Equation 1 indicates that the stress-intensity factor must be linearly related to stress and must be directly related to the square root of a characteristic length. Based on Griffith's original analysis of glass members with cracks and the subsequent extension of that work to more ductile materials, the characteristic length is the crack length in a structural member. Consequently, the magnitude of the stress-intensity factor must be directly related to the magnitude of the applied nominal stress and the square root of crack length c. In any case, the general form of the stress-intensity factor can be written

$$K = f(g) \cdot \sigma \cdot \sqrt{c} \tag{2}$$

where f(g) is a parameter that depends on the specimen and crack geometry and has been subjected to extensive investigation and research.

Buranarom (5) has developed a general finite element computer program that calculates stress-intensity factors for any given geometry, crack length, and loading condition. The analysis is two-dimensional and assumes the existence of plane strain conditions. Normalized stress-intensity factors were related to normalized crack lengths. Before the factors are defined, it should be mentioned that the relations are valid for all thicknesses and all diameter sizes of specimens as long as the ratio of the wedge depth to the radius of the specimen is 3 to 8 (1.905 to 5.08 cm).

The terms in the relation are defined as follows:

$$K = (F_{stress})(F_{geom})(\sqrt{c})(P/tR) \tag{3}$$

where

F_{stress} = stress factor,
F_{geom} = geometry factor,
P = vertically applied load,
t = thickness of sample, and
R = radius of sample.

$$F_{stress} = 6.153\,078 \exp[4.305\,77(c/R)^{2.475}] \tag{4}$$

$$F_{geom} = 3.950\,373 \exp[-3.071\,03(c/R)^{0.25}] \tag{5}$$

So, given c, R, t, and P, the stress-intensity factor K is calculated. The recommended range for c/R to be used in Equations 4 and 5 is $0 \leq c/R \leq 1$. This range is a practical one for almost all conditions of testing.

Experimental Verification of New Method

Different tests were performed to investigate the validity of the new test method, and the results were compared with those obtained through tests of beams on elastic foundations. Each test is described below, and the results are discussed.

Fracture Toughness Testing

Tests were conducted on specimens that had different thicknesses but a uniform crack length of 0.254 cm (0.1 in). Figure 3 summarizes the results. The tests were conducted so that the rate of stress application at the crack tip was maintained at a constant value for all specimen sizes. This was intentionally done so as to have a common basis of comparison for the results.

From Figure 3, it is reasonable to assume that the K_{Ic} values maintain a constant value for a specimen thickness of about 5.08 cm (2 in) and above. Since fracture toughness is a material property and is supposed to be constant at a given temperature and rate of stress, it can be concluded that tests conducted on samples 5.08 cm thick and thicker simulate the theoretical model assumed in the analysis. Specimens with thicknesses of 5.08 cm or more represent plane strain conditions assumed in the analysis. This is an important conclusion since it sets a lower limit of specimen thickness where the fracture toughness tests are valid.

Figure 4 summarizes the results of tests of the effect of loading rate. The tests were conducted on specimens 5.08 cm (2 in) thick that had an original crack length of 0.254 cm (0.1 in). The comparison of these results with the results obtained on beam tests is excellent. As expected, the results show that fracture toughness is a rate-dependent parameter. The stress rates are calculated based on the stress values in the vicinity of the crack tip.

Fatigue Testing

Two sets of fatigue tests were performed, the first to establish the sensitivity of fatigue life to the level of loads and the second to determine the fatigue parameters A and n in the fatigue model.

The first set of tests was performed at room temperature on specimens 5.08 cm (2 in) thick. Each specimen was subjected to a different load level until failure, and the number of cycles of load application to failure was noted. As expected, the fatigue life increased as the magnitude of the cyclic load decreased. The results, which

Figure 3. Variation of fracture toughness with thickness of specimen.

1 centimeter (cm) = 3.937×10^{-1} inches (in.)
1 pascal $\sqrt{M}$ = 9.1×10^{-4} psi $\sqrt{in.}$

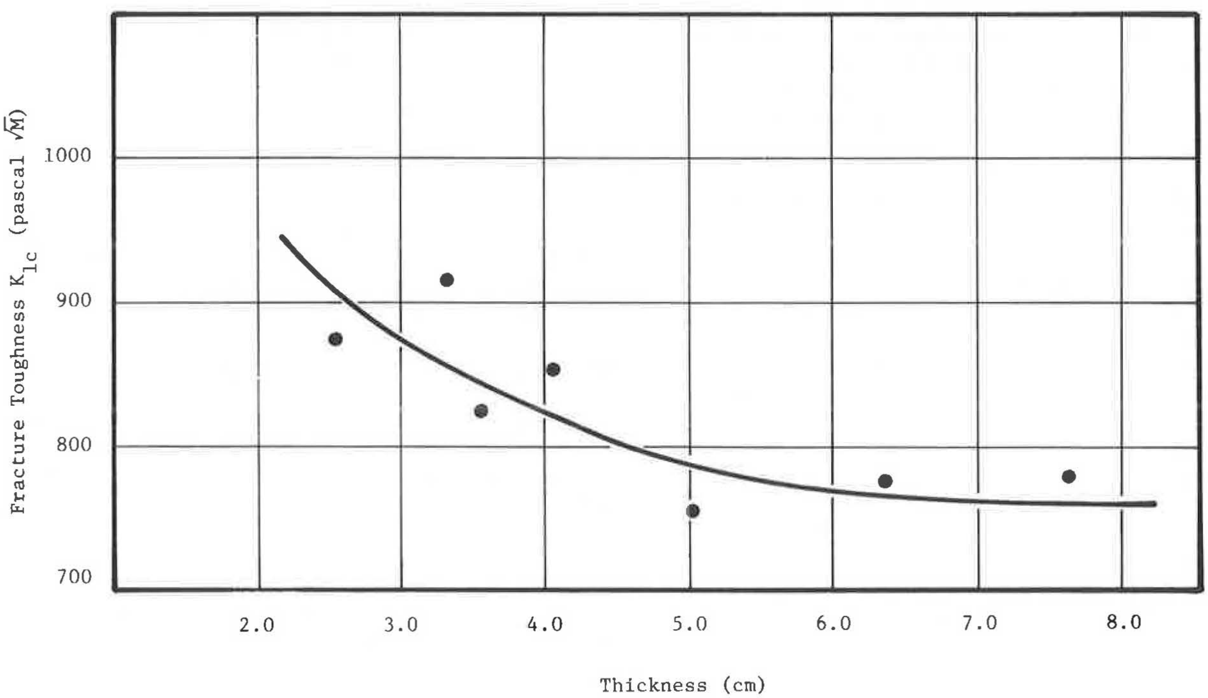


Figure 4. Variation of fracture toughness with rate of stress.

1 pascal/second = 1.45×10^{-4} psi/second
1 pascal $\sqrt{M}$ = 9.1×10^{-4} psi $\sqrt{in.}$

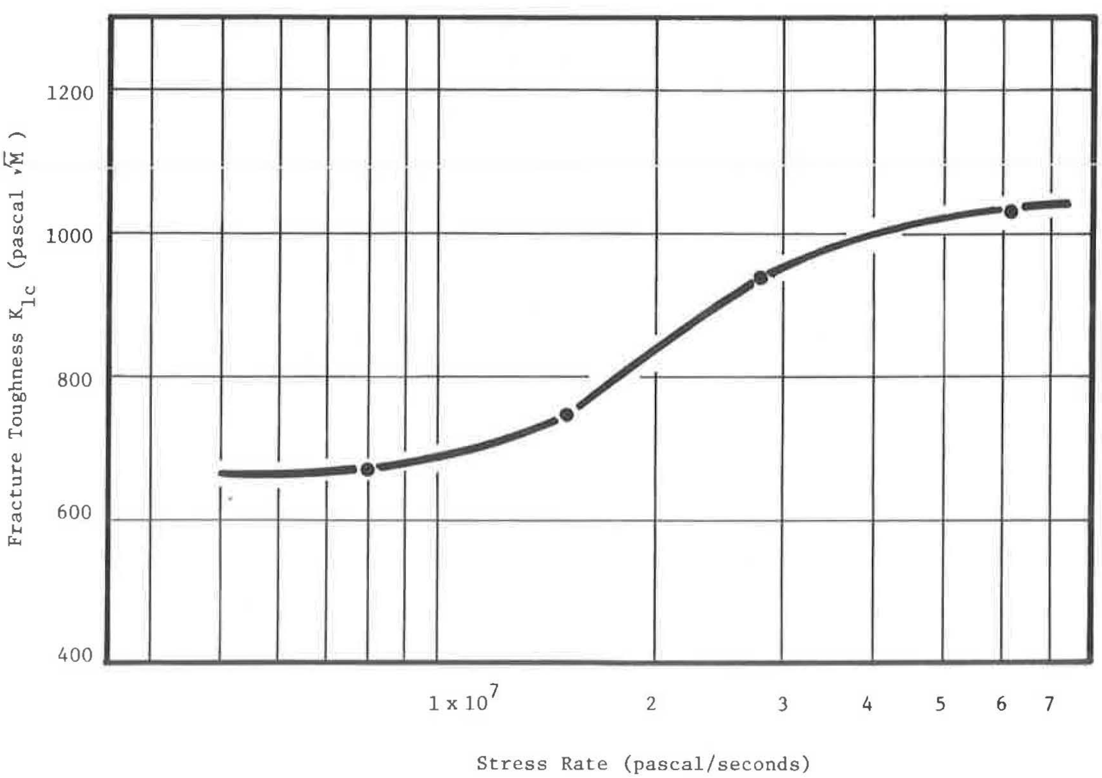


Figure 5. Load versus cycles to failure.

1 Newton = 2.25×10^{-1} pounds force (lbf.)

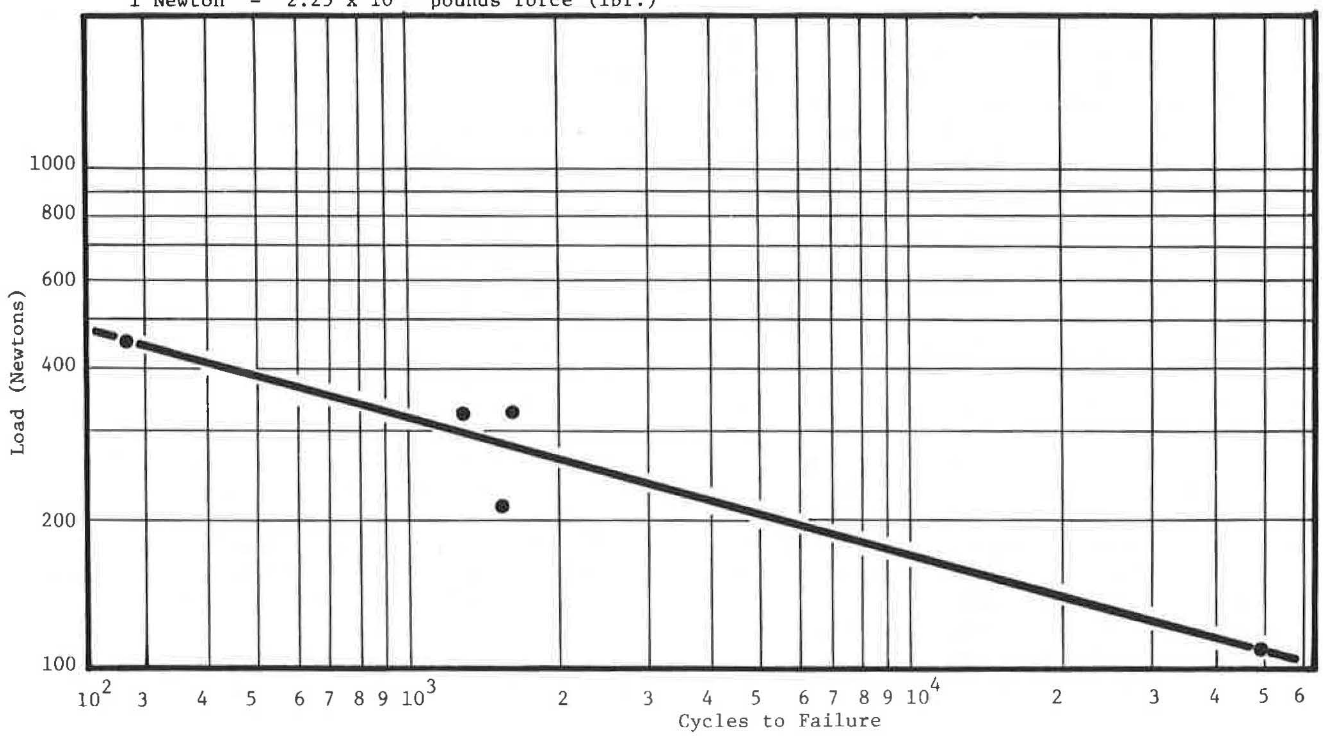
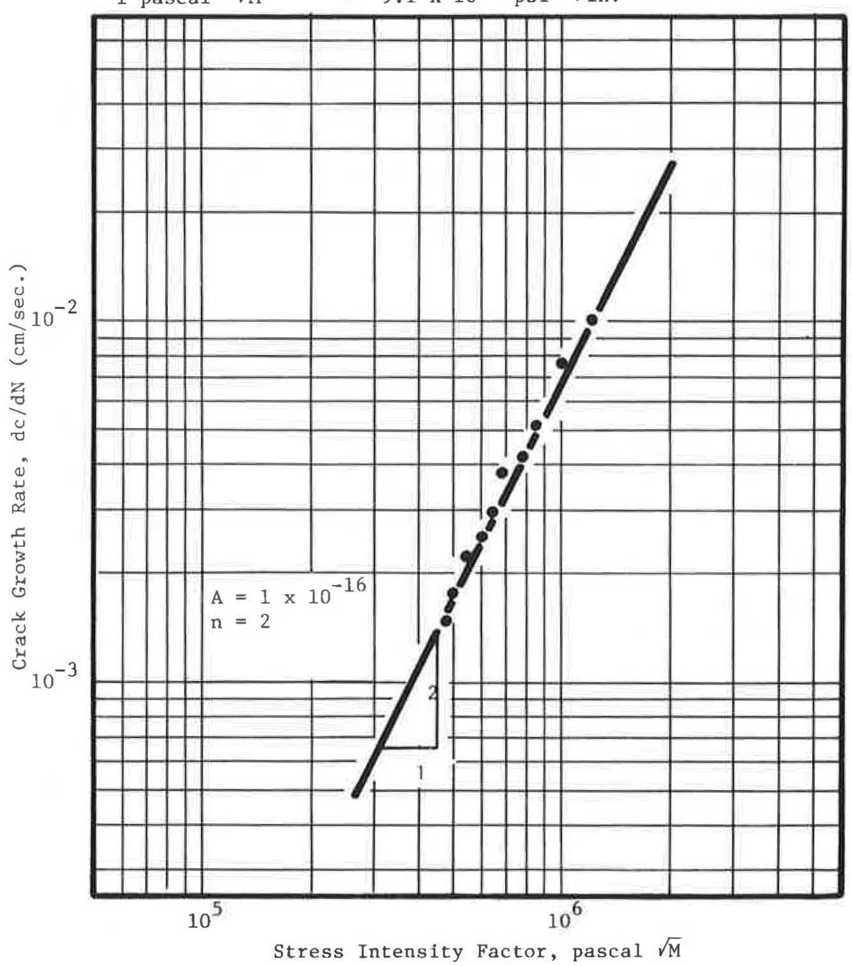


Figure 6. Crack growth rate: dc/dN versus stress-intensity factor.

1 centimeter (cm) = 3.937×10^{-1} inches (in.)

1 pascal $\sqrt{M}$ = 9.1×10^{-4} psi $\sqrt{in.}$



are shown graphically in Figure 5, confirm known relations of load to life.

Tests for fatigue parameters A and n were also performed at room temperature on samples 5.08 cm (2 in) thick. An original crack of 0.254 cm (0.1 in) was introduced in the specimen before testing. For the specific specimen in question, the c - N data were analyzed by following the procedure described earlier. For a value of $n = 2.0$, the value of A was 3.4×10^{-7}. These values compare well with those previously obtained by testing, on elastic foundation, beam specimens that had the same asphalt mix properties. Figure 6 shows the graphical analysis of the data.

CONCLUSIONS AND RECOMMENDATIONS

It is obvious that the new method for testing fatigue and fracture toughness is an applicable and feasible one that has certain advantages over the old one. The following conclusions and recommendations can be made:

1. The new method makes it feasible for actual cores taken from highways to be tested for fatigue and fracture in the laboratory, whereas the method that used beams on elastic foundations required a beam cut from the pavement, a task too difficult to be done properly.
2. The theoretical analysis of the specimen geometry and the experimental setup are much simpler in the new method than in the old one. The stress-intensity factor K is independent of the elastic modulus E of the material.

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Petrographic Insights Into the Susceptibility of Aggregates to Wear and Polishing

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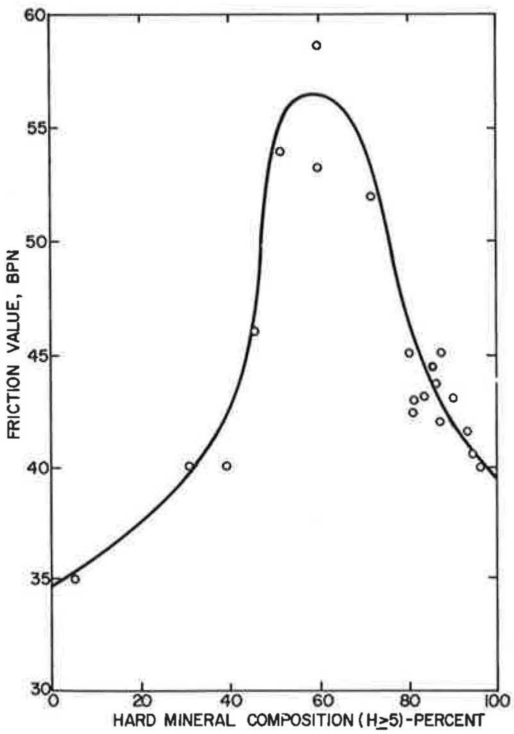
Results from three studies confirm that a strong relation exists between the petrographic properties of an aggregate and its susceptibility to wear and polishing. Constituent mineral composition; hardness; differential hardness; porosity; grain shape, size, and distribution; and bonding between grains or crystals and matrix are all important properties that contribute to aggregate wear resistance and polish resistance. Hard, well-bonded minerals will resist wear but will eventually polish though at a slower rate than softer minerals. Loosely bonded materials will resist polishing but will wear at a rate that may render them not durable. Two subtasks in the studies revealed that an inverse relation exists between the rate of wear of an aggregate and its susceptibility to polishing. To resist both polishing and wear, an aggregate should ideally contain a high percentage of hard, coarse, angular crystals well bonded into a matrix of softer, finer grains, or the hard crystals should be well bonded together in a porous structure so that slow, gradual, irregular fracture of the crystals will occur. Based on the findings from the three studies and other research, a table has been prepared that includes suggestions for aggregate property values that will result in high resistance to both wear and polishing.

It is generally conceded by those concerned with pavement surface skid resistance and wear resistance that these properties are largely a function of aggregate performance, particularly in bituminous surfaces in which the coarse aggregate constitutes the major portion of the surface that comes in contact with vehicle tires. Other factors such as particle size, shape, and gradation; mix design; binder properties; and construction practices are also important, but these factors play a lesser role.

In the past two decades, several studies have been done by concerned agencies and interested researchers to predict the skid-resistance performance of surface aggregates by laboratory tests before the aggregates are used in field construction, particularly in bituminous surfaces. Most of the methods used in these studies were laboratory polishing procedures intended to simulate aggregate polishing by traffic (1, 2, 3, 4). Other studies involved the testing of field installations to obtain a history of skid-resistance performance on the aggregates that were used in these installations (4, 5, 6). Results of both field and laboratory tests often showed significant performance variations, not only between one general group of aggregates and another—for example, between limestone and granite—but also between aggregates of the same group that come from different sources—for example, between one limestone and another or between one granite and another (7, 8, 9, 10). These variations aroused an interest among several researchers in investigating basic factors that influence the skid-resistance performance of various aggregates. Accordingly, several petrographic studies were undertaken

Figure 1. BPN friction values (ASTM E303) versus hard mineral content.



to investigate the intrinsic properties that may control the resistance of aggregates to polishing. Some of these studies were concerned with the investigation of different types of aggregates (9, 10, 11, 12, 13, 14, 15), and others were principally concerned with investigating carbonate aggregates, limestones, and dolomites because they are widely used and because they generally tend to polish faster than most other types of aggregates (7, 8).

INITIAL FINDINGS

Several papers published in the late 1950s and in the 1960s (7, 8, 10, 11, 12) showed that the polish susceptibility of aggregates was associated with the percentage content of soft carbonate minerals, particularly calcite and dolomite in carbonate rocks, and with the fineness and uniformity of grains in other rocks, as in the case of fine-grained serpentines, basalts, and some rhyolites. On the other hand, aggregates composed of minerals that have differential hardness—such as most sandstones, some granites, and some limestones with high silica content—and aggregates that are composed predominantly of one hard mineral but have a porous structure—such as scoria, vesicular slag, expanded shale, clay, or slate—tended to retain polish resistance under prolonged exposure to traffic provided that they could resist premature wear.

RECENT RESEARCH

In the past few years, further work by Dahir and others (9, 13, 14, 15) has confirmed earlier findings that pertain to constituent mineral hardness and to the fineness and uniformity of grains; it has also added some refinements to include the proportion of hard minerals, the degree of differential hardness, crystal size, shape, and distribution in matrix, and the susceptibility of some of the constituent minerals to wear or attrition caused by extreme softness of matrix.

To summarize, it has been found that the optimum hard mineral content that is needed to maintain a high level of long-lasting skid resistance lies in the range of 50 to 70 percent (Figure 1) (9) and that differential hardness between the hard crystals and the softer matrix grains should be at least two numbers on the Mohs hardness scale. Examples of this group include most sandstones and some granites and gneisses. In contrast, some diabase rocks that have not been altered by weathering are composed of minerals that range in hardness from Mohs H = 5 to H = 6, and most dolomitic limestones are composed of minerals that range in hardness from H = 3 to H = 4. Both of these types were found to be more polish susceptible than sandstones and granites (Tables 1 through 3).

Figure 2 shows examples of polish-resistant aggregates and of polish-susceptible aggregates that contain hard minerals. Polish-susceptible aggregates (limestone) that contain soft minerals are not shown because photographs of such material reproduce poorly.

For an aggregate to be both skid resistant and wear resistant, its matrix should consist of minerals that are not so soft and friable as to wear readily or weather easily and thus render the material unusable because of lack of durability. Examples of this type of aggregate include clayey siltstones and some geologically young sandstones (SS-1 in Tables 1 and 3), both of which provide high friction but lack durability. On the other hand, if hard crystals are highly bonded together by a medium of equal hardness or by a strong interlock, they will eventually polish to a smooth surface regardless of their hardness. Examples of this type include high-content quartz aggregate (>90 percent), as in some quartzites and quartz gravels, and some unweathered diabase (Figures 2 and 3 and Tables 1 through 3). Furthermore, to produce and maintain high friction, the hard crystals should be relatively coarse—100 to 250 µm (16)—and have sizes larger than those of grains in the bonding matrix; they should also be of angular shape, neither rounded nor flakey, with protruding asperity angles of 90° or less (17). The hard crystals should have fairly even distribution in the softer matrix and should not occur in concentrations separated by relatively large, smooth matrix patches.

COMPARISON OF RESULTS OF THREE STUDIES

Three independent studies that involve petrographic analysis of aggregates were undertaken at different times. The results of the studies are summarized and compared below.

Study 1

Samples from 20 aggregates were incorporated in 150-mm (6-in) diameter specimens prepared in the laboratory by using an open-graded asphalt mix design (3). The specimens were cured, their surfaces were cleaned with a solvent, and they were then polished dry for 16 h in a circular track apparatus that used four small rotating go-cart pneumatic tires. British pendulum numbers (BPNs), according to ASTM E 303-69, were measured after each 2 h of polishing. Generally, BPNs appeared to reach a stable condition before 16 h of polishing had elapsed. Thin sections were prepared from the aggregates used in the testing, and photomicrographs were made. Tables 1 through

Table 1. Physical properties and skid resistance of sample aggregates.

Study[a]	Aggregate Symbol	Aggregate General Classification	Bulk Specific Gravity[b]	Los Angeles Abrasion Loss (%)	Water Absorption (%)	Polishing Passes Laboratory	Polishing Passes ADT per Lane[c]	Skid Resistance BPN[d]	Skid Resistance SN_{40}
1	LS-1	Limestone	2.85	18	0.30	115 000		35	
	LS-2	Dolomitic limestone	2.87	25	0.40	115 000		39	
	MB	Marble	2.95	29	0.30	115 000		40	
	DB-1	Diabase	2.77	15	0.30	115 000		41	
	RH	Rhyolite	2.67	27	0.30	115 000		42	
	GT	Granite	2.66	41	0.50	115 000		43	
	GN	Gneiss	2.67	29	0.41	115 000		43	
	SL	Slate	2.78	24	0.33	115 000		45	
	SS-1	Arkosic sandstone	2.66	NA	2.55	115 000		59	
2	LS-3	Limestone	2.82	16	0.27	48 000		15	
	LS-4	Dolomitic limestone	2.72	29	0.40	48 000		18	
	DB-2	Diabase	2.78	16	0.30	48 000		21	
	QZ	Quartzite	2.64	36	0.30	48 000		19	
	SS-2	Arkosic sandstone	2.58	20	2.50	48 000		48	
	SS-3	Lithic sandstone	2.65	21	1.20	48 000		34	
3	LS-5	Limestone	2.82	20	0.49		800		36
							7000		35
							2500		40
	LS-6	Limestone	2.72	20	0.27		6500		27
	GL	Gravel (SS)	2.59	24	1.22		2500		63

[a]Study 1 was performed at North Carolina State University, and studies 2 and 3 were performed at Pennsylvania State University.
[b]Physical properties for study 1 were provided by the North Carolina Department of Transportation, and those for study 2 were provided by the Pennsylvania Department of Transportation.
[c]Trucks ranged from 2 to 4 percent on the three pavements of LS-5 and GL; there were no trucks on LS-6.
[d]Measured on laboratory-prepared pavement specimens in study 1 and on ground rock panels in study 2.

Table 2. Mohs hardness of minerals in sample aggregates.

Mineral	Name	Hardness Range
M-1	Chlorite, kaolinite, or sericite	2-2.5
M-2	Mica (biotite or muscovite)	2-3
M-3	Calcite	3
M-4	Dolomite	3.5-4
M-5	Pyroxene (augite)	5-6
M-6	Feldspar (orthoclase or plagioclase)	6
M-7	Limonite, hematite, or magnetite	5-6.5
M-8	Olivine	6.5-7
M-9	Quartz	7
M-10	Others (apatite, amphibole, epidote, pyrite, zircon)	5-7.5

Table 3. Percentage mineral composition, level of bonding, and skid resistance of sample aggregates.

Aggregate	Mineral M-1	M-2	M-3	M-4	M-5	M-6	M-7	M-8	M-9	M-10	Skid Resistance BPN[a]	Skid Resistance SN_{40}	Bonding
LS-1			93						5	2	35		Good, uniform
LS-2			65	30					5		39		Medium to good
MB		5	25	30		5			30	5	40		Medium to good
DB-1					40	50	5	5			41		Highly interlocking laths
RH		5				40	15		40		42		Fine grains, well bonded
GT		10				50			35	5	43		Good, interlocking grains
GN		10				40			40	10	43		Good bonding
SL	55					20			15	10	45		Medium to loose
SS-1	40					10	10		40		59		Loose
LS-3			85	15							15		Good, uniform
LS-4			70	30							18		Good
DB-2		2			45	48	5				21		Highly interlocking laths
QZ									99	1	19		Very well cemented
SS-2	30						10		60		48		Medium to loose
SS-3	20								80		34		Medium to good
LS-5			59	30					10	1		37	Good
LS-6			90	10								27	Good, uniform
GL							30		70			63	Medium to good

[a]Measured on surfaces identified in text and in footnote to Table 1.

3 give pertinent petrographic data and friction data (BPNs) measured after 16 h of polishing. Because of the prolonged polishing, BPNs measured after 16 h were close to SN_{40} (within 5 to 10 numbers) measured with a skid trailer (ASTM E 274) on pavements that incorporate the same aggregates (18). BPNs versus percentage hard mineral content for all 20 aggregates tested are shown in Figure 1. Photomicrographs of representative samples of the aggregates tested are shown in Figure 3.

Study 2

Rock panels 100 by 150 mm (4 by 6 in) were mounted in steel frames, planed by grinding, and then polished with silica abrasive and water by using a reciprocating rubber pad to accelerate the polishing process (19). Eight silica abrasive gradations that ranged in size from 5 to 105 microns were used. Each surface was polished by using 6000 passes of the rubber pad for each abrasive size gradation, starting with the coarsest size and followed successively by the finer sizes. The surface friction of specimens was measured by the British portable tester (BPT) after polishing with each abrasive size. By the end of the polishing cycle, friction appeared to stabilize to a fairly constant state of polish. Friction measurements (BPNs) after the final polishing cycle and corresponding petrographic data are given in Tables 1 through 3. Photomicrographs of thin sections are shown in Figure 3.

Study 3

Five bituminous surfaces in Centre County, Pennsylvania, were designated for routine skid-resistance testing by a full-scale tire skid trailer that conformed to ASTM E 274-70. Representative coarse aggregate particles were taken from each surface. Bihourly skid tests according to ASTM Method E 274 were made in late September for 30 h. The average skid number (SN_{40}) and the petrographic data on the surface aggregates and other pertinent data, including average daily traffic (ADT), are given in Tables 1 through 3. Corresponding photomicrographs of thin sections of the coarse aggregates used in the surfaces are shown in Figure 3.

Figure 2. Aggregates of varying polish susceptibility.

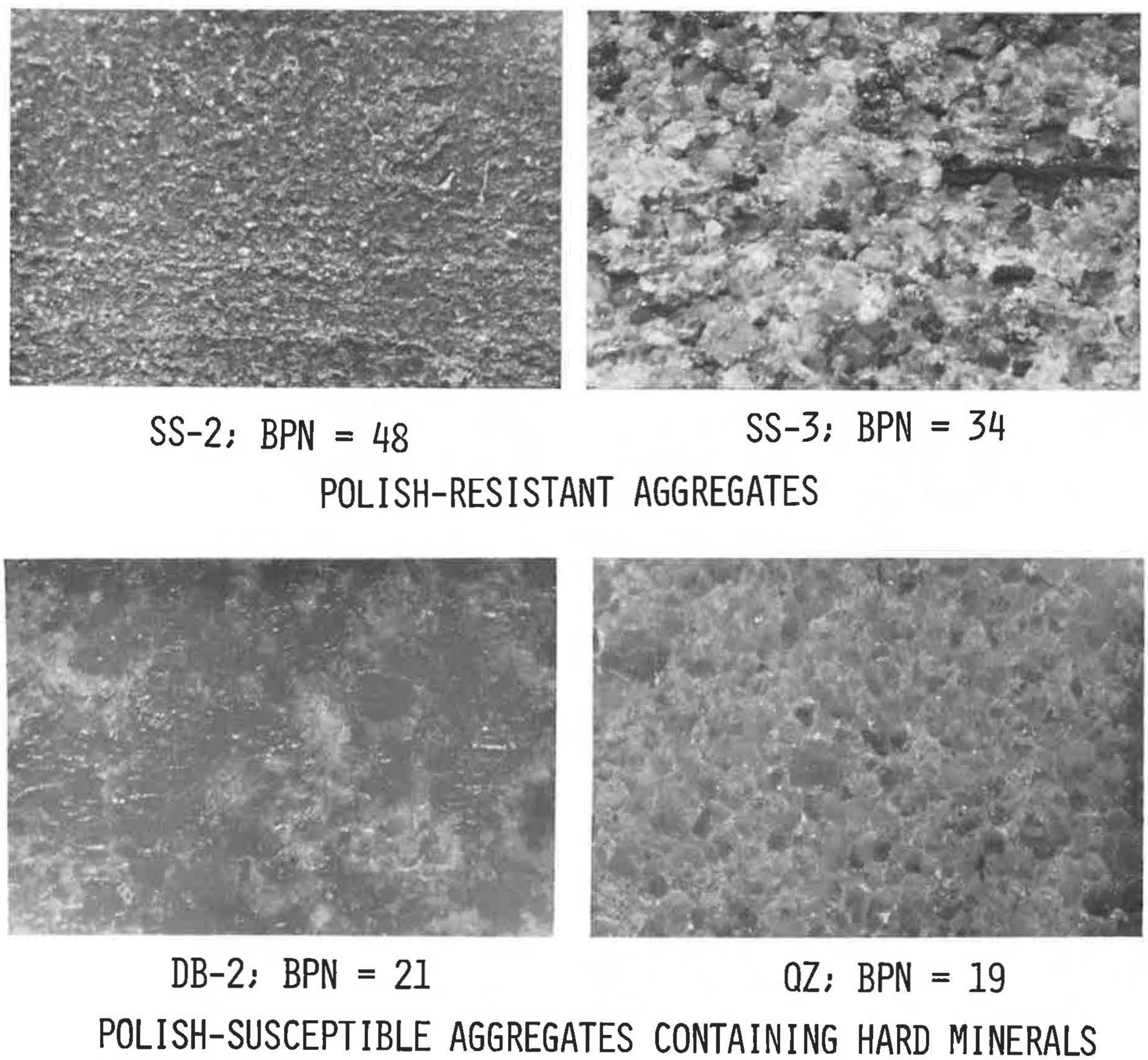
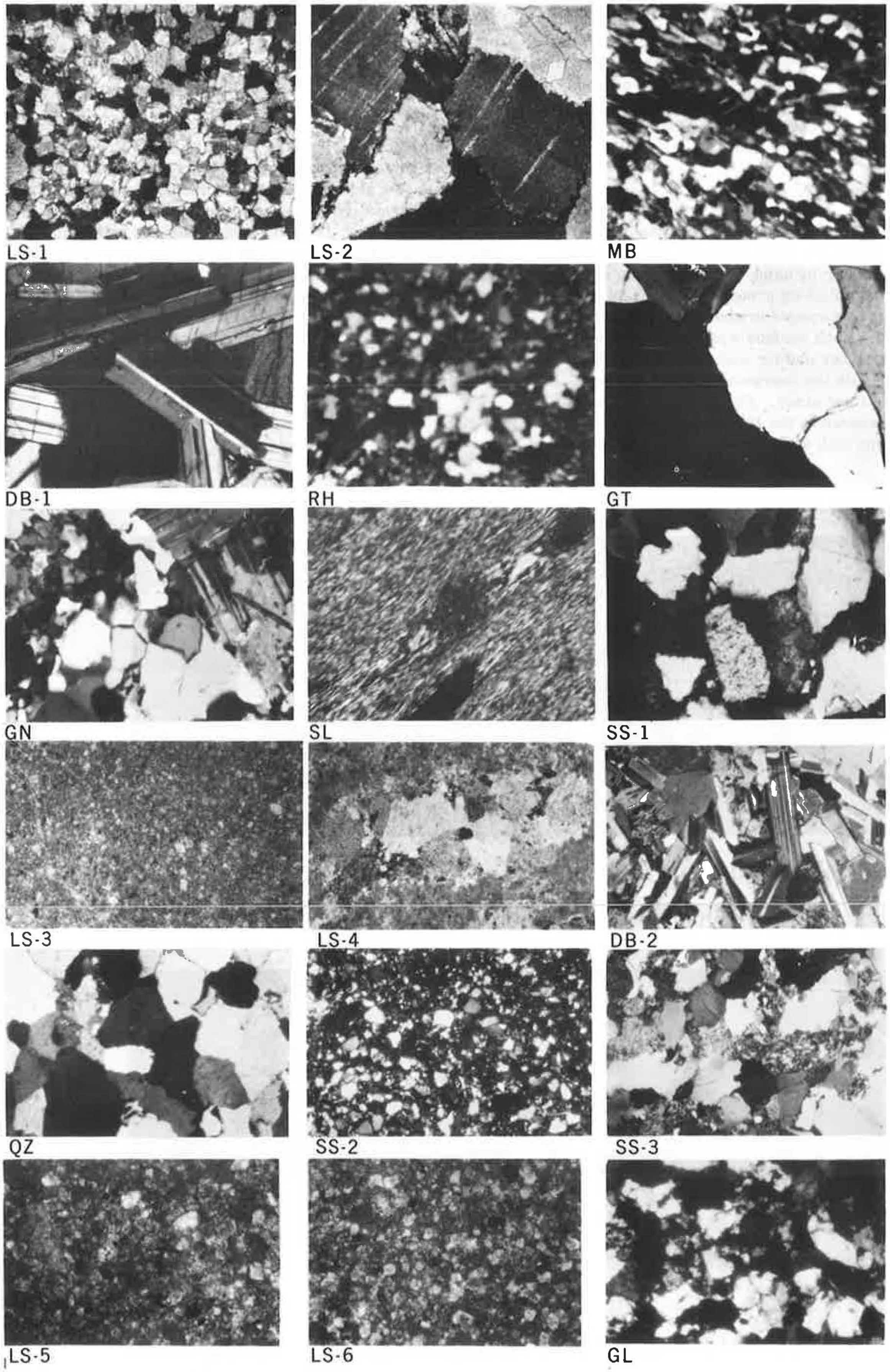


Figure 3. Thin-section photomicrographs of the sample aggregates under crossed nicols (22.4x).



DISCUSSION OF RESULTS

Data in Tables 1 through 3 and Figures 1 through 3 show that a strong relation exists between the petrographic properties of an aggregate and its susceptibility to polishing. Constituent mineral composition; hardness; differential hardness; porosity; grain shape, size, and distribution; and bonding between grains or crystals and matrix are all important properties that contribute to the friction performance an aggregate will exhibit in service.

To provide and maintain a high friction level while enduring the effect of traffic and environmental influences through the expected pavement surface life, an ideal aggregate should have a high composition (50 to 70 percent) of coarse, sharp, hard mineral crystals well distributed and strongly bonded into a matrix of softer mineral(s) of finer grains. Alternatively, the crystals should be well bonded together in a porous structure and optimally have a porosity in the range of 25 to 35 percent (20) so that gradual, irregular fracture of the crystals will occur at a rate sufficiently slow that it will not cause undue surface wear. Table 4 includes suggested target properties for an ideally skid- and wear-resistant aggregate. It is realized that such an aggregate hardly exists in nature and may not be feasible or economical to manufacture with currently known technology. However, this fact does not preclude possible future developments for which the target values in Table 4 or similar values may be used. In the meantime, the closer the properties of an aggregate come to these target values, the better the aggregate will be expected to perform as a pavement surface aggregate.

Obviously, as established in the specifications of the Pennsylvania Department of Transportation (21, 22), the skid-resistance requirements of pavement surfaces that have different levels of traffic and different environmental conditions may be satisfied by aggregates of varying properties. But the data given in Table 4 may serve as a reference and do point to the important properties expected in an aggregate intended for use in a pavement surface, particularly in a bituminous pavement. It is hoped that more attention will be directed by concerned and interested agencies and researchers to investigating the petrographic properties of surface aggregates and that sufficient quantitative data will be generated to permit the development of a specification for surface aggregate based on the study of aggregate petrography.

Table 4. Target values for properties that would enhance skid resistance and wear resistance of aggregates.

Property	Value Range	Reference
Mohs hardness of hard fraction	8-9	(16, 23, 25)
Mohs hardness of soft fraction	6-7	(16, 23)
Differential hardness, min	2-3	(7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 23)
Percentage of hard fraction		
Natural aggregate	50-70	(9)
Artificial aggregate	20-40	(24)
Hard grain or crystal size	150-300 μm, average 200	(24, 25)
Hard grain or crystal shape	Angular tips (≤90°)	(9, 17, 24)
Percentage porosity (vesicularity)	25-35	(20, 25)
Pore size, optimum	125 μm	(25)
Aggregate particle size range	3-13 mm	(6)
Aggregate particle shape	Conical, angular (≤90°)	(17, 25)
Los Angeles abrasion, percent	≤20	(9, 21)
Aggregate abrasion value, percent[a]	≤8	(15, 16, 23, 25)
Aggregate impact value, percent[a]	≤20	(15, 23, 24)
Polished stone value, BPN[b]	≥75	(15, 16, 23, 25)

[a]According to British Standards Institution BS812:75.
[b]According to BS812:75 or ASTM D3319-74T and E303.

COROLLARY TO THE STUDY

To illustrate the dilemma that the highway engineer must face, tests were made to investigate whether a relation exists between aggregate skid resistance and aggregate wear by abrasion. In study 1 (3), 1000-g samples between 9.5 and 4.75 mm (passing the 3/8-in sieve and retained on the no. 4 sieve) in size from each of eight aggregates were abraded by tumbling them dry in a rotating jar-mill for 120 h; 19-mm (0.75-in) hard flint pebbles were used as abrasive. The abraded aggregate particles were then glued in a 150-mm (6-in) diameter frame and tested with the BPT (ASTM E 303-69). Wear as percentage loss and friction in BPN measurements are given in Table 5, and a correlation is shown in Figure 4. The high coefficient of correlation indicates that some relation exists between wear and the friction performance of an aggregate: High friction is associated with a high rate of wear. This relation was not found to be generally true when Los Angeles abrasion test results (ASTM C 131) were correlated with BPN. However, it did hold true for aggregates that were of the same type but came from different sources, as in the case of four granite samples tested in study 1 (9).

A recent limited study at Pennsylvania State University appears to confirm the finding that high friction is associated with high rate of wear. Ten individual particles 12.7 to 9.5 mm (0.5 to 0.375 in) in diameter from each of seven aggregate samples were weighed, mounted in steel holders, reweighed, and then polished for 30 min in a small, rubber-covered drum machine that rotated at 110 revolutions/min (4). A slurry of silica abrasive and water was used to aid the polishing. Friction force was measured by an electronic force cell, and the instantaneous average friction was recorded as a continuous trace by an oscillograph recorder. Initial and final friction forces were read and recorded, and the specimens were reweighed after polishing. A decrease in friction attributable to aggregate polishing

Table 5. Wear, friction, and other physical properties of eight typical paving aggregates.

Aggregate	Symbol	Bulk Specific Gravity	Water Absorption (%)	Los Angeles Abrasion Loss[a] (%)	120-h Jar-Mill Wear Loss (%)	BPN
Arkosic sandstone	SS-1	2.66	2.55	NA	40.2	62.0
Expanded slate	SO-1	1.58	3.50	40	31.1	60.0
Granite gneiss	GN-1	2.67	0.41	29	22.8	54.0
Slate	SL-2	2.78	0.33	24	21.2	54.0
Limestone	LS-1	2.85	0.30	18	14.5	48.5
Granite	GT-1	2.79	0.31	36	13.0	54.0
Dolomitic limestone	LS-2	2.87	0.40	25	10.9	47.5
Expanded glass	SP-1	2.05	2.40	23	7.8	45.0

[a]ASTM C 131.

Table 6. Results of wear and polishing of seven aggregates in Pennsylvania State University small-drum machine.

Aggregate	Initial Weight (g)	Final Weight (g)	Weight Loss (%)	Friction Force (N) Initial	Final	Drop
Juniata, Pennsylvania, red bed	20.8	17.8	14.4	75.58	55.87	19.71
Maryland granite	19.8	18.3	7.6	44.35	25.49	18.86
Texas red rock	17.0	16.5	2.9	70.64	49.29	21.35
Connecticut traprock	18.8	18.5	1.6	87.10	34.52	52.58
Expanded shale	22.6	17.8	21.2	90.39	90.30	0.09
Blast furnace slag	16.4	16.1	1.8	82.16	39.46	42.70
Fused refuse	18.6	18.2	2.2	64.90	34.52	30.38

Note: 1 N = 0.2248 lbf.

Figure 4. Jar-mill wear versus friction number.

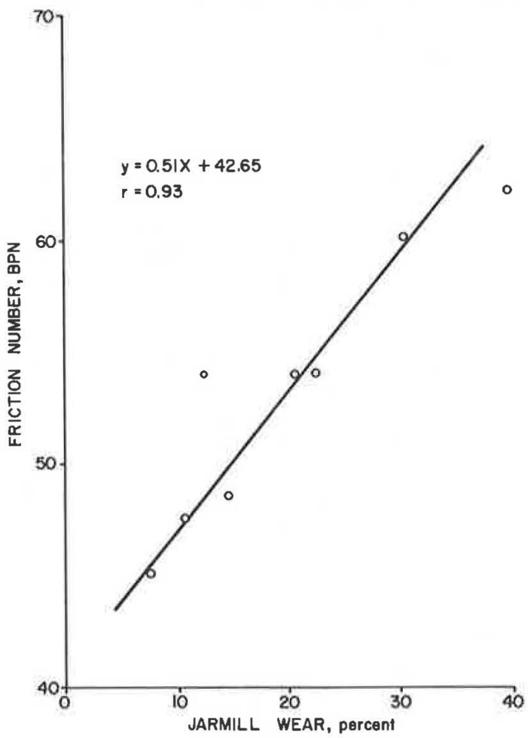
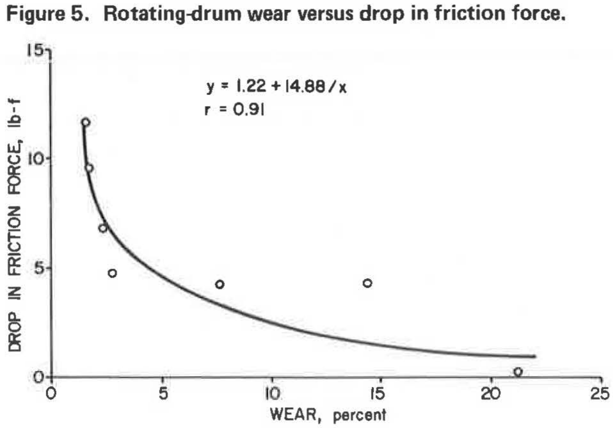


Figure 5. Rotating-drum wear versus drop in friction force.



was indicated by the drop in friction force from initial to final: High polishing is associated with a high drop in friction and vice versa.

The testing results are summarized in Table 6, and drop in friction force versus percentage of wear is shown in Figure 5. Although it is nonlinear, the good correlation confirms that an adverse relation exists between the rate of wear and the friction properties of an aggregate, a fact that poses a dilemma for the highway engineer and requires attainment of a balance between the two parameters—polish resistance and wear resistance—until some aggregate can be economically manufactured to optimize both. Thus far, high resistance to both wear and polishing has been reported only in the production of some relatively expensive synthetic aggregates in Britain (23, 24, 25).

CONCLUSIONS

Three studies by Dahir have indicated that aggregate wear and polish susceptibility may be determined in the laboratory by using petrographic analyses. Constituent mineral properties and bonding largely determine aggregate performance. Hard, well-bonded minerals will resist wear but will eventually polish, though at a slower rate than softer minerals. Loosely bonded, coarse-grained, hard minerals will resist polishing but will wear at a rate that may render them not durable. To resist both wear and polishing, an aggregate should ideally contain a high percentage of hard, coarse, angular crystals that are well bonded into a matrix of softer, finer grains, or the hard crystals should be well bonded together in a porous structure in such a way that slow, gradual, irregular fracture of the crystals will occur. Since an ideal surface aggregate is currently hardly attainable, compromises must be made in the selection of aggregates for practical applications.

Much basic, useful information about the relations between aggregate petrography and the wear and polish susceptibility of aggregates is already known, but more quantitative data are needed to make possible the development of a model or a standard specification that can be used to predetermine the expected skid resistance and wear resistance of an aggregate from petrographic analysis alone.

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The contents of this paper reflect my views, and I am responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the offi-

cial views or policies of the Pennsylvania Department of Transportation, the North Carolina Department of Transportation, or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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Study of Transverse Cracking in Flexible Highway Pavements in Oklahoma

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The results of a research study to determine the nature and extent of transverse cracking on Oklahoma flexible pavements and to investigate the possible causes of this form of distress are reported and discussed. Nine test sites with various degrees of cracking were selected for comprehensive study. An indirect tensile-splitting test apparatus was developed to evaluate the low-temperature tensile properties of asphalt concrete cores obtained from the test sites. Use was made of the concept of the stiffness modulus to characterize the low-temperature behavior of recovered asphalt cements and mixtures. Measured tensile properties of field core samples, particularly tensile strains at failure, were satisfactorily correlated with the observed degree of cracking. The occurrence of transverse cracking was found to increase as failure strain decreased and failure stiffness increased. In addition, the stiffness moduli of recovered asphalts were significantly correlated with the severity of cracking in pavements at the test sites. As the stiffness of the asphalt cements increased, the degree or amount of cracking increased.

Transverse cracking is a serious highway-performance problem in Oklahoma. Open cracks permit the ingress of surface water, which can cause stripping in the asphalt-bound materials and softening of the subgrade. In extreme cases, depression occurs at these transverse cracks because of subgrade softening or pumping of fine materials or both, and secondary cracks develop parallel to the main crack.

Pavement surfaces with this kind of cracking must be repaired to prevent further deterioration and maintain the safe, smooth riding quality desired by the motoring public. If these cracks are not properly sealed, the surface condition of the pavement may deteriorate to the point that complete resurfacing is required long before the design life of the pavement is reached. Frequently, this expensive solution is unsatisfactory because these cracks tend to reflect through the new surfacing in a short time if they have not been adequately sealed before overlaying.

POSSIBLE CAUSES OF AND MAJOR FACTORS IN TRANSVERSE CRACKING

Transverse cracking can result from a variety of causes. However, many of the studies conducted in different parts of the United States and Canada (1, 2, 3, 4, 5) concluded that this type of cracking could be attributed to the thermal stresses that develop in the pavement as a result of temperature changes, particularly changes in the low-temperature range. One of the most significant variables was found to be the consistency-related characteristics of the bitumen used in the surface layer (1). In addition, factors of pavement age, subgrade type, thickness of the asphalt layer, and traffic loads have been directly related to this problem. It is important to realize that these different factors do not necessarily act independently but may be combined. For instance, the combination of traffic loads with a drop in temperature in an asphalt concrete pavement could create stress of sufficient magnitude to cause a transverse fracture of the pavement.

METHOD AND SCOPE OF STUDY

The primary objective of this study was to determine the nature and extent of transverse cracks on selected Oklahoma pavements and investigate the causes of this form of distress. The study dealt primarily with the bituminous components of the pavement and their influence on or contribution to transverse cracking.

Nine test sites with various degrees of cracking were selected for comprehensive study. Mapping and counting techniques were used to determine the severity of cracking at each site. Pavement cores 152.4 mm (6.0 in) in diameter that spanned newly developed cracks were obtained to study the depth of penetration of these cracks in the asphalt pavement layers. Other cores 101.6 mm (4.0 in) in diameter were taken at random locations in the vicinity of the transverse cracks for further laboratory testing. The low-temperature tensile properties of these 101.6-mm core specimens were determined by using an indirect tensile-splitting apparatus. The asphalt binder was recovered from the respective core specimens, and tests were performed to evaluate the rheological properties of the recovered asphalts. Use was made of the concept of the stiffness modulus to characterize the behavior of the recovered asphalt cements and field core specimens at low temperatures. Test results were analyzed, and correlations with the observed degree of pavement cracking were made by using the statistical analysis system (SAS) computer program (6).

EXPERIMENTAL PROGRAM AND TEST PROCEDURES

The experimental program of this study was divided into two phases: (a) surveying and core sampling at the field test sites and (b) laboratory testing to evaluate the properties of the respective field core samples.

Field Testing Program

Pavement test sites where various degrees of cracking had occurred were selected so that the contributing factors at the sites could be identified and compared. A total of nine test sites were selected for detailed investigation. These selected sites included four sections on US-177 and five sections on I-35 and I-40 in Oklahoma. Two of the Interstate sites had almost no cracking and were chosen for comparative purposes. Information on pavement, base, and subgrade soil at the respective test sites is given in Table 1.

At each test site, a 152-m (500-ft) length of pavement that satisfied established requirements for safe sight distance was chosen for detailed surveying, counting, and coring of cracks. Transverse crack patterns were sketched on an appropriate field data sheet. These cracks were classified according to type (multiple, full, half, and part), and the number of various types of cracks per 152 m of pavement length was used to establish a cracking index (CI) for each test site (7). Some of the "part" cracks were closely inspected, and suitable ones were chosen for further study. Large-diameter cores that

Table 1. Test site data.

		Original Surfacing		Overlay			Base		Subgrade		
Test Site	Highway	Depth (mm)	Type[a]	Depth (mm)	Type	Date	Depth (mm)	Type	AASHO Classification	Plasticity	CI
1	US-177	38.1 76.2	C A	38.1	C	1972	203.2	Sand asphalt	A-6(10)	LL = 34, PI = 14	6.5
2	US-177	38.1 76.2	C A	38.1	C	1972	203.2	Stabilized aggregate	A-4(5)	LL = 24, PI = 6	10.5
3	US-177	38.1 76.2	C A	38.1	C	1971	203.2	Stabilized aggregate	A-7-6	LL = 42, PI = 21	15.5
4	US-177	38.1 76.2	C A	38.1	C	1971	203.2	Stabilized aggregate	A-2-4	NP	24.5
5	I-35	38.1 76.2	C A				304.8	Stabilized aggregate	A-6(2)	LL = 27, PI = 12	2.0
6	I-35	38.1 76.2	C A	38.1	C	1971	304.8	Sand asphalt	A-4(5)	LL = 25, PI = 8	9.0
7	I-35	38.1 76.2	C A	38.1	C	1972	254.0	Black base	A-6(10)	LL = 36, PI = 16	20.0
8	I-35	38.6 76.2	C A				304.8	Stabilized aggregate	A-6(2)	LL = 24, PI = 11	0.0
9	I-40	38.6 76.2	C A	38.1	C	1972	254.0	Black base	A-2-4	NP	0.5

Notes: 1 mm = 0.0394 in.
LL = liquid limit, PI = plasticity index, and NP = nonplastic.
[a]Oklahoma Department of Transportation asphalt concrete mixtures.

spanned these cracks were obtained in an attempt to determine the mechanism of transverse cracking.

Another part of the field study consisted of securing pavement core samples for laboratory testing. In order to consider the effect of traffic densification on the asphalt paving materials, cores were taken from the pavements at both wheel-path and non-wheel-path locations. The tensile properties of these core samples were to be evaluated at three different low temperatures and, to increase the reliability of the results, three test replicates for each combination of location and temperature were needed. Thus, eighteen 101.6-mm (4.0-in) diameter core specimens were taken from each test site. Randomization principles were used to select the locations of these field cores within the chosen 152-m (500-ft) length of pavement.

Laboratory Testing Program

Careful examination of the large-diameter cores taken at recently developed cracks revealed that the majority of these "beginning" cracks did not extend through the pavement matrix. Apparently, the cracks had originated at the surface and had propagated to only a limited depth in the underlying layers.

Based on the hypothesis that these transverse cracks were caused by tensile forces developed in the pavement surface as a result of the low-temperature response of the asphalt paving mix, the research approach was directed toward evaluating the low-temperature behavior of the asphalt materials and mixtures and correlating this behavior with actual field cracking data, i.e., the cracking indexes of the pavement sections. The tensile properties of the field samples were determined over a range of low temperatures: 0°C, -5°C, and -10°C (32°F, 23°F, and 14°F). Some preliminary tensile-splitting tests of field specimens were done at a lower temperature [-20°C (-4°F)]. However, at this temperature a very brittle behavior was observed, and splitting occurred with little or no deformation of the specimens.

Indirect Tensile-Splitting Test

The tensile-splitting test apparatus used in the study is described in detail elsewhere (8). A load cell was used to measure applied compressive loads, which were applied at a rate of 1.52 mm/min (0.06 in/min). Horizontal deformation of the specimen was measured by series-connected linear variable differential transducers (LVDTs). The output signals from the load cell and differential transducers were fed to an X-Y recorder that plotted a continous load-deformation trace. From this load-deformation trace, the total horizontal deformation (X_{TF}) and the corresponding maximum load causing failure (P_{max}) were determined. Tensile strength was calculated as follows:

$$\sigma_{TF} = 2P_{max}/\pi td \qquad (1)$$

where

σ_{TF} = tensile strength (MPa),
P_{max} = maximum load causing failure (N),
t = thickness of the specimen (mm), and
d = diameter of the specimen (mm).

Total tensile strain at failure for a 101.6-mm (4.0-in) diameter specimen with a 12.7-mm (0.5-in) curved loading strip was determined as follows (9):

$$\epsilon_{TF} = X_{TF}[(0.1185\nu + 0.038\,96)/(0.2494\nu + 0.067\,30)] \qquad (2)$$

where

ϵ_{TF} = total tensile strain at failure (mm/mm),
X_{TF} = total horizontal deformation at failure (mm), and
ν = Poisson's ratio of the asphalt concrete material.

Previous studies have shown that Poisson's ratio for asphalt concrete materials varies between 0.25 and 0.35 and averages 0.3 (9). By substituting this value in Equation 2, the total tensile strain at failure can be expressed as

$$\epsilon_{TF} = 0.524\, X_{TF} \quad (3)$$

From these relations, ultimate failure stiffness (S_{TF}) was computed as the ratio between tensile strength and total tensile strain at failure. It was felt that failure stiffness might be a better indicator of the low-temperature behavior of the material since it combines both tensile strength and failure strain responses.

Evaluation of Field Core Samples

The field specimens used in the tensile-splitting tests were obtained by cutting the surface layer, which was approximately 50.8 mm (2.0 in) thick, from the cores with a concrete saw. Before testing, the dimensions of these specimens were measured and the bulk specific gravity was determined. After testing, the theoretical maximum specific gravity of the surface mixtures in the field specimens was determined by using ASTM D 2041.

The asphalt binder from these specimens was extracted by using 1,1,1-trichloroethane in accordance with method B of ASTM D 2172, and the asphalt cement was recovered from the extraction solution by using a modification of ASTM D 1856. Initial studies indicated that the recovered asphalt was excessively hardened during the standard recovery procedure. Tests made on asphalt samples of known penetration grades showed that this hardening could be minimized by reducing the final test temperature to 155° C (311°F) and maintaining it for only 6 min.

The recovered asphalts were tested for penetration, ring-and-ball softening point, and kinematic and absolute viscosity in accordance with standard American Society for Testing and Materials (ASTM) test procedures. These properties were used in calculating the stiffness moduli of the respective asphalts and asphalt-aggregate mixtures according to McLeod's approach (10).

DISCUSSION OF RESULTS

Tensile Properties at Low Temperatures

The results of tensile-splitting tests were statistically analyzed to test for evidence of real differences in the observed values and to estimate the magnitude of such differences. A significance level of 0.05 was considered the criterion for the rejection or acceptance of the hypothesis of no differences. Correlation studies were performed to investigate the general trend of the relation between each of the tensile properties and the cracking index. To minimize the effect of variation in material properties, these correlation studies were made separately on the test results for the wheel-path and non-wheel-path specimens. A Hewlett-Packard calculator plotter (model 9862A) was used to plot the regression lines, and the coefficients of correlation and determination (r and R^2) for the first- and second-degree polynomials respectively were determined by using the SAS computer program.

The tensile-splitting test results (8) were used as input data for a statistical analysis of variance. The results of this analysis of variance and of the correlation studies are discussed for the three tensile properties investigated.

Tensile Strength

Analysis of variance of all test results indicated a strong evidence of locational (wheel-path and non-wheel-path) differences in values of tensile strength. The observed significance level $\hat{\alpha}$ was 0.0014. The tensile strength of wheel-path specimens was considerably greater than that of non-wheel-path specimens. This can be attributed to the effect of the relatively higher pavement densities developed under traffic loads.

Test temperature had a very significant effect on the tensile strength value ($\hat{\alpha} = 0.0001$). Average tensile strengths at -10° C (14° F) were noticeably higher than those at -5° C and 0° C (23° F and 32° F) respectively. These results indicated the general behavior of the asphalt concrete mixtures at low temperatures; i.e., as temperature decreased, the mixture became rigid, lost some of its plasticity, and behaved in an elastic manner. Consequently, an increase in average tensile stress at failure could be expected.

The analysis of variance also showed that the greater variation in the tensile strength values was attributable to differences in the properties of the test sites ($\hat{\alpha} = 0.0001$). In general, higher tensile strengths were observed for the Interstate sites (sites 5 through 9). This may indicate the importance of the quality and adequacy of pavement design and construction procedures to service behavior at low temperatures.

Figures 1 and 2 show the regression lines, correlation coefficients (r), and corresponding observed significance levels ($\hat{\alpha}$) of the relation between tensile strength and cracking index for wheel-path and non-wheel-path specimens. In general, test sites with a high degree of cracking showed lower tensile strengths. However, a considerable scattering of tensile strength values was observed, and this reduced the observed correlation coefficients. Much of this data scatter was considered to be a result of the natural nonhomogeneity of the paving materials. It has also been suggested that the scatter might have been caused by the fact that the test temperatures were considerably higher than the low temperatures that initiated the transverse cracking in the pavement.

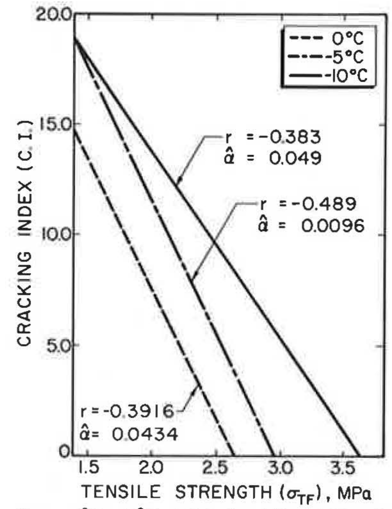
Tensile Strain at Failure

The calculated tensile strains at failure for wheel-path specimens were considerably higher than those for non-wheel-path specimens. The analysis of variance of the test results showed that the observed significance level associated with differences in location was 0.0042. It generally appeared that the average tensile strain at failure significantly decreased as temperature decreased. The observed significance level was 0.034. This substantiated the effect of service temperature on the behavior of the asphalt concrete mixtures; i.e., failure strains at low temperatures are related to the elastic response of the asphalt mixture.

Again, tensile strains at failure differed considerably between test sites ($\hat{\alpha} = 0.001$), and greater tensile strains at failure were associated with the higher quality pavements at the Interstate test sites.

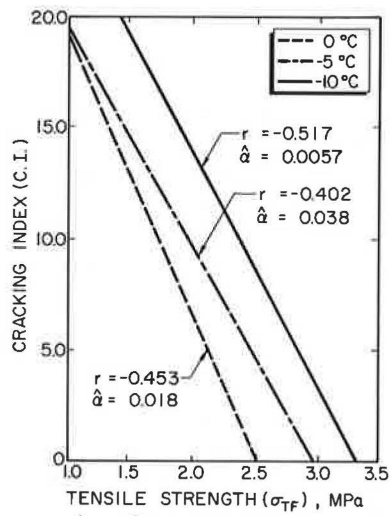
Results of the correlation analysis indicated a strong relation between tensile strains at failure and observed degree of cracking (Figures 3 and 4). The coefficients of determination (R^2) associated with this relation were considerably higher than those for the tensile strength values. The occurrence of transverse cracking was found to increase as the tensile failure strain of the pavement specimens decreased. This suggests that resistance to cracking at any low temperature may be a function of the strain

Figure 1. Tensile strength of wheel-path specimens versus cracking index.



Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

Figure 2. Tensile strength of non-wheel-path specimens versus cracking index.



Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

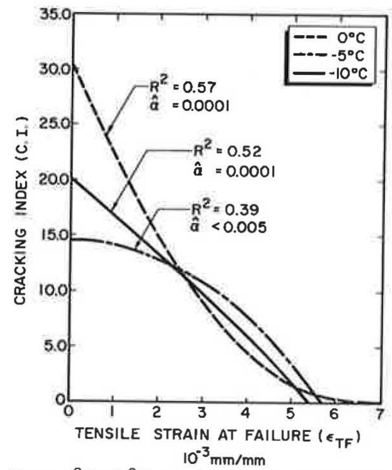
capability of the asphalt concrete mixture at that temperature. It also appears that a permissible, or standard, failure strain for pavement mixtures used in a given geographic region could be established.

Ultimate Failure Stiffness

The analysis of variance did not indicate that a difference in location influenced the failure stiffness (S_{TF}) values. However, a strong evidence of temperature differences was indicated ($\hat{\alpha} = 0.005$). In general, higher average values of failure stiffness were observed at lower temperatures. These results emphasized the earlier findings and indicated that the test sites were significantly different with respect to their ultimate failure stiffness responses.

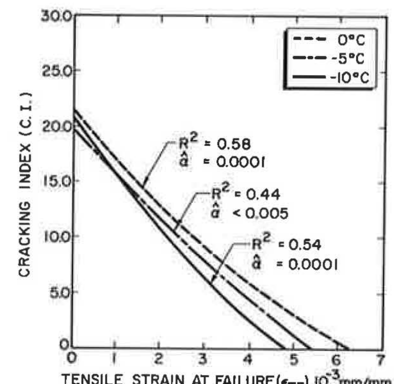
Results of the correlation analysis, shown in Figure 5, indicated that the cracking index was proportional to the ultimate failure stiffness at all test temperatures and that test sites with a high degree of cracking generally exhibited the higher failure stiffness values. However, it should be noted that the determined correlation coefficients were relatively small. The best correlation between failure stiffness and degree of cracking was that at 0° C (32° F).

Figure 3. Tensile failure strain of wheel-path specimens versus cracking index.



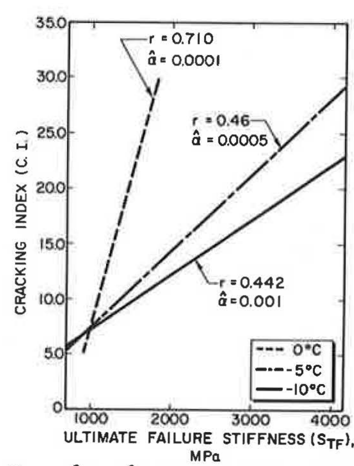
Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 mm = 0.039 in.

Figure 4. Tensile failure strain of non-wheel-path specimens versus cracking index.



Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 mm = 0.039 in.

Figure 5. Ultimate failure stiffness versus cracking index.



Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

Stiffness Moduli of Recovered Asphalt Cements and Mixtures

McLeod's method (10) was used to calculate the stiffness moduli of recovered asphalt cements and mixtures. A study of climatological data over a 73-year period (11) revealed that the lowest minimum air temperature recorded in Oklahoma City was -27.22° C (-17° F). Oklahoma City is more or less centrally located in the state as were the pavement sites studied. Based on temperature data reported in another research study (12), the temperature

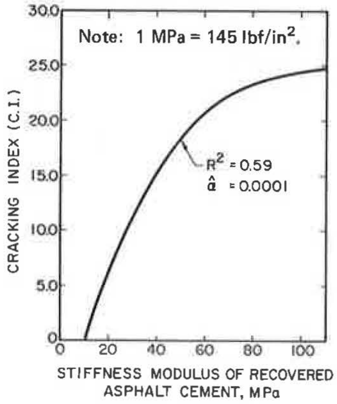


Figure 6. Stiffness moduli of recovered asphalt cements versus cracking index.

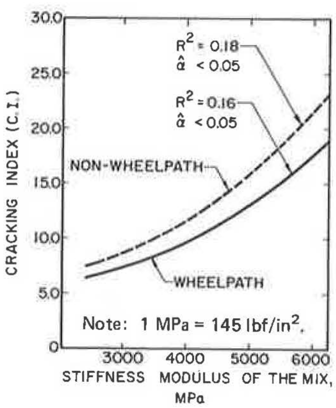


Figure 7. Stiffness moduli of field mixtures versus cracking index.

at a pavement depth of 50.9 mm (2.0 in) was about 3.9° to 4.4°C (7° to 8°F) higher than the air temperature. Consequently, modulus of stiffness values were calculated at a temperature of -23.33° C (10° F).

Correlation studies indicated that the low-temperature stiffness moduli of the recovered asphalt cements could be related to the observed degree of cracking or the cracking index (Figure 6). Pavement sections with a high degree of cracking were generally those that had the stiffer asphalt cements in the surface layer. Only site 7 on I-40 was an exception to this trend. The cracking index of this section was 20.0, but the stiffness modulus was relatively low. It is possible that the high frequency of cracking at this site was associated with a subgrade problem or other related factors. If the data for this site are disregarded, the coefficient of determination goes up to 0.82. This is a remarkably high value considering the variables involved in this generalized relation.

A similar relation was found for the stiffness moduli of the field mixtures (Figure 7). However, the coefficients of determination associated with this relation were smaller than those of the previous one. This can be attributed to the variation in the other mix properties of the surfacing at the individual test sites—i.e., variation in asphalt content, specific gravity, and percentage of density. Again, if the stiffness moduli of the mixtures at site 7 are disregarded, the coefficients of determination increase to 0.31 and 0.32 respectively for the wheel-path and non-wheel-path relations.

The previous findings were substantiated by the results of another research study performed in Texas (13). Transverse cracking in pavements in central and west Texas was significantly related to hardening of the asphalt cement. As asphalt cement hardens with time, a considerable increase in its stiffness occurs. This results in an increased low-temperature cracking susceptibility of the pavement surface.

CONCLUSIONS

The following major conclusions were derived from this study:

1. Examination of recently developed transverse cracks revealed that, in most cases, the cracks had originated at the pavement surface. Thus, the major cause of these cracks appears to be the cold-temperature contraction of the asphalt concrete surface layer.
2. Temperature had a highly significant effect on the measured tensile properties of the paving mixtures. As temperature decreased, tensile strengths and failure stiffness remarkably increased and tensile strains at failure decreased. This is primarily a result of the increase in stiffness of the asphalt binder.
3. A satisfactory correlation was found between the results of the tensile-splitting tests and the observed degree of cracking. The occurrence of transverse cracking was found to increase as failure strain decreased and failure stiffness increased.
4. A permissible or standard failure strain can be established for a pavement mixture in a given geographic region. Such a value could be used in future mix design procedures in which asphalt viscosity, asphalt content, and aggregate gradation are modified to meet design criteria for failure strain.
5. The stiffness moduli of recovered asphalts, determined at the expected minimum temperature in central Oklahoma, were significantly correlated with the cracking indexes of the pavement test sites. The stiffer or harder the asphalt cement in a pavement was, the greater was the degree of transverse cracking.

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Minnesota Heat-Transfer Method for Recycling Bituminous Pavement

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A method for hot-mix-recycling of in-place bituminous pavement that was developed for use on a roadway project in Maplewood, Minnesota, is described. An urban four-lane street was reconstructed to a four-lane divided roadway with turn lanes and transit bus turnouts. The bituminous material was salvaged and recycled along with the upper 0.33 m (1 ft) of gravel base. The old material was scarified, picked up, and hauled to the mixing plant where it was run through a normal three-crusher plant (one jaw and two rolls) and stockpiled. The upper 0.33 m of gravel base was loaded off the roadway, hauled to the plant site, and stockpiled. The salvaged gravel was conveyed from the stockpile to the dryer where it was heated to 232° C to 260° C (450° F to 500° F). The unheated, crushed, salvaged bituminous material was conveyed directly from the stockpile to the weigh hopper above the pug-mill mixer. For the base course, 50 percent salvaged bituminous material was blended with 50 percent salvaged gravel at the mixer for 20 s; then 3 percent asphalt was added and wet-mixed for 30 s. The proportions for the binder course were 40 percent salvaged bituminous, 60 percent salvaged gravel, and 3.5 percent asphalt. The paving was done by use of conventional equipment and conventional methods. Test results for the finished product and the performance of the pavement to date (winter, spring, and summer) indicate that the structure is comparable to a conventional full-depth asphalt base.

Recent developments in the road-building industry have prompted a reassessment of the recycling of bituminous pavements and base aggregates on construction projects. These developments were the rising cost of asphalt, an awareness of the need to conserve finite deposits of nonrenewable natural resources, and a difficulty in finding environmentally and politically acceptable disposal sites for the debris that results from construction and demolition.

Recycling of in-place road materials is not a new concept. In the past, however, when pavement surface and base materials were reused in selected subgrade or base applications, no benefit was realized from the old asphalt binder. Whether the reused material was a gravel base or a pavement surface material, the assigned structural value was no more than that of gravel. A more cost-effective alternative would be to rejuvenate the old asphaltic binder of the in-place bituminous material. Recycling and placing this material as a hot mix rather than an aggregate base would produce a higher strength pavement structure.

The opportunity to hot-mix-recycle came when Raymond Hite, then superintendent of public works in Maplewood, Minnesota, came to the Minnesota Department of Transportation (DOT) for assistance with a project he felt should include the recycling of the in-place bituminous pavement. This project was an urban street of conventional flexible pavement design that was to be reconstructed as a full-depth asphalt pavement. Shortly thereafter, the Minnesota Local Road Research Board authorized funds for setting up and evaluating an urban and a rural bituminous recycling project.

In reviewing the experience of other agencies, which used direct heat in the softening and mixing process, it was noted that the major problem was air pollution in the form of smoke—a problem caused, for the most part, by the burning of asphaltic cement. In at least one other case, a patented process had solved the air pollution (smoke) problem. That process called for a new, completely redesigned drum mixer that used an indirect method of heating the salvaged bituminous mix. A proprietary softening agent was also added to the salvaged bituminous material during mixing operations. The use of that process would have meant buying or leasing the indirect-heating drum mixer and the payment of royalties. It was determined that this would not be cost-effective for the Maplewood project, which called for only 18 144 Mg (20 000 tons) of recycled mix. Coupled with these problems of pollution and expense was the fact that the recycling done thus far was limited to quite low production rates at the mixing plants.

Thus, it was concluded that on this project, because of the small amount of bituminous material that was to be produced, the only cost-effective method would be to use conventional equipment with minimal modifications; this led to the development of the Minnesota heat-transfer method.

This report describes the Minnesota heat-transfer method and the design and construction of this project, which used salvaged bituminous pavement and salvaged aggregate for the full-depth bituminous pavement. This project was located in Maplewood, adjacent to and partially within the headquarters complex of the Minnesota Mining and Manufacturing (3M) Company. The project was a joint effort of the city of Maplewood, the Minnesota DOT, C.S. McCrossan, Inc., and the Minnesota Local Road Research Board.

HEAT-TRANSFER CONCEPT

The problem on the Maplewood project was how to recycle the in-place roadway materials into a usable recycled bituminous mixture cost-effectively and yet without excessive smoke pollution. The materials available for recycling were approximately 4536 Mg (5000 tons) of old bituminous material and 13 608 Mg (15 000 tons) of gravel base. The project required approximately 18 144 Mg (20 000 tons) of recycled bituminous mixture. It became evident that cost-effectiveness required using conventional equipment with minimal modification.

Initially, it was proposed that the salvaged bituminous material and aggregate be blended before they were introduced into the dryer of the mixing plant. But a review of past experience with this method indicated that this could not be done without smoke pollution and reduced plant production. At one of several meetings, the idea of heat transfer was conceived. Instead of blending the salvaged bituminous material and aggregate before introducing them into the dryer, it was proposed to "super heat" the salvaged aggregate in the dryer and combine it with the salvaged bituminous material away from the intense heat of the dryer.

A laboratory experiment was set up to test the concept. In the experiment, the clean aggregate was heated to 193° C (380° F) and then combined and mixed with salvaged bituminous material that was at room temperature. The results of the heat transfer showed that the different proportions of salvaged bituminous material and salvaged aggregate would combine readily within 2 to 3 min. The temperature of the combined mix was almost the theoretical straight-line relation between the room temperature of the salvaged bituminous material and the hot salvaged aggregate.

The concept was discussed with several contractors. Some felt that the concept could be adapted easily to conventional mixing plants at a minimal cost. A few were sure that the plant would get plugged up when the materials were combined and thus significantly reduce mixture production.

The only remaining problem was determining the mixing time and the temperature of the recycled mixture during the initial stages of production. Otherwise, plant operation was excellent, production was normal, and there was no smoke.

DESCRIPTION OF THE PROJECT

The existing roadway was a 14.6-m (48-ft) wide, four-lane bituminous roadway adjacent to the 3M Company headquarters complex east of St. Paul. It was constructed in a series of projects from 1959 to 1964. The structure was nominally 10.2 cm (4 in) of bituminous material and 30.5 cm (12 in) of gravel base over a sandy loam subsoil (A-3) with an R-value of 75 (Hveem stabilometer). It had served traffic well, but increased traffic volume and higher axle loads indicated the need for structural and geometrical upgrading. The roadway was to be upgraded to a four-lane expressway with channelization, concrete curb and gutter, and bus turnouts. The new structural design was 18 cm (7 in) of bituminous base, 3.8 cm (1.5 in) of bituminous binder (leveling), and 1.9 cm (0.75 in) of taconite-tailings wearing course (a good, hard rock waste product with good frictional properties).

PRELIMINARY LABORATORY TESTS

Before the mix was designed and special contract provisions were written, five samples were taken of the bituminous material, gravel base, and subgrade to a depth of 1.2 m (4 ft). These samples were submitted to laboratory analysis and testing. The samples of bituminous material were softened and remolded.

The tests run were (a) density and stability values by the Marshall method, (b) percentage voids by the Rice method, and (c) percentage asphalt by AASHTO T 164-74, method C. The recovered asphalt was tested for (a) penetration (AASHTO T 49) and (b) ductility (AASHTO T 51). These results are given in Table 1.

Sieve analysis was run on the aggregate after asphalt extraction (AASHTO T 30-74). Results of these tests are given in Table 2. The five samples of gravel base were also tested by sieve analysis (AASHTO T 27-74). The tabulated gradation results are given in Table 3.

The test results showed that all materials available for recycling were comparable to virgin material with the exception of the percentage of material passing the 0.075-mm (no. 200) sieve that would be specified for a conventional mix. Therefore, it was felt that the salvaged bituminous material and the salvaged gravel would provide an adequate recycled bituminous mixture. The special contract provisions that pertained to the recycling operation were then drawn up.

SPECIAL PROVISIONS OF CONTRACT

The following is a condensed, slightly edited version of the special contract provisions for the project.

S-1302: Award of Contract

As a condition precedent to award of this contract, the successful bidder shall provide a plan for eliminating possible air pollution. This plan shall meet with the letting authority's approval before award of the contract and be in accordance with the rules, regulations, and standards adopted and established by the Minnesota Pollution Control Agency and in accordance with the provisions of Minnesota Highway Department (MHD) 1717.

S-2105: Excavation and Embankment

Excavation and embankment construction shall be performed in accordance with the provisions of MHD 2105 except as modified below.

Table 1. Samples of existing bituminous material before construction.

Test	Sample 1	2	3	4	5
Bituminous mix					
Density,[a] kg/m³	236.6	229.9	241.4	227.6	228.6
Stability,[a] N	5536	7454	10 382	7160	6626
Voids,[a] percent	7.0	9.8	5.4	12.2	9.5
Asphalt,[b] percent	3.8	3.6	3.9	3.1	4.0
Recovered asphalt					
Penetration[c]	80	31	127	29	40
Ductility,[d] cm	150+	126	150+	93	150+

Note: 1 kg/m³ = 0.062 lb/ft³; 1 N = 0.2248 lbf.
[a]Test procedures on file at Materials Office, Minnesota DOT.
[b]AASHTO T 164-74, method C.
[c]AASHTO T 49.
[d]AASHTO T 51.

Table 2. Sieve analysis of aggregate after extraction of existing material (AASHTO T 30-74).

Sieve Size[a] (mm)	Percentage Passing: Sample 1	2	3	4	5	Average	BA-2 Specification[b]
19	100	100	100	100	100	100	100
16	99	99	99	100	98	99	95-100
9.5	89	83	81	87	88	86	65-95
4.75	74	71	69	74	76	73	
2.0	62	59	59	60	64	61	35-65
0.425	28	26	34	29	32	30	10-35
0.075	7.7	6.3	10.7	6.4	6.9	7.6	1-7

[a]Corresponding U.S. sieve sizes: 0.75, 0.625, and 0.375 in and nos. 4, 10, 40, and 200.
[b]Current Minnesota DOT gradation specification for aggregate used in bituminous mixtures.

Table 3. Sieve analysis of in-place gravel.

Sieve Size[a] (mm)	Sample 1	2	3	4	5	Average	C1-5 Specification[b]
25	100	100			100	100	100
19	98	99	100	100	99	99	90-100
16	94	97	99	96	98	97	
9.5	81	85	85	80	85	83	50-90
4.75	69	72	69	68	69	69	35-80
2.0	61	63	55	60	56	59	20-65
0.425	33	34	29	35	29	32	10-35
0.075	8.0	7.6	8.2	8.9	7.4	8.0	3-10

[a]Corresponding U.S. sieve sizes: 1, 0.75, and 0.625 in and nos. 4, 10, 40, and 200.
[b]Current Minnesota DOT gradation specification for aggregate used in base construction (AASHTO T 27-74).

Salvaged Aggregate

Salvaged aggregate shall include the existing sand, gravel, or crushed rock materials that lie between the bottom of the existing bituminous surface and a plane parallel to and 30.5 cm (12 in) below the bottom of the existing bituminous surface that can be salvaged and used without pulverization. The gradation of the salvaged aggregate material shall be reasonably uniform from fine to coarse with 100 percent passing the 37.5-mm (1.5-in) sieve.

Salvaged Bituminous Mixture

Salvaged bituminous mixture shall include the bituminous mixture in the existing bituminous courses. As a part of the salvaging operations, the bituminous mixture shall be processed or pulverized so as to provide a reasonably uniform gradation from fine to coarse with 100 percent of the material passing the 37.5-mm (1.5-in) sieve.

Salvage Materials

Salvage materials will be measured by the ton. Salvaging material from the existing roadway, processing the material as specified, and placing the processed material in stockpiles shall be one operation constituting a complete unit of measure; removing material from stockpiles and placing it in the work as specified shall be a separate operation constituting another complete unit of measure. The salvaged aggregate and the salvaged bituminous mixture after processing shall be placed in two separate, distinct stockpiles. If necessary, to facilitate crushing and processing, up to 20 percent salvaged aggregate may be incorporated into the salvaged bituminous mixture.

S-2331: Plant-Mixed Bituminous Pavement

A plant-mixed bituminous pavement shall be constructed in accordance with the provisions of MHD 2331 and as shown in the plans, except as modified below or supplemented by using asphalt cement (AC) with the appropriate penetration for producing the specified mixtures:

Mixture	AC Penetration
Binder course	85/100 or 120/150
All base courses	120/150

S-2331.3: Construction Requirements

Under 2331.3B, add the following:

The contractor shall submit, prior to the award of the contract, an acceptable proposal for preventing or eliminating excessive air pollutants.

Under 2331.3C1a(2), Feeder for the Drier, add the following:

A means shall be provided for adding the salvaged bituminous mixture, when required, to the heated aggregate after the aggregate has left the drier. This means shall provide for positive control on proportioning the salvaged bituminous material into the mixture.

Under 2331.3C1a(3), Drier, add the following:

When it is required to add the salvaged bituminous mixture for the bituminous base and binder courses mixtures, it may not be necessary to run the salvaged bituminous mixture through a drier.

Under 2331.3E, the provisions of MHD 2331.3E(1) are supplemented as follows: The approximate mixture proportions of salvaged bituminous mixture and salvaged aggregate to be used in the bituminous base and binder courses shall be as given below. These courses shall be placed in the areas shown in the plans.

Mix	Approximate Mixture Proportions: Salvaged Bituminous	Salvaged Aggregate
Base	20-40	60-80
Binder	20-40	60-80

In addition, a control section shall be constructed where shown in the plans.

Under 2331.3F1, delete the first three sentences of the third paragraph and use the following in lieu thereof:

The aggregate shall be heated to a temperature as designated by the Engineer. This temperature may be in excess of 107°C (225°F). When the aggregate reaches the mixer, either by itself or in combination with the salvaged bituminous mixture, it will be at a temperature which will not cause damage to the asphalt being added.

All costs of equipment modification for the bituminous mixture composed partially of the salvaged bituminous mixture, including the adoption of any of the alternatives included in the bidder's contingency plan for elimination of air pollution, shall be paid for as a lump sum not to exceed $15 000. The contractor shall submit to the engineer before final payment an itemized list of costs incurred for this equipment modification.

Payment at the contract price per ton of mixture for each of the types of mixture shall include all costs of preparation, mixing, and placing the respective mixtures.

REMOVAL AND PROCESSING OF OLD BITUMINOUS MATERIAL AND AGGREGATE BASE

The contractor tried several methods for scarifying and reducing the size of chunks before hauling the old bituminous material 50 km (31 miles) to the plant site at Osseo, Minnesota. The contractor concluded that the most effective method for this project was to scarify and windrow the old bituminous material with a motor grader and process it through a conventional crushing operation at the plant site in Osseo (Figures 1 through 4).

The salvaged bituminous material was hauled to stockpiles, either temporarily at the job site by a self-loading scraper or to the plant site by trucks. The contractor was allowed to pick up a portion of the aggregate base with the salvaged bituminous material to facilitate removal and crushing operations.

The salvaged bituminous material was crushed to pass the 37.5-mm (1.5-in) sieve at the contractor's conventional crushing plant without any unusual difficulties. The crushers were able to handle the size of chunks (Figure 3) as they came from the job without any intermediate steps. No problems were encountered in the crushing operation even at an ambient air temperature as high as 38°C (100°F). The crushing operation provided a uniform, well-graded material.

The remainder of the upper 30.5 cm (1 ft) of aggregate base was windrowed and hauled to separate stockpiles in the same manner as the salvaged bituminous material (no crushing was required).

Figure 1. Motor grader ripping up old bituminous pavement for recycling.

Figure 2. Overall view of section of ripped-up pavement.

Figure 3. Closeup view of ripped-up chunks of pavement.

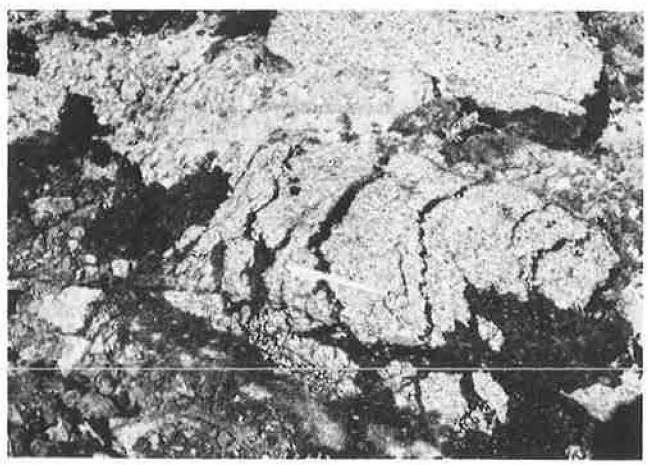

Figure 4. Overall view of crushing plant.

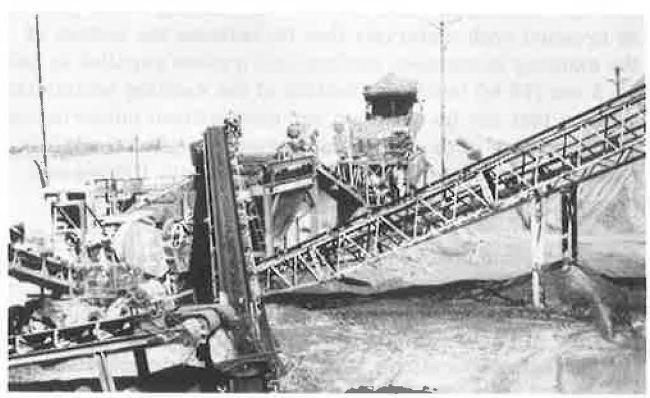

Table 4. Laboratory results for trial mix.

	Mix							
Property	1	2	3	4	5	6	7	8
Proportion bituminous to aggregate	20/80	30/70	40/60	50/50	20/80	30/70	40/60	50/50
Density, kg/m³	216.8	218.8	223.0	221.1	224.8	225.9	228.5	226.8
Stability, N	2755	3849	3827	4330	3889	4779	4730	4704
Voids, percent	Test not completed because of stripping				10.7	11.4	8.5	9.7
Cold water abrasion loss,[a] percent	Test briquets crumbled, failure				27.5	24.6	21.2	21.8

Notes: 1 kg/m³ = 0.062 lb/ft³; 1 N = 0.2248 lbf.
For mixes 1 through 4, 2 percent asphalt added by weight (AC-1, penetration 120/150); for mixes 5 through 8, 4 percent asphalt added by weight (AC-1, penetration 120/150).

[a]Minnesota test procedure for indication of stripping; less than 12 percent loss desired.

RECYCLED BITUMINOUS MIX DESIGN

After a sufficient amount of crushed, salvaged bituminous material and salvaged gravel had been stockpiled (August 1976), samples were submitted to the Minnesota DOT materials laboratory for mix design.

The Marshall mix design method of testing (AASHTO D 1559) was used on eight mixes as follows (percentage by weight):

	Salvaged		AC-1	
Mix	Bituminous (%)	Gravel (%)	Percentage	Penetration
1	20	80	2	120/150
2	30	70	2	120/150
3	40	60	2	120/150
4	50	50	2	120/150

Mixes 5 through 8 used the same proportions of salvaged bituminous and gravel as mixes 1 through 4, but 4 percent AC was added instead of 2 percent. The results of these mixtures are given in Table 4.

As a result of these tests, it was recommended that 3 percent AC-1 (penetration 120/150) be added for bituminous base design and 3.5 percent for bituminous binder, based on Marshall design criteria, and that mix proportions use 40 to 50 percent of the salvaged bituminous material.

As a further check, a set of trial mixes was done with 3.5 percent added AC and 40 and 50 percent salvaged bituminous materials. In a second identical set, the salvaged bituminous was heated slightly to 66° C (150° F) in response to the contractor's suggestion on plant modification. Another mixture was mixed in a routine manner by using 100 percent salvaged gravel and 4.5 percent AC. The test results for these mixes, given below, indicate no change in the mix when the salvaged bituminous is heated to 66° C compared with mixing at the ambient temperature of 28° C (82° F) [t° C = (t° F - 32)/1.8]; 1 kg/m³ = 0.062 lb/ft³; and 1 N = 0.2248 lbf:

	3.5 Percent AC		
Characteristic	28°C	66°C	4.5 Percent AC
Proportion bituminous to aggregate	40/60 50/50	40/60 50/50	0/100
Density, kg/m³	234.4 234.1	234.1 235.7	231.8
Stability, N	5843 5718	5945 6163	3832
Voids, %	6.3 6.3	7.2 6.3	7.7
Cold water abrasion loss, %	14.7 12.8	12.9 11.8	14.1

These results compared favorably with those for the conventional mixes that used 4.5 percent AC and virgin aggregate. Some stripping was noted during the test for voids, probably because of loss of bonding characteristics from oxidation of the recycled bituminous mix.

Each mix was tested for asphalt content [t° C = (t° F - 32)/1.8]:

	3.5 Percent AC		
Characteristic	28°C	66°C	4.5 Percent AC
Proportion bituminous to aggregate	40/60 50/50	40/60 50/50	0/100
Added AC, % (by weight)	3.5 3.5	3.5 3.5	4.5
Recovered AC, %	4.1 4.2	4.1 4.1	4.6

The recovered asphalt was then tested for penetration and ductility (1 cm = 0.39 in):

	3.5 Percent AC		
Characteristic	28°C	66°C	4.5 Percent AC
Penetration	36 60	53 55	82
Ductility, cm	120+ 120+	120+ 120+	120+

PLANT MODIFICATION AND OPERATION

The plant used for this project was a 6.8-Mg (7.5-ton) batch plant with a pug-mill mixer. The plant operation was essentially the same as that for a conventional plant mix except that the aggregate leaving the drier was hotter than normal. The batch plant was modified to feed the cold salvaged bituminous material into the weigh hopper above the pug mill (Figures 5 and 6). The cost of plant modifications was minimal, and the mix was produced without burning the asphalt and thus without smoke pollution.

The quantity of heat added to the aggregate as it moves through the drier must be sufficient to provide a recycled mix that is workable for paving operations. Sufficient heat must be available in the heated aggregate to drive off the moisture in the salvaged bituminous material, melt the old asphalt (thus mixing the old and the new asphalt), and allow for the normal heat losses from the plant operation. On this project, the temperature of the salvaged aggregate as it left the drier was approximately 232° C to 260° C (450° F to 500° F). Figure 7 shows the steps involved in the production of the recycled mixture.

It was planned to produce a recycled mixture with 30 percent salvaged bituminous material and 70 percent salvaged aggregate. However, because some of the salvaged aggregate was used for subgrade corrections, a 50/50 proportion was used for the bulk of the project (all references to proportions or blends of recycled mixtures are for salvaged bituminous material to salvaged or clean aggregate). Even at the 50/50 proportion, enough heat was available to thermally break down the old asphalt mix. Later in the fall, as the weather became colder, a 40/60 proportion was used to get a higher laydown temperature to ensure adequate compaction of the mixture. A 152-m (500-ft) control section of conventional, unrecycled mix was placed on the project for comparison with the recycled mixture.

PAVING OPERATIONS

Paving operations were conventional in all respects except that the recycled bituminous mixture seemed to be "fluffier" than a normal bituminous mixture. The paver screed was raised somewhat to obtain the same compacted thickness as that of a conventional mixture. The plant operator commented that the storage silo would not hold quite as much recycled mixture as conventional mixture according to weight. This appears to be caused by lower than normal mixture temperatures.

The eastbound base was placed in three lifts. The westbound base was placed in one 18-cm (7-in) lift. This was done to hold the heat since the paving was done late in the construction season. A 152-m (500-ft) control section was placed in the eastbound lane for purposes of comparison. Additional asphalt was added to the 3.8-cm (1.5-in) binder or leveling course because it was to carry traffic over the winter months. The final 1.9-cm (0.75-in) taconite-tailings wearing course was placed in the summer of 1977.

TEST RESULTS FOR MATERIALS DURING CONSTRUCTION

Preliminary laboratory tests of the in-place material (samples taken by hand from the road) showed a 3.7 percent asphalt content. The laboratory tests on the crushed

Figure 5. Loading crushed, salvaged bituminous material onto conveyor, which feeds pug-mill stockpile in background.

Figure 6. Point at plant where conveyor feeds cold salvaged bituminous material into pug mill.

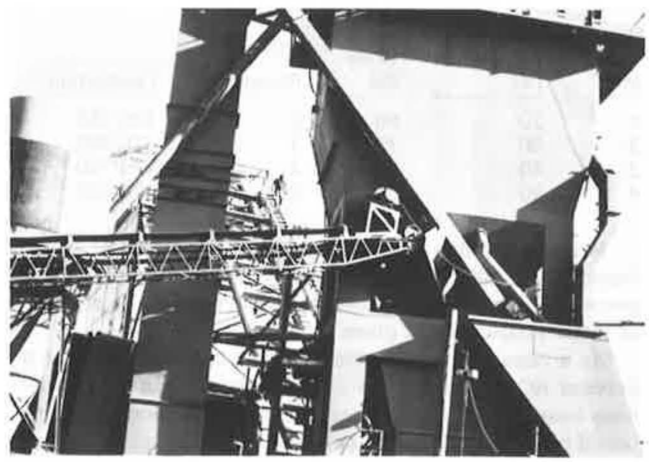

Figure 7. Steps involved in production of recycled bituminous material for the Maplewood project.

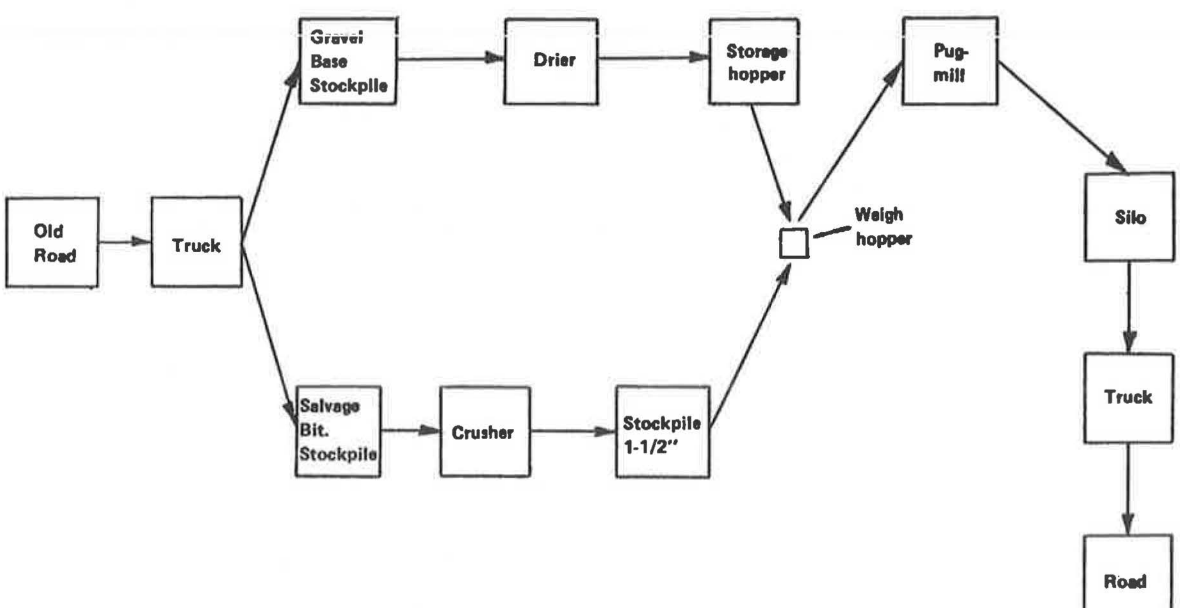

Table 5. Summary of test results: recovered asphalt.

Property	Recycled Base Mix		Recycled Binder Mix		Cold, Crushed Salvaged Mix	
	$\bar{x}$[a]	s	$\bar{x}$[b]	s	$\bar{x}$[c]	s
Bituminous material, percent	4.1	0.36	4.7	0.43	1.9	0.39
Penetration	68	9.75	66	7.72	32	4.02
Ductility, cm	120+	0.00	120+	0.00	120+	0.00
Softening point	123	3.79	123	2.11	138	2.96

Note: 1 cm = 0.39 in.
[a]n = 64. [b]n = 16. [c]n = 27.

Table 6. Summary of test results: new asphalt in storage.

Test	$\bar{x}$	s	n
Original asphalt			
Penetration	128	0.01	21
Ductility, cm	120+	0	16
Absolute viscosity, Pa·s	67.8	19.25	14
Thin-film oven[a]			
Loss on heating, percent	0.42	0.03	16
Penetration of residue	69	1.77	16
Percentage of original penetration	54	1.77	16
Ductility of residue, cm	120+	0	16

Note: 1 cm = 0.39 in; 1 Pa·s = 10 poises; t°C = (t°F - 32)/1.8; 1 mm = 0.039 in.
[a]At 163°C, 3.2 mm, 5 h.

Table 7. Sieve analysis of salvaged aggregate at plant site.

Sieve Size[a] (mm)	Percentage Passing: Cold Feed $\bar{x}$[b]	Cold Feed s	BA-2 Specification[c]	Hot Bins (composite)[d] $\bar{x}$[e]	Hot Bins s
19	97.4	1.21	100	99.5	0.78
16	94.5	1.52	95-100	97.3	1.73
9.5	83.6	2.59	65-95	87.5	1.10
4.75	70.9	3.26		77.1	1.39
2.00	60.6	3.15	35-65	66.2	3.32
0.425	32.2	3.04	10-35	35.5	3.12
0.075	6.78	2.08	1-7	6.21	1.56

[a]Corresponding U.S. sieve sizes: 0.75, 0.625, and 0.375 in and nos. 4, 10, 40, and 200.
[b]n = 24.
[c]Current Minnesota DOT gradation specification for aggregate used in bituminous mixtures.
[d]The heated, dried aggregate in this plant is split into three segments going into storage bins—coarse, intermediate, and fine—by using the 19-, 16-, and 4.75-mm (0.75-in, 0.625-in, and no. 4) sieves. It is then recombined at batching time according to requirements.
[e]n = 24.

salvaged material produced by the contractor and ready for recycling showed only 1.9 percent asphalt content. This difference is explained by the amount of gravel base mixed with the material during scarifying and loading operations. This was borne out by the results of tests run on mix samples taken during paving (Tables 5 and 6). The 50/50 recycled mix with 3 percent new asphalt AC-1 (120/150) had an extracted asphalt content of 4.1 percent. The 40/60 recycled mix with 3.5 to 4 percent new asphalt AC-1 (120/150) had an extracted asphalt content of 4.7 percent.

Table 5 gives the results of tests on the recovered asphalt. These were compared with the results of tests run on the new asphalt used in the mixes (Table 6). It was considered highly desirable, as a gauge of oxidation, that the penetration values of 66 and 68 for the asphalt extracted from the recycled mixture be close to the penetration value of the new asphalt (using the thin film oven test result of 69).

Table 8. Summary of test results for hot mix.

Property	Base Mix $\bar{x}$[a]	Base Mix s	Binder Mix $\bar{x}$[b]	Binder Mix s
Marshall density, kg/m³	236.0	1.79	236.6	3.14
Marshall stability, N	6978	1205.1	6230	966.5
Voids, Rice method, percent	6.4	1.27	5.1	1.81
Cold-water abrasion loss, percent	8.7	2.19	6.6	2.91

Note: 1 kg/m³ = 0.062 lb/ft³; 1 N = 0.225 lbf.
[a]n = 58. [b]n = 13.

Table 9. Sieve analysis of aggregate recovered after extraction: recycled hot mix.

Sieve Size[a] (mm)	Percentage Passing: All Samples $\bar{x}$[b]	All Samples s	Base Samples $\bar{x}$[c]	Base Samples s	Binder Samples $\bar{x}$[d]	Binder Samples s
19	99.9	0.80	99.8	0.89	100.0	0.00
16	97.5	1.97	97.4	2.05	98.1	1.50
9.5	87.4	3.04	87.3	3.14	87.9	2.60
4.75	75.8	3.47	75.5	3.42	77.1	3.50
2.00	64.3	2.34	63.9	3.56	65.7	3.20
0.425	35.4	3.31	35.1	3.49	36.4	2.22
0.075	9.66	1.40	9.63	1.79	9.77	0.60

[a]Corresponding U.S. sieve sizes: 0.75, 0.625, and 0.375 in and nos. 4, 10, 40, and 200.
[b]n = 71.
[c]n = 58.
[d]n = 13.

The objective was to recycle the gravel base as it came off the road and the processed salvage bituminous as stockpiled. Gradations of the gravel base were taken to determine if the materials were uniform going into the dryer. The results show that the material was uniform. They were also near compliance with the bituminous aggregate specification except for the minus 0.075-mm (no. 200) material. Gradation summaries and the bituminous aggregate specification are given in Table 7.

As noted in Table 8, the hot mix produced with these salvaged materials, by this method, exhibits the qualities called for in a full-depth bituminous base. The resultant mean values in the mix of 4.1 percent asphalt content and 6.4 percent voids are within a tolerable variance of the design criteria of 4.5 percent asphalt content and 4 to 6 percent voids. The results of the Marshall tests for density and stability are more than adequate at 235.5 kg/m³ (14.6 lb/ft³) and 6978 N (1570 lbf) respectively. The test results indicate that there is little difference in quality between the base and the binder. The sieve analysis of the aggregate recovered from the hot-mix samples is summarized in Table 9. These test results show that there is variance from sample to sample—probably a result of using the materials from the road without controls for uniformity.

TEST RESULTS AFTER PAVING

Pavement cores, Benkelman beam deflections, and a crack survey were taken before the final wearing course was placed. In general, the tests indicated that the base constructed with recycled mix was comparable to base constructed with conventional mix.

Cores were taken from the roadbed shortly after paving. These cores were taken at locations where samples had been taken during paving. The average test

Table 10. Laboratory test results for road cores (recycled mixtures): bituminous mixture and recovered asphalt.

	Base Cores		Binder Cores	
Test	$\bar{x}^a$	s	$\bar{x}^b$	s
Bituminous mixture				
Marshall density, kg/m³	227.0	5.24	222.7	6.22
Voids, Rice method, percent	9.9	1.95	10.6	3.71
Bitumen, percent	4.0	0.23	5.1	0.54
Recovered asphalt				
Penetration	54	7.34	52	6.85
Ductility, cm	120+	0.00	120+	0.00
Softening point	127	3.21	129	4.31

Note: 1 kg/m³ = 0.062 lb/ft³; 1 cm = 0.39 in.
[a] n = 24. [b] n = 7.

Table 11. Laboratory test results for road cores (recycled mixtures): sieve analysis of recovered aggregate.

	Percentage Passing					
	All Cores		Base Cores		Binder Cores	
Sieve Size[a] (mm)	$\bar{x}^b$	s	$\bar{x}^c$	s	$\bar{x}^d$	s
19	99.7	0.62	99.7	0.56	99.6	0.89
16	97.2	1.52	97.0	1.50	98.2	1.30
9.5	86.5	3.31	86.1	3.51	88.0	1.87
4.75	75.0	3.89	74.6	4.25	76.4	1.14
2.00	63.0	3.69	62.7	4.03	64.4	1.14
0.425	34.7	2.68	34.4	2.75	36.0	2.12
0.075	9.53	0.86	9.49	0.84	9.68	1.03

[a] Corresponding U.S. sieve sizes: 0.75, 0.625, and 0.375 in and nos. 4, 10, 40, and 200.
[b] n = 26.
[c] n = 21.
[d] n = 5.

values and their standard deviations are given in Tables 10 and 11.

Marshall stability tests could not be run on the cores taken from the road because of their diameter and the roughness of the sidewall. However, many of the cores were tested by triaxial compression to determine the resilient modulus of the unconfined cores. The results show the recycled mix to have a lower modulus than historical data gathered on several state highways in Minnesota. It is a reasonable modulus and not unusual for conventional mixes. This resilient modulus indicates a more flexible pavement and one with more voids. Data gathered by the bituminous office of the Minnesota DOT verify that statewide nonwearing courses have about 9 percent voids compared with 10 percent on this project.

In June 1977, after the binder course had carried traffic over the winter, Benkelman beam deflections were run on the road surface before the wearing course was placed. The deflections were taken at approximately 152-m (500-ft) intervals throughout the length of the recycled paving. An additional three locations were tested in the 152-m conventional section. In the evaluation of these deflections, there is a slight difference between the eastbound roadway where the base was placed in three lifts and the westbound roadway where the base was placed in one lift. The westbound roadway has slightly higher deflections, greater range, and larger deviation. Despite this, the average deflections of the three sections are similar:

Test Site	Number of Tests	$\bar{x}$	s
Eastbound, recycled	9	0.46	0.09
Eastbound, conventional	3	0.48	—
Westbound, recycled	10	0.51	0.10
All		0.51	0.09

When these deflections are evaluated according to criteria developed in Minnesota DOT investigation 183 (Application of AASHO Road Test Results to Flexible Pavement Design in Minnesota) and investigation 195 (Full-Depth Asphalt Pavement Structures Evaluation), a total "gravel equivalency" of about 23 is the result. A unit gravel equivalency of 2.7 can then be calculated for the base and binder. An R-value of 75 for the sand and gravel subgrade was assumed. The conventional section, which is limited in length, and the eastbound roadway behave as would be expected for 23.4 cm (9.2 in) of full-depth asphalt base. The westbound roadway behaves similarly but is not quite as strong; it compares with 2.18 cm (8.6 in) of conventional full-depth asphalt base. The strength is equal to or greater than the design thickness of 21.6 cm (8.5 in).

In a crack survey made in June 1977, the only cracks noted were in the eastbound roadway. This appears to be a result of the different methods of paving (multiple lift as opposed to single lift). Five full-width cracks and one that crossed one traffic lane and the turn lane were noted. All cracks were at catch basins, electrical "pull boxes," or where some sort of utility passed under the road.

APPLICATION OF METHOD TO OTHER CONVENTIONAL PLANTS

Although this method of pavement recycling was developed for use at a batch plant, the Minnesota heat-transfer concept can also be applied to conventional dryer-drum and continuous-mix plants. Several equipment manufacturers have modifications for drum mixers and batch plants. Some contractors have made modifications to their plants. The only limitations appear to be the minimal proportion of clean aggregate, which is used as the heat-transfer medium, and the maximum teperature of the dryer. Contractors appear to be reluctant to increase the temperature of the aggregate leaving the dryer to more than 260° C (500° F).

BENEFITS OF RECYCLING

Observations to date indicate that it is feasible to produce a hot bituminous mixture by following the method described in this paper and using recycled bituminous material and aggregate. The recycled hot mix can be placed by conventional methods and appears to be similar to a conventional mix.

The benefits of this process are

1. Its use of conventional equipment with minor modifications,
2. Reuse of all aggregates,
3. Reuse of the old asphalt as an effective binder,
4. Use of less new asphalt,
5. Fuel savings,
6. Minimal cost for plant modifications,
7. Normal or nearly normal production rates, and
8. Process emissions that are within pollution standards.

GUIDELINES FOR DETERMINING FEASIBILITY OF RECYCLING OLD BITUMINOUS PAVEMENT AND AGGREGATE BASE

To determine the cost saving of reclaiming an old bituminous pavement, the following should be known:

1. The amount of asphalt in the old bituminous pavement by the extraction method or by using 80 percent of the actual asphalt used when the pavement was constructed;
2. The cost of new asphalt;
3. The cost of aggregate (BA-2) for bituminous mixture;
4. The cost of salvaging, loading, hauling, and stockpiling existing aggregate base;
5. The cost of scarifying, loading, hauling, stockpiling, and crushing salvaged bituminous pavement; and
6. The cost or profit of disposing of the existing gravel base and bituminous pavement [this should include scarifying, loading, hauling, leveling, landscaping, and the cost of dumping (royalty) or may include payment for the material being dumped].

Hauling costs are a major factor in determining the cost of a project. A preferred method for determining hauling costs is the cycle time method. The following information is needed: (a) rental rate of hauling unit per hour, (b) capacity of hauling unit, (c) cycle time of hauling unit. This can be expressed as (rental rate × cycle time) ÷ (capacity × 60) = cost per megagram. For example, if rental rate = \$26/h operated, capacity = 15.5 Mg, and cycle time = 45 min, then (26 × 45)/(15.5 × 60) = \$1.25/Mg.

Hauling costs can be reduced significantly on recycling projects by backhauling salvaged bituminous material and salvaged gravel base.

CONCLUSIONS

The Minnesota heat-transfer concept has wide application for cost-effectively recycling old bituminous pavements and aggregate bases. The modification to conventional batch, drum-mixer, and continuous-mix plants is minimal. This method requires clean aggregate for heat transfer, which in turn requires additional new asphalt. By using additional asphalt with higher than normal penetration, the effective penetration of the recycled asphalt binder is improved without the use of rejuvenators. The production rate of the plant is not seriously reduced. No smoke is emitted from the modified batch plant operation. There are some smoke emissions from the modified drum-mixer operation, but this can be held within present pollution standards. Although no continuous-mix plants have actually been modified, it is felt that they would work much like a modified batch plant.

Practical proportions for the design of future recycling projects would appear to be 50/50 for batch and continuous-mix plants. Since there appears to be some additional heat in the modified drum-mixer plants, the practical proportion limit would appear to be 60/40.

Many roadways, streets, and airports have been constructed with several centimeters of bituminous surfacing and several centimeters of gravel base. In many cases, no new aggregates would be required to produce a recycled mix that would result in a higher strength structure. Only the addition of new asphalt would be required, and this would be less than that required for a new conventional mixture.

Although there is some question as to the durability of recycled mix versus new conventional mix, all conventional testing shows recycled mix to be comparable to new mixes.

If the salvaged bituminous material and the salvaged aggregate are uniform and well graded, the gradation of the recycled bituminous mixture will also be uniform and well graded provided the contractor uses reasonable care in handling the stockpiled materials. The only change in gradation is the slight increase in the amount of materials passing the 0.075-mm (no. 200) sieve.

The savings attributed to recycling seem to be positive in all cases. Even if recycling were equal in cost to conventional construction, there are environmental and social benefits of extending and preserving the nonrenewable asphalt and aggregate resources. The biggest challenge left is to make recycling work for us by looking at every project to determine if the benefits of recycling are positive.

ACKNOWLEDGMENTS

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Determination of Moisture Contents in Bituminous Mixtures by a Nuclear Method

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A study undertaken to evaluate the effect of moisture on nuclear-gauge data and to explore whether the nuclear gauge can be used to determine moisture content when asphalt content is held constant in a paving mixture is reported. A Troxler model 2226 gauge was used in the study. Four wearing-course mixtures that contained slag, gravel, and limestone aggregates were studied, and moisture content was varied from 0 to 3.47 percent. Statistical analysis of the

data indicates that the nuclear gauge has the potential to read moisture content within ±0.3 percent of actual value (at a 95 percent level of confidence). The nuclear gauge would provide a rapid means of testing and monitoring moisture content in the mixtures produced by the drum-dryer process. Tests run on emulsified asphalt-aggregate mixtures indicate that the nuclear gauge can be used effectively to monitor total liquid content (emulsified asphalt plus water) in a mixture to facilitate compaction at the optimum liquid content.

Studies undertaken to determine asphalt content in bituminous mixtures by using a nuclear gauge have been very promising. However, the presence of absorbed moisture in the aggregate can pose problems since the hydrogen in the water will affect the nuclear gauge counts and the moisture will thus be read as an additional asphalt content in the mix. Absorptive aggregates such as slags can contain significant amounts of moisture (more than 1 percent) without any apparent visual signs of steaming or slumping in the bituminous concrete.

Moisture content in the hot aggregate after drying operations varies from day to day according to the condition of the aggregate stockpiles and the prevailing weather. Even if the gauge is calibrated with the aggregate from the hot bins to allow for retained moisture, some of the moisture might be lost in the mixing operation. There is a need, therefore, to investigate the extent to which moisture in bituminous mixtures affects nuclear gauge counts and to explore the possibility of determining moisture content in bituminous mixtures when asphalt content is considered to be constant. Since mixtures produced by the drum-dryer process at relatively lower mixing temperatures contain a significant amount of moisture to aid in compaction, it is believed that such investigations would have useful field applications. The current standard method of determining water content by distillation (AASHTO T 55 or ASTM D 95) is very time consuming and can take up to 4 h if the moisture content is very high. Use of the nuclear gauge could reduce the testing time to 15 min.

Bituminous mixtures that contain emulsified asphalt are normally mixed with a liquid content (emulsified asphalt plus water) that is higher than the optimum liquid content needed to achieve optimum density. Such mixtures are allowed to cure after spreading until the optimum liquid content is obtained, and then the rolling is begun. Curing time depends on the characteristics of the mix and the prevailing weather conditions. There is a need to continuously monitor total liquid content in the mixture so that rolling can be started when the optimum liquid content is reached.

LITERATURE REVIEW

The principle of using nuclear radiation to measure the asphalt content of bituminous mixtures was established several years ago by Lamb and Zoller (1). Subsequent studies by Varma and Reid (2), Howard and Covault (3), Walters (4), Qureshi (5), Hughes (6), and Grey (7, 8) have contributed to modifications of the nuclear gauge. Further studies to evaluate the effect of gradation, type of aggregate, and asphalt source on asphalt content have been reported by Klotz (9) and Hughes (10).

However, most of the research has been conducted on bituminous mixtures that contain relatively dry aggregate even though, in actual field conditions, some moisture is usually encountered in aggregates. Finding moisture is even more likely if absorptive aggregates are used in the mix or if the mix is produced by the drum-dryer process.

THE NUCLEAR GAUGE

The 40.6- by 40.6- by 40.6-cm (16- by 16- by 16-in) Troxler model 2226 gauge (Figure 1) consists of a one-piece unit that weighs about 56.7 kg (125 lb). All components are enclosed within the single unit, and a sliding-drawer arrangement is provided so that the stainless steel pans that contain the bituminous test sample can be inserted into the gauge. Three He_3 neutron detector tubes are used to monitor the thermal neutrons from the test specimen; two of the tubes are sample detector tubes positioned beneath the test specimen pan in the sliding drawer. The other tube sits near the top of the gauge and acts as a reference detector.

The counts monitored by the reference detector are used as a continuous, internal standard count and are electronically compared with the sample count. Thus, any electronic drift caused by variation in ambient temperature or aging of components can be accounted for during the actual test count. Because the system is continuously standardized, no auxiliary standard was provided.

The operation of a nuclear asphalt-content gauge of this type is based on neutron thermalization. The 11.1-GBq (300-mc) americium-241 source produces neutrons in the 0.4-pJ (2.5-MeV) range. Elastic and inelastic scattering collisions occur between the neutrons and the material under investigation. After a series of collisions, the "fast neutrons" [energies from 0.08 to 1.6 pJ (0.5 to 10 MeV)] are slowed to the "thermal" level [0.004 aJ (0.025 eV)], at which they can be counted by the gauge detector tubes. The scattering collisions that occur over a timed counting period are a function of the nuclei of the test material. In a typical bituminous mixture, it is the added hydrogen atoms present with increased asphalt that produce a higher count. Therefore, any increase in the number of hydrogen atoms, from either asphalt or the addition of moisture (H_2O, two hydrogen atoms per molecule), would produce more scattered thermal neutrons and yield a higher count rate on the gauge.

If the count obtained on a mixture of bituminous material can be separated into two portions—that attributable only to the materials that compose the mix (aggregate and asphalt) and that attributable to residual moisture in the aggregate—the gauge could be successfully used to determine the moisture content of a typical bituminous mixture.

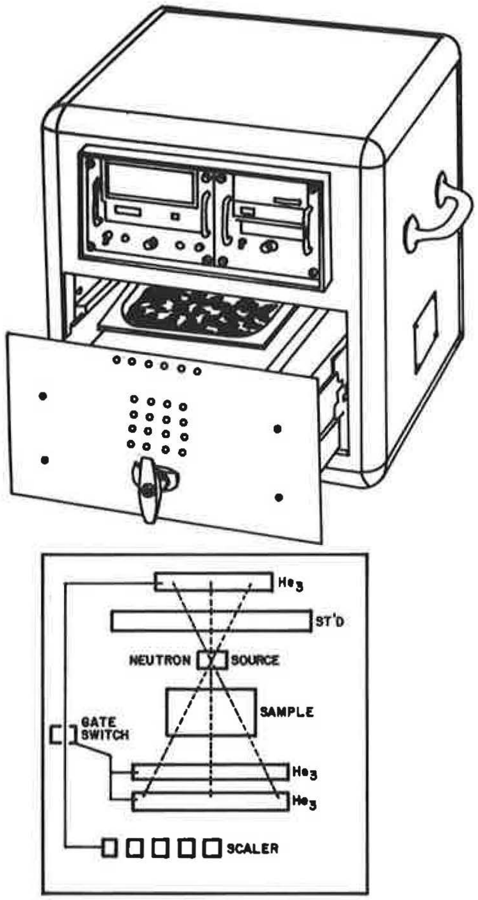
EXPERIMENTAL PROCEDURE

This study consisted of producing pans of bituminous mixtures that contained essentially the same asphalt content for a particular type of aggregate but various percentages of moisture. Three types of aggregate (slag, gravel, and limestone) were used. Slag aggregates from two different sources were investigated. The pores in the slag aggregates were more easily filled with water and aided in getting higher percentages of moisture.

The following general procedure was used:

1. An accurately weighed sample pan of aggregate material only (no asphalt added) was thoroughly dried (0 percent moisture) until a constant weight was achieved, and a count was made on it in the nuclear gauge.
2. Asphalt was then added to the above pan to obtain a precise mix with a known asphalt content, and a count was made again. A straight-line plot of count versus asphalt content (no moisture present) was made for the two data points so that the slope of the line would indicate the counts

Figure 1. Troxler model 2226 asphalt-content gauge.



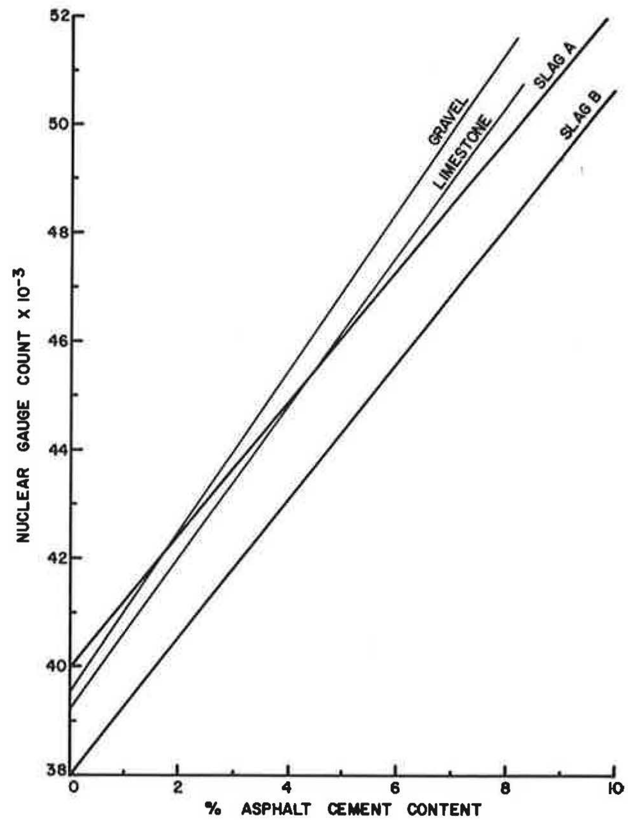
per percentage of asphalt content (Figure 2).

3. The same aggregate and asphalt were used in making up a series of pans of bituminous mixtures with accurately determined moisture contents by weight in the aggregates used. These were made by soaking aggregate of predetermined weight for several hours and then pouring off excess water and oven drying until the percentage weight of water reached a desired value. Asphalt was then quickly added and mixed to the same asphalt content as in step 2 above, and the mixed pan sample was read by the nuclear gauge, the count being a sum of asphalt and moisture. Weighing was done just after mixing to allow for any loss of moisture during the mixing operation. Counts were taken as quickly as possible after mixing to keep the loss of moisture during testing to a minimum. Tests of water distillation run on the bituminous mixtures immediately after the nuclear test indicated minimal loss of moisture during the nuclear testing operation.

4. Asphalt content was held as constant as possible in the series of mixtures, and only moisture content was varied. However, the actual asphalt contents incorporated in the mixtures varied ± 0.2 percent from the target asphalt content after the asphalt that stuck to the mixing bowl was taken into account. The data acquired in step 2 (asphalt content versus count) were used to correct all counts to a count value that corresponded to the target asphalt content. In most cases, slight corrections were needed.

5. A statistical analysis of the data was then made to

Figure 2. Percentage asphalt content versus gauge count.



show the relation between gauge count and moisture content of typical bituminous mixes.

TEST DATA AND INTERPRETATION OF RESULTS

The mixtures in test series 1 through 4 met the gradation requirements of 1973 Pennsylvania Department of Transportation specifications for ID-2 wearing course. The gradation is given below (corresponding U.S. sieve sizes are 2, 0.75, 0.5, and 0.375 in and nos. 4, 8, 16, 30, 50, 100, 200):

	Percentage Passing	
Sieve Size (mm)	ID-2 Wearing Course	Base Course
50	100	100
19	100	76
12.5	100	–
9.5	90	53
4.75	62	37
2.36	45	27
1.18	32	20
0.6	22	–
0.3	15	–
0.15	9	–
0.075	5	5

Test Series 1

Nineteen mixtures consisted of slag aggregate A supplied by Sheridan Slag Company and AC-20 asphalt cement supplied by United Refining Company. It was desired to incorporate 9 percent asphalt by weight of the mix, but the actual asphalt content ranged from 8.85 to 9.23. As men-

Figure 3. Percentage moisture versus gauge count for slag A.

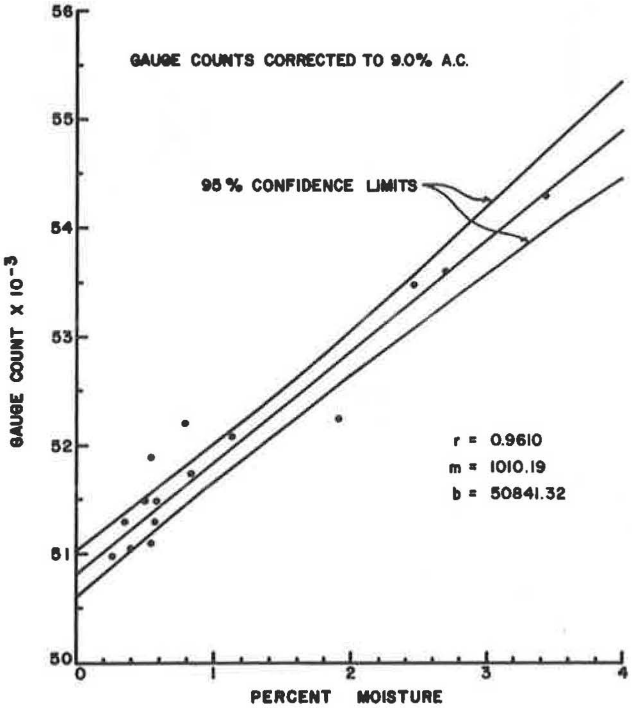


Figure 4. Percentage moisture versus gauge count for slag B.

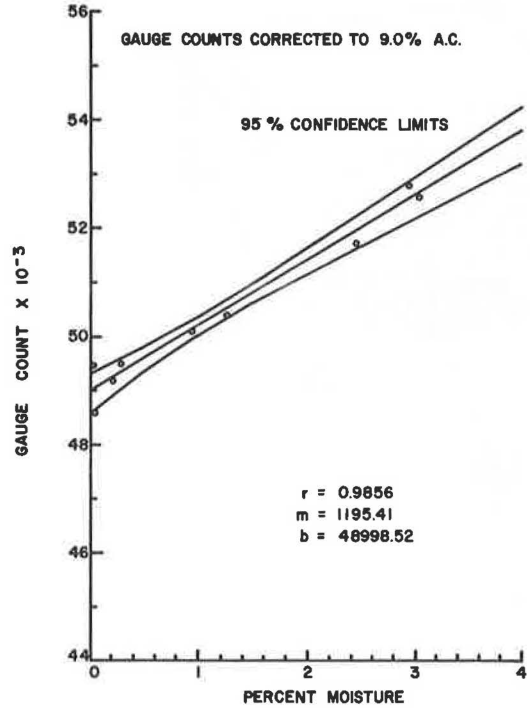


Figure 5. Percentage moisture versus gauge count for gravel.

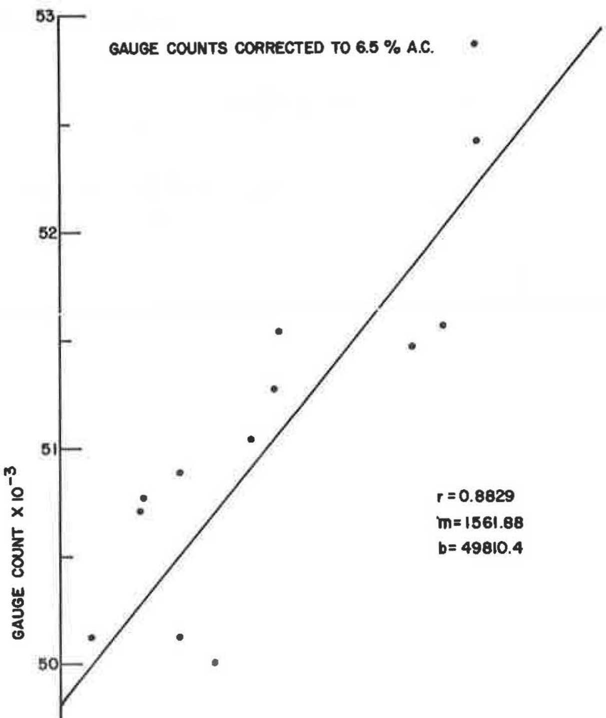
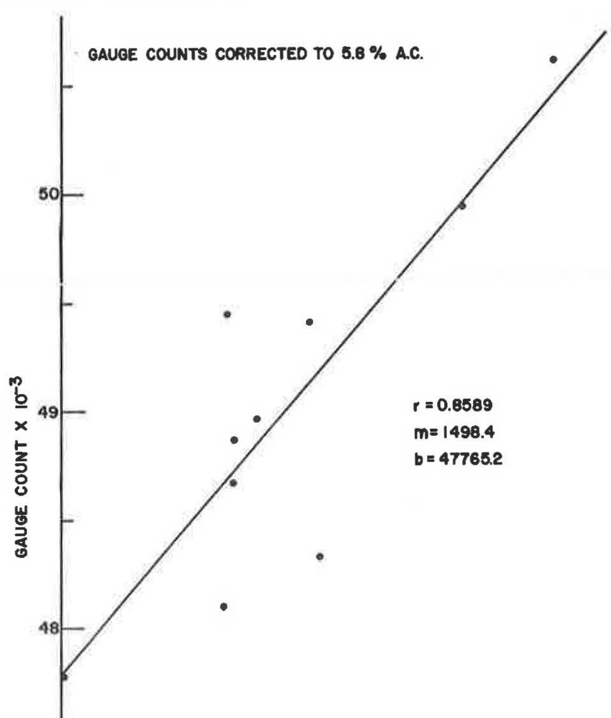


Figure 6. Percentage moisture versus gauge count for limestone.



tioned earlier, the nuclear gauge counts were corrected to 9 percent asphalt content by using Figure 2. Moisture in the mix just after mixing ranged from 0.07 to 3.47 percent by weight in 19 test runs.

Figure 3 shows a plot made for percentage moisture in the aggregate (or mix) versus the nuclear gauge reading. Statistical analysis of the data was performed to establish the 95 percent confidence belt on the plot (11). The results are very encouraging. As expected in statistical analyses, the belt is narrowest at the mean value of the independent variable (approximately 1 percent moisture content). At this level, the nuclear gauge can read moisture content within ±0.15 percent of actual value at a 95 percent confidence level. The divergence of the belt at higher moisture contents is probably attributable to the following:

1. At higher moisture contents, the mix might have been losing moisture while it was being tested in the nuclear gauge so that the exact moisture content is difficult to determine.
2. Fewer tests are performed at higher moisture contents than are performed below 1 percent.

However, the gauge can read moisture content within ± 0.3 percent in the 0 to 3.5 percent moisture range, which appears acceptable.

Test Series 2

Eight mixtures consisted of slag B supplied by Duquesne Slag Products Company and AC-20 asphalt cement supplied by Chevron Asphalt Company. The asphalt content was held constant at 7.93 percent in all mixtures. However, the gauge counts were corrected to 9 percent by using the slag B line from Figure 2 so that a comparison could be made with the data from test series 1. Moisture content was varied from 0 to 3.05 percent.

Figure 4 shows the plot for percentage moisture versus gauge count. Again, the correlation is very good. For the entire moisture content range of 0 to 3 percent, the gauge would give results within ±0.3 percent of the actual value, which appears acceptable.

Test Series 3

Thirteen mix samples consisted of gravel aggregate from Oil City Sand and Gravel Company and AC-20 asphalt cement. It was desired to incorporate 6.5 percent asphalt by weight of the mix, but the actual asphalt content ranged from 6.46 to 6.60 percent. The gauge counts were corrected to 6.5 percent. Moisture in the mix just after mixing ranged from 0.11 to 1.54 percent by weight.

The plot for percentage moisture in the aggregate (or mix) versus the nuclear gauge reading is shown in Figure 5. Statistical analysis of the data indicates that the nuclear gauge can read moisture content within ±0.3 percent of actual value at the 95 percent confidence level.

Test Series 4

Ten mixtures consisted of limestone aggregate and AC-20 asphalt cement. It was desired to incorporate 5.8 percent asphalt by weight of the mix, but the actual asphalt content ranged from 5.74 to 5.95 percent. The gauge counts

Figure 7. Comparison of percentage moisture versus gauge count for slags A and B, gravel, and limestone.

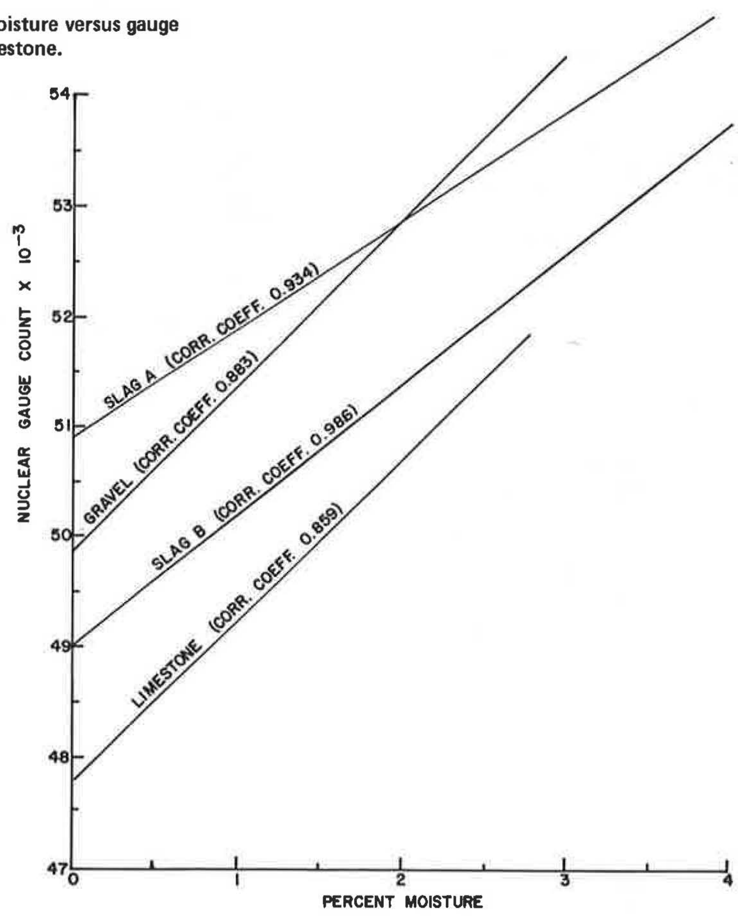
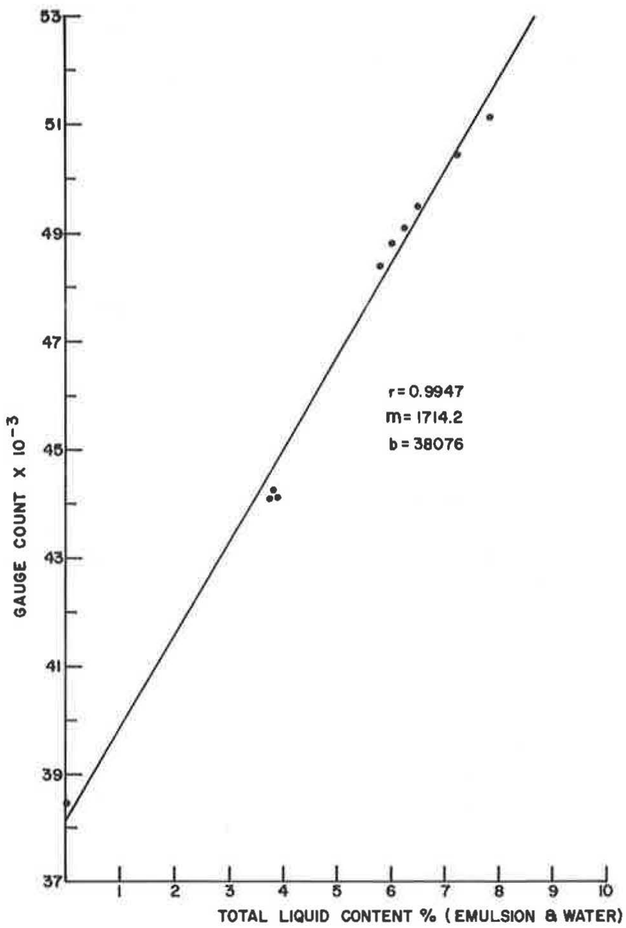


Figure 8. Total percentage liquid content versus gauge count for mix A.



were corrected to 5.8 percent asphalt content. Moisture in the mix just after mixing ranged from 0.60 to 1.81 percent of the weight.

The plot for percentage moisture in the aggregate (or mix) versus the nuclear gauge reading is shown in Figure 6. Statistical analysis of the data indicates that the nuclear gauge can read moisture content within ±0.3 percent of actual value at a 95 percent confidence level.

Comparison of Four Test Series

Figure 2 shows the plot of percentage asphalt versus gauge reading for all aggregates. The apparent shift between the straight lines can be attributed to the difference in aggregates, asphalts, and gauge backgrounds. The same shift can be seen in Figure 7 when percentage moisture versus gauge count is plotted. This indicates the need, also established by other researchers, for recalibration of the gauge for each type of aggregate and asphalt cement.

Relative gauge counts per 1 percent of asphalt cement and moisture are given below:

Material	Asphalt Cement	Moisture
Slag A	1226	1004
Slag B	1269	1195
Gravel	1503	1562
Limestone	1418	1499

Therefore, 1 percent moisture in the mix could be read as 0.82, 0.94, 1.04, and 1.06 percent asphalt content in test series 1, 2, 3, and 4 respectively. The average would be 0.96 percent.

Test Series 5

A base course mixture (mix A) that met the gradation given in the first text table above was prepared by using a medium-setting cationic emulsified asphalt (CMS-2) and limestone aggregate. The total liquid content of 7.87 percent consisted of 3.52 percent residual asphalt in the emulsion, 1.96 percent water in the emulsion, and 2.39 percent free moisture or water. Nuclear gauge counts were taken just after mixing. A count was also made on thoroughly dried aggregate (0 percent liquid content) before the mixing operation. As the emulsified asphalt-aggregate mixture was allowed to cure, weight losses attributable to evaporation of water were accurately determined and gauge counts were taken at several intervals until the mixture was completely cured. The data from this initial run are plotted in Figure 8; the best fit straight line was obtained. The correlation between total liquid contents and gauge counts was excellent (r = 0.995).

To verify whether this relation could be used to determine total liquid content, another base-course mix (mix B) was prepared with a known liquid content of 9.07 percent, consisting of 3.49 percent residual asphalt in the emulsion, 1.95 percent water in the emulsion, and 3.63 percent free moisture or water. The mix was then allowed to cure. Total liquid contents were determined at several intervals from the gauge counts by using the mix A calibration straight line (extrapolated when necessary) and from actual weight losses by weighing. The following results were obtained:

Actual Liquid Content (%)	Gauge Liquid Content (%)	Difference
9.05	8.70	+0.35
8.00	8.05	-0.05
6.90	6.85	+0.05
5.90	6.20	-0.30
5.00	4.90	+0.10
3.50	3.30	+0.20

Statistical analysis of differences indicates that the difference is not significant at the 95 percent confidence level and that the gauge can read the total liquid content within ±0.3 percent in the 3.5 to 9.0 percent total liquid content range, which appears acceptable for use in the field.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations resulted from this study:

1. If asphalt content is held reasonably consistent, which is possible in contemporary automated asphalt plants, the nuclear asphalt-content gauge has the potential to read moisture content over the range from 0 to 3.5 percent.
2. The nuclear gauge must be recalibrated whenever there is a change in aggregate source, asphalt source, mix type, and test background.
3. Undetected moisture in the bituminous mix would be read by the gauge as asphalt content. According to this study, 1 percent moisture would be read as 0.82 to

1.06 percent asphalt by weight, depending on the aggregate composition.

4. Some moisture is required in the mixtures produced by the drum-dryer process to aid in compaction. However, it is necessary to regulate the moisture content within a working range. Conventional methods of determining moisture content are very time consuming. The nuclear asphalt-content gauge would provide a rapid means of testing and monitoring moisture content in mixtures produced by this process.

5. Tests run on mixtures that contain emulsified asphalt and aggregate indicate that the gauge can be used effectively to monitor total liquid content (emulsified asphalt plus water) in the mixture within ± 0.3 percent. The gauge could thus be used to indicate when the optimum moisture content has been reached for proper compaction.

ACKNOWLEDGMENTS

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Abridgment

Recycling Asphaltic Concrete: Arizona's First Project

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Interest in Arizona in the possibility of recycling old asphaltic concrete pavement stemmed initially from a paper by Dunning, Mendenhall, and Tischer (1). In addition, the increase in the price of paving asphalt from approximately \$44.20/Mg (\$40/ton) in 1971 to \$110.50/Mg (\$100/ton) in 1975 caused us to realize the great potential savings represented by the approximately 4 percent residual asphalt in old asphaltic concrete. Consequently, Materials Services of District 3 of the Arizona Department of Transportation (DOT) began a testing program on old asphaltic concrete to determine if recycling of this material was feasible.

In the southeastern portion of Arizona, between Willcox and the New Mexico state line, part of the old asphaltic concrete being removed and disposed of on an \$8.5 million Interstate highway project was salvaged, crushed, recycled, and used to overlay 8.53 km (5.3 miles) of US-666 from I-10 north to the Graham County line. The material was crushed, heated, and remixed and was checked by the Marshall method of determining asphaltic concrete mix designs. It was determined that adding 1.5 to 2 percent AR 2000 paving asphalt to the old asphaltic concrete resulted in a good mix.

On the basis of these preliminary tests, approximately 16 819 m^3 (22 000 yd^3) of old asphaltic concrete was salvaged from project I-10-6(50). This material was stockpiled near the south end of the proposed overlay project. It was later crushed by state maintenance forces with a Pioneer model 358S roll crusher.

Further laboratory testing indicated that blending coarse mineral aggregate with the crushed old asphaltic concrete at a ratio of 20 to 80 percent would give a job-mix formula with the desired gradation, percentage voids in mineral aggregate, and effective voids. The average gradation for these two materials is given in the following table (corresponding U.S. sieve sizes are 1, 0.75, 0.50, 0.375, and 0.25 in and nos. 4, 8, 40, and 200):

Sieve Size (mm)	Percentage Passing: Old Asphaltic Concrete	Percentage Passing: Coarse Aggregate
25	–	100
19	100	80
12.5	93	33
9.5	85	11
6.3	–	2
4.75	72	1
2.36	60	
0.425	26	
0.075	8.8	

The average residual asphalt in the old crushed asphaltic concrete was 3.2 percent.

It was determined that the cost of salvaging, hauling, crushing, and stockpiling the old asphaltic concrete was \$1.65/Mg (\$1.49/ton). In addition, coarse mineral aggregate was purchased, hauled, and stockpiled, ready for blending with the salvaged asphaltic concrete, for \$3.14/Mg (\$2.84/ton).

Arizona DOT Materials Services completed the design for the proposed recycling and overlay project on August 10, 1976. This design called for the 20-80 blend of coarse mineral aggregate and old asphaltic concrete previously mentioned. These materials were to be combined and mixed with approximately 2.2 percent of an AR 2000 paving asphalt and aromatic extender oil blend. Approximately 18 percent of aromatic extender oil, based on the weight of the asphalt, was recommended for the blend.

On October 14 and 15, 1976, representatives of District 3 and Materials Services contacted Robert Mendenhall of the Las Vegas Paving Corporation and discussed recycling of asphaltic concrete. They also observed the recycling procedure used on Nevada project I-015-1(50)0. Requirements in the special provisions for this project were later revised and adopted for the Arizona project.

The Arizona project was finally advertised for bid with a bid opening date of January 7, 1977. A special provisions addendum called for a prebid conference to be held in Phoenix on December 28, 1976, and all prospective bidders were requested to attend. As a result of this conference, the requirement that aromatic extender oil and paving asphalt be introduced into the mixer through separate gallonage meters was deleted. In addition, after much discussion, air quality standards were relaxed because of the experimental nature of the project. Specifically, the particulate count was waived, and the allowable maximum opacity was 40 percent.

The bid opening was postponed until January 13, 1977, when it was learned that a U.S. patent had been approved for recycling of asphaltic concrete and for the aromatic extender oils used in the process. This delay gave the bidders an opportunity to review the patent and to make provisions, if necessary, in their bids for royalties claimed by Mendenhall, the patent owner.

The successful bidder, A.C. Speer Construction Company, participated in a preconstruction conference held in the District 3 office. Speer was granted a delayed starting date to March 7, 1977, so that necessary modifications to the company's drum mixer could be made.

The initial modifications to the drum mixer included installation of a special steel alloy "pyro-cone" 1.8 m (6 ft) in diameter and perforated with 25-mm (1-in) holes. The second major modification was the addition of a high-speed underfeed belt. The purpose of this belt was to throw the blended cold-feed material about 0.9 m (3 ft) into the drum so that the asphalt-coated particles of the crushed asphaltic concrete would be removed from the high temperatures near the inlet end of the drum as quickly as possible.

Another modification was the installation of a meter for blending the aromatic extender oil with the paving asphalt as the two materials were pumped into the asphalt storage tank. This method of metering worked fairly well but for one serious drawback: The percentage of extender oil in the blend of asphalt and extender oil could not be changed with any degree of exactness until the asphalt storage tank was almost empty.

Another important addition was six 9.375-mm (0.375-in) water sprays, three each to two 25-mm (1-in) feed lines, for the purpose of adding moisture to the cold-feed material. Each feed line was equipped with a valve and a pressure gauge for adjusting the amount of added moisture to help control emissions from the stack.

On March 14, 1977, the contractor began production at a rate of 136 Mg/h (150 tons/h). Water was applied to the cold feed by means of three water sprays at 483 kPa (70 lbf/in^2). The amount of added blend of paving asphalt and extender oil was 2.2 percent. The amount of extender oil in the blend was 18 percent, based on the weight of the paving asphalt.

The result of this first production was not satisfactory. Heavy smoke was emitted from the plant. The mix had a dry, lifeless appearance and lacked cohesiveness; the aggregate coating was not good; and the fine particles had a burned look. The water pressure on the cold feed was reduced from 483 to 207 kPa (70 to 30 lbf/in^2) and was finally discontinued; there was no apparent change in the opacity of emissions as a result of any changes that were made.

On the second day of operation, the contractor moved the discharge end of the asphalt line in the drum from 3.2 to 3.66 m (10.5 to 12 ft) from the inlet end of the drum. Several combinations of mix ingredients were tried in an effort to reduce the opacity of emissions but without success. It was apparent from all of these combinations that most of the smoke problem was caused by the burning of the finer asphalt-coated particles in the crushed old asphaltic concrete. It was soon determined that the plant would not meet the air quality standards for the project unless major changes were made.

A decision was made to further modify the drum mixer. The steel alloy pyro-cone was moved from 1.83 to 0.91 m (6 to 3 ft) from the inlet end of the drum. The burner was moved back an additional 0.61 m (2 ft) from the end of the drum, and a 0.61-m-wide steel collar was placed around the circumference of the drum extending toward the burner. The purpose of the modifications was to confine the burner blast and also to produce the effect of moving the cold feed farther away from the burner blast and the high temperature at the inlet end of the drum.

Operations began again on March 22, 1977. Adjustments were made to the exhaust damper and the air-fuel mixture used in the burner. The opacity of emissions still exceeded the allowable 40 percent. In the afternoon, 2 percent moisture was added to the 4 percent stockpile moisture of the blended cold-feed material. The application of this additional moisture considerably reduced emissions. At the end of the day, the addition of moisture to the cold feed was stopped, and emissions increased drastically.

The next day, a number of different production rates were tried. For all rates of production, 2 percent moisture was added to the cold feed. We also began experimenting with different allowable temperatures of the mix at the outlet of the drum. The contractor discovered that, to increase production and maintain allowable opacities, the outlet temperatures had to be reduced.

On the basis of these results, an order for a change in the project contract was initiated to lower the required temperature of the mix at the outlet of the drum. The contractor also agreed to cover the surge hopper and the conveyor belt from the outlet of the drum to the surge hopper to prevent loss of heat from the mix.

As the project progressed, further experimenting was done with temperatures and it was determined that, by lowering the temperature of the mix to between 93°C and 96°C (200°F and 205°F) at the drum outlet, production rates of 294 Mg/h (324 tons/h) could be achieved without serious smoke problems as long as 2 percent moisture was being added to the cold feed. Higher production rates might have been achieved, but the ultimate capacity of the exhaust fan for this particular plant was reached at this rate of production.

The gradation consistency of the mix was very uniform. And, with 50 percent aromatic extender oil in the blend of AR 2000 paving asphalt and extender oil, the most desirable mix was achieved with the addition of 2.7 percent of this blend. The resultant extracted asphalt content was 5.3 percent with a penetration of approximately 64 and a viscosity of about 218 Pa·s (2180 poises).

SUMMARY

1. The asphaltic concrete recycled on this project was dry and brittle with a penetration of 7 on the residual asphalt. It was easily crushed and did flow back together in the stockpile.

2. It was economically feasible to blend crushed asphaltic concrete and mineral aggregate and paving asphalt and aromatic extender oil in a modified drum mixer. The average production rate was 222 Mg/h (245 tons/h) based on actual plant running time. Most of the production was within the air quality standards established for the project. Production rates as high as 293.2 Mg/h (324 tons/h) were achieved, without opacity problems, when the mix temperatures at the drum outlet were lowered to 93°C to 96°C (200°F to 205°F) and 2 percent moisture was added to the cold feed.

3. Properties similar to new asphaltic concrete were obtained by 2.7 percent of a blend of AR 2000 and 50 percent aromatic extender oil with a blend of 80 percent crushed old asphaltic concrete and 20 percent new coarse mineral aggregate.

4. There was a savings of \$3.73/Mg (\$3.38/ton) when a comparison was made between the actual cost of the recycled asphaltic concrete on this project and the estimated cost of new asphaltic concrete on similar projects.

5. If the blend of AR 2000 and 50 percent extender oil, which was used in the later stages of this project, had been used throughout the project, an additional savings of \$0.044/Mg (\$0.04/ton) could have been realized. In addition, if the contractor's production rate in the later stages of the project could have been achieved throughout the project, an additional savings of \$0.663/Mg (\$0.60/ton) could have been achieved, or a total savings of \$4.44/Mg [\$4.02 (\$3.38 + 0.04 + 0.60) per ton].

RECOMMENDATIONS

1. Temperatures of the mixture should be lowered to 93°C to 104°C (200°F to 220°F) at the drum outlet. Allowing lower temperatures undoubtedly reduced the flashing or igniting of the finer asphalt-coated particles, and smoking or opacity was greatly reduced.

2. Recycling projects of this type should be scheduled during warmer periods of weather to allow the lower mix temperatures suggested above without causing problems in obtaining density in the compacted mix.

3. It should be required that paving asphalt and aromatic extender oil be introduced into the mixer through separate, interlocked positive displacement meters. This will provide the capability of varying the percentage of extender oil as job conditions require.

4. More investigation is needed to determine the best possible location of the discharge end of the asphalt line in the drum mixer. In addition, more work needs to be done in determining the required exhaust for capacity for a drum mixer modified with a perforated steel alloy cone. Fan capacities must be increased so that proper dust collectors can be used in the future.

CONCLUSIONS

The process of recycling asphaltic concrete has not been perfected. The capabilities of other types of drum mixers need to be proven and modifications made as required. We feel confident that production rates will go up, costs will go down, and air quality standards will be met. We are only on the threshold of realizing great potential savings in both funds and natural resources.

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Abridgment

Construction of Open-Graded Asphalt Friction Courses by Using Asphalt Cement and Asphalt Emulsion

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Both the microtexture and the macrotexture of pavement influence the level of skid resistance available to a tire rotating on the roadway surface (1). To increase the level of friction, several state highway departments have constructed open-graded asphalt friction course (AFC) layers as wearing surfaces (2, 3, 4, 5, 6). Much information on AFC mix design, in which asphalt cement is used as the binding agent, has been published by the Federal Highway Administration (FHWA) (7, 8) and by the American Association of State Highway and Transportation Officials (AASHTO) (9).

In Indiana, hot asphalt emulsion pavement mixtures have been used in the surface courses of highway pavements. This paper reviews the mix design methods, construction procedures, and performance of two AFC projects in which asphalt emulsion was used instead of asphalt cement as the binding agent in most of the AFC material.

IN-43 EXPERIMENTAL PROJECT

Mixtures for Test Sections

The Indiana State Highway Commission (ISHC) constructed an experimental AFC project on a 1.6-km (1-mile) section of IN-43 in White County in June 1974. The four different AFC mixtures that were placed used three coarse aggregates and two types of asphalt. A fifth section, a control section, used an ISHC standard emulsified asphalt hot sand mix.

Limestone, gravel, and slag coarse aggregates were used. The gradation of each met the requirements for ISHC 11 material, which is similar to AASHTO 2.36-mm (no. 8) grading. Coarse aggregate made up 90 percent of the AFC mix by weight of aggregate, and natural sand (ISHC 14-2) and mineral filler (ISHC 16 limestone dust) each made up 5 percent of the total aggregate weight. The gradations of the aggregates used are given below (corresponding U.S. sieve sizes are 0.50 and 0.375 in and nos. 4, 8, 16, 30, 50, 100, and 200):

	Percentage Passing by Weight of Aggregate		
Sieve Size (mm)	Coarse Aggregate	Fine Aggregate	Mineral Filler
12.5	100	–	–
9.5	75-95	100	–
4.75	5-20	98-100	–
2.36	0-5	75-95	–
1.18	–	50-75	–
0.6	–	20-53	100
0.3	–	6-25	–
0.15	–	1-17	–
0.075	0-3	0-3	65-100

Both asphalt cement and asphalt emulsion were used. The asphalt cement was a standard 85-100 penetration grade material. The asphalt emulsion, Indiana grade AE-60 (a high-float material), was manufactured by using a 50-100 penetration base asphalt cement.

Each of the five test sections was about 304.7 m (1000 ft) long and 1.90 cm (0.75 in) thick. The gravel coarse aggregate mix contained 6.5 percent asphalt cement. Slag coarse aggregate was used in two sections, one with 7.8 percent asphalt cement and one with 7.8 percent residual AE-60 asphalt emulsion (the amount of emulsion added to the mix was 11.1 percent, assuming 30 percent water in the emulsion). The limestone coarse aggregate mix incorporated 6.5 percent residual AE-60 asphalt emulsion. The hot sand mix control section, which contained 100 percent ISHC 14-2 natural sand, had a residual asphalt emulsion content of 7.5 percent.

All asphalt contents were determined by using modified Hveem mix design laboratory methods. The actual asphalt content for each mix was chosen based on a combination of stability, void content, and visual appearance according to Colorado (6) and FHWA (7) guidelines.

Construction

All five mixtures were produced in a Stansteel 3.6-Mg (4-ton) batch plant. A 60-s wet cycle time was used for all mixes, and target plant temperatures were 124° C and 110° C (255° F and 230° F) for the asphalt cement and asphalt emulsion mixtures respectively. The hot emulsion sand mix was made at a standard 93° C (200° F) at time of plant discharge.

Because of the small quantities of each mixture that were produced and because of the varying moisture contents in the three coarse aggregate materials, a wide variation occurred in the mix temperature of different mix batches in the plant. These variable temperatures caused the asphalt (particularly the asphalt cement) to drain from the coarse aggregate particles during the haul to the jobsite. In several truckloads, asphalt and mineral filler accumulated 5.1 to 7.6 cm (2 to 3 in) deep on the bottom of the truckbed.

Many fat spots occurred in the mixes when they were placed on the roadway by a conventional asphalt finishing machine. The fat areas were especially prevalent in the gravel and slag coarse aggregate sections that used asphalt cement as the binding agent. Because of the extreme number of spots, it was impractical to remove them, and they were allowed to remain in place.

Two steel-wheel rollers, a three-wheel and a tandem, were initially used to compact the AFC mixtures. Much crushing of the aggregate occurred during the rolling operation, especially under the three-wheel roller. Compaction was finally accomplished by using only one or two passes of the tandem roller. Some pickup of the mix occurred when the roller passed over a fat spot in the pavement surface.

Performance

The performance of the four AFC test sections and one sand mix test section has been monitored visually since 1974. Test data on mix performance have not been gathered because of the variations that existed at the time of construction, particularly the spots of excess asphalt on the AFC surfaces. In early 1978, the fat spots were still quite visible. Some raveling of the coarse aggregate has occurred in all test sections, but more coarse aggregate has been lost from the two sections that used asphalt cement (gravel and slag coarse aggregate) than from the two sections that used asphalt emulsion (slag and limestone coarse aggregate).

Based on visual observations only, the AFC test sections can be ranked as follows in order of quality of performance from best to worst: (a) slag and emulsion, (b) limestone and emulsion, (c) slag and asphalt cement, and (d) gravel and asphalt cement. The performance of each mix, however, seems to be directly related to the condition of each mix at the time of initial placement.

I-64 PROJECT

Mix Design

In the fall of 1975, an AFC surface course mixture was placed on a 5.72-km (3.56-mile) long section of I-64 in Crawford County, Indiana, at IN-37. The material was placed 1.90 cm (0.75 in) thick on top of 40.6 cm (16 in) of full-depth asphalt concrete. Hot emulsion sand mix was placed as the wearing course on an additional 4.98 km (8.10 miles) of the project at a rate of 1.58 cm (0.625 in).

A 90 percent mechanically crushed gravel that met an AASHTO 2.36-mm (no. 8) grading was used as the coarse aggregate in the AFC mix. This material, barged and trucked over 160.9 km (100 miles) to the paving site, was used for 90 percent of the mix. Natural sand (ISHC 14-2) and a limestone dust mineral filler (ISHC 16) each made up 5 percent of the total weight of the aggregate blend. The specification limits for the mixture are given below (corresponding U.S. sieve sizes are 0.50 and 0.375 in and nos. 4, 8, 16, 30, 50, 100, and 200):

Sieve Size (mm)	Percentage Passing by Weight of Aggregate: Specification Limits	Percentage Passing by Weight of Aggregate: Typical Grading
12.5	100	100
9.5	88-100	92
4.75	19-37	37
2.36	9-19	13
1.18	7-13	9
0.6	6-12	7
0.3	5-10	5
0.15	4-8	4
0.075	3-6	3

Grade AE-60 asphalt emulsion was used in the AFC material. The laboratory asphalt content of the mix was determined by the centrifuge kerosene equivalent test.

Different residual asphalt contents, mix temperatures, and methods of compaction were used in several trial mix designs made by both ISHC and FHWA. An optimum asphalt content of 6.2 percent residual asphalt emulsion (8.9 percent emulsion, by weight of mix, added to the plant pug mill) was selected for the project. Mixture temperature was set at 110° C (230° F).

Construction

The AFC mixture was produced in a 4.5-Mg (5-ton) Cedarapid asphalt batch plant. Because of the volume of water in the emulsion and the size of the plant's asphalt weigh bucket, only 3.6 Mg (4 tons) of AFC were mixed in one batch. A 2-s dry mix, 70-s wet mix, and 82-s total mix cycle was used. DC-200 silicone was added to the AE-60 asphalt emulsion. The mixture was discharged into the haul vehicles at a temperature between 104° C and 116° C (220° F and 240° F).

The mixture, which was hauled an average distance of 11.3 km (7 miles), was spread with a Blaw-Knox PF-180 paver. At the time of laydown, mix temperatures ranged between 102° C and 104° C (215° F and 220° F) except at the start of paving. No unusual placement problems occurred. The mix flowed from under the paver screed with no pulling or tearing. Compaction was accomplished in four passes over each point in the pavement surface with a Tampo RS-188A double-drum vibratory roller. The roller operated in the static mode without vibration. No nuclear density readings or compaction cores were taken on the AFC layer.

Temperature variations occurred in the first few loads of mix delivered because of starting difficulties at the plant. Some minor drainage of asphalt emulsion from the aggregate occurred in these loads during the truck haul. Several small areas of excess asphalt appeared in the placed mix; these were left in place. Once the plant temperature was stabilized, no further problems were encountered with asphalt drainage.

Performance

Traffic Volumes

In the summer of 1977, approximately 6000 vehicles/d, including 15 to 20 percent trucks, were traveling the test section of I-64. The eastern half of the AFC surface has been under traffic a year longer than the western half because of the location of an interchange in the middle of the job and a delay in the opening of the western half as a result of uncompleted construction beyond the project. Significant differences exist in the two halves of the AFC material; the section that has been under traffic for the extra year shows more damage and poorer performance.

Flushing

The eastern part of the AFC mixture has not performed satisfactorily. Minor flushing occurred in the outside lane at the start of the paving project early in the life of the pavement, primarily because of fluctuations in the initial mix temperature and problems with drainage of asphalt emulsion. This early flushing stabilized during 1976 and remained stable until mid-1977. The remainder of the AFC mixture initially exhibited no flushing or bleeding characteristics.

During the summer of 1977, major flushing occurred in

Table 1. Skid resistance of I-64 surface mixtures.

Date	Test Speed (km/h)	Skid Number: Asphalt Friction Course: Eastbound	Westbound	Average	Emulsion-Sand Mix: Eastbound	Westbound	Average
November 1975	64	39.2	43.4	41.3	50.9	47.3	49.1
June 1977	64	43.5	39.7	41.6	50.5	47.2	48.8
	80	40.6	40.5	40.5	47.7	45.2	46.4
	96	42.5	37.6	40.0	40.4	41.8	41.1

Note: 1 km = 0.62 mile.

both directions in both wheel paths of the travel lanes on the eastern half of the project. No flushing occurred in the wheel paths of the passing lane in either direction. No flushing occurred in any of the AFC mix on the western part of the project even though that section was opened to traffic in late 1976 and was subjected to the same traffic during all of 1977 that caused the flushing on the eastern half.

The flushing in the wheel paths on the eastern portion is not continuous. Although several areas show excessive asphalt for a distance of over 0.81 km (0.5 mile), some locations between the badly flushed locations exhibit only minor or spotty flushing. This flushing has occurred both in the AFC mix placed at the start of paving and in the mix laid at the end of the job (3 d of total paving).

Consolidation and Shoving

Several of the badly flushed AFC areas have consolidated or rutted. In some locations, the depth of rutting is almost 0.95 cm (0.375 in); the average is 0.63 cm (0.25 in). Consolidation has taken place in a layer only 1.9 cm (0.75 in) thick. Visual observation indicates that lateral shoving has occurred: The AFC mix adjacent to the flushed wheel paths is slightly humped up above the surrounding pavement surface. The rutting is directly related to the flushing of the AFC mix.

Skid Numbers

The first skid tests on the I-64 project were conducted in November 1975, about 1 week after the eastern half of the project was opened to traffic. An American Society for Testing and Materials E-17 skid trailer was used. The measured skid numbers (Table 1) were obtained at 64 km/h (40 mph). The skid values for the AFC mixture were lower than those measured on the adjacent hot asphalt emulsion sand mix.

Skid tests were again conducted in June 1977. There was no improvement in the AFC values. The skid measurements were taken on the eastern portion of the project before the extensive flushing in the wheel paths took place. The AFC material exhibits little decrease in skid number with an increase in vehicle speed. The speed gradient of this mix is essentially zero. Although the hot asphalt emulsion sand mix has a greater speed gradient, it also has better levels of skid resistance at all test speeds. The 64-km/h (40-mph) skid numbers for the AFC mixture are slightly less than the average values obtained on densely graded asphalt concrete surface course mixtures in Indiana that use either asphalt cement or asphalt emulsion as the binding agent. To date, the AFC mixture has not demonstrated any benefits in relation to skid resistance.

Discussion of Results

The open-graded asphalt friction course material on the eastern half of the I-64 project deteriorated badly, in terms of rutting and flushing, during the second summer under traffic. The texture of the AFC surface has closed up; this significantly reduces the macrotexture of the surface in the wheel paths of the travel lane. Measured skid numbers are low even in the unflushed areas.

Visual observations made of the AFC material indicate that the mix design procedure used for this type of material is not adequate for use with asphalt emulsion as the binding agent in the mix. The performance of the AFC mix has been less than satisfactory, primarily because of flushing of the mix and rutting of the material under traffic. In addition, the skid numbers of the mix, using crushed gravel coarse aggregate, are below those of Indiana's standard hot asphalt emulsion sand mix or normal, densely graded asphalt concrete mixtures.

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