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Asphalt: Materials, Mixes, and Construction

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Effects of Soil Binder on Blackbase Mixture

WEI-CHOU V. PING AND THOMAS W. KENNEDY

The findings of a study to evaluate the effects of soil-binder content on the behavior of blackbase mixtures used in Texas are summarized. The evaluation was based on a comparison and analysis of engineering properties, obtained by using the static and repeated-load indirect tensile tests, of mixtures with various soil-binder contents. For this study two blackbase mixtures, a rounded gravel with field sand and a crushed limestone, were investigated. Various engineering properties were evaluated at various soil-binder contents and asphalt contents. The engineering properties evaluated were tensile strength, static modulus of elasticity, fatigue life, and resilient modulus of elasticity. Most of the tests were conducted on air-dried specimens; however, a limited number of tests were conducted on pressure-wetted specimens to evaluate the influence of moisture content. Generally, the results indicate that the various engineering properties were maximized at relatively low soil-binder contents and at correspondingly lower asphalt contents. The optimum asphalt contents tended to decrease as the soil-binder content decreased. The optimum soil-binder contents for the various engineering properties ranged from 5 to 10 percent.

The primary objective of the study summarized in this paper was to evaluate the effect of soil-binder content on the behavior and design of blackbase paving mixtures used in Texas. Soil binder is defined as material that will pass the U.S. standard no. 40 sieve.

The acceptability of an aggregate gradation for use in asphalt mixtures often is judged by its conformity to specified size limits. These limits have generally been established either on the basis of satisfactory experience with materials that meet selected gradation specifications or in terms of selected gradation patterns of natural or crushed materials that are readily available. Thus, it is possible to have gradation limits that vary significantly but that will still produce satisfactory asphalt mixtures.

In Texas, a range of binder contents is specified as a part of the gradation requirements (specification no. 346, 1962). To determine the effect of soil-binder content on asphalt paving mixtures and to determine whether improved or less costly mixtures can be produced by specifying a binder content or by eliminating all specification requirements concerning binding content, a limited study (1) was conducted at the request of the Texas State Department of Highways and Public Transportation.

EXPERIMENTAL PROGRAM

The basic experimental approach was to determine the relationships between asphalt content and the engineering properties and to determine the optimum asphalt content for each property. These relationships and optima were then evaluated with respect to soil-binder content to determine whether properties could be improved by controlling the binder content. Finally, the effect of moisture on these relationships was evaluated.

The experimental study was a comparison of the engineering properties of asphalt mixtures composed of two representative types of aggregate, each with various soil-binder contents. By changing the quantity of soil binder, each selected aggregate gradation was mixed to produce laboratory specimens that had asphalt contents in the range generally used for design.

Materials

Two aggregates were used, a rounded river gravel and a crushed limestone, both of which are commonly used in pavement construction in Texas and have performed satisfactorily. The two aggregates were obtained from Eagle Lake and Lubbock, Texas, respectively.

The Eagle Lake aggregate was a mixture of four different aggregates, the combination of which can be generally described as a smooth-surfaced, angular, nonporous, crushed siliceous river gravel. The other material was a rough, subangular, porous crushed-limestone caliche obtained approximately 10 miles southeast of Lubbock.

The soil-binder contents selected for the Eagle Lake material were 0, 5, 10, 20, and 30 percent, and the binder contents for the Lubbock limestone were 0, 5, 10, and 25 percent. The various gradations and percentages of soil binders were obtained by the addition or removal of material finer than the no. 40 sieve while a constant amount of the coarser material was maintained. Gradations of the mixtures are shown in Figures 1 and 2.

The asphalt cement used with the Eagle Lake materials was an AC-20 produced at the Exxon refinery in Baytown, Texas; the asphalt cement used with the Lubbock limestone was an AC-10 produced by the Cosden Oil refinery in Big Springs, Texas.

Specimen Preparation

All specimens prepared for this investigation were mixed and compacted according to test method Tex-126-E by using the Texas motorized gyratory compactor (2). Subsequent to compaction, all specimens were allowed to cool in the compaction mold for 1 h before extrusion to prevent slumping of the specimen. After extrusion from the mold, the specimens were cured overnight at room temperature. Smaller test specimens for the indirect tensile test were then cut from the top and bottom portions of the compacted specimen. The densities of these test specimens were calculated from their weights and physical dimensions.

The top and bottom specimens were cured overnight at a room temperature of approximately 24°C (75°F). Thus, the total curing time was two days. The sawed specimens were 152 mm (6 in) in diameter and about 84 mm (3.3 in) in height and were tested by using the indirect tensile test.

To evaluate the effects of moisture, the exposed sawed faces of the test specimens were coated with a thin film of the asphalt cement used in the mixture. Then the specimens were subjected to pressure wetting (test method Tex-109-E, part IV). This procedure subjects a specimen to an 8274-kPa (1200-psi) hydrostatic water pressure at a water temperature of 65°C (150°F) for 15 min prior to actual testing by means of the indirect tensile test.

Indirect Tensile Test

The indirect tensile test is conducted by loading a cylindrical specimen with a single or repeated compressive load that acts parallel to and along the vertical diametral plane. The loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametral plane, which ultimately causes the specimen to fail by splitting along the vertical diameter. The basic testing equipment was

Figure 1. Gradations of Eagle Lake gravel mixture.

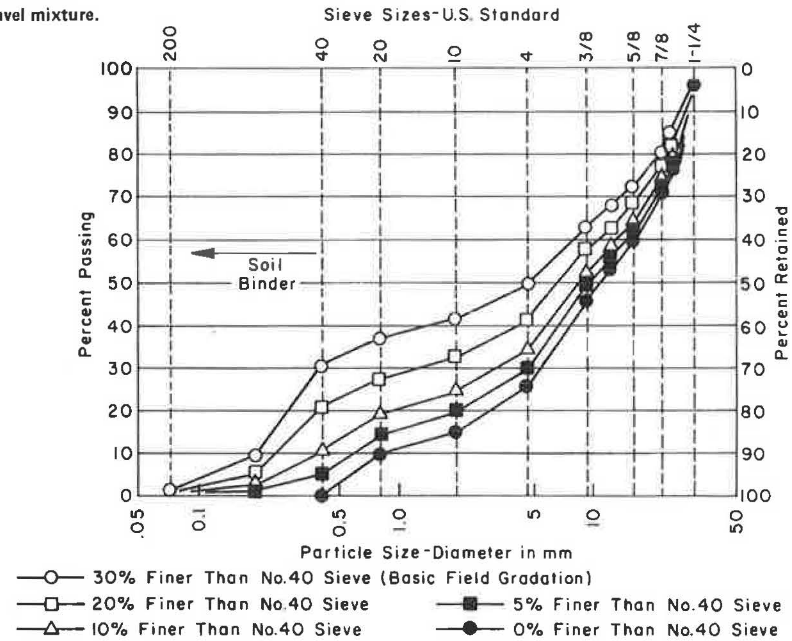
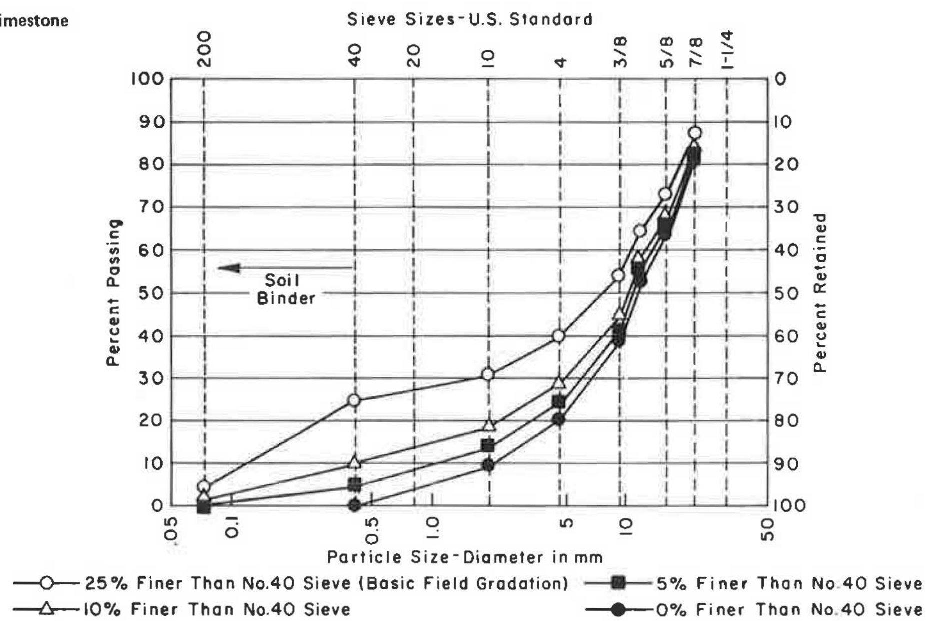


Figure 2. Gradations of Lubbock limestone mixture.



a Material Testing System (MTS) closed-loop electrohydraulic loading system.

The basic test procedure is described by Kennedy and Anagnos (3). A preload, which produced a tensile stress of approximately 5 kPa (0.6 psi), was applied to the specimens in the static tests to prevent impact loading and to minimize the effect of seating of the loading strip. The specimen was then loaded at a constant deformation rate of 51 mm/min (2 in/min). Vertical deformations were measured by a dc linear variable differential transducer (LVDT). Horizontal deformations were measured by two cantilevered arms with attached strain gauges. Both the load-vertical deformation and load-horizontal deformation relationships were recorded by a pair of X-Y plotters.

For the repeated-load tests, a light preload was also applied. The desired load was applied at a frequency of 1 Hz with a 0.4-s load duration and a 0.6-s rest period. Both the horizontal and vertical deformations were measured by dc LVDTs and were recorded on the X-Y plotters. All tests were conducted at 24°C (75°F)

Properties

Several of the properties analyzed are related to the relevant pavement distress modes of (a) thermal or shrinkage cracking, (b) fatigue cracking, and (c) permanent deformations, or rutting. The properties analyzed were

1. General: (a) density and (b) total air voids,
2. Static characteristics: (a) tensile strength and (b) static modulus of elasticity, and
3. Repeated-load characteristics: (a) fatigue

life and (b) resilient modulus of elasticity.

Density and Air Voids

The asphalt-voids ratio (AVR) density was calculated by using the mold diameter and the measured height, which was obtained while the specimen was subjected to the final compaction load of 3447 kPa (500 psi). The specimen weight was determined after extrusion from the mold. This density is also referred to as the in-mold density and is used to calculate percentage of total air voids as defined by test method Tex-126-E.

Tensile Strength

The ultimate tensile strength was calculated by using the following relationship for specimens 152 mm (6 in) in diameter and the load-deformation information obtained from the static indirect tensile test:

$$S_t = 0.105P_{ult}/t \quad (1)$$

where

S_t = ultimate tensile strength (psi),
P_{ult} = maximum load carried by the specimen (lb), and
t = thickness of the specimen (in).

Tensile stresses produced by loads less than the maximum load P_{ult} can also be calculated by using the above equation.

Static Modulus of Elasticity

The static modulus of elasticity was determined from the load-deformation relationships for static tensile tests. The equation used to calculate the static modulus was

$$E_s = (S_h/t)(0.27 - \nu) \quad (2)$$

where

E_s = static modulus of elasticity (psi);
S_h = slope of the relationship between axial load and horizontal deformation, i.e., the ratio of axial load to horizontal deformation within the linear range (lb/in); and
ν = static Poisson's ratio = (4.09/DR) - 0.27,

where DR = deformation ratio, i.e., slope of the relationship between vertical deformation and horizontal deformation (inches of vertical deformation per inch of horizonal deformation).

Fatigue Life

Fatigue life is defined as the number of load applications at which the specimen will no longer resist load or at which deformation is excessive and increases with essentially no additional loads.

Resilient Modulus of Elasticity

The resilient modulus of elasticity was calculated by using the resilient horizontal and vertical deformations, which are characteristic of the elastic deformations produced by repeated loads of short duration.

The equation used to calculate the resilient modulus was

$$E_R = (P_R/tH_R)(0.27 + \nu_R) \quad (3)$$

where

E_R = resilient modulus of elasticity (psi),
P_R = applied repeated load (lb), and
ν_R = resilient Poisson's ratio = 4.09 $(H_R/V_R) - 0.27$

where H_R and V_R are the resilient horizontal and vertical deformations for the loading cycle that corresponds to 0.5 N_f. The equation is the same as that used for the static Poisson's ratio; however, since the relationship between load and deformation is essentially linear, the equation was modified.

Testing Program

The variables included in this study were aggregate type, soil-binder content, asphalt content, and moisture content. For the repeated-load tests, two stress levels that would produce reasonable fatigue lives were selected. Only a limited number of mixtures were tested to evaluate the effects of moisture. These mixtures contained the optimum asphalt contents for maximum tensile strength. All tests were performed at room temperature, 24°C (75°F).

ANALYSIS AND DISCUSSION OF TEST RESULTS

Density and Design Optimum Asphalt Content

The in-mold AVR densities were generally larger than the densities of the top and bottom sections of the specimens, and the densities of the bottom specimens were generally greater than those of the top specimens. Mixtures that had high soil-binder contents had the lowest in-mold AVR densities, whereas the mixtures that had relatively low binder contents had the highest in-mold AVR densities. The relationships between asphalt content and total air voids were determined for each aggregate gradation (Figure 3). The laboratory AVR-design optimum asphalt contents (2) are slightly greater than the asphalt contents that correspond to the inflection point on the straight-line section of the AVR curves.

Eagle Lake Gravel

These relationships indicate that (a) as the amount of soil binder decreased from 30 percent to 5 percent the total air voids decreased and (b) the total air voids increased appreciably as the amount of soil binder decreased from 5 percent to 0 percent. The total air voids were approximately the same for binder contents of 5 and 10 percent. The relationships between soil-binder content and total air voids at asphalt contents of 3.0, 3.5, and 4.0 percent are shown in Figure 4. These relationships indicate that the minimum total air voids occurred at a binder content of about 7 percent for mixtures containing 3.0 and 3.5 percent asphalt and at a binder content of about 10 percent for mixtures containing 4.0 percent asphalt. The laboratory AVR design optimum asphalt content decreased from 4.5 percent for a 30 percent binder content to a minimum value of 3.5 percent for a 5 percent binder content and then increased slightly to 3.6 percent for 0 percent binder content (Figure 5a). The corresponding total air voids at laboratory AVR design optimum asphalt content remain constant at 1.6 percent for values of soil-binder content ranging between 5 and 20 percent but increase appreciably for 0 percent and 30 percent binder contents (Figure 5b).

Figure 3. Relationships between asphalt content and total air voids.

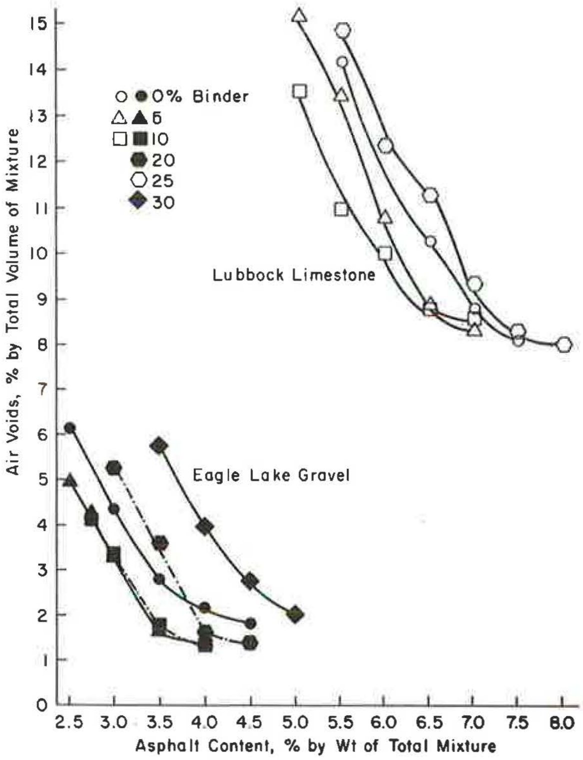


Figure 4. Relationships between soil-binder content and total air voids.

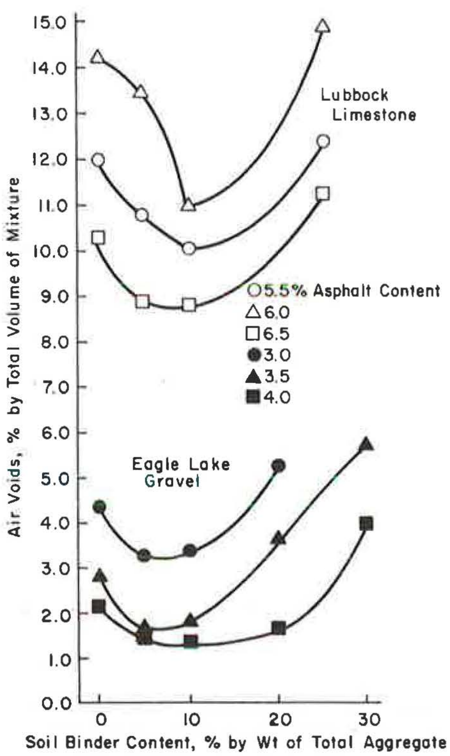
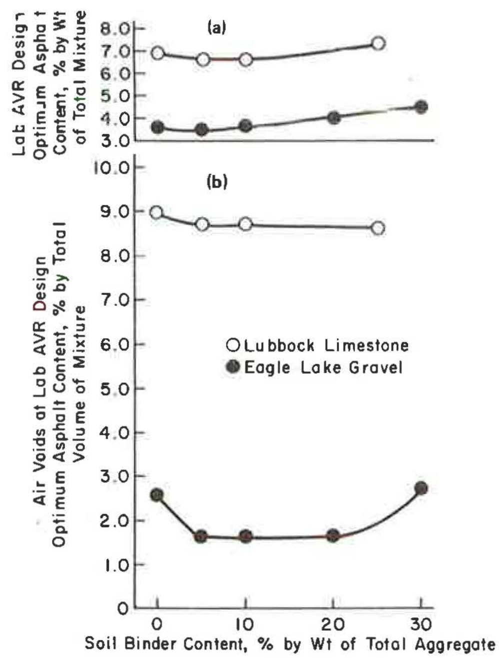


Figure 5. Relationships between soil-binder content and (a) AVR design optimum asphalt content and (b) total air voids.



Lubbock Limestone

The total air voids relationships for Lubbock limestone shown in Figure 4 indicate that, as the amount of soil binder decreased from 25 percent to 10 percent, the total air voids decreased to a minimum and then increased as the amount of soil binder decreased further from 10 to 0 percent. The mixture containing 10 percent binder had the lowest total air voids, regardless of the percentage of asphalt content.

The relationships between soil-binder content and (a) laboratory AVR design optimum asphalt content and (b) total air voids at laboratory AVR design optimum asphalt content are also shown in Figure 5. Laboratory AVR design optimum asphalt contents (Figure 5a) were 6.6 percent for both 5 percent and 10 percent soil-binder contents; however, the optimum asphalt contents are higher for 25 percent soil-binder content (7.3 percent) and 0 percent soil-binder content (6.9 percent). For each soil-binder content, the total air voids at laboratory AVR design optimum asphalt content are very close; they ranged from 9.0 percent for 0 percent soil-binder content to 8.6 percent for 25 percent soil-binder content.

Static Indirect Tensile Test Results

The engineering properties, tensile strength, and static modulus of elasticity were estimated by using the static indirect tensile test.

Tensile Strength

Optimum asphalt contents for maximum tensile strength were found for each soil-binder content and each aggregate type. Values of optimum asphalt content for the Eagle Lake mixture ranged from 3.0 percent for soil-binder contents of 5 and 10 percent to 4.0 percent for a binder content of 30 percent and from 5.5 percent for a binder content of 10 percent to 7.0 percent for a binder content of 25 percent for the Lubbock limestone mixtures (Figure 6a). The maximum tensile strength was 1365 kPa (198 psi) for the Eagle Lake gravel mixture and 1367 kPa (200 psi) for the Lubbock limestone mixture, both of which occurred at a binder content of 5 percent (Figure 6b).

Figure 6. Relationships between binder content and (a) optimum asphalt content and (b) corresponding maximum tensile strength.

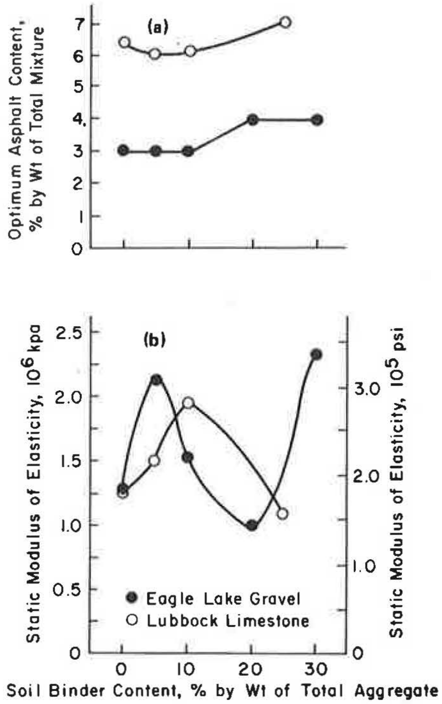
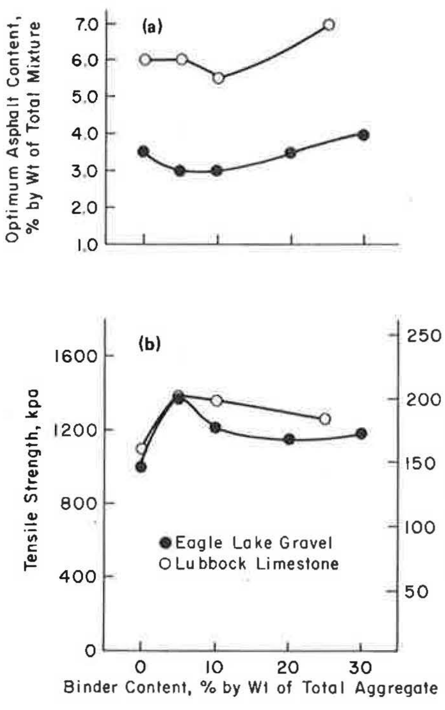


Figure 7. Relationships between soil-binder content and (a) optimum asphalt content and (b) corresponding static modulus of elasticity.



The maximum tensile strengths of the Eagle Lake mixtures at binder contents of 30, 20, and 10 percent were essentially equal at about 1190 kPa (173 psi); however, the optimum asphalt contents were 4.0, 3.5, and 3.0 percent, respectively. As the soil-binder content decreased from 10 to 5 percent, the strength increased by 180 kPa (26 psi), and the optimum asphalt content remained constant at 3.0 percent. Without any soil binder, the ultimate tensile strength of the Eagle Lake gravel mixture decreased significantly, and the mixing optimum asphalt content increased from 3.0 to 3.5 percent.

Similar trends were found for the Lubbock limestone mixtures, except that the optimum asphalt contents for 5 and 0 percent binder contents were the same (6.0 percent).

Static Modulus of Elasticity

For all mixtures there were optimum asphalt contents for maximum static moduli of elasticity (Figure 7). These optimum asphalt contents for Eagle Lake gravel mixtures ranged from 3.0 percent for 0, 5, and 10 percent binder contents to 4.0 percent for 20 and 30 percent binder contents (Figure 7a). For Lubbock limestone mixtures, the optimum ranged from 6.0 percent for 5 percent soil-binder content to 7.0 percent for 25 percent soil-binder content.

For the Eagle Lake gravel mixtures, the maximum static modulus of elasticity probably occurred at 5 percent (Figure 7b). The optimum binder content was 10 percent for Lubbock limestone mixtures.

Repeated-Load Indirect Tensile Test Results

Repeated-load indirect tensile tests were conducted to evaluate the fatigue life and resilient modulus of elasticity. A discussion of the resistance to permanent deformation for each of the mixtures may be found in Ping and Kennedy (1).

Fatigue Life

To eliminate the effect of stress and to determine the effect of asphalt content and binder content, the fatigue lives of the mixtures were evaluated for a tensile stress of 100 kPa (14.5 psi) for each binder content.

The optimum asphalt content for maximum fatigue life was found for each of the mixtures of Eagle Lake gravel and Lubbock limestone. The relationships between soil-binder content and the optimum asphalt contents are shown in Figure 8. Optimum asphalt contents for the Eagle Lake mixtures ranged from 2.9 percent for 5 percent binder content to 4.6 percent for 30 percent binder content and from 4.5 percent for 10 percent binder to 7.5 percent for 25 percent binder for the Lubbock limestone mixtures. For both mixtures the optimum asphalt content for 0 percent soil binder was higher than the optimum for mixtures with 5 and 10 percent soil binder.

The optimum soil-binder content for maximum estimated fatigue life was 5 percent for both types of aggregate. The maximum estimated fatigue life was about 8.7 kHz for the Eagle Lake gravel at 2.9 percent asphalt content and about 980 kHz for the Lubbock limestone at 6.0 percent asphalt content.

Resilient Modulus of Elasticity

The relationships between asphalt content and the resilient modulus of elasticity at 0.5 N_f indicated that the optimum asphalt content for maximum resilient modulus was not well defined; most of the relationships are flat. This behavior is consistent with the behavior previously reported (1,4).

Nevertheless, in order to analyze the effects of binder content, an attempt was made to pick an asphalt content that produced the maximum modulus. The resulting relationships between soil-binder content and optimum asphalt content for maximum resilient modulus of elasticity for the loading cycle corresponding to 0.5 N_f are shown in Figure

Figure 8. Relationships between binder content and both optimum asphalt content and the corresponding fatigue life for (a) Eagle Lake and (b) Lubbock limestone mixtures.

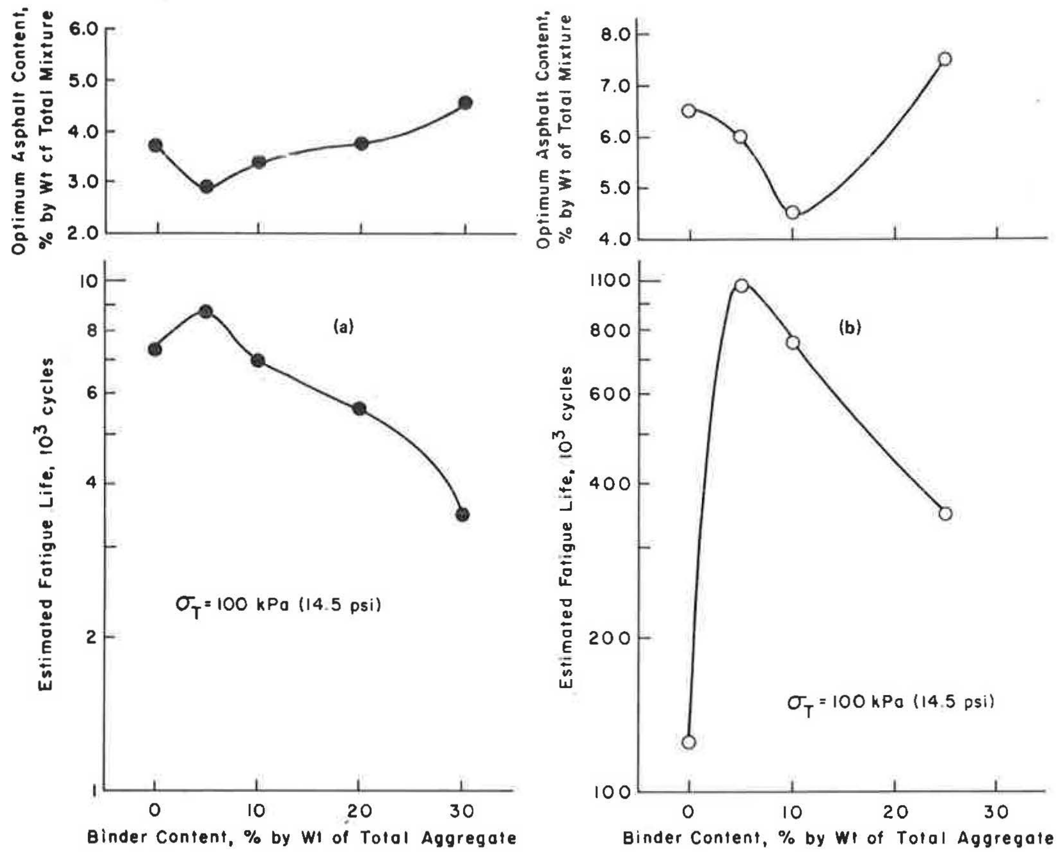
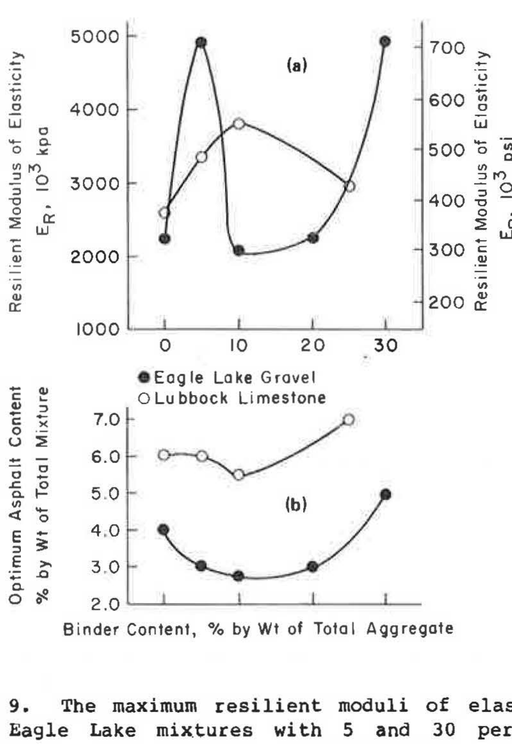


Figure 9. Relationships between soil-binder content and (a) maximum resilient modulus of elasticity at 0.5 N_f and (b) corresponding optimum asphalt content.



9. The maximum resilient moduli of elasticity for Eagle Lake mixtures with 5 and 30 percent soil-binder contents were about 2.5 times those for 0, 10, and 20 percent soil-binder contents. Thus, either 5 or 30 percent may be chosen as the optimum soil-binder content; however, the curve was not well defined. The optimum binder content for maximum resilient modulus of elasticity for the Lubbock limestone mixtures was 10 percent with 5.5 percent asphalt content.

Since the actual asphalt content is not critical to the resilient modulus of elasticity and since the actual values of modulus were relatively constant, the minimum values of asphalt content should be used.

Moisture Damage

This study generally indicated that the optimum soil-binder contents for maximum engineering properties were relatively low, in the range of 5 to 10 percent. In addition, low binder contents required less asphalt and, therefore, improved the economy of the mixtures. However, the specimens tested were dry and had not been subjected to moisture. A limited series of tests was therefore conducted to evaluate the effects of moisture on the engineering properties of the two materials. A series of specimens for each aggregate type at the optimum asphalt content for the maximum ultimate tensile strength was subjected to pressure wetting and then was tested to obtain static indirect tensile results and resilient moduli of elasticity.

Test results are shown in Figures 10 through 13. The asphalt contents of tested specimens were lower than the optimum asphalt contents for the maximum densities, and the air-void contents were higher. Contents after pressure wetting were proportional to the total air voids, i.e., the higher the total air voids, the higher the water contents.

A definite effect of moisture was apparent on the static modulus of elasticity and the ultimate tensile strength (Figures 10 and 11). A strength loss of about 250 kPa (36 psi) occurred for Eagle Lake gravel mixtures with 5 percent soil binder and a loss of about 500 kPa (72 psi) for mixtures with 30 percent soil binder. However, pressure wetting did not produce a loss of tensile strength for

Figure 10. Relationships between binder content and moisture content on (a) the static modulus of elasticity and (b) the ultimate tensile strength for Eagle Lake gravel mixtures.

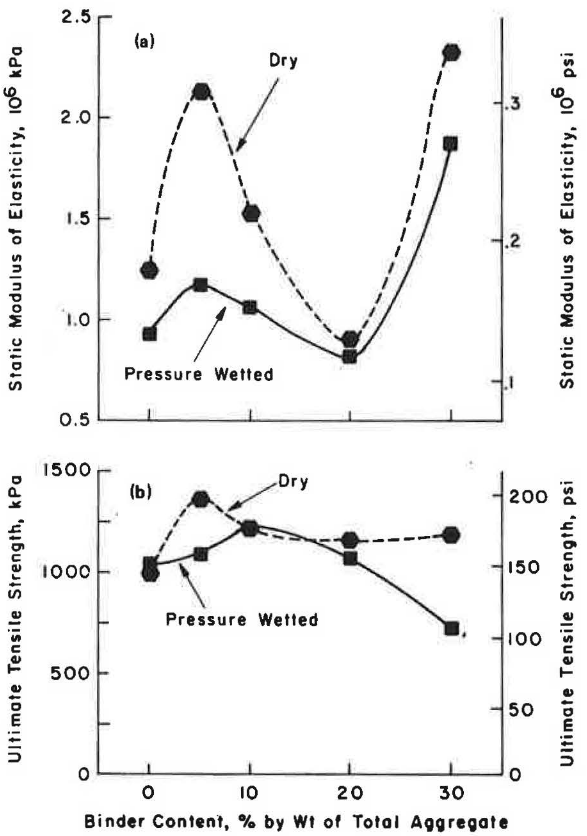


Figure 11. Relationships between binder content and moisture content on (a) the static modulus of elasticity and (b) the ultimate tensile strength for Lubbock limestone mixtures.

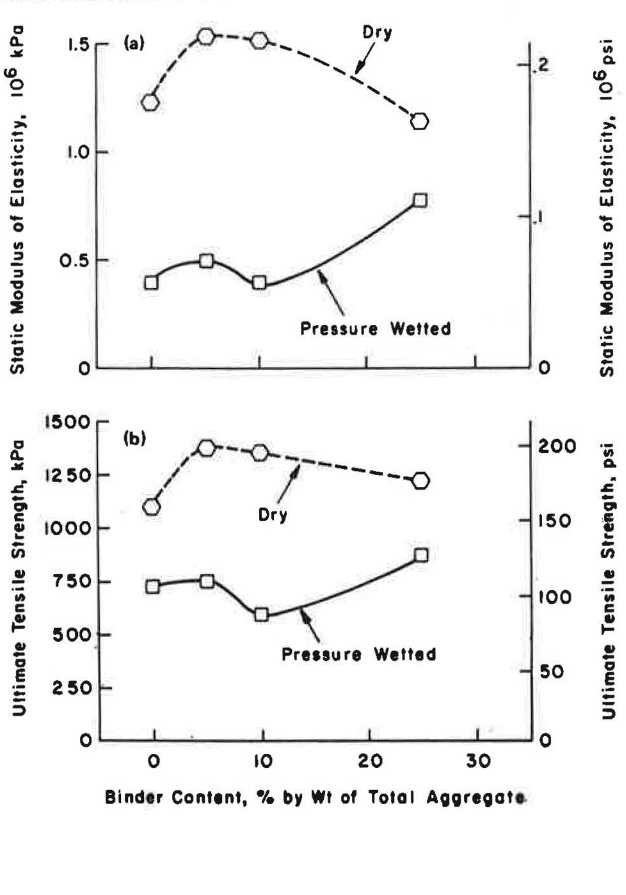


Figure 12. Relationships between binder contents and air voids, water content, and resilient modulus of elasticity for Eagle Lake gravel mixtures.

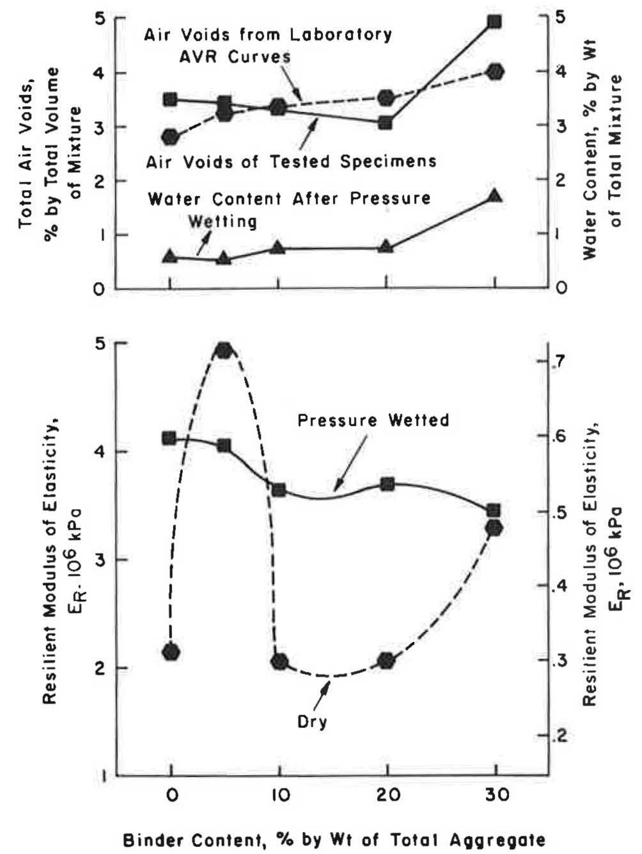
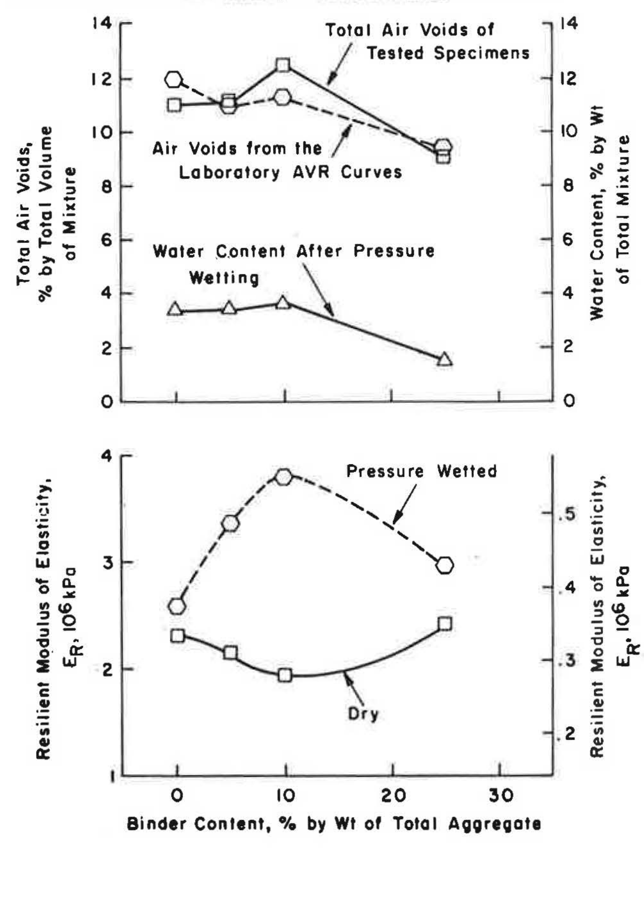


Figure 13. Relationships between binder content and air voids, water content, and resilient modulus of elasticity for Lubbock limestone mixtures.



mixtures with 0, 10, and 20 percent soil binder (Figure 10). For the Lubbock limestone mixtures the losses were more consistent, varying from 750 kPa (110 psi) to 400 kPa (58 psi). The effect of pressure wetting on static modulus of elasticity was more significant (Figure 11). Losses in modulus for the Eagle Lake mixtures ranged from 100 000 kPa (14 500 psi) to slightly less than 1 000 000 kPa (145 000 psi). Similarly, for the Lubbock limestone the losses ranged from about 400 000 kPa (58 000 psi) to 1 000 000 kPa (145 000 psi). No consistent or explainable relationships were observed for the resilient modulus of elasticity (Figures 12 and 13). In most cases the pressure-wetted specimens exhibited higher moduli than did the dry specimens. This was especially true for the Lubbock limestone mixtures.

The shape of the density relationships for tested specimens and the shape of the relationships for tensile strength and the static modulus of elasticity after pressure wetting (Figures 10 and 11) were similar.

Moisture damage apparently was dependent on the density of the mixtures or air void content. The highest density for Eagle Lake gravel mixtures was achieved at 5 percent soil-binder content and at 10 percent soil-binder content for Lubbock limestone mixtures. This would suggest that, as long as the mixture has adequate density, substantial moisture damage will not occur; however, additional study is required.

CONCLUSIONS

The conclusions based on the findings of this investigation are summarized below.

General

1. The laboratory AVR design optimum asphalt contents ranged from 3.5 percent to 4.5 percent for Eagle Lake gravel mixtures and from 6.6 percent to 7.3 percent for Lubbock limestone mixtures. Total air voids were affected by both the soil-binder content and the asphalt content. With proper asphalt content, the minimum total air voids occurred at soil-binder contents between 7 and 10 percent for Eagle Lake gravel mixtures and at 10 percent soil-binder content for Lubbock limestone mixtures. The corresponding total air voids for Lubbock limestone mixtures with the laboratory AVR design optimum asphalt content were from 8.6 percent to 9.0 percent, which was much higher than those for Eagle Lake gravel mixtures (1.6 percent to 2.7 percent).
2. An optimum binder content for the maximum AVR density existed for the two materials. The Eagle Lake gravel mixture with 5 percent soil-binder content had the lowest optimum asphalt content (3.5 percent) and the highest density, and the Lubbock limestone mixture with 10 percent soil-binder content had the lowest optimum asphalt content (6.5 percent) and the highest density.
3. There was a tendency for the optimum asphalt content to decrease as the soil-binder content decreased; however, when the mixture contained little or no soil binder, the optimum asphalt content increased.

Static Characteristics

1. Both Eagle Lake gravel and Lubbock limestone mixtures exhibited essentially equal maximum static tensile strength; however, the asphalt content required for the Lubbock limestone mixtures (6.0 percent) was much higher than that for Eagle Lake gravel mixtures (3.0 percent).
2. Within the limits of this study, no definite relationship could be found between static modulus of elasticity and soil-binder content for Eagle Lake gravel mixtures; however, an optimum soil-binder content (10 percent) for maximum static modulus of elasticity existed for Lubbock limestone mixtures.

Repeated-Load Characteristics

1. A definite optimum binder content for maximum fatigue life existed for both the Eagle Lake gravel and the Lubbock limestone mixtures. At the same stress level of 100 kPa (14.5 psi), the fatigue life of Lubbock limestone mixtures was much higher than that of Eagle Lake gravel mixtures; i.e., the maximum fatigue life of Lubbock limestone mixtures at 5 percent binder content was about 110 times that of Eagle Lake gravel mixtures.
2. For resilient modulus of elasticity and static modulus of elasticity, no well-defined optimum soil-binder content existed for the Eagle Lake gravel mixture; however, an optimum soil-binder content (10 percent) was found for the Lubbock limestone mixture.

Moisture Damage

1. The moisture damage for the Lubbock limestone mixture was more severe than that for the Eagle Lake gravel mixture.
2. The moisture damage appeared to be directly related to the total air voids of the mixture; i.e., the damage resulting from water was greater for mixtures with higher total air voids.

Optimum Asphalt Content

1. Optimum asphalt contents were found to occur for the following material properties: (a) AVR density, (b) tensile strength, (c) static modulus of elasticity, and (d) fatigue life. No well-defined optimum occurred for the resilient modulus of elasticity.
2. The optimum asphalt content for mixtures with higher soil-binder content was generally larger than the optimum for mixtures with lower soil-binder content.
3. In general, the lowest optimum asphalt content occurred at 5 percent soil binder for the Eagle Lake gravel mixture and at 10 percent soil binder for the Lubbock limestone mixture.

Optimum Soil-Binder Content

1. For the Eagle Lake gravel mixture, the optimum soil-binder content was 5 percent for AVR density, tensile strength, fatigue life, and permanent deformation.
2. For the Lubbock limestone mixture, the optimum soil-binder content ranged from 5 to 10 percent for the various engineering properties.

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The contents of this paper reflect our views, and we are solely responsible for the facts and the ac-

curacy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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Permanent Deformation Characterization of Bituminous Mixtures for Predicting Pavement Rutting

J. BRENT RAUHUT

A general overview of rutting models available for predicting rutting in flexible pavements, the permanent deformation characterizations used by these rutting models or subsystems, and the general limitations of these characterizations is given. Discussed in detail are VESYS and DEVPAV. The most intensive development of a rutting model has been for the VESYS subsystem through support of the Federal Highway Administration. Since numerous researchers are involved in its further development and implementation, the published results of permanent deformation testing by a number of researchers have been transformed to the VESYS material characterization parameters ALPHA(1) and GNU(1). The relationships between these and other significant variables such as deviator stress, temperature, resilient modulus, and confining stress are reviewed and summarized. ALPHA(1) and GNU(1) were found to generally decrease with increasing deviator stress or temperature; the increase in ALPHA(1) dominated the effect on permanent strain predictions. ALPHA(1) increases with increasing confinement, and GNU(1) appears to decrease. Both parameters appear to increase as the material stiffness increases.

The advent of pavement models capable of making predictions of permanent deformations in pavement materials has created a need for characterizing materials in terms of permanent strains that result from repeated wheel loadings. A number of researchers have studied this problem both in the laboratory and through theoretical studies and comparisons with field measurements, and several procedures have been applied with varying degrees of success.

Virtually all models that offer predictions of rutting in pavements use elastic-layer theory to arrive at predictions of stresses or strains and then use these predictions and characterizations of the permanent deformation potential of pavement materials to arrive at predictions of rutting on a pavement surface. All of these procedures are limited (a) because the models are not as yet developed to the level of complexity imposed by the real physical system and (b) because of shortcomings in the characterizations of the permanent deformation potentials of the pavement materials. This does not imply that the characterizations are not useful but simply means that further development and improvements are needed to increase the confidence level in their predictions. The same is certainly true of fatigue-life predictions and, to a lesser extent, of most other outputs from analytical models used for pavement analysis and design.

The purposes for this paper are twofold. The first is general in nature and is intended to offer a brief overview of available pavement models developed for predicting rutting in flexible pavements and the characterizations used for permanent deformation potential of bituminous mixtures. The second is to update the state of knowledge of those characterization parameters used specifically by the VESYS rutting prediction subsystem. The former should be useful to researchers interested in flexible pavement models and the latter to the Federal Highway Administration of the U.S. Department of Transportation, state departments of transportation, and private and academic researchers working to improve and implement VESYS.

RUT-PREDICTION MODELS AND THEIR MATERIAL CHARACTERIZATIONS

A number of general approaches to the permanent deformation characterization of bituminous materials have been applied; they may be classified as follows:

1. Creep compliance characterization: These compression tests are generally run with one long-term static load but have also been run incrementally with specified load and unload durations (1) and with repetitive static loads. Characterization is in terms of a creep modulus as a function of load duration.
2. Repetitive-load test characterization: Many cycles of stress are applied for very short intervals (usually 100 000 or more cycles at 0.06- to 0.2-s load durations) in a factorial format to take into account the effects of stress state and sample temperature. Characterization is in terms of permanent strain as a function of cycles of loading.
3. Regression model characterization: Rutting is predicted on the basis of parameters such as stress and material stiffness. Permanent deformation potential is implicitly considered through its correlations with the significant parameters used.

The creep compliance test characterization is used by Shell in its design procedures (2) for predicting rutting, whereas multiple-regression model characterizations are used by Meyer and Haas (3,4) in their computer programs OPAC and WATMODE. Most other procedures used repetitive-load test characterizations, and the remainder of this paper will be limited to this type of characterization. However, discussions of the Shell procedure, OPAC, and WATMODE may be found in Rauhut, Roberts, and Kennedy (5,6).

VESYS RUTTING-PREDICTION SUBSYSTEM

The VESYS rutting-prediction subsystem uses a characterization derived from repetitive-loading compression tests to define fraction of predicted total strain that is permanent as a function of number of load cycles. The two parameters used are called ALPHA(1) and GNU(1); these parameters and their use are derived and explained in full elsewhere (7,8). They are also reviewed briefly below.

When permanent strain is plotted versus load cycles on log-log paper, a straight line may be drawn tangent to the curve to approximately predict permanent strain over the range of loadings of interest (usually to a portion of the curve that represents the highest range of loading numbers). ALPHA(1) is one less the slope of this line and thus represents primarily the rate at which permanent strain is increasing. GNU(1) is the product of the intercept value of permanent strain after the first load cycle and the slope divided by the average resilient strain; thus GNU(1) is the more complex parameter and is less prone to predictable behavior than is ALPHA(1). In general, increasing ALPHA(1) or decreasing GNU(1) individually results in reduced rutting predictions.

As with any slope-intercept representation of a function, a region on the graph may be reached by means of a variety of combinations of two parameters; therefore they may vary somewhat according to some relationship and still give reasonable results. In this case, neither variable is directly a slope or an intercept but, rather, are both functions of the slope and therefore somewhat interdependent.

Originally, one set of these parameters was used in VESYS IIM to characterize the bituminous material in the pavement structure for the entire year. An improved version called VESYS A (9) included a capability for separate seasonal characterizations, and a version called VESYS IIA added additional layers so that surface and bituminous base layers could be characterized separately.

At the present stage of development, selection of values of ALPHA(1) and GNU(1) requires previous knowledge of the expected stress state so that the effects of this important parameter on the characterization parameters may be considered. An approximation of the stress state may easily be obtained with elastic-layer solutions, but the effects of the stress state are difficult to apply both because of the limited data developed to date and because of the sensitivity of ALPHA(1) and GNU(1) to stress state and a number of material characteristics. I have previously attempted to assemble the results of all known permanent test results in terms of ALPHA(1) and GNU(1) (7,9) to alleviate this problem. More recent additional data will be added subsequently in this paper.

It should be kept clearly in mind that the permanent deformation potential of a bituminous mixture [characterized by ALPHA(1) and GNU(1)] varies daily and seasonally much as its dynamic modulus or stiffness does. As for stiffness, average values of ALPHA(1) and GNU(1) are used for discrete periods of time called "seasons" and are made consistent with average pavement temperatures in the bituminous layer. However, there is a further assumption or averaging effect for ALPHA(1) and GNU(1) that involves the average stress state throughout the depth of the bituminous layer. Whereas the material stiffness is not strongly stress dependent, the permanent deformation potential is very stress dependent and varies greatly from the top to the bottom of the pavement (7,10). To select average values of ALPHA(1) and GNU(1) in terms of stress state requires averaging across the upper zone of lateral compression and a lower zone of lateral tension.

While both repetitive-load triaxial and indirect tensile testing have been conducted to approximate the stress state in the tension zone of a bituminous layer, it is my opinion that such testing has not really as yet been successful. The reasons for this are that

1. No test has been devised that truly simulates the stress state in the lower or tensile zone of a bituminous layer and
2. Important lateral deformations that would contribute heavily to the vertical deformations called rutting in the bituminous layer are generally sufficient to mobilize substantial passive resistance to the lateral deformation; this phenomenon has not been accounted for in any test procedure I know of.

The shortcomings in the material characterizations are mentioned here simply because the VESYS model is being discussed first, not because they are unique to VESYS. They are just as applicable to the other models discussed subsequently since they all use essentially the same test technique for characterization.

My associates and I have attempted on two occasions to obtain generalized definitions of ALPHA(1) and GNU(1) through multiple regressions. The first attempt (7) used deviator stress, temperature, and percentage of asphalt content as independent variables. The resulting equations explained less than 50 percent of the variance of the dependent variables and were considered unacceptable for general application. A recent attempt that used a much broader data base met with little more success, although several likely equation forms were tested.

RUTTING SUBSYSTEM OF MONISMITH AND OTHERS

The design subsystem presented by Monismith and others (11) estimates the amount of permanent deformation or rutting that results from repeated traffic loading. Relationships between applied stress and permanent strain defined by repeated-load triaxial compression tests are used for fine-grained soils, granular materials, and asphalt concrete. Stresses resulting from the wheel loads are estimated through use of one of the ELSYM computer programs as a structural model. The stresses in turn permit estimation of permanent deformation in each layer of a specific pavement by

1. Computing the permanent strain at a number of points within the layer, the depth being sufficient to define the strain variations with depth, and
2. Estimating the deformation by summing the products of the average permanent strains and the corresponding differences in depths between the

locations at which the strains were determined; total rut depth is estimated by summing the contributions from each layer.

While this is a fairly straightforward method and indicates promise, the material characterizations used are quite complex and are dependent on a very detailed test program for specific materials used in order to arrive at the values of the many parameters considered. Few data are currently available for use with this procedure.

The consideration of discrete increments of depth in the bituminous layer and the characterization of materials in terms of stress state offer the opportunity to consider variations in stress state throughout the layer and, thus, the variation of permanent strain with stress. If properly characterized, this would allow the effects of lateral tension to come into play. In general, it is my opinion that this offers some theoretical advantage over the current generation of VESYS subsystems, but it would require a very large research commitment to sufficiently develop the approach and to broaden the material characterization data base for general use, even for research.

Similar procedures have also been developed and applied by Singh and Hamdani (10), Barksdale (12), and Huschek (13).

DEVPAV

DEVPAV is a finite-element program that has been under development by Kirwan and others (14) for some years in Ireland. In addition to the usual loading information, it uses permanent deformation characterizations for the various layers that were apparently developed by multiple regressions on repetitive-loading compression test results for each specific material considered. The following equation was given for a dense bituminous macadam tested at the University of Nottingham:

$$\epsilon_n\,(\%) = [0.0015(0.68 + 0.0008\,T^2\,\mathrm{Log}_{10}N)^{1.9}\nabla]^{1.75} \qquad (1)$$

This equation is in terms of number of applications N, the applied stress ∇ (kPa), and the temperature T. This is an interesting development because this equation is in the form of a constant times stress to a power in lieu of the constant times number of applications to a power used in the VESYS system. This characterization, like those used by Monismith and others (11), appears to be more complete than those for VESYS.

PERMANENT DEFORMATION MATERIALS CHARACTERIZATIONS

The use of stress state and pavement temperature directly in the material characterization, as reported by several researchers and discussed above, offers a rather direct application of these important parameters to rutting predictions. This is, in effect, the object of studies by me and by others to define the relationships of the VESYS parameters ALPHA(1) and GNU(1) to stress state and material temperature. If these relationships are established, ALPHA(1) and GNU(1) may be easily expanded to input arrays as functions of distress and temperature and their values selected internal to the computations. This would require modifications to the computer programs to generate ALPHA(1) and GNU(1) rather than simply obtain them directly from input arrays.

Recent Permanent Deformation Test Results

I have for some years transformed all repetitive-load permanent deformation test data available into the ALPHA(1)-GNU(1) characterization in order to gain as much insight as possible about relationships between these material variables and other significant variables, especially deviator stress and material temperature (7,9). Since ALPHA(1) and GNU(1) are not independent of each other, their relationship to each other has also been studied and reported (7). My intent here is to expand the published data to include results of more recent testing and to add relationships between ALPHA(1) and GNU(1) and resilient modulus.

The primary sources of new information on permanent deformations in bituminous mixtures are the following:

1. A factorial of 42 compression tests on cylindrical specimens run by Barksdale (15,16) on base mixtures used in Georgia [these samples were compacted by the 50-blow Marshall procedure and included variations in air content (4.5-6.0 percent), asphalt content (4.2-5.5 percent), deviator stress, lateral stress, and temperature; the load duration was 0.06 s with a loading frequency of 45 cycles/min],
2. A factorial of compression tests for a particular sand-asphalt mixture reported by Singh and Hamdani (10) [this mixture had an asphalt content of 7 percent and was tested with a loading frequency of 60 cycles/min; this factorial included three levels of deviator stress and three levels of temperature, and the lateral stress was held constant at 138 kPa (20 psi)], and
3. Tests on a dense bituminous macadam reported by Brown and Bell (17) (these test results are for a single temperature and a mixture with 4.7 percent asphalt content and 5.7 percent voids; these results are reported to show the effects of changing lateral pressure with deviator stress and temperature constant).

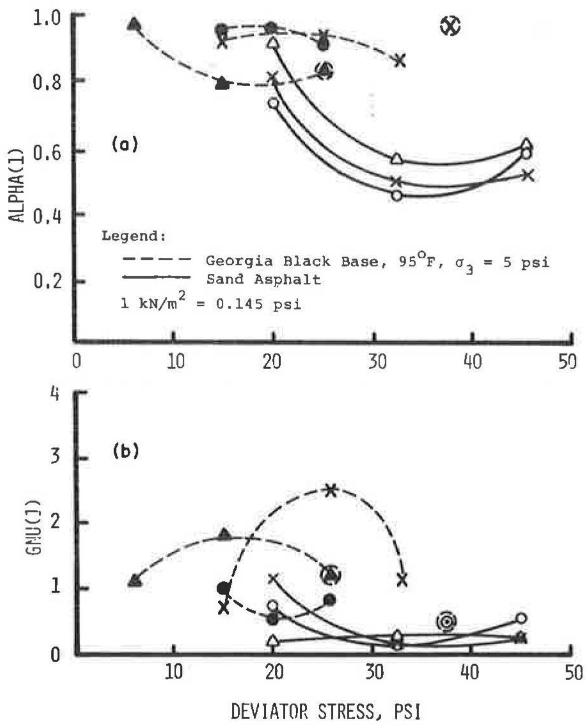
Although reported previously with deviator stress and temperature as independent variables (9), the results of tests by Rauhut that have resilient modulus as the independent variable are also discussed briefly again. Each set of test results is discussed separately below and then summarized in combination. The general trends or relationships developed through previous studies and reported in Rauhut (9) are also listed below:

1. GNU(1) and ALPHA(1) both decrease as stress increases. The decrease in ALPHA(1) generally dominates to cause an important increase in permanent deformation as stress increases.
2. It appears that the mechanisms that affect permanent deformation differ above and below some temperature on the order of 15.6°C (60°F) to 21.1°C (70°F). Below 15.6°C, (a) GNU(1) increases with temperature and (b) ALPHA(1) increases slightly with temperature for the higher stress levels but decreases at the lower stress level. Between 15.6 and 21.1°C, (a) GNU(1) decreases with increasing temperature and (b) ALPHA(1) decreases with increasing temperature.

Georgia Base Mixture

ALPHA(1) and GNU(1) are first displayed as functions of deviator stress in Figure 1 for Georgia black base (15) and sand asphalt (10). The general decrease in ALPHA(1) as deviator stress increases (noted in previous studies) is indicated in Figure 1a, although there appears to be an increasing trend at the lower deviator stresses. And although the results for GNU(1) are somewhat scattered, it appears to increase at lower deviator stresses and to decrease at higher ones (see Figure 1b). Most other test data indicate a decrease in GNU(1) as deviator stress increases over the full range of deviator stress.

Figure 1. Plots of ALPHA(1) and GNU(1) versus deviator stress for Georgia black base and sand asphalt.



Three of the four curves in Figure 2a for temperature approximate the trends from previous studies that showed ALPHA(1) decreasing as temperature increased, but the fourth, at a higher lateral pressure, increases as temperature increases. It is not clear why the increased lateral pressure would have this effect, but it appears likely that the higher lateral pressure may have caused more vertical rebound (and thus less measured vertical permanent strain) as the mix became less stiff at higher temperatures.

The curves in Figure 2b indicate that two of the test series displayed the anticipated downward trends in GNU(1) as temperature increases, whereas the other two increased as temperature increases. The increase in GNU(1) with temperature for the mix tested at 207-kPa (30-psi) lateral pressure is consistent with the increase in ALPHA(1) just discussed for this same test series.

To summarize, Barksdale's test results (15) displayed, in general, the same trends as other previous test results, but some of the test series indicate different trends. These differing trends may indicate a factor or factors not accounted for or yet explained, or they may be test related.

Figure 3 provides graphs of these same test results for resilient modulus with the addition of data for Texas surface mixture (9). Although there is considerable scatter, it appears that ALPHA(1) increases rapidly as mixture stiffness increases in the range of very low mixture stiffness below about 690 000 kPa (100 000 psi) and then decreases at a slow rate as further increases in stiffness occur. GNU(1) appears to generally increase as stiffness increases.

Sand Asphalt for Desert Use

The sand asphalt data in Figure 1 also indicate general decreases in ALPHA(1) and GNU(1) with increasing deviator stress. However, the trends

Figure 2. Plots of ALPHA(1) and GNU(1) versus temperature for Georgia black base and sand asphalt.

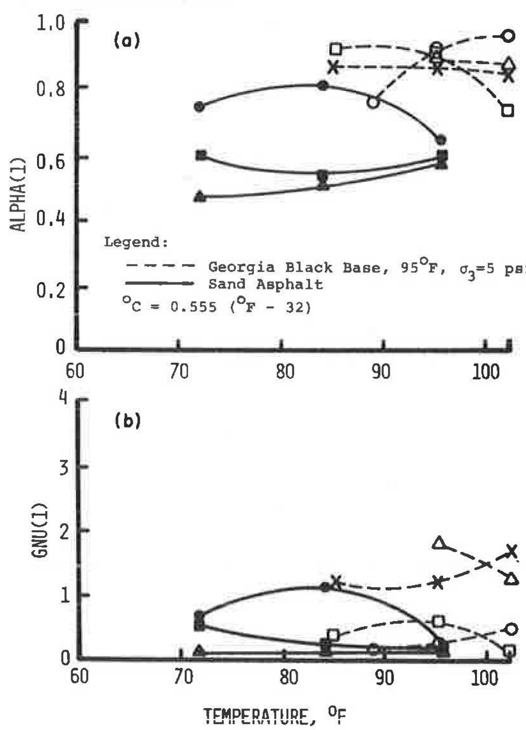
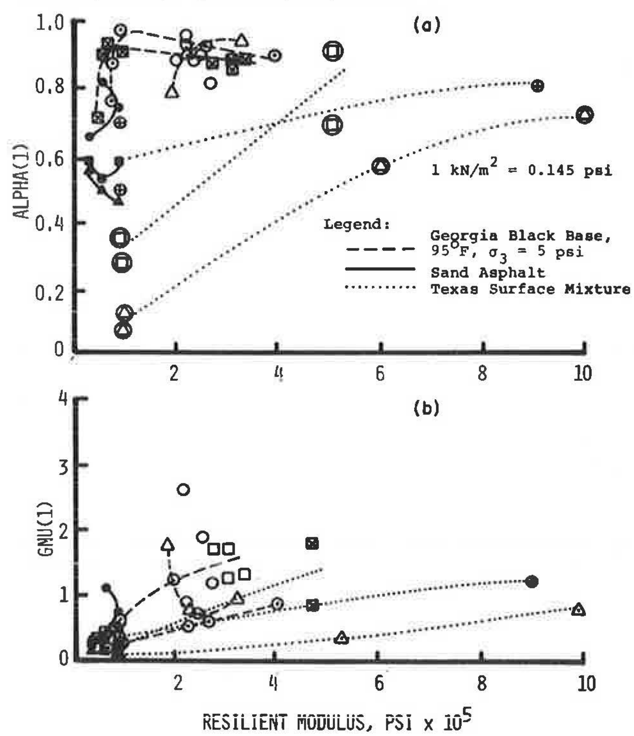


Figure 3. Plots of ALPHA(1) and GNU(1) versus resilient modulus for Georgia black base, sand asphalt, and Texas surface mixture.



with temperature for this material (Figure 2) were less clear. The increases in permanent deformation with temperature across this range of 25-40°C (72-96°F) were not dramatic, so it is hypothesized that the effects of interactions between ALPHA(1) and GNU(1)--to decrease ALPHA(1) and to increase GNU(1) individually results in increased permanent strain--are not readily discernible in the graphs.

The graphs for sand asphalt in Figure 3 of ALPHA(1) and GNU(1) versus resilient modulus do not show any discernible trends. It should be noted that the resilient modulus was quite low at all temperatures, ranging from 572 000 kPa (83 000 psi) at 25°C (72°F) to 173 000 kPa (25 000 psi) at 40°C (96°F).

Dense Bituminous Macadam

Figure 4 indicates the effect of increasing both confining and vertical pressures to maintain a constant deviator stress and an increasing lateral stress [temperature = 30°C (86°F), asphalt = 4.7 percent, voids = 5.7 percent (17)]. As might be expected, the permanent strain was considerably decreased by the increase in lateral resistance, and ALPHA(1) reflected this as an increasing trend. GNU(1) decreased with increasing confinement for the lower deviator stress but increased for the higher deviator stress.

It appears, based on these limited data, that the dominant effect of lateral confinement is reflected in changes in ALPHA(1); trends for GNU(1) are more erratic.

Texas Surface Mixture

Graphs of trends for variations in ALPHA(1) and GNU(1) with deviator stress and temperature are given in Appendix A of Rauhut and Jordahl (9), and

Figure 4. Plots of ALPHA(1) and GNU(1) versus lateral stress for dense bituminous macadam.

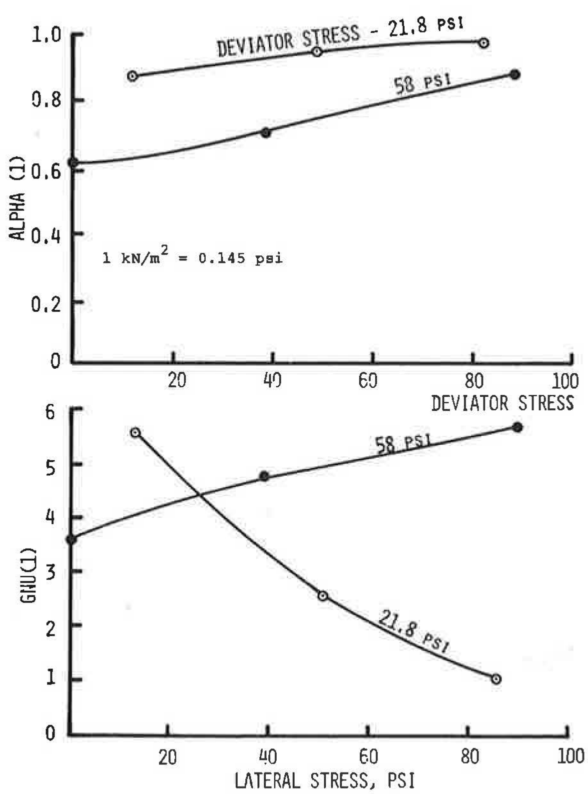
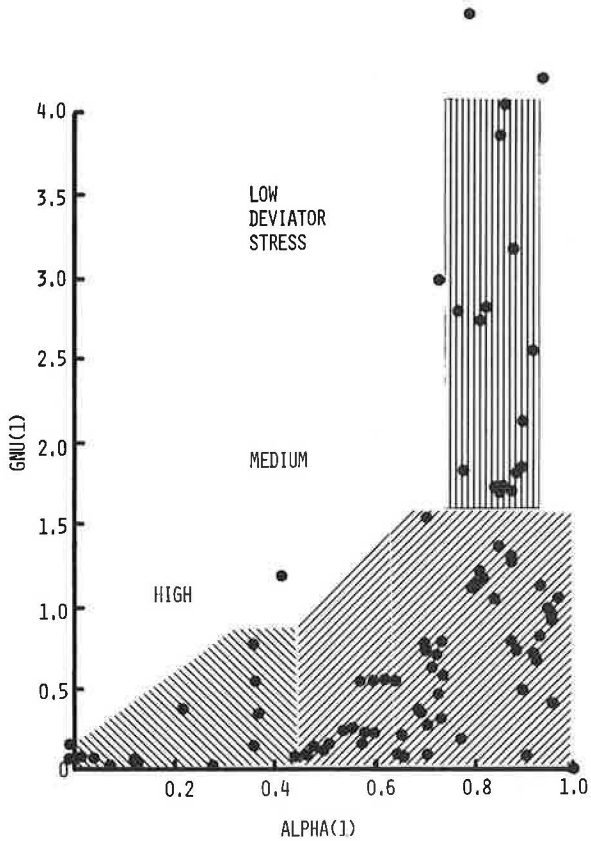


Figure 5. Relation between ALPHA(1) and GNU(1).



the summary of results of that study are given above. However, Figure 3 shows these same test results (dotted lines) plotted as a function of resilient modulus. For these tests and this surface material, ALPHA(1) and GNU(1) both generally increase as the resilient modulus increases.

Relationship Between ALPHA(1) and GNU(1)

The relationship between these two parameters has been studied in the past, and a plot from Rauhut and others (7) has been augmented to include results from testing reported above (see Figure 5). Virtually all of the new data fit the region occupied by previous test results. It appears that GNU(1) is generally in the range of 1.5 or less but may increase to higher values associated with relatively high values of ALPHA(1). Since rutting generally occurs during periods of higher pavement temperature [when ALPHA(1) would generally have relatively lower values], it appears probable that values of GNU(1) greater than 1.5 will not have much significance.

SUMMARY

The principal pavement models currently in use or under development for predicting rutting in flexible pavements have been discussed briefly. Also discussed have been some of their limitations, their materials characterizations, and the parameters on which their values depend.

The results of recent permanent deformation testing were reported, and the relationships between the VESYS material characterization parameters ALPHA(1) and GNU(1) and deviator stress, temperature, resilient modulus, and confining stress were explored. The insights gained from past studies and from this more recent testing have been combined and are as follows.

1. ALPHA(l) and GNU(l) generally decrease as deviator stress or temperature increases; the decrease in ALPHA(l) dominates to result correctly in more predicted permanent strain.
2. ALPHA(l) increases as confinement increases to result in less predicted permanent strain. GNU(l) is also believed to decrease as confinement increases, but there are not sufficient data to strongly support this hypothesis.
3. Both ALPHA(l) and GNU(l) appear from the limited data to increase as material stiffness increases (predicted total strain would decrease), but ALPHA(l) may decrease for some mixes.

Some of the relationships between the material characterization parameters and other significant variables appear fairly clear; however, others are not as clear. This is believed to be partly the result of the sensitivity of these parameters to numerous material factors and properties, to stress state, and to test conditions. However, the interdependence and nature of these two parameters also affect the clarity of the trends.

The trends noted above for ALPHA(l) are consistent with the well-established increases in permanent strain and rutting in pavements that result from increased pavement temperature or from increased stresses caused by heavier axle loads. The trends for GNU(l) are not always consistent, but the net effect on rutting predictions is generally dominated by ALPHA(l).

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Modifier Influence in the Characterization of Hot-Mix Recycled Material

SAMUEL H. CARPENTER AND JOHN R. WOLOSICK

The hot-mix recycling operation for bituminous mixes commonly uses a modifier to restore the aged asphalt cement to a condition that resembles a virgin asphalt cement. In the laboratory this is done on the extracted asphalt cement, thus assuring a thorough mixing. In the actual recycling operation, however, the modifier is added to the material during the mixing process and merely coats the salvaged asphalt concrete particles that are being recycled. It takes a certain amount of time for the modifier to combine with the old asphalt, but the exact nature of this time period and its influence on the structural behavior of the mix has not been reported. This paper investigates the influence of this diffusion process on material behavior and shows that the diffusion process exerts a large influence on the material properties required for long-term performance predictions in computer models such as VESYS. Immediately after sample preparation, the mix may have high stiffness and excellent resistance to rutting. For the material investigated in this paper, a week after preparation the stiffness had decreased by a factor of two, and the resistance to rutting had decreased accordingly. This behavior is explained from a conceptual consideration of the diffusion process and is physically verified by extracting the outer and inner layers of the modifier-asphalt cement combination prepared in a simulated recycling operation and by comparing their consistency. Consideration of this phenomenon has serious implications in characterization for long-term performance predictions and in evaluating the effects of various laboratory conditioning procedures.

Recycling operations have experienced a rapid growth in recent years. This growth has resulted from increased awareness of the potential for cost savings and material conservation. More importantly, the effort put forth by the equipment manufacturers has increased. The recent years have seen rapid advances in pulverizers, millers, and hot-mix plants that facilitate recycling operations. With the increase in recycling operations has come an increased awareness that the recycled material must be characterized in order to ensure a quality pavement. To accomplish this the Federal Highway Administration has funded (a) research to investigate the effect of rejuvenators on recycled binders (Texas A&M University), (b) a test procedure for examining the efficiency of the mixing process (University of Washington), (c) the design and characterization of recycled mixtures (University of Texas), (d) a study to compare the performance of recycled shoulders versus all-new construction (University of Illinois), and (e) a study to characterize and predict performance of recycled pavements (Resource International). These studies will help indicate whether the long-term performance of recycled pavements will be comparable to the performance of new construction. If the recycled pavements show excessive deterioration, the cost and energy savings obtained during construction may be lost through excessive maintenance. Initial indications are that a quality pavement is being constructed. However, these pavements have not been in service long enough to permit judgment of their long-term performance.

The present study undertaken at the University of Illinois--Design Properties of Recycled Shoulders: Performance Characteristics--has as its objective the one-on-one comparison of recycled and new construction. The economical method to generate these data involves the use of computer models for performance predictions and extensive characterization in the laboratory. The main computer model selected for these comparisons is VESYS-G (1). This model predicts fatigue cracking, rutting, roughness, and the serviceability index with time. Traffic loads, temperature, moisture conditions, and material property descriptions can be varied over the design life.

To obtain accurate comparisons with the VESYS program, extensive laboratory characterization is necessary. The necessary tests include

1. Fatigue: controlled stress, beam flexure;
2. Creep compliance: cylindrical compression;
3. Permanent deformation: cylindrical compression; and
4. Resilient modulus: indirect tensile, diametral loading.

Another damage mechanism not included in the VESYS analysis is thermal fatigue, which requires (a) creep compliance (tensile loading) and (b) thermal coefficient of contraction. Thermal fatigue is analyzed by a modified viscoelastic procedure (2).

REQUIREMENTS FOR PERFORMANCE COMPARISONS

To examine the performance of two asphalt-concrete materials in order to evaluate the influence of the recycling operation, the materials must be as similar as possible. The items that must receive consideration include (a) density and air voids, (b) gradation, (c) type of aggregate, (d) fluids content, and (e) viscosity of fluids (or penetration).

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

In order to compare laboratory performance predictions for new construction and recycled materials, the voids, gradation, and aggregate type must be similar. These parameters, which represent variables that can be altered, can have a large influence on test results; they also indicate quality control in the recycling process. Fluids content and consistency must also be the same, for these two values represent the quantities that are most critically influenced by the recycling operation. The consistency of the recycled asphalt cement should be similar to that used in the samples that represent new construction and to the virgin asphalt cement added during the recycling operation in order to emphasize the influence of the recycling operation.

The procedures used for determining the amount of modifier and for mixing during the recycling operation represent the focal point for the most serious problem in characterizing a recycled material. This problem reduces to one question, How long after sample preparation should the specimens be allowed to cure before the data will be representative of the pavement over its design life? The necessity for this question arises from the fact that the modifier does not instantaneously combine with the old asphalt cement that coats the salvaged material. A diffusion process takes place over a period of time during which this combination occurs.

THE DIFFUSION PHENOMENON

In order to determine the amount of modifier required, the asphalt is extracted from the salvaged material and thoroughly fluxed with various percentages of the modifier. The classification properties are then measured on this rejuvenated asphalt cement. These test results are used to select the amount of modifier needed to match classification data of a virgin asphalt cement (penetration at 25°C or viscosity at 60°C).

In a typical hot-mix recycling project, the modifier is added during the mixing process along with the virgin asphalt cement and new aggregate. The actual addition process varies with the recycling technique being used, but the general process involves addition of modifier during mixing. During this procedure the modifier is attracted to the virgin asphalt cement and to the old asphalt cement surrounding the aggregate. Both materials will begin to soften at different rates because of the different affinities exerted on the modifier by the virgin asphalt cement and the old oxidized asphalt cement.

This presents a very complicated picture indeed. To simplify the discussion, the remainder of this paper will be concerned with a mixture containing 100 percent recycled material (no virgin materials added). The process that begins when the modifier is added in the recycling operation can be outlined as follows:

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

These phases and time steps are depicted schematically in a plot that illustrates the variation in viscosity with time (Figure 1). This penetration is a diffusion of the modifier or select components of the modifier into the aged asphalt.

The gradual softening, or rejuvenating, of the asphalt in a mixture once it has been compacted is a major problem in the characterization of a recycled material. The time between mixing and testing of the samples may be critical because of the softening effect. The structural parameters may or may not be sufficiently developed to provide any resistance to the wheel loads during the initial portion of the recycled pavement's life, when it will be softening because of the diffusion process. Thus, it is necessary to characterize the material by its long-term parameters as well as its short-term parameters. In order to examine the importance of this diffusion process in material characterization and in performance predictions, a testing program was set up to examine the property changes following sample preparation.

SAMPLE PREPARATION

The material used in this study was removed from a city street in Champaign, Illinois, in June 1976 by using a Roto-Mill made by the CMI Company of Oklahoma City. The material had been in service for 12-15 years and had developed a rough ride surface (as a result of transverse cracking), a low skid resistance, and a poor drainage profile. The surface, a hot-mix asphaltic concrete, was milled to a 1.9-cm depth, left open to traffic for approximately four months, and overlaid with a 2.5-cm overlay. The material removed from the roadway was stockpiled by the city of Champaign and used as a cold-weather patching material.

The material removed from the roadway was less than 2.5 cm in size because of the abrasive action of the Roto-Mill. The oversized material was crushed to pass the 1.25-cm sieve. Approximately 6000 g of asphalt cement were extracted and recovered from the salvaged material. The aggregate and fines were saved for later use in preparation of the rejuvenated samples. The properties of the asphalt cement were

Item	Value
Viscosity at 60°C	4490 Pa•s
Penetration at 25°C	2.6 mm
Penetration at 4°C	2.2 mm
Softening point	63°C
Asphalt content	5.3 % (total weight basis)
Specific gravity	1.198

The salvaged pavement aggregate had a specific gravity of 2.71 (bulk). The aggregate gradation (wet sieve on recovered samples), which very closely followed an Illinois CA-12 specification, was as follows:

Sieve Size	Percent Passing	Sieve Size	Percent Passing
12.5 mm	100	850 μm	34
9.5 mm	81	425 μm	23
6.3 mm	78	150 μm	13
4.75 mm	68	75 μm	9
2.00 mm	58		

In order to study the influence of the modifier alone, a standard modifier (Paxole 1009) was chosen. Various percentages of the modifier were mixed with the reclaimed asphalt cement. The results of viscosity testing are presented in Figure 2 as a plot of viscosity at 60°C versus the percentage of the modifier. A 20 percent modifier by weight of asphalt cement was selected to produce a viscosity of 100 Pa•s at 60°C, corresponding to an AR-1000. The properties of this rejuvenated combination are given below.

Item	Value
Viscosity at 60°C	99.4 Pa•s
Penetration at 25°C	11.2 mm
Penetration at 4°C	3.7 mm
Softening point	44°C
Specific gravity	1.164

The amount of modifier (Paxole 1009) in the rejuvenated asphalt cement was 20 percent (by weight of asphalt); its viscosity at 60°C was 234 mm²/s, and its specific gravity was 1.028. The properties are not dissimilar to those of a virgin asphalt cement. In the mixing process used for laboratory preparation, this asphalt-modifier combination hardened to an approximate viscosity of 240 Pa•s at 60°C.

Rejuvenated Samples

The 6000-g quantity of extracted asphalt cement was

Figure 1. Schematic of modifier coating an aggregate particle during recycling to illustrate viscosity variation.

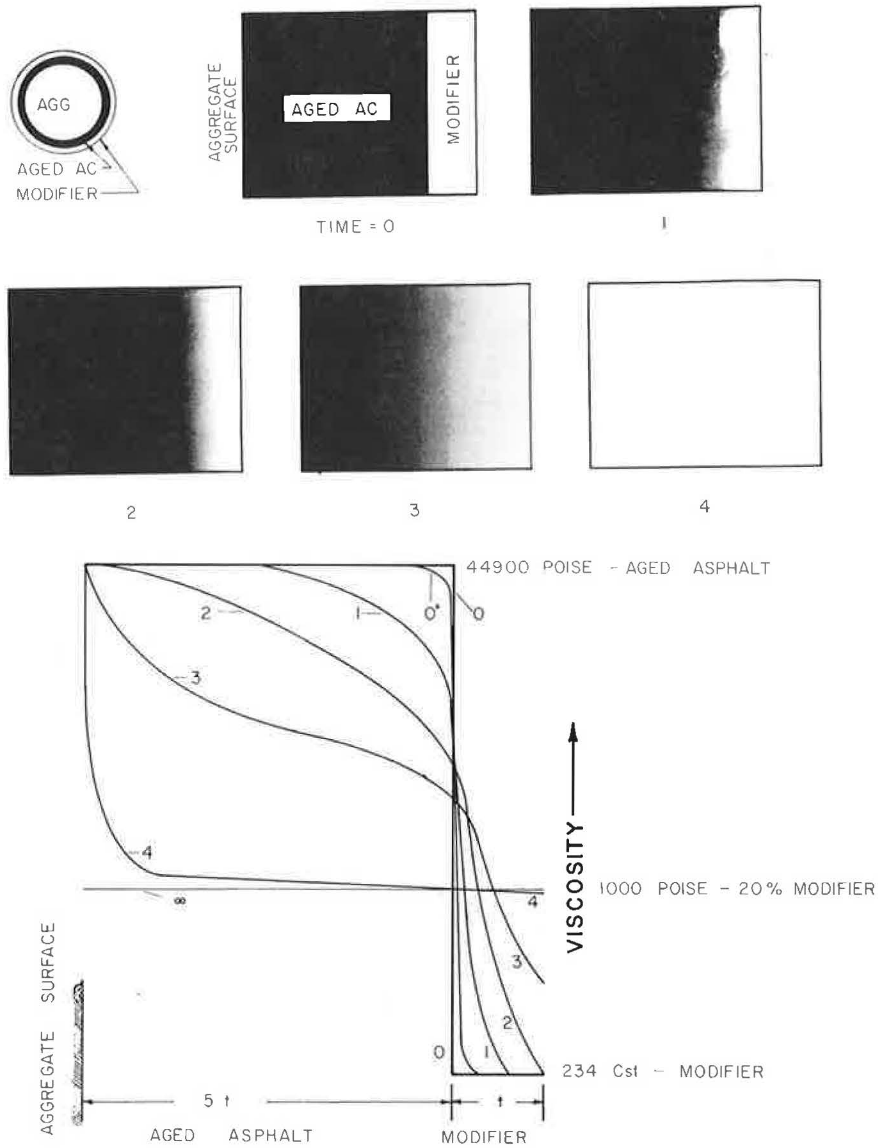
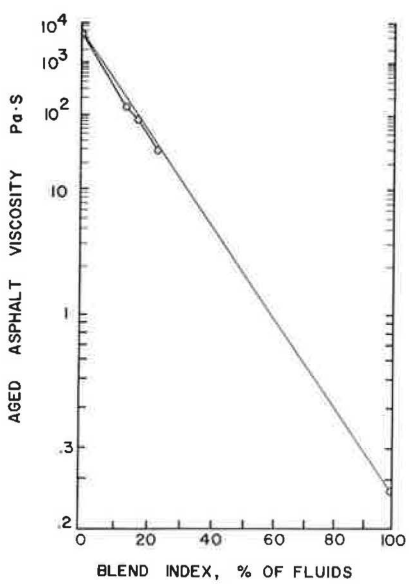


Figure 2. Viscosity of rejuvenated asphalt cement as a function of percentage of modifier.



fluxed with 20 percent modifier. This rejuvenated combination was then used as a normal asphalt cement. It was added to the recovered aggregate from which it was originally removed, and six Marshall samples were formed by using 75 blows per side. Six cylindrical samples (5.08 x 10.16 cm) were also formed by using double-ended static compaction. The analysis of the rejuvenated samples, including bulk specific gravity of mixture (Gmb) and coefficient of variation (CV), is given below.

Property	Marshall Samples	Cylindrical Samples
Gmb	2.414	2.401
σ	0.0091	0.0094
CV (%)	0.38	0.39
Air voids (%)	3.4	3.9

The fluids content represents the original asphalt cement, 5.3 percent by weight of aggregate, plus the 20 percent modifier. These samples represent a rejuvenated material with which samples formed by the recycling process may be compared. All subsequent samples were constructed by using double-ended static compaction to match the density of these samples as closely as possible.

Recycled Samples

The salvaged material was heated to 116°C, and 11.2 g of the modifier were mixed with 1056 g of the salvaged material. These percentages match those used in the rejuvenated samples. The recycled material was then compacted to a predetermined height to produce the desired density for 16 Marshall samples. The same procedure was followed in preparing four cylindrical samples. The resultant analyses are shown below.

Property	Marshall Samples	Cylindrical Samples
Gmb	2.368	2.343
σ	0.024	0.014
CV (%)	1.03	0.060
Air voids (%)	5.3	6.3

The samples were tested at various times after preparation to illustrate the influence produced by the diffusion process of the modifier into the asphalt cement in the recycled samples.

TEST RESULTS

The complete test sequence necessary for a VESYS characterization has been detailed earlier. We did not feel that the entire series of tests was necessary for studying the effects of diffusion. Resilient modulus in diametral loading on Marshall samples, creep compliance and permanent deformation tests on cylindrical samples, and Marshall stability were deemed sufficient, since they are all influenced to a large degree by the type and amount of asphalt cement present.

Resilient Modulus

The resilient modulus values for the rejuvenated samples showed no variation with time after compaction, as expected. The relatively low value of 358 540 kPa does not indicate a poor-quality material but, rather, reflects the influence of a low-viscosity material tested at a relatively high temperature (25-27°C). The fluids content of 6.3 percent by total weight is also above what normally would be used and would be reduced by adding new aggregate.

The resilient modulus values for the recycled samples showed a rather dramatic variation with time (Figure 3). The day following preparation the modulus is high. With time it decreases and finally increases again. There is a critical period when the resilient modulus is at a low value.

Creep Compliance Results

The creep compliance tests showed essentially no variation with time (Figure 4). The recycled samples show the same value for creep compliance as do the rejuvenated samples. The lack of variation with time in the compliance curves indicates that diffusion, if it is occurring, has a minimal influence on creep compliance. The explanation of this phenomenon is given below in the discussion of the implications of the test data collected.

Permanent Deformation Results

The permanent deformation data are developed from the incremental-static procedure outlined in the

Figure 3. Variation in resilient modulus as a function of time.

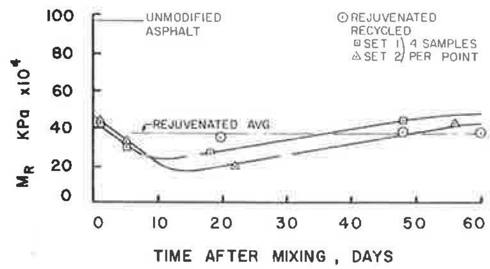


Figure 4. Creep compliance data for rejuvenated and recycled samples.

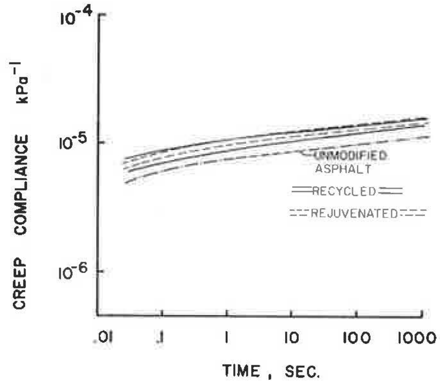
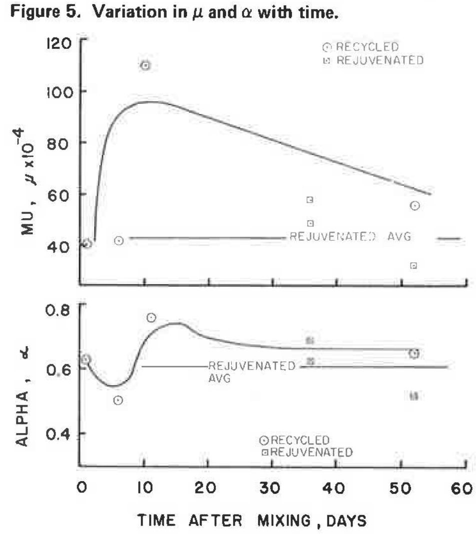


Figure 5. Variation in μ and α with time.



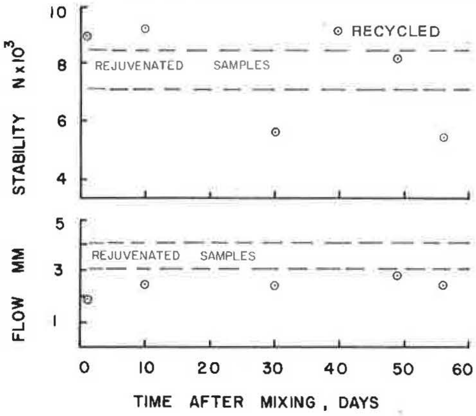
VESYS User's Manual (3). Although they are not based on as many samples as the resilient modulus data, a trend may be inferred from the time variation of μ and α, the values used in the rutting subsystem of VESYS (Figure 5). The rejuvenated sample values again show little variation with time. The recycled samples again show a softening followed by a hardening.

Briefly, permanent deformation is characterized by the following equation in the VESYS program:

$$F = \mu N^{-\alpha} \quad (1)$$

where

Figure 6. Variation in Marshall stability and flow with time.



F = fraction of total deformation that becomes permanent deformation,
N = number of wheel loads, and
μ,α = coefficients determined from test data.

In general, as μ increases and α decreases, the accumulation of permanent deformation increases. The data indicate that increased permanent deformation may be accumulated over the initial short-term portion of the pavement's life. The interpretation of these data will assist in explaining the influence of the proposed diffusion model on material characterization.

Marshall Test Results

The Marshall stability and flow values for the rejuvenated and recycled samples are presented in Figure 6. Test values for the rejuvenated samples fell within the range indicated by dashed lines on Figure 6. Little can be inferred from these data because of the inherent insensitivity of the Marshall test. It is important to note, however, that the stability and flow values are acceptable, even though no attempt was made to obtain a suitable mix from the Marshall standpoint. It is unlikely this mix would perform satisfactorily as a normal pavement mix, however, because of the other performance parameters.

INTERPRETATION OF TEST RESULTS

The test data support a softening effect caused by the diffusion of the modifier into an old asphalt cement. This is most apparent in the resilient modulus test, which is measured entirely in tension. The creep compliance, a compressive test, showed no interpretable variation with time, whereas the permanent deformation characteristics, which rely on a rebound to place the asphalt films in tension, showed variation similar to that of the resilient modulus. The following discussion explains the test data in terms of the diffusion process discussed earlier.

1. The modifier is mixed with the salvaged material (as shown in Figure 7a) and compacted to a predetermined density.
2. Compaction forces the old asphalt-coated aggregate into intimate contact, high viscosity in contact with high viscosity and even aggregate in contact with aggregate, as shown in Figure 7b.
3. The modifier begins diffusing into the old asphalt cement. This produces an enlarging layer of lower-viscosity material. The asphalt in the recessed contact areas begins to soften only slightly because of the distances the modifier must penetrate. This is illustrated in Figure 7c.
4. As the modifier continues to diffuse, it reaches a balance point at which all the raw modifier is gone and the driving potential for diffusion is reduced. At this point, the outer layer is at its softest consistency, and stiffness measurements may be at their lowest values.
5. With the decrease in the driving force, the diffusion process slows down appreciably. An imbalance of viscosities still exists between the inner and outer layers. At this time the viscosity of the inner layer begins to decrease and the outer layer's viscosity begins to increase as a result of the continuing diffusion. Stiffness measurements begin to show an increase at this point.
6. The long-term equilibrium point might be similar to that depicted in Figure 7d. The viscosity will have stabilized, slightly lower on the surface and slightly higher at the aggregate interface compared with the totally rejuvenated value determined on extracted asphalt. Because of distance effects, the modifier could be expected to take a very long time to penetrate to the contact areas. Thus, stiffness measurements may be slightly higher for a recycled sample than for a rejuvenated sample.

This explanation of the proposed diffusion model accurately models the test data measured on recycled samples. At stage 2, when stiff asphalt is in contact with stiff asphalt, the recycled samples are stiffer than the rejuvenated samples are. At stage 3, a larger volume of the asphalt cement has been softened and, when put in tension, it deforms more than at stage 2. In compression, however, old asphalt cement is still being forced into old asphalt cement, and compression testing produces nearly the same results that it did in the previous stages. However, there is less old asphalt that resists deformation under the load, and when the load is released there will be less rebound and more permanent deformation. At stage 5, the outer layer begins to increase its viscosity and the sample will stiffen, which was observed in all tests except the creep compliance test. Some softening in the creep compliance may occur, but this trend may be masked by data scatter. At stage 6, the "long-term equilibrium point," the average viscosity may be similar to that of the rejuvenated material. However, because the viscosity of the inner recesses will still be higher, the recycled data should be stiffer. After a sufficiently long time, the recycled data should asymptotically approach the rejuvenated values.

PHYSICAL VALIDATION

Although the test data substantiate the proposed diffusion model quite well, a physical verification was attempted. This process involved mixing a recycled sample with the modifier in the same manner that was used to prepare the recycled samples. The proper amount of modifier (11.2 g) was added to 1056 g of the salvaged material. The sample was mixed at 116°C and allowed to cure, uncompacted, for a specified time. Three samples were used for each determination.

At predetermined time intervals, an incremental extraction process was initiated. The recycled mix was immersed in trichloroethelyne and left to sit for 3 min, and the solution was decanted. The solution for each of three samples was combined, and the asphalt cement was recovered by using the Abson method. The remaining mixes were washed with the

Figure 7. Schematic of diffusion in a compacted sample.

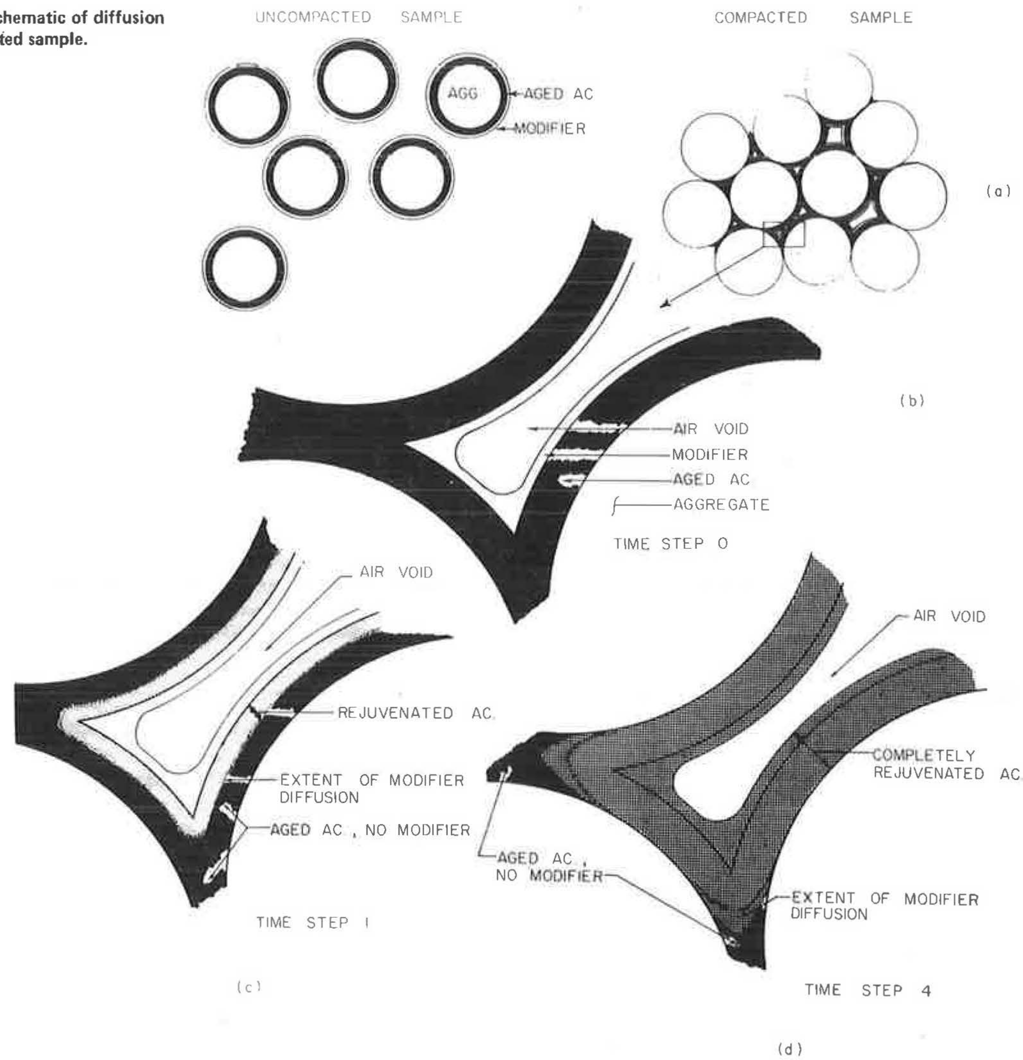
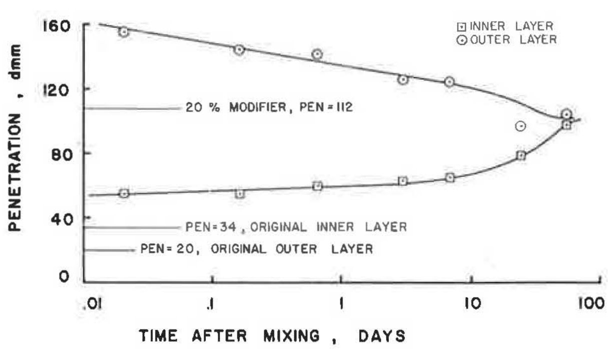


Figure 8. Penetration of the outer and inner layers as a function of time.



solvent to remove all of the remaining asphalt cement. The asphalt cement in these washings was also recovered. This procedure provided two samples of asphalt cement: The first sample represented the outer layer, and the second represented the inner layer. The consistency of each layer should vary with time if the diffusion process was acting as hypothesized. This procedure was used by Zearley of Iowa to determine whether a soft asphalt cement would penetrate an aged asphalt cement after two years of service (4).

The data from these extractions clearly indicate that the outer and inner layers are not of the same consistency for an appreciable time following mixing. The penetration at 25°C for each layer is plotted as a function of time after mixing (Figure 8). The data clearly show the outer and inner layers to be approaching the same consistency, indicated by the penetration value of the rejuvenated sample. The long-term consistency is stiffer than that of the rejuvenated asphalt cement and indicates the hardening produced in the laboratory mixing procedure.

DISCUSSION

The data developed in this study pose some serious problems for anyone attempting to characterize a recycled bituminous mix that contains a modifier. The recognition and acceptance of the diffusion process that occurs over an extended time period compared with normal time frames for material testing imply that the handling of the samples as well as the time between mixing and testing is critical. Different modifiers in different concentrations may diffuse at different rates. The composition of the old asphalt cement could also affect the diffusion rate of the modifier.

Studies that have cured a recycled mixture for a given period in a specified manner (5) are open to skepticism about the results and how well such results indicate the performance of that material over the long term. A process of artificial aging by placing the sample in an oven for a specified time is meaningless if the process is undertaken before diffusion is complete: The material undergoing aging will be the modifier, not the combination of modifier and old asphalt cement. It is this combination that will age over the extended life of the actual pavement, so it is this combination that must be aged in the laboratory. The same argument holds for accelerated moisture and freeze-thaw conditioning. To be indicative of long-term performance of the mixture, the conditioning must not begin until the diffusion process is completed.

Therefore, studies that have sampled recycled paving mixtures at some time after placement and compared the results of the characterization tests with similar materials compacted in the laboratory may not be comparing similar sets of data (6). Even though the results may indicate an acceptable mix, it cannot be stated that these results might not have changed appreciably if the samples had been tested several days before or after the date of their actual testing. Further, if samples are used in tests that require an extended period of time for completion (such as fatigue testing), the results cannot be interpreted in a meaningful manner. For one thing, the tests on each sample will run for different lengths of time, depending on physical test inputs such as load or strain magnitude. It is also unlikely that each sample test will begin at the same time after sample preparation.

The main point is that by no stretch of the imagination can data taken on a sample that is in the transition phase of the diffusion process be used for long-term performance predictions. During this transition phase, the material is highly bimodular because of the viscosity variation with thickness. The compression modulus will be high, and the tensile modulus will be low. The fact that previous testing on recycled mixes has indicated good mixes, even though some of the data might have been collected during the transition phase, is testament to the fact that a well-planned recycling operation can produce an excellent mix.

In an actual recycling operation that employs any of the commonly used recycling equipment, the problem of time dependency may be accentuated even further. In the actual recycling operation, virgin asphalt cement is added at the same time as the new aggregate and the modifier. The modifier will then diffuse at different rates into each asphalt, since their diffusivity coefficients will be different. This could remove modifier from the old asphalt and excessively soften the virgin asphalt. It would take much longer for this system to diffuse to a uniform-viscosity asphalt cement than it would take the simple asphalt-modifier system investigated in this paper. The considerations are exactly the same, however, and the material must be characterized after the diffusion process is complete in order for long-term performance predictions to be meaningful.

The structural properties of the pavement during the diffusion process must be investigated to make certain that the pavement will not fail during the time that the mix is at its softest. Any accumulation of damage during this diffusion period should be predicted so that it can be combined with the long-term performance. If the mix becomes too tender, it may accumulate excessive rutting and roughness when it is exposed to traffic; it then may have to be redesigned or the pavement closed to traffic for a period immediately after construction.

It must be recognized that the data discussed in this paper were developed on a recycled mix prepared from 100 percent salvaged material. No new aggregate or asphalt cement was used. Therefore, these data may represent the most severe condition for softening produced by the diffusion process. When a relatively large amount of new aggregate, typically 30 to 50 percent, is used in an actual recycling process, the dramatic softening will be reduced somewhat. Because the economics of recycling commonly dictate using the lowest amount of new material possible, the diffusion process will still be present to some extent. It is this extent that must be determined.

RECOMMENDATIONS

The diffusion process must be recognized and accounted for in the laboratory characterization for field performance predictions of recycled pavements that use a modifier. These considerations must encompass the following.

1. Diffusion effects must be examined for all modifier-asphalt combinations during the mix-design stage of the recycling project in order to assess the structural effects that the diffusion process will produce. If these effects will lead to serious structural problems, the mix should be redesigned. The most common redesign would include increasing the amount of new aggregate in the mix, which reduces the influence of the modifier.
2. If redesign of the mix is not practical because of economic factors, the influence of the diffusion process should be lessened by cold mixing the modifier with the salvaged pavement after crushing. The material could be stockpiled for a sufficient time to allow the diffusion process to pass the critical period. Material handling problems should be considered, since the modifier may accelerate the congealing of material in the stockpile.
3. For research purposes, recycled materials should be tested throughout their initial life period after construction. These data should be developed whenever recycling procedures are being investigated and a modifier is being used. These tests should be made to quantify this behavior with as many material combinations as possible.

ACKNOWLEDGMENT

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Abridgment

Evaluation of Selected Recycling Modifiers

RICHARD J. HOLMGREEN, JR., AND JON A. EPPS

A study was undertaken to establish guidelines for use of modifiers in recycling pavement materials for use by the field engineer. Several modifiers were subjected to physical laboratory tests both alone and in blends with aged asphalts. The results indicated that an aged asphalt could be reconstituted but would not necessarily meet all specifications for a virgin asphalt of the same grade. Indications for modifiers blended with asphalts from Woodburn, Oregon; Rye Grass, Washington; and Abilene, Texas, are that the recycled binders should perform satisfactorily.

Materials used to alter properties of asphalt cements have been called softening agents, reclaiming agents, modifiers, recycling agents, fluxing oils, extender oils, bromatic oils, etc. Suppliers of these oils market their products under such names as Cyclogen, Dutrex, Paxole, Reclaimite, and RejuvAcote (1). The term "modifier" will be used to designate this type of material in the report and originates from the American Society for Testing and Materials Subcommittee on Modifier Agents for Bitumen in Pavements and Paving Mixtures. The general definition of modifier is "a material that when added to asphalt cement will alter the physical-chemical properties of the resulting binder". A more specific definition has been developed by the Pacific Coast User-Producer Group for the term "recycling agent"; their Specification Committee defines a recycling agent as "a hydrocarbon product with physical characteristics selected to restore aged asphalt to requirements of current asphalt specifications". It should be noted that soft asphalt cements as well as specialty products can be classified as recycling modifiers or agents.

The purpose of the modifier in asphalt-pavement recycling is to (a) restore the recycled or "old" asphalt characteristics to a consistency level appropriate for construction purposes and for the end use of the mixture, (b) restore the recycled asphalt to its optimal chemical characteristics for durability, (c) provide sufficient additional binder to coat any new aggregate that is added to the recycled mixing, and (d) provide sufficient additional binder to satisfy mixture design requirements.

Methods must therefore be developed to permit the engineer to define the type and amount of modifier to use for a particular asphalt-pavement recycling operation. Findings of a study performed at the Texas Transportation Institute for the National Cooperative Highway Research Program aimed at aiding the engineer in the field are reviewed below.

MODIFIER PROPERTIES

Modifier properties of interest to the engineer are those that can be used for specification purposes to ensure that the modifier will perform the following functions:

1. Be easy to disperse in recycled mixture (2),
2. Alter viscosity of old recycled asphalt cement to the desired level (2-4),
3. Be compatible with the old recycled asphalt to ensure that syneresis (exudation of maltenes from asphalts) will not occur (3),
4. Have the ability to redisperse the asphaltenes in the old recycled asphalt (4),
5. Improve the life expectancy of the recycled asphalt mixture (2-4),
6. Be uniform in properties from batch to batch (2), and
7. Be resistant to smoking and flashing if used in hot-mix operations (2,3,5).

In an effort to classify the modifiers, four possible properties emerged that had enough information on each modifier to use as a classification test. The four tests were viscosity at 77°F (25°C) and at 140°F (60°C), the percentage loss from the thin-film oven test, (TFOT), and viscosity at 140°F on the thin-film oven residue. Since asphalts are graded by viscosity at 140°F either before or after the TFOT, these two tests were considered more significant than the thin-film oven loss and viscosity at 77°F. Also, the design procedure requires the input of viscosity at 140°F of both old asphalt and modifier. Since the old binder was to be reconstituted to an AC-10 grade asphalt, for the purposes of this program, viscosity at 140°F before the TFOT was chosen as the classifying test.

Based on these test results, the seven modifiers to be used for blending with an artifically aged asphalt were chosen. The modifiers were selected first by viscosity; one was chosen from several ranges. In addition, both commercial availability and probability of use were considered major factors. Since the mixing procedure was to simulate a hot-mix operation, emulsions were omitted. After all considerations had been taken into account, the seven modifiers were chosen.

Of the seven modifiers produced, only modifier 1 is a commercially produced additive manufactured for recycling of asphalt pavements. Modifier 2 was motor oil reclaimed from the Texas A&M University Transportation Center after being drained from university vehicles. Modifier 3 is commercially produced as a lube stock, and modifier 4 is a paving grade asphalt, AC-5. Like modifier 4, modifier 5 is also a paving grade AC-5, but it was the highest paraffinic AC-5 available from the source involved. Modifier 6 is commercially available as a slurry oil (carbon black oil), and modifier 7 is available as a roofing asphalt flux.

BLENDS OF MODIFIERS AND AGED ASPHALTS

It is assumed that the field mixing process together with a reaction time (of unknown length) will allow the modifier and the aged recycled asphalt to be completely mixed. If this supposition is accepted, the problem of blending old asphalts and modifiers is greatly simplified. The basic laboratory steps consist of extraction and recovery of the old asphalt, followed by the mixing of various percentages of modifier until the desired consistency is obtained. This process is basically a trial-and-error procedure; however, methods of predicting modifier contents to produce desired viscosities have been developed by the Arizona Department of Transportation, Chevron Research Company (2), Dunning and Mendenhall (4), the Navy (6), the above-mentioned Pacific Coast User-Producer Group, and Witco Chemical Company (3). The basis for all of these methods is basically the same, in that the viscosity of a blend of asphalts of different viscosity can be characterized by equations (1-4) of the following form:

$$\log (V) = a + bp \tag{1a}$$

$$\log - \log (V) = a + bp \tag{1b}$$

$$\log - \log (V) = a + b(\log p) \tag{1c}$$

where V = viscosity of blend (normally measured at 140°F in centistokes) and p = volume by percentage of modifier in blend. If no modifier is used, the viscosity is that of the old asphalt. If 100 percent modifier is used, the viscosity is that of the modifier. Hence, the constants a and b must be determined for each old asphalt-modifier blend.

Laboratory Blends

The seven modifiers chosen for blending were blended with an air-blown Los-Angeles-basin asphalt cement (prepared by Douglas Oil Company) in small quantities in an attempt to achieve the viscosity of an AC-10, which is in the range of 800-1200 P (8000-12 000 Pa) at 140°F. Once the blend for the required viscosity was established, larger quantities were blended for further testing.

Of the seven modifiers tested, all except modifier 3 fell within the specification range of an AC-10 when blended in larger proportions (see Figure 1). It is interesting to note in Table 1 that although the viscosity at 140°F was controlled within a fairly narrow range, the penetration at 77°F (25°C) for all blends that conform to AC-10 requirements ranged from a penetration of 4.2 to a penetration of 14.2; penetrations at 60°F (15.5°C) ranged from 1.0 to 7.0; and viscosities at 210°F (98.8°C) ranged from 14.7 to 29.9 P (147-299 Pa). Note that in the case of modifier 7, an asphalt cement is produced that does not meet the AC-10 requirements (ASTM D3381) for penetrations at 77°F.

At this point, it is important to note that generally the renewed binders did not meet all specifications for a conventional AC-10. This would indicate that it is as important to evaluate the modified binders that are not within normal specifications as it is to evaluate those that do meet specifications. If the reconstituted binder performs as well as virgin binder, even though specifications are not met, then new requirements specially designed to control recycled binders will need to be derived.

Based on an extensive review of the literature, Halstead (7) has proposed a relationship between penetration and ductility that relates asphalt properties to pavement performance. In a 1978 letter, L. C. Krchma has proposed a slight modification to this relationship. Figure 2 illustrates the criteria proposed and the results of the seven modifiers tested. According to Halstead's and Krchma's criteria, blends of modifier 2 (reclaimed motor oil) and modifier 3 (lube stock) with the laboratory-aged asphalts produce asphalt cements that will have unsatisfactory field performance.

Field Blends

From the seven modifiers selected for study with the laboratory-aged asphalt, four were selected for blending with asphalts extracted and recovered from pavements located near Woodburn, Oregon; Rye Grass, Washington; and Abilene, Texas. These pavements were then used in hot-mix recycling projects; modifiers 1 and 2 were selected.

Properties were determined for the recovered asphalts from each of these projects from various locations within the projects. The three projects produced a range in viscosity and penetration typical of many in-service pavements.

The mixtures from each location were then subjected to a laboratory recycling procedure. Modifier 1 was added in an amount determined by mixing several locations and by using that recovered asphalt for blending purposes for each project.

Properties of modifier blends from several locations on the field projects were determined. Blends of materials from location 3 of the Rye Grass, Washington, project consistently gave lower viscosity and higher penetrations than did other locations. This can be explained by the relatively soft nature of the extracted and recovered field-aged asphalt from location 3. Similar behaviors were noted on results of location 5 blends, where the original asphalt had a relatively low viscosity. Blends of modifier 2 result in the largest variation in results among locations.

Asphalt extracted from the various locations in the Woodburn, Oregon, project were more consistent in their properties. However, viscosity measured at 140°F varies by more than ±100 P (1000 Pa) among locations. Thus, the importance of careful laboratory techniques is apparent.

Extraction and recovery tests were performed on laboratory-recycled mixtures by using materials from the three projects. One set of tests was performed after laboratory mixing, compacting, and Marshall testing. The second set of tests was performed after

Figure 1. Viscosity of the artificially aged asphalt plus modifier at 140° F for the seven selected modifiers.

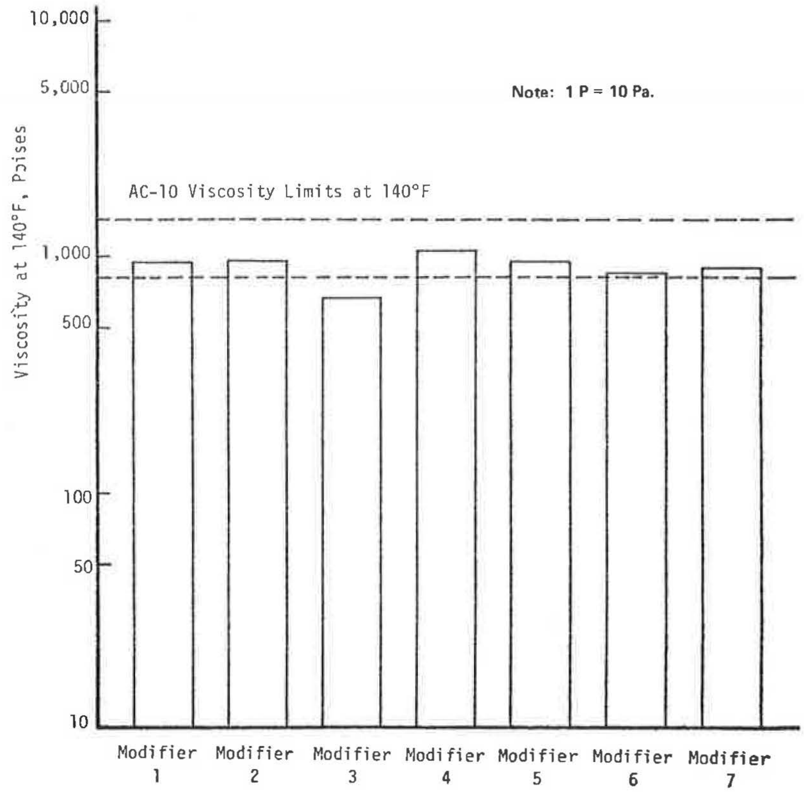


Table 1. Properties of modifier-asphalt blends.

Modifier	Modifier Content (%)	Penetration (mm) 60°F	68°F	77°F	Viscosity (P) 140°F	210°F	Ductility at 77°F (cm)	Ring-and-Ball Softening Point
1	26	3.5		11.2	940	17.1	150+	111
2	27	7.0		14.2	958	14.7	9	116
3	27	6.2		15.8	673	16.8	44	112
4	87	4.0	7.5	12.5	1052	29.9	145	118
5	84	3.1	6.0	10.7	952	24.8	188	116
6	27	3.2	6.8	11.0	853	15.4	115	115
7	47	1.0		4.2	890	21.8	115	122

Note: 1 mm = 0.04 in; ±°F = (t°C·1.8) + 32; 1 P = 10 Pa.

Figure 2. Penetration-ductility relationship of artificially aged asphalt-modifier blend after thin-film oven exposure with Halstead's critical penetration-ductility line.

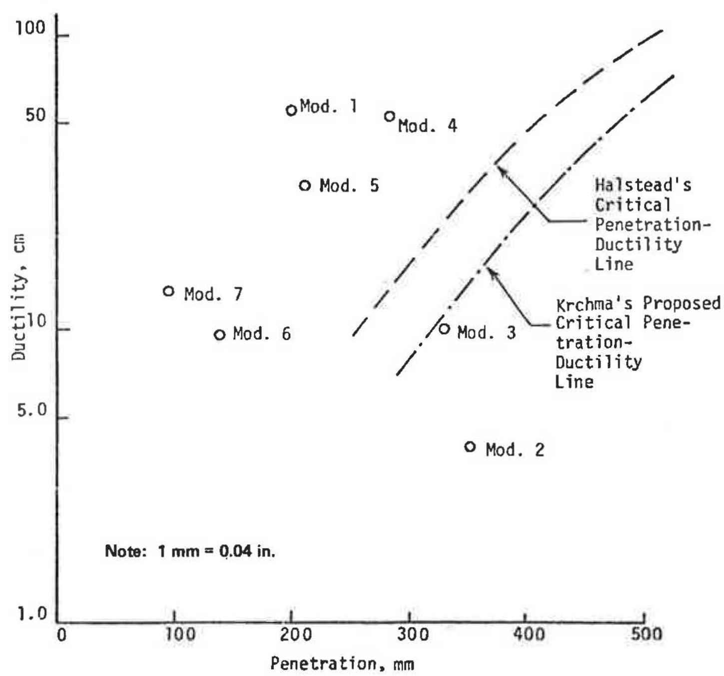
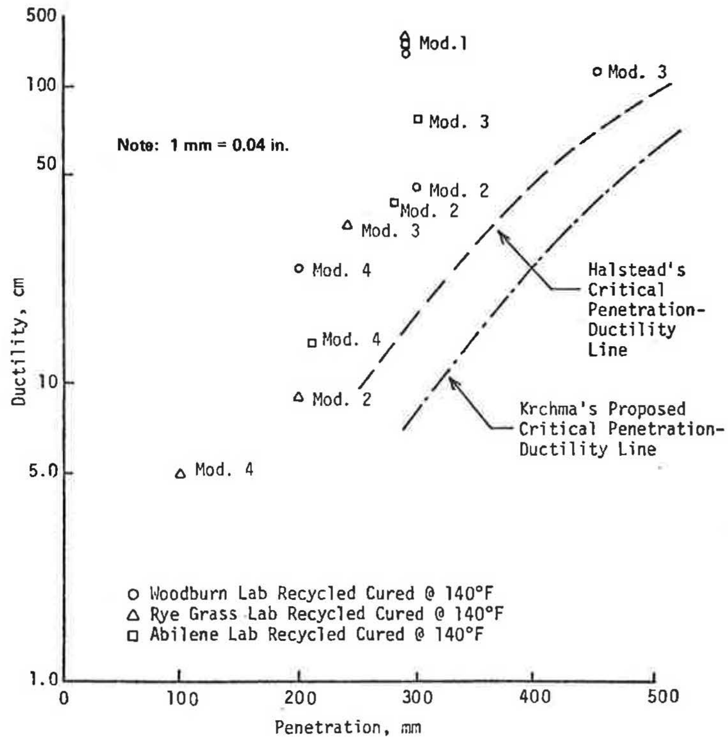


Figure 3. Penetration-ductility relationship of asphalt-modifier blend from 140° F curing samples with Halstead's critical penetration-ductility line.



laboratory curing of the compacted sample for 150 days at 140°F. The mixture that contained modifier 3 hardened excessively during the mixing and compaction operations. Mixtures that contained modifier 1 exhibited the lowest amount of hardening during mixing and compaction.

Penetration-ductility test results obtained after 150 days of curing at 140°F are shown in Figure 3. All combinations meet the established criteria; that is, the blended asphalts can be expected to produce a paving mixture that gives satisfactory performance. The result appears to be contrary to that shown in Figure 2, where results of modifier and laboratory-aged asphalt blends were reported. However, several significant differences between these two materials must be recognized.

1. The modifier and laboratory-aged asphalt blends were subjected to the TFOT.
2. The modifier and field-aged asphalt blends were subjected to laboratory mixing, compaction, and curing, which attempts to simulate field aging.
3. Halstead's criteria were based on extracted and recovered properties of field-aged asphalts and not TFOT results.
4. Laboratory-aged asphalts may produce a different type of material than a field-aged asphalt (2,5).

CONCLUSIONS

As a result of extensive literature review and limited testing at the Texas Transportation Institute, the following conclusions were drawn:

1. Testing techniques must be carefully controlled when asphalt modifying agents are evaluated.
2. Viscosity at 140°F (60°C) is an excellent physical property for classifying modifiers, since blending procedures require this information.
3. Laboratory evaluation tests must be performed on a project-by-project basis.
4. Existing methods to predict asphalt-modifier contents are sufficiently accurate to allow the engineer to estimate required modifier quantities.
5. Modified binders will not necessarily meet specifications of similar conventional binders, and special considerations must be given to each recycled binder.
6. The compatibility of asphalt modifiers and old recycled asphalts needs to be more accurately defined.
7. The best sampling procedure discovered as a result of this study is to sample at various locations and to mix in equal proportions.

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Laboratory Evaluation of Asphalts from Shale Oil

JOE W. BUTTON, JON A. EPPS, AND BOB M. GALLAWAY

The objective of this study was to determine the suitability of shale-oil asphalts for paving purposes. Selected shale-oil asphalt cements were characterized both by tests commonly used to specify paving asphalt and by certain special tests. Asphalt-aggregate mixtures were made by using these asphalts, and they too were subjected to tests that are used in specifying paving mixtures. The test results were compared with similar characteristics of petroleum asphalt cements and petroleum asphalt-aggregate mixtures. Based on the laboratory test results, these shale-oil asphalts exhibit somewhat higher temperature susceptibility and lower water susceptibility than the petroleum asphalt, and the properties of the mixtures are shown to be satisfactory when compared with standard specifications.

This research was undertaken to determine the suitability of shale-oil asphalt for paving purposes. Tests of selected shale-oil asphalt cements were made and the results compared with similar characteristics of petroleum asphalt cements and petroleum asphalt-aggregate mixtures.

ASPHALT CEMENT PROPERTIES

Crude shale oil was produced from oil shale from the Green River formation in Colorado by the gas combustion process. A sample of the resulting shale-oil residue (LERC #SOA-71-98) was used by selected vendors to produce three grades of asphalt cement. A soft asphalt cement labeled SO AC-5 was produced by vacuum distillation, and a solvent-extracted asphalt cement labeled SO SC-10 was prepared by Kerr-McGee Company through a high-pressure process that uses an aliphatic solvent. The first attempt to produce the third asphalt, an AC-20, by vacuum distillation resulted in a material that was much too hard. There was only enough original residuum for one trial. Since the unfractionated distillate from the residuum had been retained, a predetermined portion was reblended with the hard asphalt to produce a material with the appropriate viscosity at 60°C (140°F); it was labeled SO AC-20. It should be emphasized that the process or processes by which shale oil might be produced commercially have not been determined. The properties of a shale-oil asphalt will undoubtedly depend on the type of process. Therefore, the properties of the shale-oil asphalts reported in this paper should be considered as tentative. For a more detailed discussion, see Button, Epps, and Gallaway (1). The material selected as the control asphalt (2) was a viscosity-graded AC-10 petroleum asphalt cement produced by vacuum reduction by the American Petrofina Company at their Mt. Pleasant, Texas, refinery.

Laboratory Tests and Results

American Society for Testing and Materials (ASTM), American Association of State Highway and Transportation Officials (AASHTO), and other (3) standard laboratory tests were performed on each asphalt to determine the basic physical and chemical characteristics, including consistency, durability, purity, and safety.

Two nonstandard tests were also conducted: the thermal neutron activation analysis, used to determine the vanadium content of the asphalt, and the actinic-light hardening test, used to determine the asphalt-hardening effects of chemically active (ultraviolet) light (4). The hardening index was computed by dividing viscosity at 25°C (77°F) of the asphalt after exposure to actinic light by its initial viscosity.

The types of tests performed and the results are presented in Table 1. The appropriate properties of each asphalt are displayed on bitumen test data charts (Figures 1-4). The arrows indicate ASTM and AASHTO specification limits for the particular viscosity-graded asphalts.

Discussion of Test Results

It should be pointed out that the SO AC-20 should not be considered a "normal" asphalt primarily because of the aforementioned method of production. The addition of the unfractionated distillate to the hard asphalt introduced material of higher volatility than would otherwise have been present in a normal vacuum-distillation product. The calculated penetration index (-0.5) and penetration ratio (44 percent) indicate that the material is typical of a normal asphalt that has a relatively low temperature susceptibility. The asphalt is, however, quite susceptible to heat damage, as evidenced by its properties after the thin-film oven test (TFOT) (Table 1). A 2 percent loss on heating indicates the presence of volatile materials; after they were evaporated, the viscosity at 60°C (140°F) became too high to be measured by means of conventional test equipment, and the penetration and ductility fell below specified limits for an AC-20. Also, the flash point and fire point were even lower than those of SO AC-5. In view of the previous discussion, it is not recommended that the results from tests on SO AC-20 be generally applied to evaluate the performance of hard shale-oil asphalts.

Another relatively hard shale-oil asphalt (SO AC-10), prepared by using conventional techniques, was resistant to heat damage, as evidenced by the properties after the TFOT (Table 1). The loss on heating was negligible, and the ductility remained greater than 150 cm (59 in). After the TFOT, changes in viscosity and penetration are what might be expected and are of the order of the corresponding changes in the laboratory standard asphalt. Overall, the properties of the SO AC-10 actually fell nearer to ASTM and AASHTO AC-20 specifications; however, it was termed SO AC-10 primarily because of the viscosity at 60°C. With a penetration index of -1.9 and a penetration ratio of 19 percent, SO AC-10 may be described as a normal asphalt with a high temperature susceptibility.

The soft shale-oil asphalt (SO AC-5) possessed a temperature susceptibility in the higher temperature range almost identical to that of the SO AC-10 and SO AC-20, which is to be expected since they have a common origin. The penetration index (+0.25) and the penetration ratio (26 percent) indicate a normal asphalt. Results from the TFOT indicate a fairly durable asphalt that will resist excessive hardening during mixing and compaction.

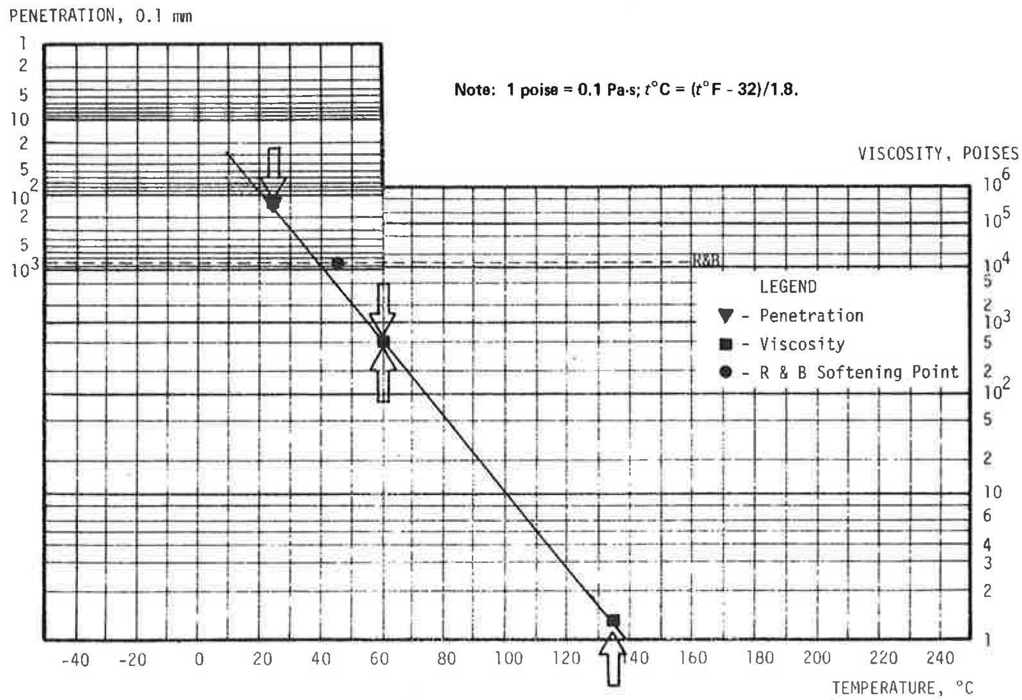
In comparison with the results of tests conducted by Traxler and others (4), the shale-oil asphalts and the laboratory standard asphalt both have very low vanadium contents. Since damage by ultraviolet light in the sun's rays apparently increases with vanadium content, these asphalts may be expected to resist surface hardening that results from exposure to sunlight; very low hardening indexes were determined from the actinic-light hardening tests.

Table 1. Original asphalt cement properties.

Characteristic Measured	Laboratory Standard AC-10	SO AC-5	SO AC-10	SO AC-20
Viscosity (Pa·s)				
25°C	5.8 x 10^4	4.8 x 10^4	2.6 x 10^5	2.5 x 10^5
60°C	158	49	130	199
135°C	0.38	0.13	0.23	0.22
Penetration (mm)				
25°C	11.8	12.3	4.3	7.0
4°C	2.6	3.2	0.8	3.1
Softening point, ring and ball (°C)	42	46	48	49
Penetration index	-1.4	+0.25	-1.9	-0.5
Specific gravity at 25°C	1.02	1.01	1.03	1.03
Ductility at 25°C (cm)	150+	127	150+	93
Solubility (CH Cl:CCl_2) (%)	99.99	100	99.97	100
Flash point (°C)	324	306	294	271
Fire point (°C)	370	355	334	308
Spot test	Negative	Negative	Negative	Negative
Thin-film oven test				
Penetration of residue at 25°C	68	48	24	22
Ductility of residue at 25°C	150+	148	150+	9
Viscosity of residue at 60°C	3050	2070	3650	Too high
Loss of heating (%)	Negative	Negative	Negative	2
Hardening index (actinic light)	1.9	2.5	2.2	1.7
Vanadium content (ppm)	3.4	2.6	–	3.2

Note: 1 Pa·s = 10 poises; t°C = (t°F − 32)/1.8; 1 mm = 0.04 in.

Figure 1. Bitumen test data chart showing properties of SO AC-5.



AGGREGATE PROPERTIES

Before discussion of the mixture properties contributed by asphalt cements, the basic characteristics of the aggregates should be presented. The two types of aggregates selected for use in this reseach study are laboratory standard aggregates used at the Texas A&M University materials laboratory (2).

The subrounded siliceous gravel was obtained from a Gifford-Hill plant near the Brazos River at College Station, Texas. A very hard crushed limestone was obtained from White's Mines at a quarry near Brownwood, Texas. Standard sieves (ASTM E-11) were used to separate the aggregates into fractions sized from 19 mm (0.75 in) to less than 75-µm (no. 200) mesh. Before the various aggregate sizes were mixed with asphalt, they were recombined according to the ASTM D3515-77 5A grading specification. Standard tests were conducted to determine various physical properties of these aggregates, such as bulk specific gravity, saturated surface-dry (SSD) bulk specific gravity, apparent specific gravity, absorption capacity, abrasion resistance, and unit weight. One additional test (5) was conducted to estimate the optimum asphalt content.

The types of tests and results are presented in Table 2.

DETERMINATION OF OPTIMUM ASPHALT CONTENT

The optimum asphalt content for each of the two laboratory standard aggregates was determined by using the laboratory standard asphalt. Then the identical asphalt content was used when each of the

Figure 2. Bitumen test data chart showing properties of SO AC-10.

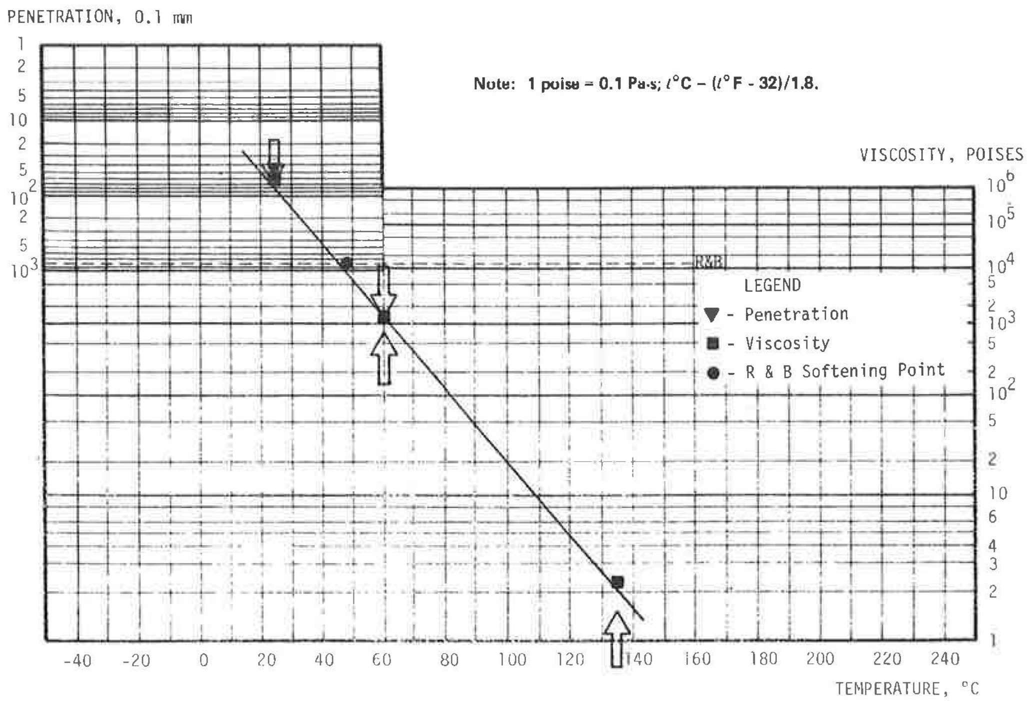
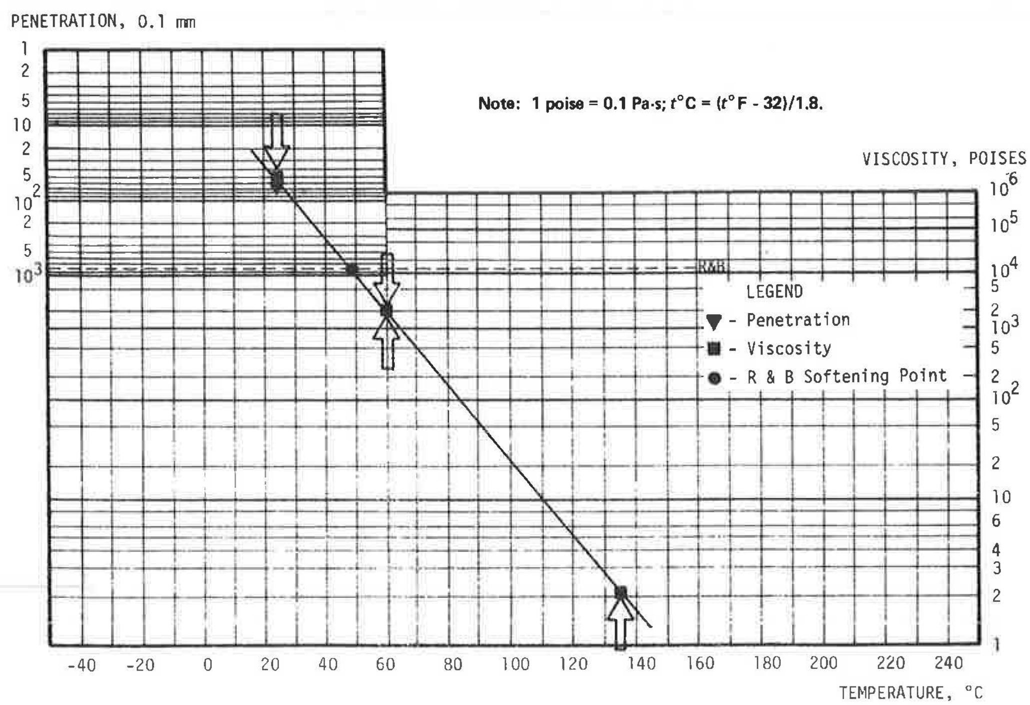


Figure 3. Bitumen test data chart showing properties of SO AC-20.



shale-oil asphalts was mixed with these aggregates, although some design procedures would indicate a somewhat different optimum for different viscosities of binder. Determination of optimum asphalt content was accomplished in accordance with the test program shown by the flowchart in Figure 5.

The selection of the optimum was based primarily on the results of the test series conducted on the Marshall specimens by using the mixture design selection procedures described by the Asphalt Institute (6). However, both the results of the test series conducted on the Hveem specimens and engineering judgment also entered into the final selection. The optimum contents were 3.8 percent for the gravel and 4.5 percent for the limestone.

It should be noted that some of the properties of the compacted mixtures at optimum asphalt content did not meet the criteria established by the Asphalt Institute (6). Undoubtedly, the quality of these mixtures could have been improved by adjusting the aggregate gradation and/or the asphalt content. However, since these mixtures were to be used as laboratory standards for test comparisons and not for highway paving, no attempt was made to further adjust the mixture design.

Figure 4. Bitumen test data chart showing properties of laboratory standard asphalt.

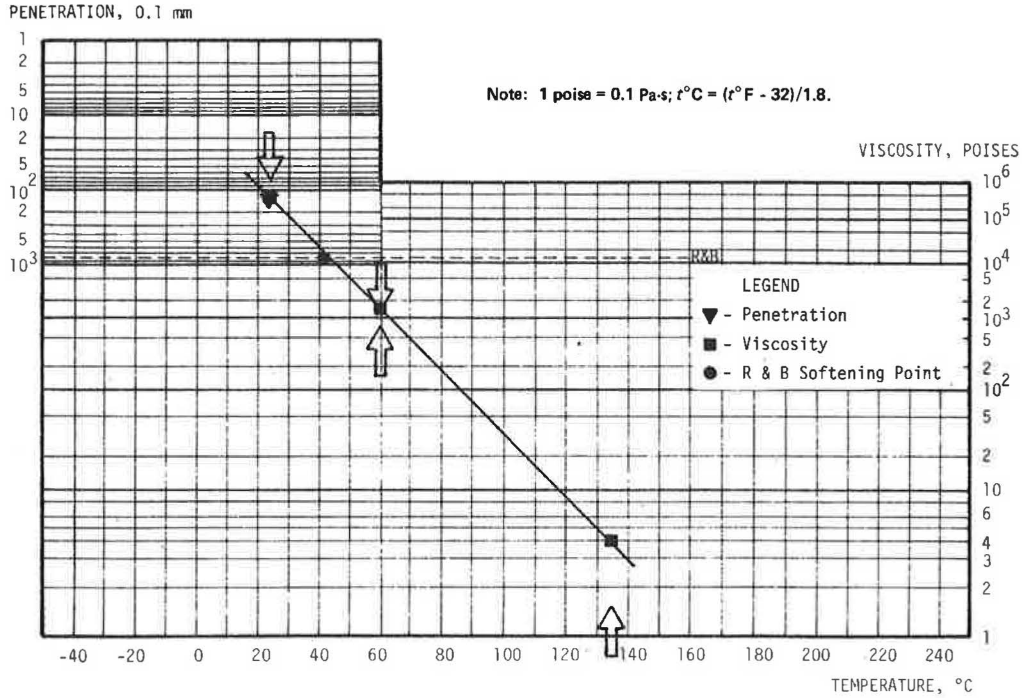


Table 2. Physical properties of aggregates.

Aggregate Grading	Test Designation	Physical Property	Test Results: Gravel	Test Results: Limestone
Coarse material[a]	ASTM C127, AASHTO T85	Bulk specific gravity	2.261	2.663
		SSD bulk specific gravity	2.640	2.678
		Apparent specific gravity	2.672	2.700
		Absorption (%)	0.72	0.7
Fine material[b]	ASTM C218, AASHTO T84	Bulk specific gravity	2.551	2.537
		SSD bulk specific gravity	2.597	2.597
		Apparent specific gravity	2.675	2.702
		Absorption (%)	1.8	2.2
	Centrifuge kerosene equivalent	Surface capacity (% by weight of dry aggregate)	3.0	4.1
Project design gradation	ASTM C127 and C128, AASHTO T84 and T85	Bulk specific gravity	2.580	2.589
		Apparent specific gravity	2.671	2.701
		Absorption (%)	1.3	1.56
	ASTM C29, AASHTO T19	Compacted unit weight (kg/m^3)	2066	1954
	Centrifuge kerosene equivalent and oil equivalent	Estimated optimum asphalt content (% by weight of dry aggregate)	4.7	5.5
Grading C <9.5 mm to >4.75 mm	ASTM C131, AASHTO T96	Abrasion resistance (% loss)	19	23
	Oil equivalent	Surface capacity (% oil retained by weight of dry aggregate)	1.8	2.3

Note: 1 kg/m^3 = 0.06 lb/ft^3.

[a] Material retained on 4.75-mm (no. 4) sieve from project design gradation. [b] Material passing 4.75-mm (no. 4) sieve from project design gradation.

PERFORMANCE OF SHALE-OIL ASPHALTS IN PAVING MIXTURES

Test Results on Gyratory-Compacted Specimens

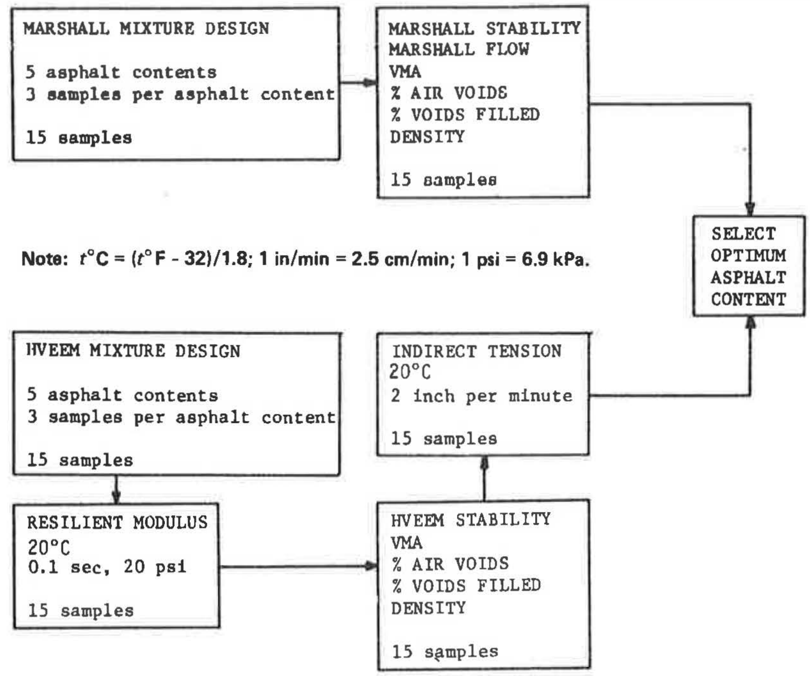
Table 3 presents the basic physical properties of the gyratory-compacted specimens. The test sequence performed on the gyratory-compacted specimens is presented in the flowchart in Figure 6 and is discussed below.

1. Resilient modulus--By using the optimum asphalt contents previously determined for each of the aggregates, 30 specimens of each of the eight asphalt-aggregate mixtures (four asphalts with two aggregates) were compacted in accordance with test method TEX-206-F. The resilient modulus of each of these specimens was measured at 20°C (68°F) by using the Schmidt device (7) (see Table 4).

2. Tensile strength--Twenty-seven of the 30 specimens were selected and divided into three groups of 9 each and conditioned at temperatures of -25, 1, and 20°C (-13, 33, and 68°F), respectively. Then they were subdivided into groups of 3 each, and the splitting tensile test (8) was conducted at loading-head displacement rates of 5.1, 0.51, and 0.051 cm/min (2, 0.2, and 0.02 in/min). A computer program with a plotting subroutine was used to reduce the data. A summary of the test results is presented in Table 5; each value represents an average of three specimen values, unless otherwise indicated.

3. Recovered asphalt properties--After the splitting tensile test, certain specimens were se-

Figure 5. Test program for determination of optimum asphalt content.



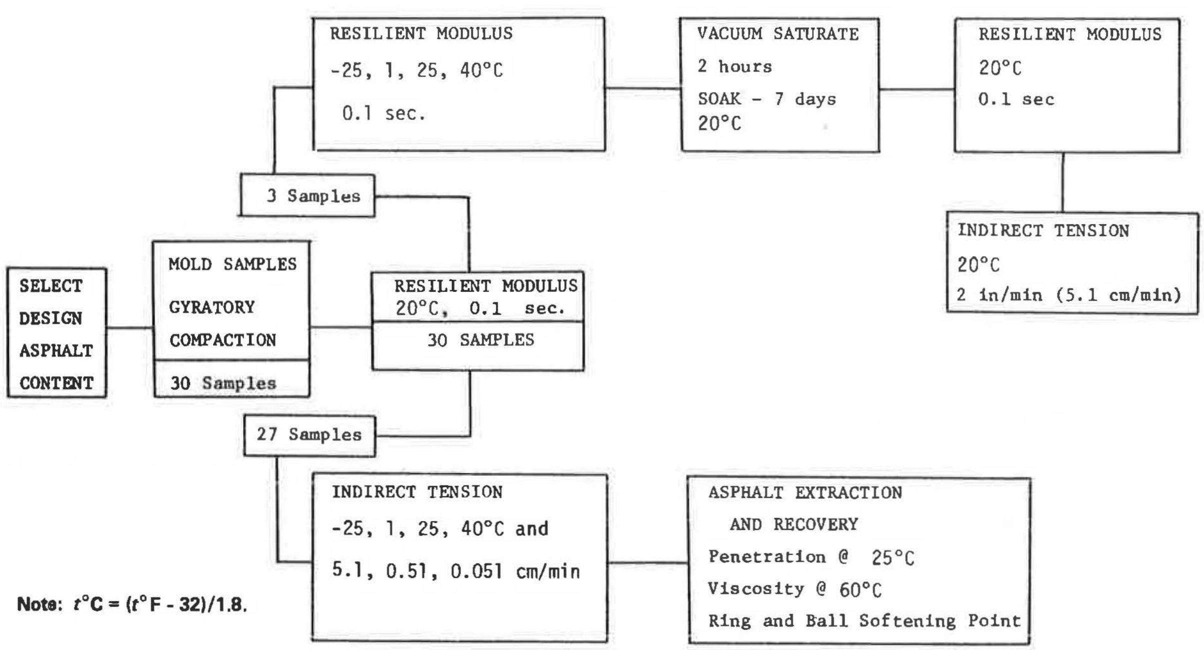
Note: $t°C = (t°F - 32)/1.8$; 1 in/min = 2.5 cm/min; 1 psi = 6.9 kPa.

Table 3. Basic physical properties of gyratory-compacted specimens.

	Rounded Gravel Aggregate				Crushed Limestone Aggregate			
Physical Property	Laboratory Standard	SO AC-5	SO AC-10	SO AC-20	Laboratory Standard	SO AC-5	SO AC-10	SO AC-20
Bulk specific gravity of compacted mix	2.43	2.40	2.42	2.42	2.42	2.45	2.45	2.46
Maximum specific gravity of mixture	2.50	2.51	2.50	2.50	2.51	2.50	2.52	2.51
Asphalt absorption (% by weight of aggregate)	1.0	1.2	0.91	0.91	1.6	1.3	1.6	1.3
Effective asphalt content (% of total mix)	2.7	2.5	2.8	2.8	2.8	3.1	2.8	3.0
Voids in mineral aggregate (VMA) (% of bulk volume)	9.3	10.4	9.6	9.6	10.6	9.5	9.5	9.1
Air void content (% of total volume)	2.8	4.4	3.2	3.2	3.6	2.0	2.8	2.0
VMA filled with asphalt (% of VMA)	76	67	73	73	74	84	79	84

Note: Each value represents an average of 30 specimens.

Figure 6. Test program to determine strength and water susceptibility of mixes.



Note: $t°C = (t°F - 32)/1.8$.

lected for extraction and recovery of each of the asphalt cements. Extraction was conducted in accordance with ASTM D2172-75 (method B). Penetration at 25°C (77°F), viscosity at 25°C and 60°C (140°F), and ring-and-ball softening point were measured to quantify any asphalt hardening that might have taken place during the mixing and compacting procedures. The properties of the asphalts recovered from gravel and limestone are given in Table 6. Although hardening occurred, it was not excessive.

4. Resilient modulus and water susceptibility--The remaining 3 specimens of the original 30 were tested to determine whether or not the asphalts were susceptible to damage by water. The resilient modulus of the specimens was measured at -25, 1, 20, 25, and 40°C (-13, 33, 68, 77, and 104°F) by using a load of approximately 320 N (72 lbf) for a duration of 0.1 s. Figure 7 shows resilient moduli as a function of temperature for the gravel specimens. The curve shapes are similar for the limestone specimens, but the values are a little higher at the higher temperatures. Note the higher temperature susceptibility exhibited by SO AC-5 and SO AC-10 between 10 and 40°C (50-104°F), which corresponds with viscosity data in this temperature range. Then the specimens were submerged in water and vacuum saturated at approximately 25 mm (1 in) of mercury (absolute pressure) for 2 h and allowed to soak at atmospheric pressure for seven days. After soaking, while the specimens were still in the saturated

Table 4. Simple statistics of resilient modulus of gyratory-compacted specimens at 20° C.

Aggregate	Asphalt	Mean Resilient Modulus (kPa x 10^6)	SD (kPa x 10^6)	Coefficient of Variation (%)
Gravel	Laboratory Standard	3.55	0.414	12
	SO AC-5	6.55	1.13	17
	SO AC-10	13.0	1.07	8
	SO AC-20	8.47	1.31	16
Limestone	Laboratory Standard	4.98	0.69	14
	SO AC-5	7.35	0.73	10
	SO AC-10	13.4	1.47	11
	SO AC-20	9.79	1.04	11

Note: 1 kPa = 0.145 psi.

Table 5. Summary of splitting tensile test data.

Displacement Rate (cm/min)	Temperature (°C)	Laboratory Standard Stress (Pa)	Laboratory Standard Strain (cm/cm)	Laboratory Standard Modulus (kPa)	SO AC-5 Stress (Pa)	SO AC-5 Strain (cm/cm)	SO AC-5 Modulus (kPa)	SO AC-10 Stress (Pa)	SO AC-10 Strain (cm/cm)	SO AC-10 Modulus (kPa)	SO AC-20 Stress (Pa)	SO AC-20 Strain (cm/cm)	SO AC-20 Modulus (kPa)
Gravel													
5.1	20	110	0.0029	38	140	0.0026	58	310	0.0038	82	160	0.0025	75
	1	390	0.0027	170	410	0.0013	354	450	0.0007	984	400	0.0009	470
	-25	490	0.0012	418	360	0.0006	625	340	0.0004	1042	370	0.0006	668
Soak	20	100	0.0050	21	200	0.0026	76	200	0.0038	55	230	0.0020	114
0.51	20	50	0.0043	12	80	0.0032	25	230	0.0032	87	100	0.0023	46
	1	250	0.0020	130	380	0.0018	212	400	0.0016	257	300	0.0014	232
	-25	380	0.0009	498	460	0.0008	578	370	0.0009	457	430	0.0009	519
0.051	20	20	0.0041	5	30	0.0037	9	80	0.0048	18	60	0.0022	30
	1	110	0.0018	59	110	0.0021	61	250[a]	0.0024[a]	102[a]	340	0.0011	348
	-25	340	0.0012[a]	331[a]	270	0.0011	246	390[b]	0.0014[b]	271[b]	410	0.0011	385
Limestone													
5.1	20	150	0.0025	60	130	0.0023	69	250	0.0029	89	150	0.0017	94
	1	520	0.0018	290	480[a]	0.0011[a]	462[a]	590	0.0006	1089	500	0.0011	479
	-25	630[a]	0.0012[a]	553[a]	500	0.0011	553	470	0.0005	955	590	0.0010	598
Soak	20	90	0.0059	16	120	0.0038	32	190	0.0031	63	240	0.0022	109
0.51	20	90	0.0041	23	70	0.0034	19	270	0.0030	97	120	0.0017[a]	70[a]
	1	310	0.0022	150	420	0.0013	337	490	0.0014	361	400	0.0014	280
	-25	630	0.0030[a]	226[a]	540	0.0012	479	470	0.0011	456	600	0.0012	500
0.051	20	40	0.0040	11	40	0.0028	12	90	0.0042	21	70	0.0023	32
	1	140	0.0021'	70	470	0.0014	340	380[a]	0.0020[a]	200[a]	200	0.0017	120
	-25	410	0.0030	156	500	0.0011	481	480[a]	0.0024[a]	205[a]	570	0.0013	462

Notes: 1 cm = 0.4 in; t°C = (t°F -32)/1.8; 1 kPa = 0.145 psi.
All values measured at the point of failure.

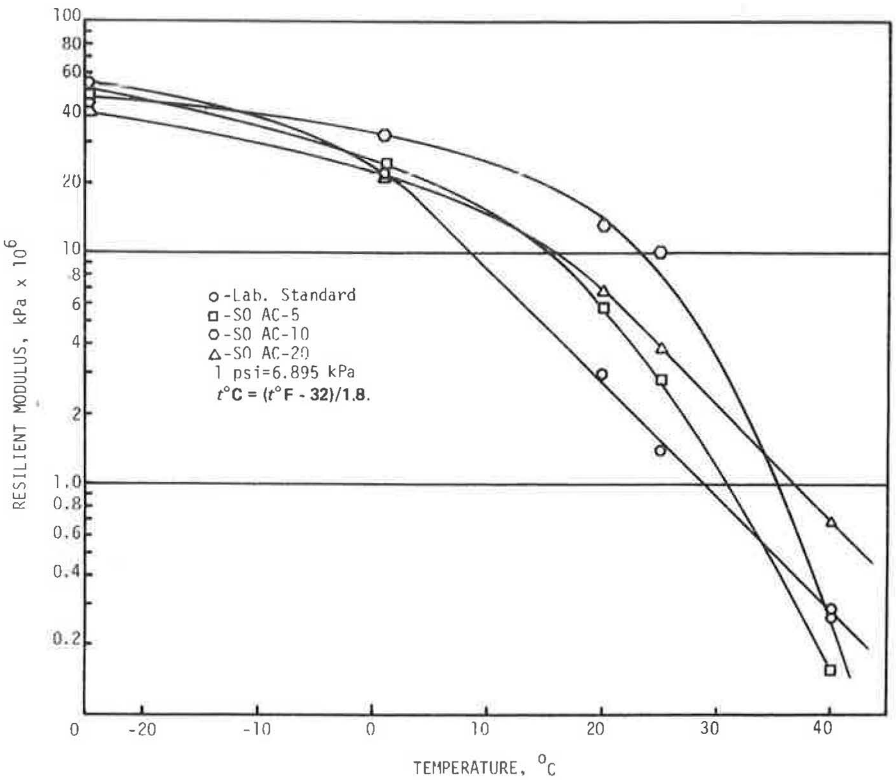
[a]Average of two specimen values. [b]Single specimen value.

Table 6. Recovered asphalt properties.

Aggregate	Test	Laboratory Standard	SO AC-5	SO AC-10	SO AC-20
Extracted from gravel	Penetration at 25°C (mm)	5.5	4.6	1.9	3.0
	Viscosity (Pa·s)				
	25°C	3.9 x 10^5	2.8 x 10^5	2.3 x 10^6	2.0 x 10^6
	60°C	463	143	881	3300
	Ring and ball softening point (°C)	54	57	57	67
Extracted from limestone	Penetration at 25°C (mm)	5.3	5.0	1.7	3.5
	Viscosity (Pa·s)				
	25°C	3.8 x 10^5	3.2 x 10^5	2.4 x 10^6	1.5 x 10^6
	60°C	432	152	801	1310
	Ring and ball softening point (°C)	54	49	58	61

Note: t°C = (t°F -32)/1.8; 1 Pa·s = 10 poises.

Figure 7. Resilient modulus of gravel specimens as a function of temperature.



condition, the resilient modulus of each specimen was again measured at 20°C; then the splitting tensile test was conducted at 20°C and 5.08 cm/min (2 in/min). Figures 8 and 9 show comparisons of mixture characteristics before and after soaking in water.

Test Results on Marshall-Compacted Specimens

Marshall tests were performed to determine the compactibility and stability of mixtures containing shale-oil asphalt and to afford a direct comparison of Marshall specimens containing shale-oil asphalt with Marshall specimens containing the laboratory standard asphalt.

After the three shale-oil asphalts had been mixed at the optimum asphalt contents, each was combined with two laboratory standard aggregates to prepare Marshall specimens by the application of 50 blows to each face of the specimens. After the dimensions and density of each specimen had been determined, the resilient modulus was determined at 20°C (68°F) by using a load of approximately 320 N (72 lbf) for a duration of 0.1 s.

The Marshall stability test was then conducted in accordance with ASTM D1559. The test results for the Marshall-compacted specimens is presented in Table 7.

Discussion of Laboratory Test Results

Gyratory-Compacted Specimens

The resilient modulus (Table 4) indicates that the order of stiffness of the asphalt mixtures is the same for mixtures containing gravel or limestone. The order from low to high follows: laboratory standard, SO AC-5, SO AC-20, and SO AC-10.

Simple statistics for the resilient modulus tests are given in Table 4. For a laboratory test such as this, coefficients of variation of 10 percent or less are considered excellent; therefore, coefficients of variation up to 17 percent should be considered reasonable.

The results of the splitting tensile test would normally be expected to yield the highest tensile strength and highest elastic moduli at the highest loading rate and the lowest temperature, and the converse should be true regarding tensile strain. Generally, this trend is fairly consistent with the data presented herein (Table 5); however, there are specific instances in which this is not true. Because of the lack of precision inherent in data of this type, the heterogeneity of individual asphalt specimens, and the fact that only three specimens were tested at each condition, it is reasonable to expect some inconsistencies.

The mode of failure of the splitting tensile test specimens ranged from physically unnoticeable at 20°C (68°F) and 0.051 cm/min (0.02 in/min) to catastrophic at -25°C (-13°F) and 5.1 cm/min (2 in/min). At -25°C the failure plane was well defined in such a way that the larger aggregates within the failure plane were severed, which indicated that the tensile strength of the matrix equaled or exceeded that of the aggregates.

If the recovered asphalt properties (Table 6) are compared with the original asphalt properties (Table 1), it is seen that, as a result of heating during mixing and compacting, the penetration at 25°C (77°F) of each asphalt cement decreased slightly more than 50 percent and the viscosity at 25°C increased by slightly less than one order of magnitude. The viscosity at 60°C (140°F) of the "soft" asphalts (laboratory standard and SO AC-5) increased by a factor of three, whereas that of the "hard" asphalts (SO AC-10 and SO AC-20) increased considerably more. Hardening of all the shale-oil asphalts was quite comparable to that of the petroleum asphalt. Interestingly, the penetration of the recovered asphalt indicates the same order of stiffness of the asphalt cements as mentioned before

Figure 8. Resilient modulus at 20° C of gravel specimens before and after soaking.

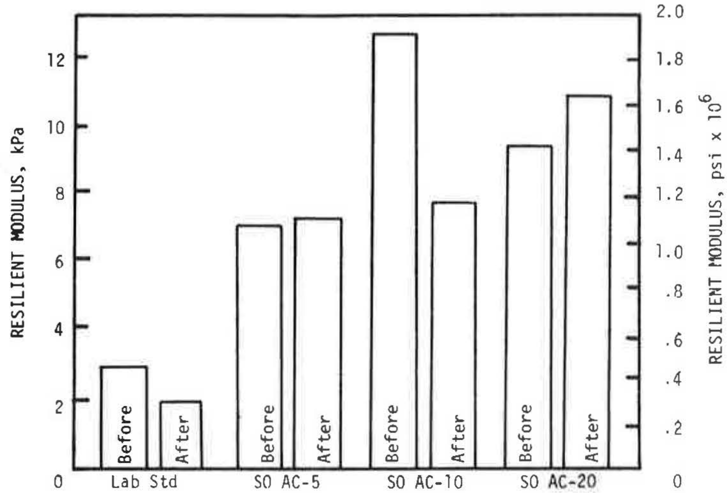


Figure 9. Splitting tensile strength of gravel specimens before and after soaking.

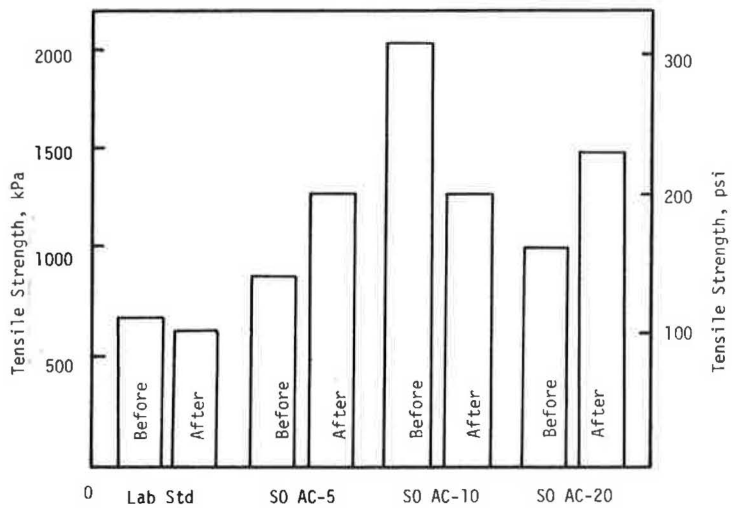


Table 7. Test results for Marshall specimens.

Physical Property	Rounded Gravel Aggregate: Laboratory Standard	SO AC-5	SO AC-10	SO AC-20	Crushed Limestone Aggregate: Laboratory Standard	SO AC-5	SO AC-10	SO AC-20
Bulk specific gravity of compacted mix	2.44	2.42	2.43	2.39	2.45	2.42	2.46	2.42
Maximum specific gravity of compacted mix	2.49	2.51	2.50	2.50	2.53	2.50	2.52	2.51
Asphalt absorption (% by weight of aggregate)	0.75	1.2	0.91	0.91	1.7	1.3	1.6	1.3
Effective asphalt content (% of total mix)	2.9	2.5	2.8	2.8	2.6	3.1	2.7	3.0
Voids in mineral aggregate (VMA) (% of bulk volume)	9.1	9.8	9.3	10.8	10.5	10.7	9.1	10.6
Air void content (% of total volume)	2.1	3.7	2.8	4.2	3.0	3.5	2.3	3.6
VMA filled with asphalt (% of VMA)	80	70	76	67	78	75	81	74
Marshall stability (N)	5650	6140	6850	10 990	12 190	10 270	11 390	15 260
Marshall flow (mm)	1.8	1.5	1.5	1.8	2.8	2	3	2.5
Resilient modulus at 20°C (kPa)	3930	7860	-	11 170	4070	8000	-	11 580

Note: 1 N = 0.225 lbf; 1 mm = 0.4 in; t°C = (t°F -32)/1.8; 1 kPa = 0.145 psi.

in discussion of resilient modulus and, generally, the splitting tensile test.

The most apparent result of the water susceptibility study was that the resilient moduli of the mixtures that used laboratory standard asphalt and SO AC-10 with both aggregates were adversely affected by soaking in water, whereas the mixtures that used SO AC-5 and SO AC-20 were not appreciably affected (Figure 8). This same trend was generally prevalent in the postsoaking results of the splitting tensile tests at 20°C (68°F) and 5.1 cm/min (2 in/min) (Figure 9). With one exception, that of SO AC-5 plus limestone, mixtures that contained SO AC-5 and SO AC-20 actually displayed an increase in tensile strength after water soaking. Consider a theory to explain these phenomena: Shale oil contains larger amounts of basic nitrogen than does petroleum. Large amounts of basic nitrogen in the shale-oil asphalts act as antistripping agents [as Kommes and Stanfield (9) and J. Claine Petersen (U.S. Department of Energy, Laramie, Wyoming) have noted] unless these compounds are removed by some procedure such as the solvent de-asphalting process. The laboratory standard and the SO AC-10 asphalts might therefore be expected to exhibit higher water susceptibility than the SO AC-5 and SO AC-20 asphalts. Further, if it is assumed that the water had little effect on the mixtures that contain SO AC-5 and SO AC-20, the increase in strength and stiffness may have been due to thixotrophy since the specimens had aged at least one week more and since the before-soaking tests were normally conducted on the day after specimen fabrication. Tests have shown that the resilient modulus of freshly made laboratory specimens will increase significantly during the first four days of curing under room conditions, as D.N. Little (Texas Transportation Institute) noted in July 1978.

Resilient modulus (stiffness) as a function of temperature of the mixtures made with shale-oil asphalt was not strikingly different from those made with petroleum asphalt (Figure 7). The slopes of these plots are indicators of asphalt temperature susceptibility. At the lower temperature, SO AC-10 exhibits the lowest temperature susceptibility. At the higher temperatures, laboratory standard and SO AC-20 exhibit significantly lower temperature susceptibilities. This illustrates the fact that asphalt temperature susceptibility depends on the temperature range within which it is defined. Mixture stiffness as a function of temperature showed that shale-oil asphalts have slightly lower temperature susceptibilities at lower service temperatures.

Marshall-Compacted Specimens

According to the Asphalt Institute (6), the medium traffic category requires 50 blows per face on each specimen and should result in a Marshall stability that exceeds 2224 N (500 lbf). The stability of all the mixtures exceeded this value (Table 7). Based on the stiffness of the SO AC-10 relative to the other asphalts tested, the Marshall stability of mixtures containing this material was surprisingly low. However, the comparatively low stability of the rounded gravel specimens was not surprising, since round, smooth aggregates usually produce mixtures that have low stabilities. The bulk specific gravity of the compacted mixtures that possess similar aggregates indicated that all the mixtures were about equal in compactibility. Since all the mixtures of a given aggregate contained identical quantities of asphalt cement, received equal compactive effort, and were in the same viscosity range during compaction, it can be stated that the air void contents indicated that SO AC-20 was the least compactible and SO AC-10 was the most compactible.

CONCLUSIONS

Based on the previous discussions of shale-oil asphalts from the Green River formation, the following conclusions appear warranted.

1. Shale-oil asphalt can be produced by conventional methods in acceptable grades for highway paving mixtures.
2. Difficulties encountered in producing the SO AC-20 asphalt from shale oil for this research were due to the vendor's problems in obtaining reliable viscosity data during sample preparation and had nothing to do with the fact that the residuum came from shale oil.
3. The vanadium content of shale-oil asphalt is low compared with that of about 65 petroleum asphalts tested by Traxler and others (4).
4. Adhesive properties of shale-oil asphalt are sufficient to produce adequate paving mixtures and compare favorably with those of petroleum asphalts.
5. Paving mixtures that contain shale-oil asphalts appear to show superior resistance to damage by water; however, mixtures prepared from the solvent-precipitated asphalt showed some water susceptibility and possibly some loss of Marshall stability.
6. Hardening of the shale-oil asphalts as a result of heating during mixing and compacting was slightly higher than that of the petroleum asphalt.
7. The stiffness as a function of temperature of mixtures made with shale-oil asphalt was not strikingly different from the stiffness of those made with petroleum asphalt.
8. The Marshall stability of mixtures made with shale-oil asphalt was more than adequate and compared well with the Marshall stability of those made with petroleum asphalt.

ACKNOWLEDGMENT

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Abridgment

Design of an Open-Graded Binder Course for Subsurface Pavement Drainage

SERGIO THEN de BARROS

The pavement of a freeway recently completed in São Paulo, Brazil, included a layer of hot-mix open-graded binder course designed mostly for subsurface drainage. The problems encountered and the solutions used in the design and construction are discussed. Revised specifications for open-graded binder course for pavement drainage and a method of density testing developed for design and construction control of open-graded asphalt mixes are presented.

The pavement design of Rodovia dos Bandeirantes, a six-lane 90-km (56-mile) freeway recently completed by the state of São Paulo, Brazil, included a layer of hot-mix open-graded binder course mostly for subsurface drainage (see Figure 1 and note the pen provided for scale). The pavement structure has a total thickness of 62 cm (24.4 in) that includes 6 cm (2.4 in) of dense-graded asphalt concrete, 8 cm (3.1 in) of open-graded binder course, 15 cm (5.9 in) of cement-treated granular base, 13 cm (5.1 in) of graded crushed stone base, and 20 cm (7.9 in) of selected soil subbase, [California bearing ratio (CBR) > 30] as well as 0-80 cm (31.5 in) of final grading course (CBR > 12). The principal functions of the binder course are

1. To drain out any rainwater that eventually infiltrates the pavement through thin cracks or minor faults,
2. To contribute structural support by spreading the load on the treated base, and
3. To absorb retraction forces and reduce reflection cracks from the treated base.

To accomplish these functions, the binder course is made of an open-graded hot mix of hard crushed stone and low-penetration asphalt cement that has a maximum size of 19-38 mm (0.75-1.5 in). The compacted mix has a void rate of about 30 percent.

MIX CHARACTERISTICS

Aggregates

The aggregates used were crushed stone from local quarries, mostly granitic, and some basaltic rock. Four grading specifications were used and tested for the binder course:

1. A--British Standards (BS:1621)--38 mm (1.5 in),
2. B--British Standards (BS:1242)--25 mm (1 in),
3. C--NCHRP Synthesis 30 (1)--25 mm, and
4. D--NCHRP Synthesis 30--19 mm (0.75 in).

The four gradations are really very similar and have only small differences in maximum size. The granulometric curves are almost vertical; they define essentially "one-size" aggregates that have an effective maximum size of 19-38 mm. Other requirements for aggregates are the usual ones for asphalt mixes, such as sound and clean matrix rock, Los Angeles wear-test results below 40 percent, and the best possible fragment form. Actually, it was not possible in this project to obtain crushed stone that had less than 30-40 percent noncubical (lamellar or elongated) fragments from the available quarries, even by using the best crushing equipment.

Asphalt

The binder used was 50/60 penetration asphalt cement to give adequate stability to the binder course. The asphalt content in the total mix was between 2.4 and 2.7 percent.

CONSTRUCTION

The conventional construction method for asphalt concrete was used in the binder course. Hauling required special attention to the cleaning of the trucks. While it was still hot, the layer received an initial compression with a light tandem roller; this was followed by compaction with variable-pressure tire compactors and then finished, again with a tandem roller. In a few sections a heavy self-propelled vibratory smooth roller alone was used after static initial compression and before vibratory final compression. The results were very good.

Heating and mixing temperatures were the usual for hot mixes. Rolling temperature, however, was much lower for binder course than for conventional asphalt concrete. The maximum temperature for the initial compression varied between 80 and 100°C (170-212°F), depending on actual grading, asphalt content, and air temperature. At higher temperatures, the uncompacted layer did not present enough stability to support the weight of the rollers, which resulted in reduced thickness and an uneven profile. The temperature of final rolling was between 50 and 60°C (122-140°F).

Compaction at these lower temperatures presented no great problem. Some pickup of aggregate particles by the roller tires occurs at the beginning of rolling, but it is of little significance. As the rolling continues, the loose particles are gradually incorporated into the surface and the pickup ceases. A tack coat applied later further binds the surface particles in place.

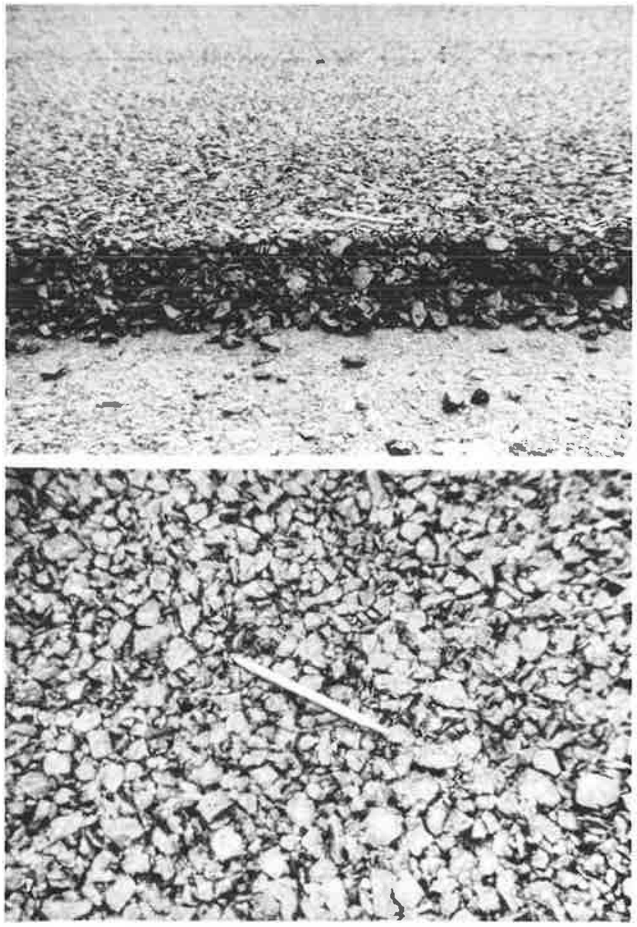
On the first or second day after construction, the surface still moves slightly under the foot action of a person standing on the binder course. This movement ceases completely after two or three days. When needed, a complementary rolling on the second day was used with good results, especially after some sun warming. The finished binder course is left to cure for a period of 7-15 days, after which a tack coat is applied and the asphalt concrete rolling course is laid. The binder course should not be left exposed for more than 15-20 days to avoid clogging by dust or earth carried by wind or construction traffic.

TECHNOLOGICAL CONTROL

The technological control methods for materials and the construction process are generally the same for the binder course as for asphalt concrete except for compaction control.

Sample extraction by rotary drilling of the compacted layer requires great care because sample specimens fall apart easily. Density testing of samples also requires special care and is not

Figure 1. Close-ups of binder course, side and top view.



practical for routine control.

For asphalt-content tests, the Soxhlet apparatus proved to be most reliable. However, granulometric analysis of the aggregate after asphalt extraction was unreliable because of the small size of the samples. The best method of obtaining larger aggregate samples for granulometric analysis was to make a dry run on the mixing plant, feeding aggregates as in normal operation but without asphalt.

EXPERIENCE OF GRADING SPECIFICATIONS

Coarse gradations of maximum size from 25 to 38 mm (1-1.5 in), used at the beginning, caused some problems of segregation, fragment breakage on rolling, harsh finishing, and surface disaggregation under construction traffic. Finer gradations of maximum size from 19 to 25 mm (0.75-1 in) eliminated all these problems and gave much easier handling and better finishing. Permeability tests showed no significant reduction of drainage capacity on the finer gradation.

It was felt, however, that the maximum size of 19 mm (0.75 in) was too small for the desired layer thickness of 8 cm (3.1 in). To achieve a better balance and also lower production costs, a somewhat larger size would be preferable. A compromise solution was proposed: Binder course thickness was reduced, and the asphalt concrete thickness was increased proportionately to maintain the same structural capacity.

Based on the experience of this project, a revised grading specification is recommended for a 5-cm (2-in) thick binder layer (see inset of Figure 2). An aggregate that has this gradation can be produced as a single crusher product that passes through a 25-mm (1-in) screen and is retained in a 9.5-mm (0.38 in) screen (see Figure 2).

MIX DESIGN METHODS

Asphalt content of open-graded mixes is not so critical as that of dense mixes. There is large void space to accommodate a little excess of asphalt, which eventually settles on the bottom without surface bleeding. A lack of asphalt is more detrimental because it affects the cohesion and workability of the mix. Therefore, the optimum asphalt content should be on the rich side.

Specific-Area Method

The specific-area formula proposed by M. M. Duriez for cold mixtures was modified to apply to aggregates that had 100 percent passing a 50-mm (2-in) sieve and less than 5 percent of fines passing a 2.0-mm (no. 10) sieve. The modified formula is as follows for aggregate specific area:

$$S = 0.01(7 + 0.07P_1 + 0.19P_2 + 0.48P_4 + 1.89P_{10}) \tag{1}$$

where

S = specific surface area (m^2/kg),
P_1 = percent passing 25-mm (1-in) sieve,
P_2 = percent passing 12.5-mm (0.5-in) sieve [or, alternatively, 1.10 x percent passing 9.5-mm (0.38-in) sieve],
P_4 = percent passing 4.8-mm (no. 4) sieve, and
P_{10} = percent passing 2.0-mm (no. 10) sieve;

for asphalt content in percentage of aggregate:

$$p' = 3.5\ \sqrt[5]{S} \tag{2}$$

and for asphalt content in percentage of total mix:

$$p = 100\ p'/(100 + p') \tag{3}$$

The results obtained by this formula were remarkably accurate for such a simple method. For the gradations used, the asphalt content calculated by the specific-area formula ranged between 2.4 and 2.7 percent, which agreed fairly well with other methods and with construction practice.

Marshall Method

The Marshall method of mix design is not applicable to open-graded mixes. The molding of test specimens by dynamic impact in the Marshall apparatus, even with only 50 blows per face, causes considerable aggregate breakage, which affects the mix conditions. It is almost impossible to extract the specimens from the mold. Most samples fall apart during extraction or soon afterward at room temperature.

An alternative method is to calculate the density by measurement and weighing of samples inside the Marshall mold without extraction. Specimens compacted with 50 blows on each face gave density values comparable with those obtained in samples drilled from the road and close to values obtained by static compression.

Static Compression

Although it is not really a design method, static

Figure 2. Recommended binder gradation.

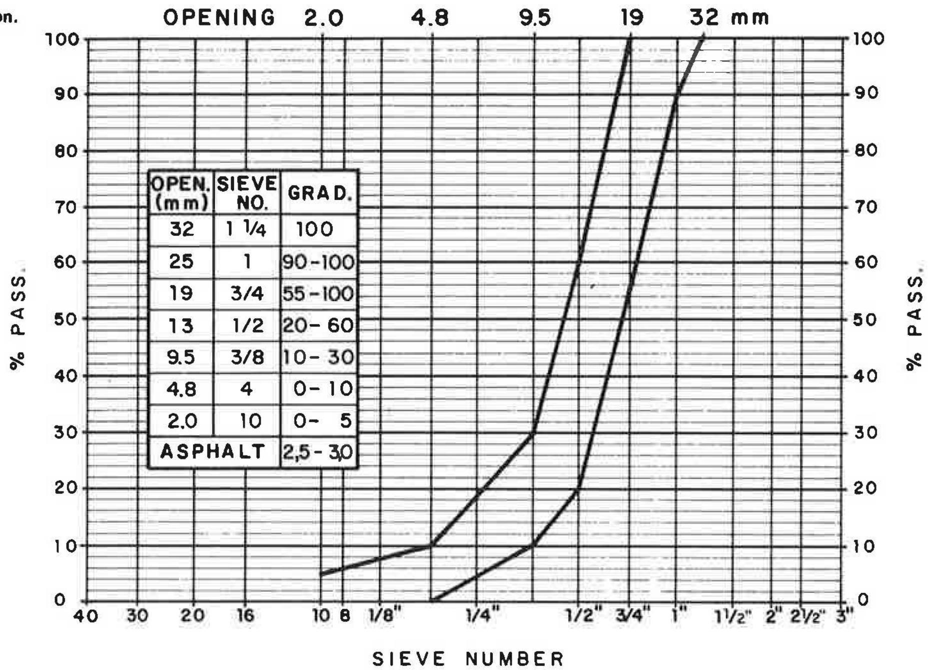


Table 1. Density of static compression samples.

Gradation	Asphalt Content (%)	Maximum Density (g/cm^3)	Average Density (g/cm^3)	Void Rate (%)
A	2.6	2.63	1.78	32
B	2.7	2.62	1.83	30
C	2.7	2.62	1.78	32
D	2.9	2.61	1.78	32

compression is a useful process of molding specimens for density and permeability tests.

In an extensive series of tests, specimens 10 cm (4 in) in height by 15 cm (6 in) in diameter were molded at various temperatures in CBR cylinders under a static load of 7000 kPa (1000 lb/in²) applied for 2.5 min. Density values were calculated by measurement and weighing of the samples inside the molds.

Density and void rate results were independent of gradation, asphalt content, and molding temperature. Density values were very close to those obtained from road samples. This series of tests confirmed field experience that compaction temperature is not critical for this type of mix. Table 1 shows the average results obtained by static compression.

Mixing-Pan Method

The trial "mixing-pan" method for asphalt content design was the most reliable. The procedure is as follows:

1. Prepare a series of trial mixtures in an open pan, starting with a low asphalt content of 2 percent and increasing by 0.2 percent in each mix;
2. Compare the visual aspect of the mixes side by side, increasing in order from lean to rich;
3. Determine the lowest asphalt content that completely covers the aggregate particles with a continuous film, without any free asphalt; and
4. Add 0.4 percent to obtain the optimum asphalt content.

It is very important that all mixtures be prepared at the same temperature, preferably at 150°C (302°F). A difference of ± 15°C (27°F) in mixing temperature can produce an error of ± 0.4 percent in asphalt content design. For the same reason, plant-run mixes that have a fixed asphalt content may appear rich or lean when the plant temperature varies by ± 15°C.

Glass-Plate Method

The glass-plate method was a verification check developed in this project to be used in connection with the mixing-pan method. The procedure is the following.

1. Spread 500 g (about 1 lb) of each of the previously prepared mixes on a thick, transparent glass plate 30x30 cm (12x12 in).
2. Cool the plates at room temperature for 1 h.
3. Raise and fix the plates in an upright position.
4. Observe the adherence of the mixes to the plates. (Lean mixes drop down in a few minutes; rich mixes adhere much longer.) The optimum asphalt content should keep the mix in place for at least 0.5 h.
5. Observe the plates by transparency from the reverse side. The optimum asphalt content should present no bleeding or free asphalt on the plate.
6. Adjust asphalt content accordingly.

DENSITY TESTING

Density testing of open-graded binder course by sample extraction from the road with diamond rotary drilling is very difficult and requires special care. The regular testing procedure that used 10-cm (4-in) sample cores proved unreliable; many cores disaggregate or break up in drilling, especially in the coarser gradations. Water used for cooling the drill penetrates the sample. After extraction, many samples deform or fall apart when left to dry at room temperature for a few hours before dry weighing. Also, paraffin coating, used for water weighing, penetrates large surface voids and affects density calculation.

Table 2. Permeability tests of static compression samples.

Gradation	Void Rate (%)	Hydraulic Flow [(L/min)/cm²]	Permeability Coefficient (cm/s)
A	32	0.20	1.50
B	30	0.25	1.72
C	32	0.21	1.58
D	32	0.15	1.13
Avg	32	0.20	1.48

Figure 3. Infiltration test on binder course surface.

A special procedure for binder-course density testing was developed in this project, as follows.

1. Extract cores by using a 15-cm (6-in) drill; use low speed and as little water as possible. Use great care to avoid breaking the samples.
2. Immediately after extraction, wrap thin adhesive tape around the lateral face of each sample.
3. Let the samples dry for a few hours in inverted position (top down) at room temperature, protected from direct sunlight.
4. Apply adhesive tape or a thin impermeable paper disc to the top and bottom of each sample.
5. Weigh samples without removing tapes. Estimate tape weight. Disregard tape volume.
6. Paint samples with a light coat of warm paraffin that is applied over the protecting tape with a brush.
7. Determine the volume by dry and immersed weighing.
8. Discount paraffin and tape weight. Compute density as usual.

Some average results of samples extracted by this method at the end of the project are shown below.

Gradation	Density (g/cm^3)	Void Rate (%)	Compaction Rate (%)
C	1.87	26	102
D	1.77	30	101

RECOMMENDED COMPACTION SPECIFICATIONS

The recommended compaction specifications for open-graded binder course are the following:

1. Compaction rate of 95 to 105 percent of density obtained in the Marshall test with 50 blows per face and
2. Void rate computed by the theoretical maximum density of 25 to 30 percent.

PERMEABILITY TESTS

A direct measurement of surface permeability on the road was not possible because of the high drainage capacity of the binder course. For comparative purposes, the permeability of Marshall and static compression specimen series was measured under constant hydraulic head and steady-flow conditions for the four gradations used. Both series show little influence of gradation in the coefficient of permeability. Table 2 shows some results of permeability tests for 15x10-cm (6x4-in) specimens with hydraulic head at 7 cm (3 in).

CONCLUSIONS

The major findings and recommendations of this study are the following:

1. The thickness of the binder course may be reduced to 5 cm (2 in).
2. A recommended gradation specification is indicated in the first text table.
3. The best method for asphalt content design is the mixing-pan followed by a glass-plate verification check.
4. The best method for gradation control tests is the dry run.
5. Special care is required for density and void rate tests.
6. The recommended compaction specification is 95-105 percent of 50-blow Marshall test.
7. Compaction temperature is not critical. Recommended compaction temperature is relatively low.
8. Heavy vibratory pneumatic compactors are the most efficient.
9. Binder permeability is very high for all open gradations tested.

ADDENDUM

A new study of binder course density was recently done on Rodovia dos Bandeirantes after one year of heavy traffic by the method described in this paper (see Figure 3). The average results found in this study for gradations C and D are included below.

Test	Average	Standard Deviation
Asphalt content (%)	2.8	0.25
Density (g/cm^3)	1.93	0.09
Void rate (%)	27.1	2.32

Densities and void rates after one year are in the same range as those found at the end of construction. So far there is no evidence of significant increase in compaction as a result of traffic.

ACKNOWLEDGMENT

I would like to thank Desenvolvimento Rodoviário S. A. (DERSA), São Paulo, Brazil, for the use of the basic data for this study and Euvaldo Dal Fabbro and Hélio O. Silva for their help in the preparation of this paper.

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

Evaluation of Low-Temperature Pavement Cracking on Elk County Research Project

PRITHVI S. KANDHAL

The Elk County Research Project in Pennsylvania consists of six test pavements constructed in September 1976 by using AC-20 asphalt cements from different sources. Two test pavements developed extensive low-temperature non-load-associated cracking during the first severe winter. After 2.5 years in service and two more severe winters, the remaining four test pavements do not exhibit any significant cracking. Periodic performance evaluation of these pavements has been conducted. Cracking of the two test pavements was attributed to high stiffness moduli of asphalt cement and asphaltic concrete, determined by indirect nomograph methods. It was felt that the stiffness moduli should also be determined by direct measurements on actual pavement cores. Split tensile tests were conducted at four below-freezing temperatures at a deformation rate of 1.27 mm/s (0.05 in/min) to determine such basic mix properties as stiffness modulus, tensile strength, and tensile strain at failure. Stiffness moduli of the aged pavements were also determined by the indirect Heukelom and McLeod methods. The data from the tensile test indicate that tensile strength or tensile strain at failure, considered independently, does not explain the low-temperature cracking phenomenon on this project. Both direct measurements and indirect methods show that the stiffness modulus of the asphaltic concrete is a better indicator of potential low-temperature cracking. Stiffness moduli determined from the tensile test were generally found to be lower than those obtained by the two indirect methods. A maximum permissible stiffness modulus of 26.9 MPa (3900 lbf/in²) for original asphalt cement (at minimum pavement design temperature and 20 000 s loading time) has been selected to develop AC-20 asphalt cement specifications for the cold regions of Pennsylvania.

The Elk County Research Project in Pennsylvania consists of six test pavements constructed in September 1976 by using AC-20 asphalt cements from different sources. This research was undertaken with the cooperation of the Federal Highway Administration (FHWA) of the U.S. Department of Transportation to study the low-temperature properties of asphalt cements and asphaltic concrete and their effect on pavement performance and durability.

The winter following the construction of these test pavements (1976-1977) was very severe in Pennsylvania. Visual observation of the six test pavements after the winter revealed that two test pavements had developed extensive low-temperature-associated cracking. The air temperature at the nearest weather station (Ridgway) was as low as -29°C (-20°F) during that winter.

The cracking of these two pavements was attributed to the high stiffness moduli of asphalt cements and asphaltic concrete determined by three indirect methods that use nomographs: the van der Poel, the Heukelom, and the McLeod methods (1).

Subsequently, a laboratory evaluation of Marshall specimens that incorporated these six asphalt cements was made to determine the basic mix properties, such as stiffness modulus, tensile strength, and tensile strain at failure, by using the indirect tensile test. A temperature range of 4-60°C (39.2-140°F) and a relatively high deformation rate of 0.84 mm/s (2 in/min) were used. The measurements indicated (2) that the mixtures in two cracked test pavements had much higher stiffness moduli values than the other test pavements.

It was felt that actual pavement cores should be tested in the lower temperature range of -29 to -12.2°C (-20 to 10°F) and at a lower rate of loading of 1.27 mm/min (0.05 in/min) to evaluate the low-temperature pavement cracking. It has been attempted in this study.

DETAILS OF TEST PAVEMENTS

The project is located in Elk County (north-central Pennsylvania) on US-219 just north of Wilcox. The average daily traffic (ADT) on this two-lane, 6.1-m (20-ft) wide highway is 3700. The research project consisted of 38-mm (1.5-in) resurfacing of the existing structurally sound pavement so that the performance of each test pavement can be studied on a comparative basis. The pavement has been built as follows: 254-mm (10-in) crushed aggregate base and 76-mm (3-in) penetration macadam (1948), 76-mm (3-in) binder and 25-mm (1-in) coarse sand mix (1962), surface treatment (1974), and 38-mm (1.5-in) bituminous concrete wearing course (1976). The subgrade consists of silty soil of AASHTO classification A-4.

The layout plan of the six test pavements and construction details are given elsewhere (1). Each test pavement is approximately 610 m (2000 ft) long. The mix composition and compaction levels were held reasonably consistent on all test pavements. The only significant variable is the asphalt type or source.

The mix consisted of gravel coarse aggregate and natural sand. The mix composition data are given below (1 mm = 0.039 in):

Sieve Size (mm)	Percent Passing	Sieve Size (mm)	Percent Passing
12.5	100	0.6	22
9.5	93	0.3	12
4.75	62	0.15	9
2.36	45	0.075	5
1.18	33		

The asphalt content by weight of mix was 7.5 percent. The Marshall design data for the mix were as follows: theoretical maximum specific gravity (ASTM D2041) = 2.326, specimen specific gravity = 2.278, percentage of voids in mineral aggregate (VMA) = 18.8, percentage of air voids = 2.1, stability = 943 kg (2075 lb), and flow = 3.3 mm (0.13 in). This mix composition has been used in the past and has given durable pavements.

Since the six AC-20 asphalt cements had different viscosities at 135°C (275°F) and 60°C (140°F), the mix temperature for each test pavement was adjusted to obtain a mixing viscosity of 170 ± 20 mm²/s (170 ± 20 cSt). This helped to obtain an almost consistent compaction level throughout the project.

PROPERTIES OF AC-20 ASPHALT CEMENTS

The AC-20 asphalt cements were supplied by five refineries. Asphalts T-1 and T-5 came from the same refinery. Table 1 gives the sources of crude, methods of refining, and chemical compositions. The properties of original asphalt cements sampled from the tankers at the bituminous concrete plant are given in Table 2, along with data from the thin-film oven test (TFOT) residue. Asphalt was also recovered from the cores taken just after construction; the recovered asphalt properties are given in Table 3. In May 1978 (20 months after construction), 102-mm (4-in) diameter pavement cores were obtained for this study. After the indirect tensile tests, asphalt was recovered from these cores. The properties of these recovered asphalts are also given in Table 3.

PERFORMANCE OF TEST PAVEMENTS

Weather Data

The Pennsylvania Department of Transportation (PennDOT) has a thermocouple installation site at Lantz Corners, 11 km (7 miles) north of the project, that is capable of recording hourly air temperature and bituminous pavement temperature at 51 mm (2 in) below the surface. According to the recorded data, the critical rapid cooling is believed to have occurred on January 28 and 29, 1977. The air temperature dropped 14°C (25°F) in 2 h. Rapid cooling of pavement 51 mm (2 in) below the surface occurred 12 h later, a drop of 5°C (9°F) in 1 h.

Table 1. Sources of crude oil, methods of refining, and chemical compositions.

Asphalt Type	Crude Oil Source	Method of Refining	Rostler Analysis[a] (%) A	N	A_1	A_2	P	$\frac{A_1+N}{A_2+P}$
T-1	49% Sahara, 21% West Texas, 21% Montana, 9% Kansas	Vacuum distillation and propane deasphalting	8.1	9.0	39.9	30.9	12.1	1.14
T-2	66.7% Texas mid-continent, 33.3% Arabian	Steam distillation	22.4	17.4	24.4	24.4	11.3	1.17
T-3	85% light Arabian, 15% Bachaquero	Vacuum distillation	17.0	23.2	18.8	31.0	10.0	1.02
T-4	75% West Texas sour, 25% Texas and Louisiana sour	Vacuum distillation	19.4	23.1	17.0	27.7	12.8	0.99
T-5	Same as T-1	Same as T-1	15.9	28.7	18.2	27.7	9.4	1.26
T-6	Blend of heavy Venezuelan and Middle East crude	Vacuum distillation	10.4	25.8	19.1	25.3	19.3	1.01

[a]A = asphaltenes; N = nitrogen bases; A_1 = first acidaffins; A_2 = second acidaffins; P = paraffins.

Table 2. Properties of original AC-20 asphalt cements.

Test	Asphalt Type T-1	T-2	T-3	T-4	T-5	T-6
Original Samples						
Penetration at 100 g, 5 s (0.1 mm)						
At 4°C	2.0	7.4	6.2	6.7	3.4	7.5
At 15.6°C	11.2	25.0	24.5	23.0	16.0	29.0
At 25°C	42	64	72	65	54	80
Viscosity						
Absolute, 60°C (Pa·s)	271.0	228.4	176.4	170.5	175.9	198.2
Kinematic, 135°C (mm^2/s)	420	402	393	355	356	406
Softening point, ring and ball (°C)	50.6	50.0	48.9	50.0	51.1	49.4
Penetration index (PI)	-2.77	-0.71	-1.51	-1.05	-2.23	-1.29
Penetration-viscosity number (PVN)	-1.04	-0.70	-0.61	-0.86	-1.03	-0.45
TFOT Residue						
Penetration at 100 g, 5 s (0.1 mm)						
At 25°C	26	38	45	38	37	44
Viscosity						
Absolute, 60°C (Pa·s)	550.1	683.5	398.2	469.4	324.8	572.1
Kinematic, 135°C (mm^2/s)	563	569	556	527	464	575
Ductility (cm)						
At 4°C, 1 cm/min	3.5	3.5	4.6	5.2	8.6	12.4
At 15.6°C, 5 cm/min	11.6	7.0	95.2	12.8	90.6	33.0

Note: t°C = (t°F -32)/1.8; 1 Pa·s = 10 poises; 1 mm^2/s = 1 centistoke.

Table 3. Properties of recovered AC-20 asphalt cements just after construction and after 20 months.

Test	Just After Construction T-1	T-2	T-3	T-4	T-5	T-6	After 20 Months T-1	T-2	T-3	T-4	T-5	T-6
Penetration at 100 g, 5 s (0.1 mm)												
At 4°C	1.5	4.5	4.5	4.0	2.0	5.8	3.0	5.0	5.0	5.7	2.7	5.0
At 15.6°C	7	17	16	13	9	20	7.0	13.0	14.1	13.0	8.0	13.0
At 25°C	24	40	43	34	29	49	19.0	25.2	36.0	31.0	20.6	29.3
Viscosity												
Absolute, 60°C (Pa·s)	552.6	572.9	378.9	382.9	401.9	461.1	847.4	2273.8	557.0	799.8	816.6	1487.1
Kinematic, 135°C (mm^2/s)	565	569	526	487	488	576	690	892	625	641	655	890
Softening point, ring and ball (°C)	56.7	53.3	53.9	53.3	54.4	53.9	58.9	64.7	57.1	61.7	59.4	62.2
Ductility (cm)												
At 4°C, 1 cm/min	0.2	4.6	13.9	5.9	0.6	14.9	1.5	1.0	5.0	5.5	4.1	4.8
At 15.6°C, 5 cm/min	8.3	7.2	48.5	10.0	15.5	34.0	1.5	3.4	12.2	4.5	4.4	5.6
At 25°C, 5 cm/min	150+	80	150+	150+	150+	150+	150+	76	150+	47.5	125.1	42.6
Penetration index (PI)	-2.24	-0.80	-0.99	-0.65	-2.03	-0.64	+0.36	+1.22	-0.12	+0.93	-0.32	+0.60
Penetration-viscosity number (PVN)	-1.13	-0.68	-0.72	-1.03	-1.16	-0.47	-1.07	-0.54	-0.65	-0.76	-1.07	-0.4

Note: t°C = (t°F -32)/1.8; 1 Pa·s = 10 poises; 1 mm^2/s = 1 centistoke.

Table 4. Survey of transverse cracking in pavements T-1 and T-5.

Pavement Section	Time	No. of Transverse Cracks Full (F)	Half (H)	Part (P)	Cracking Index[a] (500 ft)
T-1, station 205+00 to station 215+00	October 1977	5	41	102	51
	May 1978	6	44	164	69
	May 1979	7	45	184	76
T-5, station 125+00 to station 135+00	October 1977	11	26	58	38
	May 1978	14	28	88	50
	May 1979	21	35	64	54

[a]Cracking index = F + 0.5 H + 0.25 P.

The minimum air temperature recorded was -29°C (-20°F), whereas the pavement temperature 51 mm below the surface reached -23°C (-10°F). The 1976-1977 air freezing index for this site was determined to be 1509 degree-days. As mentioned earlier, test pavements T-1 and T-5 developed excessive low-temperature-associated shrinkage cracking during this winter.

Low ambient temperatures prevailed again at the experimental site during the second (1977-1978) and the third (1978-1979) winters. The following minimum air temperatures were recorded at the nearest U.S. weather station (Ridgway), 22.5 km (14 miles) south of the project [t°C = (t°F - 32)/1.8]:

Date	Minimum Air Temperature (°C)	Date	Minimum Air Temperature (°C)
1/23/78	-25.6	2/12/79	-31.7
1/24/78	-25.6	2/13/79	-27.2
2/04/78	-27.8	2/14/79	-28.9
2/05/78	-27.2	2/15/79	-27.2
1/12/79	-26.1	2/17/79	-30.6
2/10/79	-26.1	2/18/79	-30.6
2/11/79	-31.7		

It is estimated that the pavement temperature 51 mm (2 in) below the surface must have reached -23°C (-10°F), which had been considered the design temperature previously for the cold regions of Pennsylvania (1).

Visual Evaluation

When the six test pavements were constructed in September 1976, no visual differences could be seen among them. These pavements have been evaluated periodically since then by a team of 8-10 evaluators. The last inspection was made in May 1979, after the pavements had undergone three unusually severe winters. The pictures of the pavements in this paper were taken in May 1979. A periodic crack survey (Table 4) indicates that the test pavements T-1 and T-5 are developing more cracks and that the existing cracks appear to widen after each successive winter. Test pavements T-2, T-3, T-4, and T-6 have not developed any significant transverse or longitudinal cracking so far.

Figure 1 shows that asphalt T-1 in both lanes has numerous transverse and longitudinal cracks. Figure 2 shows part of the same pavement that has developed block cracking that has produced deterioration and formation of potholes at some places. Figure 3 illustrates the classic effect of C_v (the factor for volume concentration of aggregate) on the mix stiffness modulus. Both lanes have the same asphalt T-1, but the background lane was compacted to 4.4 percent average voids, whereas the foreground lane has approximately 9 percent voids (the result of an isolated cold-mix load). It can be seen that the transverse cracks are mostly confined to the background lane. The mix in the foreground lane has a lower stiffness modulus because of higher air voids and is thus less susceptible to low-temperature cracking. However, higher air voids have induced surface raveling in this area. It appears that the asphaltic mixtures designed with a high VMA by adjusting gradation should also have low stiffness moduli values and thus would help to minimize the cracking.

Figure 4 shows asphalt T-1, which has block cracking, in the foreground lane and asphalt T-2, which has no cracks, in the adjacent background lane. Figures 5 and 6 show asphalts 3 and 4, respectively, which have no transverse or longitudinal cracking.

Figure 7 shows asphalt T-5 in both lanes; it has numerous full- and half-width transverse cracks. Transverse cracks are better defined in asphalt T-5 than in asphalt T-1. Figure 8 shows cracked asphalt T-5 in the foreground lane and crack-free asphalt T-6 in the adjacent background lane.

TEST PROCEDURE AND CALCULATIONS

The indirect tensile test involves loading a cylindrical specimen with compressive loads that act parallel to and along the vertical diametrical plane. Pavement core specimens 101.6 mm (4 in) in diameter were tested in this study. The average thickness of the cores was 25.4 mm (1 in). To distribute the load and maintain a constant loading area, the compressive load was applied through a 12.7-mm (0.5-in) wide steel loading strip that was curved at the interface with the specimen and had a radius equal to that of the specimen.

This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametrical plane that ultimately causes the specimen to fail by splitting or rupturing along the vertical diameter. By measuring the applied load at failure and by continuously monitoring the loads and the horizontal and vertical deformations of the specimens, one can estimate the mix tensile strength (S_T), Poisson ratio (ν), and stiffness modulus (S_F). The equipment and test procedure are described elsewhere (3). A deformation rate of 1.27 mm/s (0.05 in/min) was employed.

The theoretical relationships used in calculating S_T, ν, and S_F are complex and require integration of various mathematical functions. However, by assuming a specimen diameter, one can make the required integrations and simplify the relationships (3,4). These simplified relationships for calculating S_T, ν, S_F, and total tensile strain at failure (ϵ_T) for a 101.6-mm (4-in) diameter specimen with a 12.7-mm (0.5-in) wide curved loading strip are as follows (since the equations are formulated in U.S. customary units of measurement, no SI equivalents are given):

$$S_T = 0.156(P_{fail}/h) \quad (1)$$

$$\nu = (0.0673DR - 0.8954)/(-0.2494DR - 0.0156) \quad (2)$$

$$S_F = (SH/h)(0.9976\nu + 0.2692) \quad (3)$$

$$\epsilon_T = X_{TF}[(0.1185\nu + 0.038\,96)/(0.2494\nu + 0.0673)] \quad (4)$$

where

P_{fail} = total load at failure (lb),
h = height of specimen (in),

DR = deformation ratio (Y_T/X_T) = slope of line of best fit between vertical deformation Y_T and the corresponding horizontal deformation X_T in the linear portion only,
S_H = horizontal tangent modulus (P/X_T) = slope of the line of best fit between load P and X_T in the linear portion only, and
X_{TF} = total horizontal deformation at failure.

The line of best fit was determined by the least-squares method. It was felt that a stiffness value obtained from the linear portion of the stress-strain relation would be more meaningful than the failure stiffness.

INDIRECT TENSILE TEST DATA

Tests were conducted at four temperatures: -29°C,

Figure 1. Asphalt T-1 showing transverse and longitudinal cracks in both lanes.

Figure 2. Asphalt T-1 showing block cracking and deterioration.

Figure 3. Asphalt T-1 showing effect of C_v on the extent of cracking.

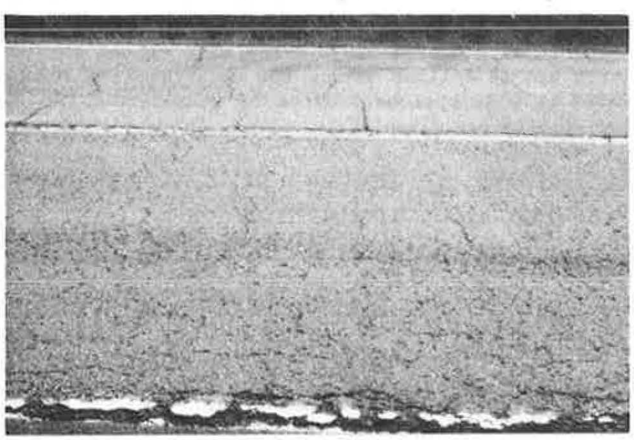


Figure 4. Asphalt T-1 (foreground lane) and asphalt T-2 (background lane).

Figure 5. Asphalt T-3 (both lanes).

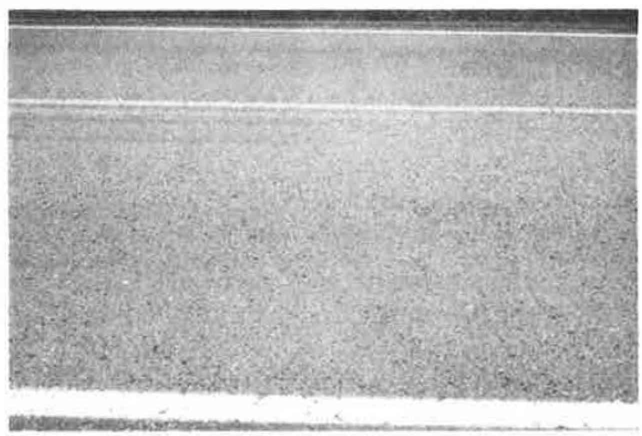


Figure 6. Asphalt T-4 (both lanes).

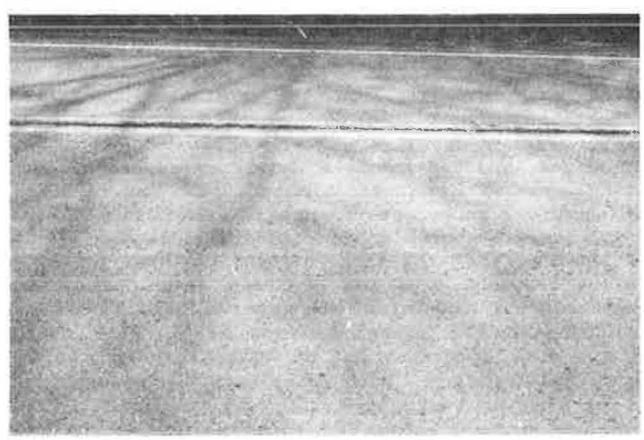


Figure 7. Asphalt T-5 showing well-defined full- and half-width transverse cracks.

Figure 8. Asphalt T-5 (foreground lane) and asphalt T-6 (background lane).

-23.3°C, -17.8°C, and -12.2°C (-20°F, -10°F, 0°F, and 10°F). A minimum of three core specimens from each asphalt type was tested at each temperature, and the results were averaged. Table 5 gives the mix test data on S_T, tensile failure strain (ϵ_F), and S_F for the six asphalts at four temperatures.

Tensile Strength (S_T)

Figure 9 shows the relationship between the temperature and mix S_T by drawing a best-fit curve for each asphalt. In all cases, the S_T of the asphaltic concrete increased as the test temperature was decreased from -12.2 to -29°C (10 to -20°F). The rate of increase of S_T as temperature decreased is lower for asphalts T-1 and T-5 than for the remaining four asphalts. These two asphalts are highly temperature-susceptible, as indicated by their penetration-viscosity numbers (PVNs). Asphalt T-6 has the highest S_T at temperatures of -17.8°C (0°F) and lower; asphalt T-2 has the lowest S_T at all temperatures.

It can be observed that the phenomenon of low-temperature cracking cannot be explained by the mix S_T alone.

Tensile Strain at Failure (ϵ_F)

Figure 10 shows the plot of temperature versus tensile strain at failure (ϵ_F). Asphalts T-1 and T-5 failed at tensile strains lower than those for asphalts T-3, T-4, and T-6 at all temperatures. Some researchers (5) have considered ϵ_F the most significant parameter; the occurrence of cracking was found to increase as ϵ_F decreased. However, asphalt T-2 has ϵ_F comparable to those of asphalts T-1 and T-5 at temperatures of -17.8°C (0°F) and lower, but it has not developed any cracking. Incidentally, asphalt T-2 has much lower S_T at all temperatures (Figure 9) and, therefore, it has a lower failure S_F than asphalts T-1 and T-5. Thus, ϵ_F does not necessarily explain entirely the low-temperature cracking.

Stiffness Modulus (S_F)

Tensile Test

Figure 11 shows the plot of temperature versus failure S_F. Figure 12 shows the plot of temperature versus mix S_F that was obtained from the linear portion of the stress-strain relation, as explained earlier.

Both figures indicate higher S_F's for asphalts T-1 and T-5, which have both developed cracking. Thus, S_F is a better indicator of potential cracking problem than tensile stress or ϵ_F examined independently.

The curves for asphalts T-2, T-3, T-4, and T-6 crisscross each other. Apparently, the differences between their S_F's do not appear to be greater than the reproducibility of the split tensile test conducted on approximately 25.4-mm (1-in) thick pavement cores. Experience has now indicated that at least five cores should be tested so that the outliers can be identified properly. It was observed that the test data on S_T were more consistent than the tensile strain data.

Indirect Methods

Stiffness moduli of the six aged asphalt cements were determined from the data in Table 3 by using two indirect nomograph methods:

1. The Heukelom method (6) uses penetration at two or three temperatures, the "corrected" softening point T_{800}, and the penetration index (PI) (pen/pen). The original van der Poel nomograph is used for determining S_F.
2. The McLeod method (7) uses penetration at 25°C (77°F) and viscosity at 135°C (275°F), base temperature, and PVN.

The S_F of the paving mixture was then determined from the S_F of asphalt cement and $C_V^{\downarrow}$ (a factor for volume concentration of aggregate) by using the chart developed by van der Poel and given by McLeod (7). Based on the core density data, $C_V^{\downarrow}$ was determined to be 0.83.

S_F's of the six paving mixtures were determined for 20 000 s loading time at the four temperatures used in the tensile test. The data are plotted in Figures 13 and 14.

Comparison of Direct Measurements and Indirect Determinations

The following observations are made after reviewing Figures 11-14 and Table 3.

1. As expected, failure S_F's (Figure 11) are higher than S_F's obtained from the straight portion of stress-strain curves (Figure 12).
2. Generally, S_F's obtained by the McLeod method are higher than those obtained by the

Table 5. Values of S_T, ϵ_F, and S_F at four temperatures.

Asphalt Type	-29°C S_T (kPa)	-29°C ϵ_F (mm/mm $\times 10^{-4}$)	-29°C S_F (kPa $\times 10^6$)	-23.3°C S_T (kPa)	-23.3°C ϵ_F (mm/mm $\times 10^{-4}$)	-23.3°C S_F (kPa $\times 10^6$)	-17.8°C S_T (kPa)	-17.8°C ϵ_F (mm/mm $\times 10^{-4}$)	-17.8°C S_F (kPa $\times 10^6$)	-12.2°C S_T (kPa)	-12.2°C ϵ_F (mm/mm $\times 10^4$)	-12.2°C S_F (kPa $\times 10^6$)
T 1	2834	2.94	2.83	2724	3.25	2.18	2648	3.42	1.70	2413	3.62	1.52
T-2	2434	2.95	1.92	2220	3.45	1.22	1903	3.49	1.33	1600	4.36	1.01
T-3	2758	4.07	2.03	2710	3.87	1.54	2537	4.49	1.59	1627	3.99	0.841
T-4	2696	3.64	1.80	2579	3.97	1.59	2331	3.92	1.50	1834	5.14	0.979
T-5	2792	3.27	2.41	2668	3.26	2.01	2599	3.66	1.74	2372	3.75	1.46
T-6	3158	4.40	1.83	2903	4.35	1.90	2751	4.88	1.59	2131	8.42	1.19

Note: t°C = (t°F -32)/1.8; 1 kPa = 0.145 lbf/in².

Figure 9. Temperature versus mix S_T.

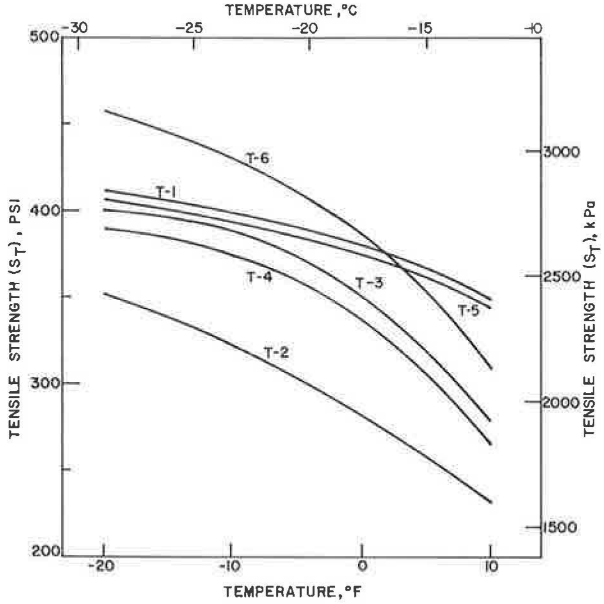


Figure 11. Temperature versus failure S_F.

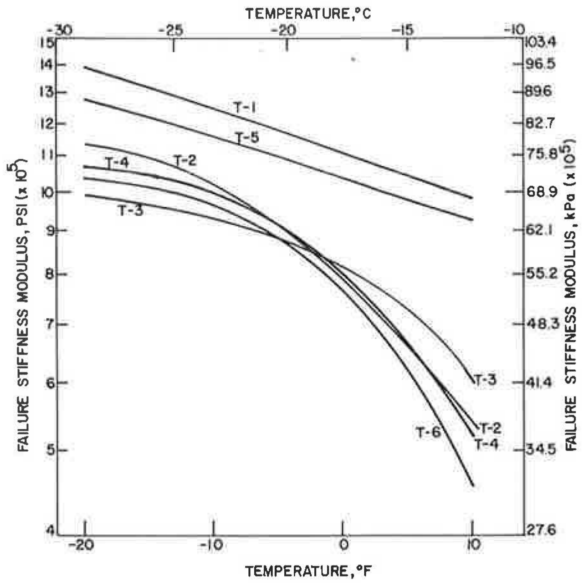


Figure 10. Temperature versus failure ϵ_F.

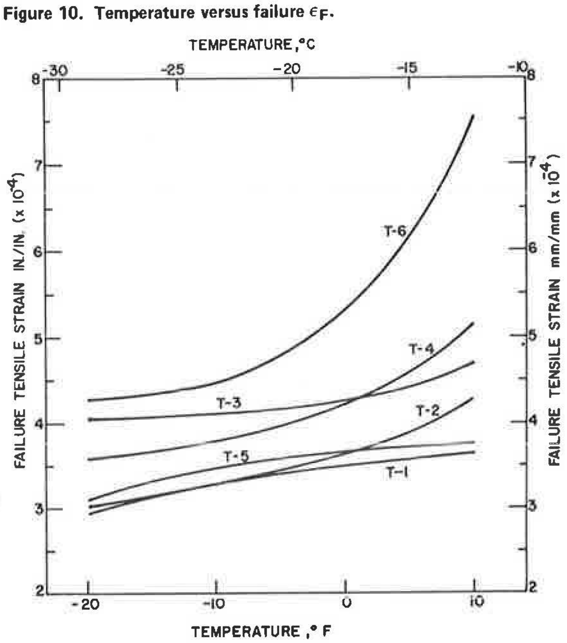


Figure 12. Temperature versus mix S_F.

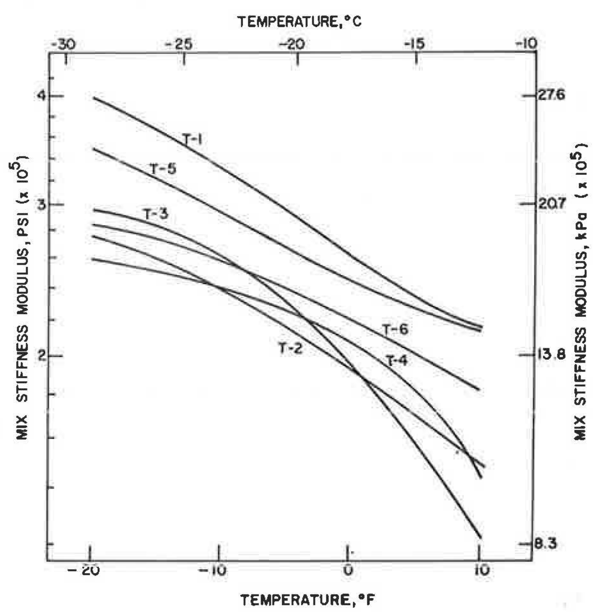


Figure 13. Temperature versus indirect mix S_F (McLeod method).

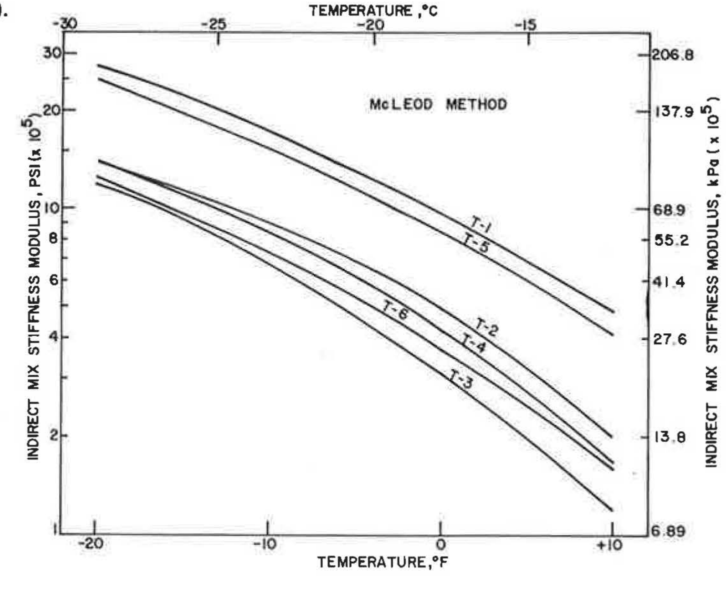
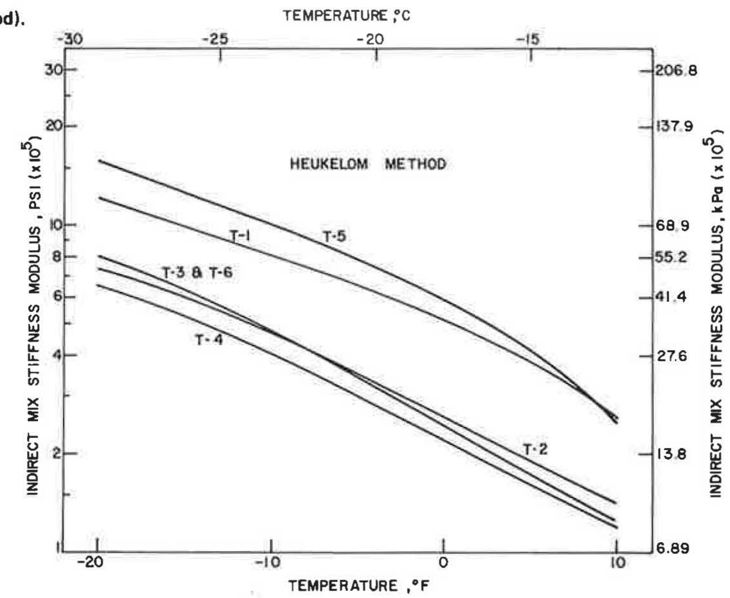


Figure 14. Temperature versus indirect mix S_F (Heukelom method).



Heukelom method, especially at lower temperatures and/or at higher stiffness levels.

3. Since the loading time is different for direct measurements and indirect determinations, it is difficult to make meaningful comparisons. Average loading time to failure in the tensile test was 60 s at a deformation rate of 1.27 mm/s (0.05 in/min). Although a much longer loading time of 20 000 s was used in the indirect determinations, the S_F's obtained are higher than the directly measured values. Generally, the failure S_F values are closer to the indirectly obtained values than are the straight-line S_F's.

Temperature Susceptibility

Temperature susceptibilities of the original and the aged asphalts are reported quantitatively by PI and PVN values in Table 6; these values are used to determine the S_F's of asphalts by the Heukelom and McLeod methods, respectively. It can be observed that PI values have changed drastically after 20 months, whereas PVN values remained essentially unchanged. Because of these drastic changes in the PI values, the S_F's of most asphalts determined by the Heukelom method have decreased rather than increased after 20 months of aging.

Limiting Stiffness Criteria

McLeod (8) has concluded that the low-temperature pavement cracking is likely to become serious if the pavement develops an S_F of 6897 MPa (1 x 10^6 lbf/in^2) at the lowest pavement temperature to which it is exposed for a loading time of 20 000 s, which corresponds to slow chilling on a cold night. This seems to have been confirmed on this project so far. Pavements T-1 and T-5 have developed stiffness moduli greater than 6897 MPa at -23°C (-10°F) and thus are exhibiting low-temperature cracking. Compared with pavement T-5, pavement T-1 has a higher S_F and thus a higher cracking index (Table 4).

It is difficult to suggest a limiting S_F value that is based on the indirect tensile test data because of the test limitations mentioned earlier.

Table 6. Temperature susceptibilities of original and aged asphalts.

Asphalt Type	PI Original	PI Just After Construction	PI At 20 Months	PVN Original	PVN Just After Construction	PVN At 20 Months
T-1	-2.77	-2.24	+0.36	-1.04	-1.13	-1.07
T-2	-0.71	-0.80	+1.22	-0.70	-0.68	-0.54
T 3	1.51	0.99	0.12	-0.61	-0.72	-0.65
T-4	-1.05	-0.65	+0.93	-0.86	-1.03	-0.76
T-5	-2.23	-2.03	-0.32	-1.03	-1.16	-1.07
T-6	-1.29	-0.64	+0.60	-0.45	-0.47	-0.40

AC-20 Asphalt Cement Specifications

Since cracking similar to that observed in pavements T-1 and T-5 was also observed on several pavements in northwestern Pennsylvania where such asphalt cements were used, it became necessary to develop new specifications for cold regions in Pennsylvania to avoid low-temperature cracking.

It became essential to establish the maximum permissible S_F of the original asphalt. Since the PennDOT specifications specify penetration at 25°C (77°F) and viscosity at 135°C (275°F), it appeared convenient to use the McLeod method for determining the S_F at a design temperature of -23°C (-10°F) and a loading time of 20 000 s. Finn and others (9) have also recommended two designer-controlled approaches to low-temperature cracking: (a) specifications for asphalt cement (penetration and viscosity) and (b) stiffness values of asphalt cement or asphaltic concrete that do not exceed limiting criteria for a particular temperature regime. At the present time, establishing the maximum permissible S_F of the original asphalt seems to be the most workable alternative. The original asphalt cements T-1 and T-5, which developed low-temperature cracking during the first winter, had S_F's of 68.5 MPa (9930 lbf/in²) and 36.7 MPa (5320 lbf/in²), respectively. Asphalt T-4 had the highest S_F--19.6 MPa (2840 lbf/in²)--among the remaining four pavements, which do not exhibit transverse cracking at the present time (2.5 years after construction).

It appeared that the maximum permissible limit for S_F was between 19.6 and 36.7 MPa. More data have to be gathered before such a limit can be fully established. It is quite possible that the other pavements may develop low-temperature cracking after aging in service. However, with the limited data it appeared reasonable to assume 26.9 MPa (3900 lbf/in²), which is midway between 19.6 and 36.7 MPa, as the limiting stiffness criterion. This limit can be lowered or increased later as more data are collected.

By using the limiting stiffness of 26.9 MPa and McLeod's nomograph method (7), minimum allowable PVNs were determined for various penetration values. Minimum kinematic viscosities were then determined from the corresponding penetration and PVN values. By specifying the minimum kinematic viscosity at 135°C (275°F) thus determined for each penetration value at 25°C (77°F), it was ensured that the PVN is not lower than the permissible value. Some values are given below (1 mm = 0.039 in; 1 mm²/s = 1 cSt):

Penetration (0.1 mm)	Minimum PVN Allowable	Specified Minimum Viscosity (mm²/s)
60	-0.80	390
65	-0.95	330
70	-1.10	290
75	-1.25	250

It can be seen that higher-temperature-susceptible asphalts can be used if their penetration values are in accordance with those listed above.

The new AC-20 asphalt cement specifications based on the maximum permissible stiffness criteria were made effective in July 1977 for the colder regions of Pennsylvania. The colder regions are those that have an air freezing index of 1000 degree-days or more based on the 1962-1963 severe winter.

CONCLUSIONS

Based on the data obtained by the direct measurements (split tensile test) and the indirect methods, the following conclusions are drawn.

1. Mix S_T or ε_F does not explain the low-temperature cracking phenomenon entirely, if each is considered independently.
2. Both direct measurements and indirect methods indicate that the S_F of the asphaltic concrete is a better indicator of potential low-temperature cracking. Asphalts T-1 and T-5, which developed such cracking, have higher S_F values than the remaining four asphalts.
3. S_F values obtained by the McLeod method are generally higher than those obtained by the Heukelom method, especially at lower temperatures and/or at higher stiffness levels.
4. Although a much longer loading time of 20 000 s was used in the indirect methods, the S_F values are generally higher than those obtained in the split tensile test that was conducted at a deformation rate of 1.27 mm/s (0.05 in/min) and had an average time to failure of 60 s.
5. Temperature susceptibility of the asphalts as indicated by PI values has changed drastically after 20 months of aging. However, the same property expressed by PVN values is essentially unchanged.
6. Limiting asphaltic concrete S_F criteria of 6897 MPa (1 x 10^6 lbf/in²) at the lowest pavement temperature for a loading time of 20 000 s to avoid low-temperature cracking has been verified on this research project so far through three unusually severe winters in succession.
7. From the available data, a maximum permissible S_F of 26.9 MPa (3900 lbf/in²) for original asphalt cement (at minimum pavement design temperature and 20 000 s loading time) was selected to develop AC-20 asphalt cement specifications for cold regions of Pennsylvania. This limit can be lowered or increased later as more data are collected.

The evaluation of these six test pavements will be continued to monitor the effect of increasing S_F of asphalt cements resulting from aging in service on the development of low-temperature non-load-associated shrinkage cracking and other forms of pavement distress.

ACKNOWLEDGMENT

This research project was undertaken by PennDOT with the cooperation of FHWA. The opinions, findings, and conclusions expressed here are mine and not necessarily those of PennDOT or FHWA.

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Discussion

H.E. Schweyer, B.E. Ruth, and C.F. Potts

We believe that this work will turn out to be a classic study in the history of asphalt pavement cracking. The work is most important to our programs in Florida in providing field confirmation of some ideas we have been investigating at the University of Florida with the help of the Florida Department of Transportation (FDOT). Kandhal has provided the opportunity for us to demonstrate that our low-temperature rheological research has been worthwhile. He sent us the bitumens and loose-mix samples from each road section. We are studying rheology data of these down to -10°C (14°F); FDOT is assisting in testing the mixtures. These test data will be correlated with the field-service evaluation to confirm a rational rheological explanation that separates different asphalts in service based on temperature susceptibility and non-Newtonian flow rheology. We expect to present a full report later.

Briefly, (a) we have confirmed the higher temperature susceptibility of T-1 and T-5 bitumens and have verified that both samples have higher

Figure 15. Extended scales for temperature susceptibility.

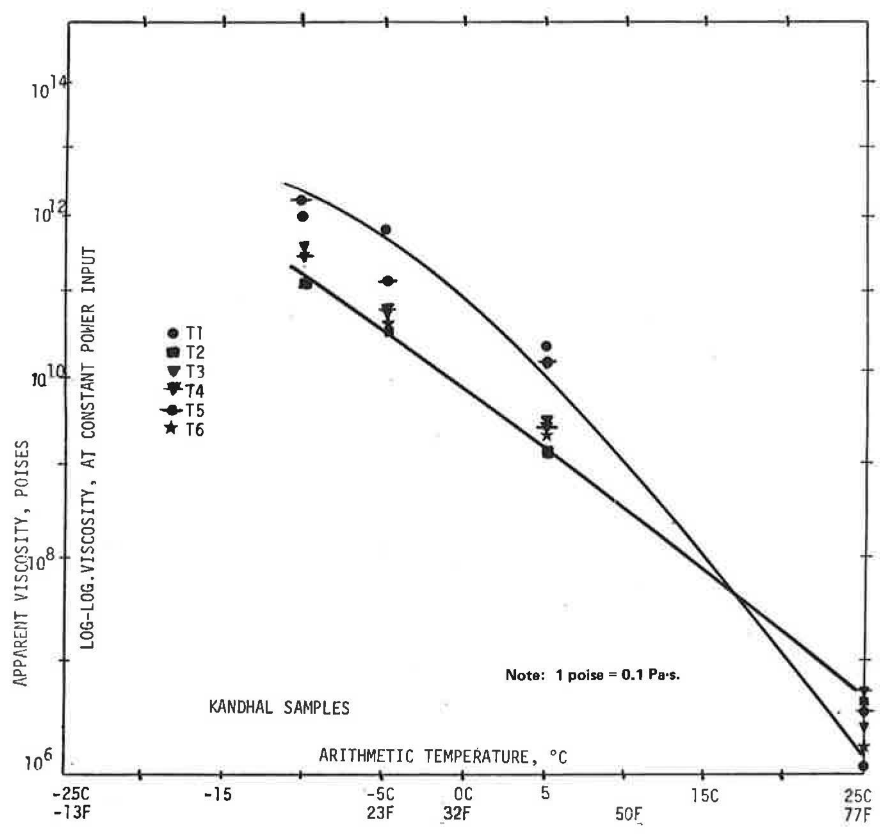

Table 7. Summary of the low-temperature rheology for the six samples.

Temperature (°C)	Property[a]	T-1	T-2	T-3	T-4	T-5	T-6	Mean
25	η (Pa·s x 10^5)	1.09	4.00	2.50	4.71	3.61	1.96	3.98
	G (Pa x 10^7)	3.42	2.48	1.90	5.90	21.1	2.04	6.14
	t_c (s)	0.003	0.016	0.013	0.008	0.002	0.010	0.0086
5	η (Pa·s x 10^8)	20.6	1.67	2.45	2.38	13.3	2.20	7.10
	G (Pa x 10^8)	8.04	5.82	13.4	6.13	10.8	7.28	8.58
	t_c (s)	2.56	0.287	0.183	0.383	1.23	0.302	0.82
-5	η (Pa·s x 10^9)	72.1	4.08	5.80	6.14	29.5	5.80	20.6
	G (Pa x 10^8)	4.14	2.23	2.15	1.37	1.85	4.08	2.64
	t_c (s)	174	18.3	26.9	44.8	159	14.2	72.9
-10	η (Pa·s x 10^{10})	9.40	1.67	4.70	3.16	12.6	2.32	5.64
	G (Pa x 10^8)	3.94	2.34	3.12	3.06	3.47	4.55	3.48
	t_c (s)	238	71.4	151	103	363	51	163

Note: t°C = (t°F -32)/1.8

[a] η = viscosity; G = shear modulus; t_c = time; all G values are in a similar range after passing the transition temperature region.

Table 8. Selected data on mix properties.

Item	T-1	T-2	T-3	T-4	T-5	T-6
Bitumen stiffness modulus[a] at -23°C (MPa)						
At 20 000 s						
Original AC	70.0	14.5	12.0	14.5	25.0	6.0
TFOT residue	58.9	121	74.5	74.2	106	153
Recovered asphalt	145	42.0	36.0	75.0	107	24.0
At 10 000 s	210	62.0	500	110	150	31.2
Viscosity at 20 months (Pa·s)	847	2274	577	800	816	1487
Mix stiffness modulus[a] at -23°C at 20 000 s (MPa)						
Original bitumen	4480	1520	1340	1520	2210	760
Pavement cores	6900	3180	2820	4690	5860	2170
Creep viscosity[b] of laboratory mix at -5°C						
Shear susceptibility index, diametral	0.72	0.79	0.98	1.0	0.78	0.94
Stiffness at 5°C (kPa x 10^6)	10.3	9.11	7.66	9.44	10.8	9.69
Rheology data[c] on original bitumen at -5°C						
Shear susceptibility index	0.72	0.41	0.65	0.47	0.38	0.32
Shear modulus (MPa)	414	223	215	137	185	408
Viscosity (MPa·s)						
η_1	4150	23.2	255	53.0	682	13.3
η_{05}	9730	136	72.2	259	446	102
t_{05} (s)	93	0.61	3.36	1.89	2.41	0.25
η_j at constant power of 100 W/m³ (MPa·s)	72 100	4080	5800	6140	29 500	5800
t_j at constant power of 100 W/m³ (s)	174	18.3	26.9	44.8	159	14.2

Note: t°C = (t°F -32)/1.8.

[a] Van der Poel method.
[b] Run at FDOT on Material Testing System (MTS) machine by Ruth.
[c] Run at the University of Florida on the Schweyer rheometer.

Table 9. Ranges of strength data for mixes.

Temperature (°C)	Tensile Strength (MPa)		Failure Strain (mm/mm x 10^{-4})		Stiffness Modulus (kPa x 10^6)	
	Range	Average	Range	Average (%)	Range	Average
-12	1.6-2.4	2.0	3.6-8.4	0.049	1.0-1.5	1.2
-18	1.9-2.8	2.5	3.4-4.9	0.040	1.3-1.7	1.6
-23	2.2-2.9	2.6	3.2-4.4	0.037	1.2-2.2	1.7
-29	2.4-3.2	2.8	2.9-4.4	0.035	1.8-2.8	2.1

Note: t°C = (t°F -32)/1.8.

stiffness limit times than the others below 5°C (41°F), and (b) we are working with diametral testing of the original mix material at -5°C (23°F) and at 5°C (41°F) to develop a correlation between the mix rheology and bitumen rheology that may help to explain the differences in service.

In connection with the temperature susceptibility, the low sensitivity for the ring and ball softening-point range should be noted. The data indicate a similar range of 49-51°C (120-124°F), which will give about the same PI for a given penetration at a specific AC viscosity grade that allows for testing variance; yet there is a definite variability in consistency. This is apparent on a sensitive viscosity-temperature chart (such as Figure 15) that has a more extended viscosity scale than the American Society for Testing and Materials (ASTM) chart.

Regarding the ductility test at low temperature, it is interesting to note that at the TFOT ductility test's lowest temperature of 4°C (39.2°F) the T-1 and T-5 bitumens that cracked had ductilities higher than or equal to at least one of those in the noncracking pavements under similar test conditions. This was also true at both 15.6°C (60°F) and 25°C (77°F); this should say something about the inadequacy of the ductility test to predict cracking. On the positive side, the data provide evidence that the shear moduli (G_o) appear similar, respectively, for the set of samples. The combination of viscosity and elasticity to give the characteristic time (t_c), as described by Schweyer and coworkers (10,11) at a constant power input of 100 W/m³, separates the samples into two groups at all three low temperatures. The results at 5°C (41°F) and below are summarized in Table 7 for the six samples. The property t_c (also known as the relaxation time) is defined as the ratio of the viscosity divided by the shear modulus at a constant power input to the sample. It represents the time required for the creep viscosity to act and permit the material to flow an amount equal to the initial

Table 10. Comparison of penetration and viscosity as determined by Kandhal and by FDOT data.

	T-1		T-2		T-3		T-4		T-5		T-6	
Property	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT
Original Samples												
Viscosity at 60°C (Pa·s)	271		228		176		171		176		198	
Penetration at 25°C (100 g, 5 s)	42		64		72		65		54		80	
TFOT Residue												
Viscosity at 60°C (Pa·s)	550	494	684	681	398	410	469	454	325	342	572	512
Penetration at 25°C (100 g, 5 s)	26	28	38	42	45	46	38	41	37	38	44	40
Recovered Samples After 20 Months												
Viscosity at 60°C (Pa·s)	847	1152	2274	1394	557	2800	800	1764	817	1037	1487	737
Penetration at 25°C (100 g, 5 s)	19	17	25	28	36	29	31	24	21	19	29	29

Note: The Kandhal data above are from the author's Tables 2 and 4: The viscosity numbers have been rounded off; t°C = (t°F −32)/1.8.

elastic strain that has occurred at a given set of conditions.

Attention is drawn to the t_c values of both the T-1 and the T-5 asphalts, which become greater in absolute value for T-1 and T-5 than for the other asphalts as the temperature is lowered. This is believed to be quite important in connection with low-temperature cracking. If the viscous creep cannot accommodate the elastic deformation before the bitumen reaches its failure strain, the pavement cannot maintain its integrity. It is believed that this particular combination of failure strain and test geometry determines the allowable stress (failure stress) at which the cracking occurs on a level that produces an irreversible effect.

The basic data on construction of the pavements, the temperatures, and the degree of cracking were given in Kandhal's paper (note that T-1 and T-5 had Ontario crack indices of 51/76 and 38/54, respectively). Later data from Kandhal and TFOT data run at FDOT [S_F at 5°C (41°F)] are presented in Table 8. Some comments about the mix properties listed in the table are in order:

1. The bitumen stiffness values found by Kandhal (van der Poel method) are essentially the same magnitude at 1000 MPa at 20 000 s at -23°C (110°F); T-1 and T-5 are higher.
2. Pavement cores for recovered asphalt values at the same conditions as those above show generally slightly higher values but still at the same order of magnitude, although at this level each number differs by 1000 MPa in absolute value.
3. Data on temperature susceptibility in Table 6 show changes that are difficult to interpret. A plot of measured viscosity over the temperature range below 25°C (77°F) appears to be of more interest and value. Such a plot appears in Figure 15.
4. In connection with the tensile strengths and failure strains in Table 5, it should be noted that for all bitumens there is not much variation. The ranges and averages for the four temperatures are summarized in Table 9.
5. The results indicate that, although there is a hardening trend as the temperature is lowered from -12°C (10°F), the average tensile strength varies only from 1.60 to 3.16 MPa.
6. The data shown in Table 8 on stiffness of the original and recovered bitumen as estimated by the van der Poel procedure at -23°C (-10°F) are at about the same level (100 MPa) as those measured at Florida at 5°C (41°F) for the shear modulus.
7. The data for the mixes also show that the Kandhal data at -23°C are at about the same stiffness levels as the Florida data at 5°C.
8. The above items indicate that after the materials enter the glass transition region there does not seem to be too much more hardening on an approximate basis. This is perhaps a logical conclusion, as indicated by the flattening of temperature viscosity plots shown in Figure 15. In addition, the differences may be greater than is apparent since, at a consistency of that level, each integer represents a change of 1000 MPa.

The above information, together with the temperature susceptibility data given in Figure 15, should permit us to determine how the rheology controls the complicated flow behavior of bitumen and, in turn, the mixtures. By using measurements from scientific procedures (instead of by using archaic empirical tests and extensive extrapolations from high-temperature data), we should be able to make better progress in improving highways. This becomes particularly important when mixed crudes from new sources and the use of additives and admixtures in recycling generate new materials whose properties certainly will be different from those of the ones that have been in common use. The work with the Kandhal samples is very interesting and will be explored further and in more detail in the future.

This preliminary comment will be supplemented by further analysis as additional information is developed in this project. As a matter of interest, Table 10 compares Kandhal's data on viscosity at 60°C (140°F) and penetration at 25°C (77°F) with those found by FDOT.

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Author's Closure

I appreciate the additional testing and analyses performed by Schweyer, Ruth, and Potts on the six asphalt cements used on this project. This is a significant contribution toward understanding the complex behavior of paving asphalts.

It has been noted that the softening points of all asphalt cements fall in a narrow range of 48.9-51.1°C (120-124°F) and thus have a low sensitivity in connection with the temperature susceptibility. However, the softening points should be evaluated in conjunction with the penetrations at 25°C (77°F), which vary from 4.2 to 8.0 mm, in order to determine the relative differences in temperature susceptibility.

Regarding the ductility test at low temperature, the TFOT ductility results (Table 2) did not predict the low-temperature cracking. However, it is interesting to note the ductility values at 4°C (39.2°F) run on the asphalt cements recovered from the project just after construction (Table 3), which indicate very low ductility values for T-1 and T-5 asphalt cements.

It would be interesting to follow up this study to ascertain which of the remaining four asphalt cements develops low-temperature cracking first.

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Seasonal Variation in Skid Resistance of Bituminous Surfaces in Indiana

B. L. ELKIN, K. J. KERCHER, AND S. GULEN

Results are reported of repetitive testing of 15 individual bituminous sections at speeds of 40, 50, and 60 miles/h by means of a skid-resistance measuring system as described by ASTM E274 to identify the surface types that provide and maintain satisfactory skid resistance independent of speed, seasonal changes, and climatic factors such as rainfall and temperature. The bituminous test sections represent surface types commonly used in Indiana, an experimental open-graded friction course, and conventional mixes modified by the substitution of slag for some or all of the conventional aggregate portion. A complete petrographic analysis that concentrated on the carbonates of the coarsest fraction was performed on individual pieces of aggregate extracted from a series of cores taken from the test sections. The report also briefly describes the calibration and standardization of Indiana's skid-resistance measurement system. The cyclic nature of skid resistance relative to season is very apparent for all of the surface types included in the study. With one exception, the skid resistance was highest in the spring, dropped off noticeably during the summer, and began to recover in late fall. Average skid values at 40 miles/h for all the test sections ranged from a high of 61.8 to a low of 23.8. Speed gradients were calculated and compared to provide an indication of seasonal sensitivity. Information obtained from the petrographic analysis and accelerated wear rates determined by means of the British polishing wheel in the laboratory revealed that slag has a greater potential for polishing than the aggregates, which are predominantly limestone. However, skid test results show that the addition of slag improves the frictional characteristics of the pavement. Dolomite appears to be more susceptible to polishing than limestone but is not as susceptible as the slag.

This report was prepared to provide for the early dissemination of information obtained midstream in a study that received Federal Highway Administration (FHWA) approval in December 1976. A sizable quantity of information has been gathered in a program of replicate sampling and testing of 15 selected test sections distributed throughout the state of Indiana. All of the routinely used conventional bituminous surface types are included, as well as some that are experimental because of modification of the mix design, aggregate type, or aggregate gradation. Field testing is to continue through the fall of 1980.

All states are required to perform pavement skid-resistance tests and to develop and provide an inventory of skid resistance for traffic safety purposes (1). Furthermore, most states obtain this information by the use of a skid-test measurement system (SMS) as described by ASTM E274. Once the measurements are obtained, the data may or may not be distributed in some fashion to maintenance, traffic, or traffic safety departments to provide a data base for future analysis or to identify those areas that appear to require immediate attention because they have low skid-resistance values and are high-accident-rate locations.

An inventory system has been established, much testing has been accomplished, and a system of reporting and follow-up has been implemented and is working; however, certain questions need to be answered. More information is needed to allow for the accurate analysis of the data obtained: Are individual test values valid? How many tests are needed to give a true indication of the surface friction under traffic? What factors affect skid-resistance values? What does it mean if there is some disparity between values obtained on what appears to be the same type of surface for the same conditions? How much disparity is normal or acceptable? Why does temperature seem to affect the skid resistance of some pavement types at some times and not at others? In short, the objective is a better understanding of the meaning of pavement skid resistance as measured by a towed trailer system. Therefore, the variability of the skid system itself must be considered. In addition, the effect of the weather, seasons, climate, pavement type and condition, and speed of the vehicle traveling on the roads must be determined. Traffic volumes obviously affect the wear rate of certain pavement types more than others, as do the type and condition of the coarse aggregate fraction incorporated in the pavement surface.

This report describes and documents the preliminary activities and initial analysis of the data collected to date on 15 bituminous sections. This information should be considered at this stage as the foundation for the development of a relationship between pavement skid resistance and seasonal changes, mix design, surface texture, and pavement wear rates that can be used to answer the questions posed earlier.

TECHNICAL APPROACH

Skid Measurement

The SMS is used to determine a skid number that can be thought of as the coefficient of friction of the pavement surface multiplied by 100. The frictional coefficient of the pavement is determined from the torsional force on a transducer that is an integral part of the axle assembly and is produced by dragging a locked wheel that has a special test tire at 40 miles/h across the wetted surface.

The SMS used in this study (in accordance with ASTM E274) is a two-wheeled towed-trailer system that uses a strain-gauge-instrumented torque tube to produce an analog signal during wheel lockup. The analog signal is digitized, processed, and converted to a skid number on board the system. Indiana's SMS is depicted in Figure 1.

Routine Evaluation of the SMS

The physical and operational characteristics of the system were initially evaluated and standardized in accordance with the requirements of ASTM E274 at the Field Test and Evaluation Center for Eastern States in East Liberty, Ohio, in August 1976 (2). Test results with the adjusted and calibrated system on the standard test surfaces at East Liberty produced a pooled standard deviation (SD) of 2.3 skid numbers for test speeds from 40 to 60 miles/h.

Special Evaluation for the Study

Sections of four different roads located close to the Research and Training Center (West Lafayette, Indiana) were selected, somewhat at random, to determine the normality and homogeneity of the skid numbers produced by the SMS. The criteria for selection after the location requirement were simple: The sections had to include a range of pavement surface types and provide a range of skid resistance, and the surface of each of the test sections had to be uniform in texture and free from defects.

Twenty tests were performed on each of the four sections within a two-day period. The tests were performed virtually at the same spot to cancel any significant variability of the pavement surface.

The Shapiro-Wilk method was used to check the normality of the skid numbers obtained on each of the four sections. Bartlett's test and the Burr-Foster Q test were used to check the homogeneity of variance (3). The distribution of skid numbers produced by the SMS was found to be normal at the 0.05 percent level for three of the special sites and normal at the 0.04 percent level for the fourth site. Results of both methods of analysis to determine the homogeneity of variances of all four sites is positive; in other words, the differences of the variances obtained on the four test sections were not found to be significant. It was inferred from this that the SMS is capable of producing skid numbers on in-service pavements that are within acceptable limits for accurate replicate testing and are not affected by pavement type and level of skid resistance.

The SMS was reevaluated at the East Liberty test center in July 1978 (4). The SD at that time was found to be 1.7 skid numbers, which is less than the initial evaluation value of 2.3. This information reinforced confidence in the system.

Selection of Study Test Sections

A preliminary list of many and various surface types that had been recently constructed was prepared late in 1976. An initial selection of 40 permanent study sections was made from this list. This list was later revised to a number of sections that could be properly and sufficiently tested with an established frequency throughout three complete annual cycles and in accordance with criteria suggested by FHWA in its review of the study plan--namely, age, traffic volume, and the guidelines set out by FHWA (5). The final list includes 15 bituminous test sections distributed about the state. Specifics of each section are summarized in Table 1. Each surface-type designation is fully described in the Indiana State Highway Standard Specifications except for the open-graded friction course (6). Briefly, however, HAE designates a hot emulsified asphaltic mixture and HAC designates a hot asphaltic concrete. The type designation refers to the maximum aggregate size fraction and gradation. Both the HAC type A and the HAE type II mixes of this study call for aggregate gradations that have 100 percent passing the 0.75-in sieve. Type B and type III are both made with aggregate that passes the 0.5-in sieve, and the type D and type IV mixes are made with aggregate that passes the no. 4 sieve. The mix design for the open-graded surface calls for a gradation that has 100 percent passing the 0.5-in sieve, 30 percent

Figure 1. Indiana's skid-test measurement system.

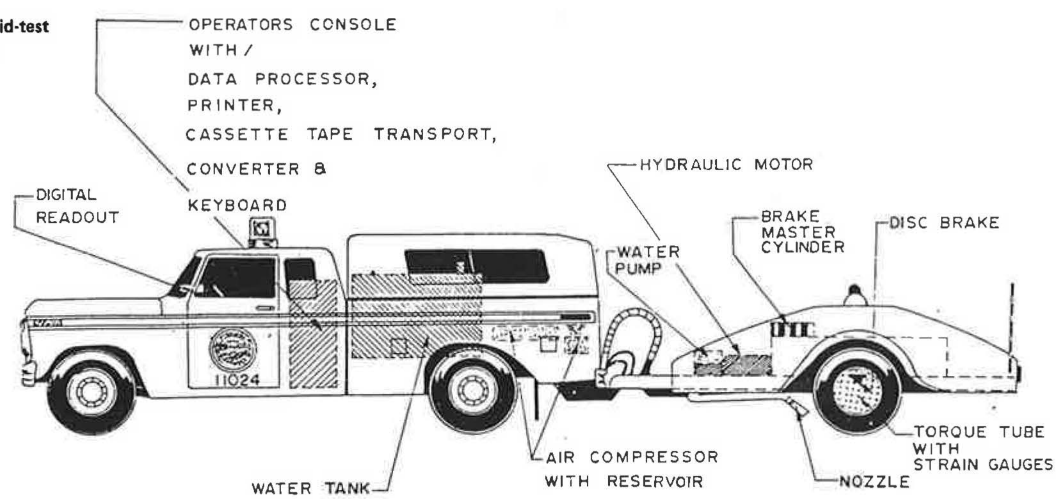

Table 1. Summary of data on bituminous test sections.

Road	Contract	No. of Lanes	Length (miles)	Surface Type	ADT per Test Lane[a]	Date Opened to Traffic
IN-26	RS-10177	2	5.0	HAE-II, no. 9 gravel	1800	9/75
IN 3	RS 9781	2	5.0	HAE II, no. 9 stone	2000	8/74
IN-9	RS-10065	4	4.7	HAE-III, no. 11 gravel	4900	7/75
US-24	RS-9592	2	5.0	HAE-III, no. 11 stone	1700	7/74
US-30	M-9314	4	6.0	HAE-III, no. 11 slag	6000	9/73
I-65	R-10209	4	5.0	HAE-IV, natural sand	8700	11/75
I-64	R-9967	4	3.5	HAE-IV, natural sand	2600	11/75
US-20	RS-10365	4	5.83	HAE-IV, slag	2800	8/76
IN-29	RS-10356	2	2.5	HAC-A, no. 9 gravel	1350	6/76
IN-14	RS-10047	2	1.5	HAC-A, no. 9 stone	1200	7/75
US-41	RS-10058	4	5.05	HAC-A[b], no. 9 stone	2100	10/75
IN-37N	R-9506	4	3.24	HAC-B, gravel	5500	10/74
IN-37S	R-9495	4	2.7	HAC-B, stone	5200	11/74
IN-2	RS-10364	2	5.5	HAC-B, slag	1500	5/76
I-64	R-9967	4	2.2	Open-graded friction course, crushed gravel	2600	11/75

[a]ADT = average daily traffic; figures are approximate. [b]US-41 used a blend of petroleum asphalt and Trinidad asphalt.

passing the no. 4 sieve, and 15 percent passing the no. 8 sieve and has a bitumen content of 6.2 percent.

In the final analysis, the permanent sections are composed of all major aggregate types--gravel, crushed stone, and slag. Surface texture includes both coarse- and fine-textured emulsified asphalt mixes, both coarse- and fine-textured hot asphaltic concretes, and an open-graded hot asphaltic friction course.

A history of each test section was compiled to include mix design, material sources, and a check for unusual occurrences during the placement of the material. As a part of this investigation, an initial material sample was obtained from each test section to verify mix properties and to determine degree of deterioration of the materials that may have occurred prior to any field testing for this study.

All test locations were initially visited to determine any unsafe areas for skid testing. After dangerous curves, hills, and residential areas had been eliminated, the remaining mileage was divided into 0.25-mile-long subsections. The subsections were consecutively numbered in each direction of travel, and a table of random numbers was employed to select three subsections in each test lane. A few of the final sections are tested in one direction only because all six subsections are in the same direction of travel. All subsections were permanently marked.

Development of the Testing Program for the Permanent Sites

SMS Testing

The following control values were used throughout the data analysis.

1. The value of α, the probability of making a type 1 error (rejecting the true hypothesis), was set at 0.05.
2. The value of β, the probability of making a type 2 error (accepting the false hypothesis), was set at 0.10.

Once these values were established, the next step was to determine the number of tests necessary to satisfy accepted statistical requirements (7). The size of a population sample is a function of (a) the value of α, (b) the value of β, (c) the SD of the population (σ), and (d) the allowable error, or tolerance (δ).

The value of σ was estimated from the results of the skid tests performed on the four special test sites as follows [average weighted SD = ±0.95 (estimation of σ)].

Road	SD	Sample Size	Road	SD	Sample Size
IN-25	±0.99	20	IN-26	±1.17	19
US-231	±0.62	20	US-41	±1.03	20

The following statistical hypotheses were used to determine the sample size:

$$H_0: \mu = \mu_0 \tag{1}$$

$$H_1: \mu \neq \mu_0 \text{ at the 5 percent significance level} \tag{2}$$

where μ_0 = mean of random sample from a normal population and μ = mean of the normal population.

When σ is estimated to be ±0.95 and the allowable difference $\delta = |\mu - \mu_0| = 2.0$ skid numbers, it is found that n = 5 is the minimum number of skid tests per site to satisfy the statistical controls and account for the inherent variability of the skid system. The skid testing program was finalized once this value was established; this required five skid-test measurements in each of the six subsections at 40 miles/h, five tests at 50 miles/h, and five tests at 60 miles/h.

It was originally intended to test each of the permanent sections at least once during each season except during winter, which is too unpredictable. However, since October 1977, when skid testing began, machine downtime and weather conditions have caused major interruptions to the established schedule. Sections close to the research facility have averaged two visits per season, whereas those at some distance average only one visit per season.

Determination of Traffic Use

Traffic use in Indiana ranges from very high volumes in the Gary and Indianapolis areas to very low volumes in the widespread rural areas found throughout most of the state. Test sections for this project reflect this diversity of traffic use.

Traffic volumes are determined by means of pressure-tube-type traffic counters placed at three or four test sites within each test section. These counts are made twice yearly for one-week periods in the spring and fall. The volumes collected are adjusted for traffic mix by 8-h traffic classification counts taken while the counter is in place. The volumes are further adjusted by a monthly factor obtained from permanent recording sites that exhibit similar traffic patterns and are monitored by the Indiana State Highway Commission Division of Planning.

Climatological Information

Permanent weather stations are maintained throughout the state under the auspices of the National Climatic Center. A copy of the monthly bulletins that summarize weather conditions at these stations is used to determine daily high, low, and average temperatures; precipitation; sky cover; humidity; and wind direction information for reporting stations near the test sites. Information is compiled for each of the seven days previous to each skid-test date by using the weather station nearest a test section. Pavement temperatures are recorded during each test by using an infrared heat sensor mounted in the floor of the tow vehicle's cab and centered over the left wheelpath.

Field Sampling and Testing

All field sampling and testing accomplished to date in each 0.25-mile subsite was done in the left wheelpath approximately 500 ft beyond the end of the skid-testing area in the following sequence. First, three 6-in-diameter cores were taken from a 10-ft strip within the wheelpath for subsequent laboratory testing. Next, five British pendulum number (BPN) values were obtained by means of the portable skid-test apparatus described in ASTM E303-74 at a convenient distance from the coring locations. These tests were performed at two setups approximately 15 ft apart. A dial-type surface thermometer was used to measure the pavement surface temperature. Finally, the sand-patch test (8) was performed at still another location beyond the portable skid-test area to provide information relative to the surface texture. Three tests were made at three different locations within 10 ft of each other.

Laboratory Testing

Laboratory testing began with the preparation of the three 6-in cores taken in the field. The surface is separated from the core by a wet cut with a diamond blade.

A determination of bulk specific gravity was performed on one of the prepared specimens (in accordance with ASTM D1188) to determine the percentage of voids in the compacted mix.

A reflux extraction test (as described by ASTM D2172) was run on the remaining samples to separate the bitumen and the aggregate. The percentage of bitumen content was calculated from this information. A sieve analysis similar to that of ASTM C136 was performed to determine the gradation of the extracted aggregate.

The extracted bitumen was recovered by the Abson method (ASTM D1856), and both kinematic viscosity (ASTM D2170) and penetration tests (ASTM D5) were performed on the recovered asphalt.

The aggregate fraction between the 0.5-in and no. 4 sieves was sent to the Division of Materials and Tests for evaluation of its polishing characteristics by using the British polishing wheel. A detailed petrographic analysis of both the coarse and fine aggregate fractions from each test site was performed by Purdue University (9).

The procedure adopted for the detailed petrographic examination began with the separation of the aggregate samples into size fractions. The percentage of each constituent was determined. A brief general description of each rock type was prepared that included grain size, particle shape, amount of weathering, cementing material, and impurities. Particle shape was described separately for each size fraction.

The distinction between limestone and dolomite for individual pieces between the 0.75-in and no. 8 sieves was made by using a 10 percent HCl solution. Brisk effervescence was taken to indicate the presence of limestone; pieces that showed slow effervescence (or produced effervescence only when scratched) were considered to be dolomite.

Fractions from the no. 30 to no. 100 sieves were examined under the binocular microscope, and it was observed in all cases that the amount of quartz increased with decreasing grain size.

Thin sections were made for the coarsest size fraction of limestone and dolomite for each sample. The sections were studied microscopically to determine the texture and mineralogy of the carbonate fraction. Carbonates were chosen for the microscopic study because they formed the main portion of each size fraction (generally more than 50 percent).

TEST RESULTS AND DATA ANALYSIS

Skid-Test Data

The results of seasonal testing for each of the sections by means of the SMS is contained in Table 2. The mean SN_{40} is the overall average skid number (SN) of the section that is the average of the five individual tests in each of the six subsites for each date at a test speed of 40 miles/h. The overall speed gradient, G_{40-60}, is defined as

$$G_{40-60} = (SN_{40} - SN_{60})/20 \quad (3)$$

The test speeds specified by ASTM E274 are 30, 40, and 50 miles/h. The test speeds selected to achieve the objectives of this study are 40, 50, and 60 miles/h for two reasons. First, evaluations at the Field Test Center showed that the system was less stable at the lower speeds. Second, the 60-miles/h tests were needed to bracket the posted speed limit of 55 miles/h. The July-September 1978 quarterly Traffic Speed Report (10) for sites in Indiana shows that the average speed for all passenger vehicles ranged from 56.4 to 59.2 miles/h, depending on highway type, and ranged from 55.2 to 59.0 miles/h for all trucks. Information for 1977 ranged from 55.9 to 60.1 miles/h for all passenger cars and 54.2 to 59.3 miles/h for all trucks. The average skid numbers at 40 miles/h (ASN_{40}) for each of the subsites were compared by using analysis of variance (ANOVA) to determine whether any of the subsites were significantly different from the group. The overall average SN_{40} and SDs shown in the tables were calculated by using only those subsites that were similar according to the ANOVA results.

The speed gradients of each subsite were also averaged for each test date. These averages and their respective SDs were also analyzed by ANOVA to determine which sites, if any, produced significantly different skid-speed gradients on any test date.

Skid Data Analysis

A linear regression analysis of skid number and corresponding speed was performed for each test section from the average skid numbers of the six subsites at the three test speeds (40, 50, and 60 miles/h). Correlation coefficients were then calculated to verify the linear relationship that apparently existed between skid number and speed. Almost all of the correlation coefficients were

Table 2. Skid results by season.

Road	Test Date	SN_{40} Mean	SN_{40} SD	SN_{40} Maximum	SN_{40} Minimum	Average Surface Temperature (°F)	Speed Gradient (G_{40-60})
IN-26	10/27/77	47.9	±0.39	49.7	45.8	71	0.221
	4/7/78	50.9	±0.96	52.0	49.5	87	0.234
	5/11/78	49.8	±0.91	52.9	45.8	69	0.287
	7/18/78	45.3	±0.96	46.9	41.5	115	0.239
	9/28/78	44.5	±0.72	47.9	42.6	87	0.267
	10/30/78	46.0	±0.64	46.9	42.3	69	0.306
	5/23/79	45.5	±1.18	46.8	40.6	105	0.278
	8/22/79	50.4	±0.86	51.2	47.3	110	0.219
IN-3	10/25/77	41.1	±0.48	42.6	35.4	64	0.149
	4/24/78	47.1	±0.38	47.4	44.8	75	0.141
	5/23/78	44.2	±1.75	46.3	41.8	65	0.204
	8/31/78	36.2	±0.29	40.8	35.8	86	0.209
	8/1/79	41.6	±0.81	46.0	40.8	86	0.258
IN-9	11/1/77	42.0	±0.83	44.3	37.7	72	0.327
	4/14/78	44.9	±0.57	49.1	40.6	68	0.284
	7/31/78	36.9	±0.67	39.9	30.6	107	0.350
	9/27/78	33.7	±1.18	37.6	28.7	92	0.340
	8/7/79	45.0	±1.49	47.0	33.9	142	0.397
US-24	10/26/77	45.1	±0.45	47.7	43.0	65	0.271
	4/25/78	46.6	±0.15	49.9	46.5	73	0.145
	5/27/78	46.3	±1.01	49.7	45.2	72	0.188
	9/1/78	41.5	±0.95	42.2	39.9	99	0.250
	8/1/79	38.2	±1.17	42.3	35.0	118	0.189
US-30	10/11/77	49.3	±0.45	49.8	44.2	64	0.223
	11/17/77	49.8	±0.83	52.3	46.4	54	0.317
	5/5/78	57.2	±0.86	58.0	50.8	76	0.280
	6/8/78	60.4	±0.42	60.9	52.7	89	0.387
	9/11/78	41.6	±0.25	45.2	37.4	110	0.327
	11/2/78	43.6	±0.84	47.6	40.5	75	0.324
	7/19/79	49.9	±1.18	55.6	48.4	117	0.275
I-65	10/5/77	32.9	±0.71	33.7	31.9	79	0.417
	11/4/77	32.3	±0.77	35.0	31.1	69	0.457
	4/5/78	36.6	±0.25	39.0	36.2	77	0.381
	5/10/78	32.9	±0.93	34.3	32.0	91	0.452
	7/17/78	24.9	±0.85	28.9	23.8	113	0.411
	9/7/78	28.5	±1.05	30.1	27.1	106	0.497
	10/3/78	30.5	±0.60	30.1	29.6	73	0.546
	11/1/78	31.1	±0.43	31.7	29.1	76	0.457
	5/7/79	28.4	±1.25	29.7	26.5	100	0.337
	8/9/79	32.3	±0.86	33.3	31.3	138	0.565
I-64[a]	10/18/77	49.5	±0.93	53.3	48.9	65	0.220
	4/11/78	54.8	±0.34	56.6	53.0	69	0.278
	5/16/78	54.7	±1.39	58.4	53.1	65	0.283
	8/17/78	49.2	±0.58	49.8	46.4	97	0.382
	11/9/78	48.7	±0.70	49.6	44.6	55	0.339
	8/14/79	58.9	±1.17	60.2	54.7	90	0.575
US-20	10/12/77	54.2	±0.78	56.5	50.5	51	0.195
	11/18/77	55.3	±1.19	56.9	51.3	42	0.269
	5/9/78	59.4	±1.40	60.8	54.6	72	0.320
	9/12/78	53.1	±0.76	53.9	47.6	75	0.385
	11/3/78	51.5	±1.41	55.0	47.6	67	0.387
	5/21/79	51.1	±0.40	54.6	50.7	95	0.217
	8/23/79	60.0	±0.83	61.1	56.8	122	0.412
IN-29	4/27/78	45.8	±0.23	47.9	45.5	83	0.322
	5/19/78	44.5	±0.93	45.8	43.0	98	0.328
	8/1/78	41.9	±0.74	42.7	39.9	96	0.426
	9/25/78	41.0	±0.59	43.4	40.4	86	0.477
	10/26/78	41.6	±0.47	42.1	39.5	53	0.361
	8/3/79	39.8	±0.98	46.3	39.0	108	0.409
IN-14	4/26/78	50.1	±0.53	50.5	44.1	63	0.211
	5/22/78	48.1	±0.87	50.3	41.6	77	0.251
	8/3/78	41.9	±0.66	43.6	38.7	92	0.237
	8/1/79	50.0	±1.06	51.0	43.3	88	
US-41	10/3/77	46.0	±1.38	47.4	44.4	79	0.135
	11/2/77	44.1	±0.93	45.4	40.7	78	0.186
	8/6/79	47.6	±0.94	48.4	43.9	122	0.298
IN-37 N	10/20/77	36.4	±1.36	41.0	30.0	72	0.267
	4/4/78	45.1	±1.26	46.8	38.2	66	0.328
	8/29/78	35.8	±0.94	37.2	33.8	96	0.379
	10/6/78	31.0	±1.20	37.0	29.8	58	0.412
	7/23/79	39.5	±1.30	40.5	34.4	127	0.540
IN-37 S	10/31/77	33.5	±1.17	34.8	32.4	63	0.243
	4/28/78	40.3	±0.95	43.7	39.0	94	0.234
	7/11/78	31.5	±0.86	32.7	30.5	103	0.284
	9/26/78	30.5	±0.73	32.5	29.5	91	0.227
	7/24/79	40.3	±1.06	42.5	39.0	111	0.358
IN-2	10/6/77	54.1	±1.57	56.3	52.2	60	0.238
	11/15/77	52.5	±0.92	53.5	51.4	62	0.228
	4/3/78	57.0	±1.43	58.7	55.3	78	0.292
	6/7/78	55.5	±1.91	61.2	53.6	105	0.261
	7/10/78	49.3	±0.58	50.0	46.3	102	0.322
	9/8/78	47.4	±0.87	49.2	46.2	119	0.288
	10/2/78	51.1	±0.91	53.3	49.8	91	0.294
	11/16/78	50.7	±0.89	51.7	47.7	46	0.332
I-64[b]	10/18/77	38.7	±0.98	39.7	36.0	69	0.189
	4/11/78	42.4	±0.93	43.7	39.2	81	0.244
	5/16/78	43.3	±1.07	44.2	37.8	68	0.361
	8/17/78	35.4	±1.01	36.6	32.3	106	0.450
	11/9/78	32.9	±0.36	33.4	29.5	55	0.375
	8/14/79	37.3	±0.38	42.9	36.9	88	0.570

[a]HAE-IV. [b]Open-graded friction course.

found to be greater than 0.900; many were in the range from 0.970 to 0.999. This indicates that there is a significant linear correlation between skid number and speed. Therefore, the speed gradient as defined in this report is valid because the slope of the regression line is constant between test speeds of 40 and 60 miles/h.

As a further step in the analysis of the speed gradients, the overall average speed gradient for skid tests obtained during each visit was calculated as shown in Table 2. These overall speed gradients were then compared by ANOVA at the 0.05 level to see whether they were significantly different from visit to visit.

Results of laboratory and field testing are presented in Tables 3 and 4. Table 3 lists the polishing data obtained from tests performed on the aggregate fraction that passed the 0.5-in to no. 4 sieves by personnel at the Division of Materials and Tests by using the British polishing wheel. Also shown are data on aggregate type from the petrographic analysis by Purdue University (9). Samples from each of the six subsites were analyzed independently. Some sections, especially those of HAE type IV, show a great variety of aggregate types. In many of the sections one type may be predominant in the sampling of one of the subsites and not in one or more of the other subsites. Because of this variability, percentages are not given. Instead, the aggregate types are listed in the order of their apparent predominance. The polishing index given in the table is the difference between the average initial BPN and the average final BPN of aggregate samples from each of the subsites.

The percentage of bitumen, penetration of recovered asphalt, percentage of voids in the compacted mix, and the sand-patch texture values of Table 4 are averages of the results of tests performed in 1977 and 1978 on sample specimens and test sites from all of the subsites within the test sections. The BPN values shown in Table 4 are the highest and the lowest of the 60 readings obtained each year for each test section in the field.

DISCUSSION OF RESULTS

Level of Skid Resistance

The test sections selected for this study exhibited a range of pavement skid numbers from 23.8 to 61.8. Three of the sections--IN-2, US-20, and I-64 (HAE type IV)--produced skid numbers that were consistently at or above 50 whereas three others--I-65, IN-37S, and I-64 (open-graded friction course)--produced skid numbers in the low 30s and even into the 20s. This situation occurs almost independently of season with few exceptions. There was noticeable improvement of the low values in the spring 1978 results, which greatly reduced the spread (except for the I-65 section). Even though there was a significant improvement in the April 1978 results over the initial values of October 1977 by an average of 4 skid numbers, the outcome of tests in May 1978, only one month later, showed a loss of the improvement, so that the average value equaled the October 1977 value of 32.9.

The HAE type IV surface on I-65 consistently produced the lowest skid numbers of all of the bituminous test sections. It should be noted that consistently low skid numbers are not typical for this type of mix. In fact, it was later found that this section was "bleeding" as a result of construction problems. An obvious factor contributing to this outcome is the very high directional average daily traffic (ADT) of 8700 vehicles, which is 1.5-7 times greater than that for any of the other sections. Results found in Table 4 reveal a bitumen content that may be too high for the traffic volume, coupled with a percentage of voids in the compacted mix that is too low. The best overall average SN_{40} for this section was 36.6 (obtained in April 1978). However, the expected average SN_{40} from season to season for this particular section appears to be more in the range of 28-32.

Another section that produced low skid numbers is IN-37S, which is an HAC type B surface with stone as the coarse aggregate. The ADT across this section is approximately 5200, which ranks it as the fourth-busiest section in the study. The coarse aggregate was identified from the petrographic analysis as a pure limestone. The section showed improvement in the spring test results, but the overall average SN_{40} is more probably 31-34, especially during the critical summer season, when pavement temperature is high and traffic is heavy.

The experimental open-graded friction course section of I-64 is producing relatively low skid numbers and appears to be sensitive to seasonal changes. These results do not compare favorably with the experience reported by other states. Results of gradation tests performed on aggregate extracted from pavement core samples reveals that the fraction that passes the 0.38-in sieve is too fine and too densely graded.

A closer inspection of the sections that have produced high average skid-resistance values reveals some significant indications. First, three of the sections are experimental mixes in which slag has been substituted for gravel or stone as the coarse aggregate fraction--namely, IN-2, US-20, and US-30. In fact, all three of the slag-modified mixes have produced high skid numbers, if not the highest, in all seasons. The HAE-IV surface on I-64 also produced high skid resistance. Another significant point to consider is that three different surface types are represented by the four sections that produced the highest skid numbers in all seasons over a two-year period. In other words, it does not appear from the information available so far that mix design by itself is responsible for high skid resistance. There are many other factors such as age and ADT that must be considered that confuse the analysis. When age and ADT are taken into account,

Table 3. BPNs and polishing indices from accelerated laboratory tests.

Road	Predominant Aggregate Type	Mean Laboratory BPN Initial	Mean Laboratory BPN Final	Polishing Index
IN-3	Dolomite, limestone	37.0	28.8	8.2
IN-26	Information not available	37.0	28.0	9.0
IN-9	Limestone	34.0	30.8	3.2
US-24	Limestone, dolomite, dolomitic-limestone	38.2	29.7	8.5
US-30	Slag	42.2	30.0	12.2
I-65	Limestone, dolomite, quartzite, siltstone, sandstone, chert, granite	31.0	26.8	4.2
I-64[a]	Information not available	36.2	27.0	9.2
US-20	Slag, dolomite, limestone, schist, sandstone, granite, chert	39.2	24.8	14.4
IN-37N	Limestone, dolomite, quartzite, chert, sandstone	31.7	28.8	2.9
IN-37S	Pure limestone	36.7	29.0	7.7
IN-2	Slag	38.8	28.5	10.3
I-64[b]	Information not available	34.7	26.3	8.4

[a]HAE-IV. [b]Open-graded friction course.

Table 4. Results of laboratory and field tests.

Road	Percentage of Bitumen 1977	Percentage of Bitumen 1978	Penetration of Recovered Asphalt 1977	Penetration of Recovered Asphalt 1978	Percentage of Voids in Compacted Mix 1977	Percentage of Voids in Compacted Mix 1978	Field BPN Values, Maximum-Minimum 1977	Field BPN Values, Maximum-Minimum 1978	Sand-Patch Texture (mm) 1977	Sand-Patch Texture (mm) 1978
IN-3	5.28	4.92	24	23	10.63	7.53	55-44	54-42	0.53	0.56
IN-26	4.20	4.60	27	28	9.67	10.10	63-46	59-42	0.41	0.49
IN-9	5.48	5.37	32	32	6.21	5.22	60-46	54-39	–	0.40
US-24	5.02	5.32	22	21	10.80	9.90	62-49	60-44	0.55	0.73
US-30	5.85	6.30	22	18	5.80	6.50	67-52	70-52	0.38	0.41
I-65	6.98	7.55	38	44	3.90	4.72	81-55	64-53	0.10	0.11
I-64[a]	6.97	7.00	22	19	7.77	8.86	72-57	68-55	0.23	0.30
US-20	5.37	6.67	14	11	10.03	10.77	65-59	65-57	0.30	0.40
IN-29	–	5.47	–	34	–	2.67	–	57.44	–	0.29
IN-14	–	5.57	–	26	–	3.17	–	59-47	–	0.63
IN-37N	5.53	5.20	35	31	4.56	4.30	58-50	59-47	0.27	0.27
IN-37S	5.35	5.80	26	28	5.34	3.70	57-40	54-37	0.31	0.41
IN-2	6.16	6.16	26	25	6.19	5.58	65-51	67-52	0.39	0.46
I-64[b]	6.22	6.12	34	33	7.51	6.22	59-39	57-41	0.51	0.34

Note: Values are averages of all subsites unless otherwise noted.

[a]HAE-IV. [b]Open-graded friction course.

US-30 is readily seen as the top performer. This section has been open to traffic almost six full years and has an ADT of approximately 6000 vehicles. It is the oldest section in the study and has the second-largest ADT (after that of I-65). The only drawback to US-30 is that it also produced the broadest range of overall average skid numbers through the seasons, from 41.6 to 60.4--an indication of seasonal sensitivity.

There is an indication that the coarser surface mixes are less sensitive to seasonal changes, since both the HAC type A and the HAE type II mixes, which have the coarsest aggregate gradation, exhibited a narrow range of average skid values throughout the seasons. The open-graded friction course has a broader range of skid numbers than the HAE-II or the HAC type A, but its level of skid resistance is low in relation to the other mix types.

Pavement Temperature and Skid Resistance

A definite relationship between pavement temperature and pavement skid resistance is apparent in almost all of the test sections. However, a number of the sections show what might be considered as the classic relationship. This relationship is especially noted if one plots skid number and pavement temperature as a function of time for IN-29 (HAC type A, gravel), US-30 (slag-modified HAE type III), I-65 (HAE type IV), IN-2 (slag-modified HAC type B), and IN-26 (HAE type II, gravel). These roads are very good examples of the loss of skid resistance as pavement surface temperature increases beyond 90°F and especially above 100°F. This situation does not appear to be dependent on pavement surface type, since all mix types are included in the examples cited. As more information becomes available, it may be possible to define this relationship more precisely.

Polishing Index and Field BPN Values

Results of the accelerated polishing tests performed in the laboratory with the British polishing wheel are given in Table 3. Laboratory polishing-index information was available for only 12 of the sections. Polishing indices for the aggregates of 3 of the sections are significantly lower than the rest (2.9, 3.2, and 4.2). The predominant aggregate type of all three is limestone. On the other hand, the polishing indices of the aggregates tested from 3 other sections are significantly high (10.3, 12.2, and 14.4). The predominant aggregate type of these sections is slag. The remaining 6 sections produced values from 7.7 to 9.2.

The field BPN values are much higher than the laboratory values. The lowest field value is 7-10 numbers greater than the initial laboratory value. The field values also appear to be quite variable; however, the variability of the field values does not appear to be directly related to a particular aggregate type or mix type. A difference in maximum and minimum values of 11-26 numbers is seen in all but two of the test sections for both years; the slag-modified HAE type IV surface on US-20 (a difference of 6-8 numbers) and the HAC type B surface with gravel coarse aggregate on IN-37 (a difference of 8-12 numbers) produced the most uniform field BPN values of the sections.

A direct comparison of the 1977 and 1978 field BPN values of all sections reveals no significant difference from year to year except for the I-65 section; however, the 81 value is not felt to be representative. It is pointed out that the mean initial laboratory value for the aggregate from this section is 31.0, which is the lowest of all the initial laboratory values. The 1979 polishing data will be compared with the 1977 and 1978 values to see whether BPN values can be used with confidence to predict pavement durability in relation to skid resistance.

An interesting observation is that those sections that had the highest average skid numbers also had the highest polishing indices, and those that had low average skid resistance had low indices. It appears that those sections that possessed the capacity to lose skid numbers (the ones with the high numbers) contain coarse aggregate that has the potential to polish. Although this may be evident from the results of accelerated polishing tests performed in the laboratory, results of skid tests, which are short term, show otherwise, as expected.

A comparison of the two HAC type B sections on IN-37 reveals that the mix that incorporates the stone as the coarse aggregate is not as variable as the section that has gravel coarse aggregate in the mix. The range of average SN_{40} on three visits to the IN-37 section that has the gravel aggregate is 8-10 skid numbers, whereas the section that incorporates the stone produced a range of 2-4 numbers. Since most of the other variables such as ADT, pavement temperature, mix type, rainfall, climate, and age are very similar, the difference of aggregate type must be considered to be the probable cause of the skid variability. The surface-texture information shows the mix that incorporates stone coarse aggregate to have more texture than the mix surface that has the gravel aggregate.

The slag-modified mixes produced a noticeably larger difference of skid numbers on most test dates. The skid values for the modified HAC type B on IN-2 are less variable than the HAE type III or type IV; however, the apparent loss of pavement skid resistance from spring to summer on IN-2, which has a low ADT, is quite dramatic.

Significant loss of skid resistance from spring through summer is found in 9 of the 15 sections. This phenomenon apparently occurs independently of mix type and is to be expected. Additional testing is needed to show the extent of recovery and to verify the relationship of skid resistance and season. The level of skid resistance remained more constant from spring through summer for 5 of the sections.

A few observations from information contained in Table 4 are worthy of mention. First of all, there is evidence that may explain the occurrence of low skid resistance on I-65. The pavement-surface texture of the I-65 section as determined by the sand-patch method in both 1977 and 1978 is only 0.0040 in (0.1 mm). Compared with the other test sections, this is a very low value. Also, the percentage of voids of the compacted mix for the I-65 sections appears to be very low in relation to the other HAE type IV sections. Second, the texture values remained the same or increased for all sections except for the open-graded friction course on I-64.

Speed Gradient

The speed gradients for most of the sections remained well under the (generally accepted) limit of 0.500. Four of the surfaces produced initial speed gradients that were less than 0.200. Six other sections exhibited initial gradients of less than 0.250. One section had an initial gradient of 0.417, and four of the sections exhibited gradients of 0.540-0.575.

The HAE type IV surface on I-64 produced a very respectable initial speed gradient of 0.220 in October 1977. The section was tested five additional times and, with the exception of November 1978, the gradient increased to a value of 0.575. The open-graded friction course on I-64 produced an even better initial speed gradient of 0.189 in October 1977. Again, the speed gradient increased on all succeeding visits except for one (November 1978) to a value of 0.570. The HAC type B surface with gravel on IN-37N started at 0.267 and continually increased to 0.540 over five visits. The HAE type IV surface on I-65 had an initial value of 0.417. This section produced varied speed gradients over 10 visits. The lowest speed gradient was a 0.337 in May 1979. A 0.546 was calculated from the October 1978 tests, and the highest gradient for the section was 0.565, achieved in August 1979.

Two of the sections produced very uniform speed gradients over the testing period. The HAC type A surface on IN-14 and the HAE type II surface on IN-26 had similar initial values. The maximum differences between their highest and lowest average speed gradients were 0.040 and 0.087, respectively.

Many sections produced improved speed gradients in the spring, and some showed improvement in the fall. The ANOVA of the speed gradients was used to see whether the average speed gradients from each set of tests were statistically similar or significantly different from visit to visit. A significant difference in G_{40-60} could imply that the surface was sensitive to seasonal changes. The following summarizes the results of the analysis of the speed gradients:

1. All three of the HAE type III surfaces produced speed gradients that did not vary significantly from season to season.
2. All three of the HAE type IV surfaces produced speed gradients that were significantly different from season to season.
3. Two of the three HAC type B surfaces exhibited speed gradients that did not vary significantly with season; only the section that contained the gravel was significantly different from season to season.
4. Of the two HAC type A surfaces, the section that contained stone showed no significant difference, but the section that had gravel was found to have speed gradients that were different from season to season.
5. Of the two HAE type II surfaces, the section that had gravel produced speed gradients that were not significantly different from season to season, but the speed gradients of the section that contained stone were found to vary significantly from season to season.
6. The speed gradients of the open-graded friction course were found to vary significantly with season.

SUMMARY OF RESULTS

It is felt that conclusions at this time would be premature. Trends are apparent from the data obtained so far. A lot of factors are involved. Additional data and further evaluation and analysis will definitely help to define and refine the relationship of the many factors involved in pavement skid resistance.

The following statements summarize the major results of the initial analysis of data that have been obtained halfway through the study:

1. It was found that five tests at each test speed are necessary to give a reliable estimate of the true average skid number of a pavement surface because of the inherent variability of the SMS.
2. The 15 bituminous test sections, representing six different mix designs and incorporating gravel, stone, or slag as the coarse aggregate fraction, produced skid numbers as high as 61.8 and as low as 23.8 between October 1977 and July 1979.
3. A high linear correlation between skid resistance and speed was found between test speeds of 40 and 60 miles/h; all but a very few of the correlation coefficients were greater than 0.900.
4. It does not appear from the available data that skid resistance is a function of mix design alone.
5. Skid resistance is improved when slag is the predominant aggregate type, regardless of mix design.
6. With few exceptions, skid resistance is highest in the spring and lowest in the summer, irrespective of mix design and aggregate type.
7. Skid resistance appears to be a function of the temperature of the pavement surface, but the exact relationship remains undefined.
8. The skid resistance of bituminous surfaces appears to be less susceptible to short-term temperature changes in the summer than in the spring and fall.
9. Results of the laboratory-accelerated polishing test indicate that slag has a greater potential for polishing than either limestone or dolomite and that limestone has the least polishing potential of the three.
10. The significant variability of the speed gradients from season to season for all of the sections of a certain type of surface and the uniformity of the speed gradients of all of the sections of another type of surface might be used to show that certain surface types are sensitive to seasonal changes.
11. It should be realized that the very small number of sections selected in this study to represent bituminous surface types common to Indiana may not, in fact, be representative or typical.

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The contents of this report reflect our views, and we are responsible for the facts and accuracy of the data presented herein.

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Wet-Pavement Friction of Pavement-Marking Materials

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A total of 39 formulations of 11 types of marking materials were studied in the laboratory and in the field. Field skid number measurements were made at three sites by using the Pennsylvania Transportation Institute Pavement Friction Tester. Laboratory and field measurements were taken to determine British pendulum numbers, microtexture, macrotexture, and static coefficient of friction. Laboratory polishing and accelerated exposure testing were also performed. A wet-friction data bank for typical pavement-marking materials was established. Based on an analysis of these data, it was found that wet friction can vary dramatically for different marking materials. The texture of the underlying pavement affects the friction of thinner marking materials (paints), but the wet-friction resistance of the thicker marking materials is unaffected by the texture of the underlying pavement. Reductions in skid resistance from the paints can persist even after the aggregate surface is exposed from wear. The wet friction of marking materials exhibits a daily and seasonal variability much like that of the pavement itself, and the variability must be accounted for when skid-resistance measurements are made.

Many types of materials are used for pavement-marking purposes. Although these materials are routinely specified and tested in order to control their durability and visibility, little is known about their friction properties. The friction, or skidding resistance, of pavement surfaces has received considerable attention in recent years; this has resulted in improved test procedures and in surfaces that are more skid resistant. Well-marked pavements and highly skid-resistant pavement surfaces are both important safety features. However, the differential skid resistance that can develop between marked and unmarked pavement surfaces may, in certain circumstances, be a potential safety hazard.

The purpose of this paper is to report on the results of an extensive study of the wet-pavement friction of all types of pavement-marking materials. Both laboratory and field data are presented for a variety of pavement surfaces. The results are discussed in terms of the different levels of friction that can be developed by the different marking materials and the implications of these differences.

WET-PAVEMENT FRICTION

The texture of a road surface is the most important characteristic that provides resistance to skidding in wet weather (1). For the purpose of designing surfaces for good skid resistance, it is convenient to define two texture classifications: macrotexture, which is related to the gradation of the coarse aggregate, and microtexture, which is determined by the surface characteristics of the exposed aggregate particles. It has been shown (2) that, for skid-resistance considerations, microtexture consists of that portion of the surface spectrum that has surface asperities <0.5 mm (0.02 in) and macrotexture consists of the portion that has surface asperities >0.5 mm.

Pavement skid resistance is quantified by its skid number (SN), which is defined as the ratio of the friction force and the vertical load that results when a locked wheel slides along a wetted pavement at a velocity V. The procedure for measuring the SN is specified by ASTM E274. For pavements it has been shown that the skid resistance decreases with speed according to the following relationship (3):

$$SN = SN_0 \exp[-(PNG/100)V] \tag{1}$$

where SN_0 is the zero speed intercept and PNG is the percentage of the normalized gradient, which is defined as

$$PNG = -(100/SN)/(dSN/dV) \tag{2}$$

It has also been shown that SN_0 can be predicted by microtexture or British pendulum number (BPN) measurements as specified in ASTM E303. The value of PNG, which determines the rate at which skid resistance decreases with speed, is determined by the macrotexture of the pavement as measured by profile analysis or by the sand-patch method (4).

EFFECTS OF MARKING MATERIALS ON SKID RESISTANCE

Pavement texture is altered at those locations where pavement marking materials are applied. As a result, there will be a difference in skid resistances between the marked and the unmarked areas of the pavement. Problems may arise when the friction forces available to all tires on a vehicle are not equal. If emergency stops, or other maneuvers, are attempted when the friction forces are significantly different at one or more of the tires, the directional control of the vehicle may be lost.

Marking materials of various types are applied in thicknesses ranging from 0.4 to 3 mm (0.016-0.125 in). Thinner applications, typical of traffic paints, permit the macrotexture to continue to show through to some extent but may alter the microtexture significantly. A surface dressing of glass spheres added for reflective purposes may also provide some texture until the spheres are worn away. At the other extreme, the thick materials such as the thermoplastics obliterate the pavement microtexture and macrotexture completely. After the surface dressing of glass spheres on thermoplastics has worn away, little texture remains, except for the texture provided by the aggregate used as a filler in the formulation of the thermoplastic.

TYPES OF MARKING MATERIALS

Conventional traffic paints applied in a typical wet-film thickness of 0.5 mm (0.020 in) have a thickness of 0.3-0.4 mm (0.012-0.016 in) when dry. Glass spheres are applied to the wet paint to provide reflectivity. In some cases, finer glass spheres are also premixed with paint. Traffic paints are formulated with an alkyd resin or a chlorinated rubber base and are designed to dry by solvent evaporation or by solvent flash evaporation when the paint is heated above the normal boiling point of the solvent prior to spraying. Glass beads are either applied under pressure (by using a pressurized glass spray gun) or are dropped by gravity through a glass dispenser onto the wet marking material.

In order to achieve longer-lasting markings, thick plastic materials (polyvinyl chloride or ethyl cellulose) were introduced in the early 1950s. These materials are applied with an adhesive that may or may not be preapplied to the material. They also may be rolled during construction into the surface of flexible pavements in the final compaction. The preformed plastic, or cold-applied plastic, materials are currently available in thicknesses that range from 1.5 to 3 mm (0.06-0.125 in).

During the past 10 years, the use of thermoplastic materials has increased in the United States. These materials are applied in a molten state, either extruded onto the surface or sprayed under pressure. Glass and hard aggregate are included in the formulation to provide reflectivity and structural integrity to the material. Glass spheres are also applied to the surface while the material is still molten. Thermoplastics are applied at film thicknesses of 2.3-3.2 mm (0.090-0.125 in). These materials flow into the texture of the pavement, fill the irregularities of the surface, and cover the pavement aggregate completely. As in the case of the cold-applied plastic, the hot-applied thermoplastic produces a surface that, when newly applied, has its own frictional properties independent of the surface over which it is applied.

"Temporary" tapes, originally designed only for temporary use during construction projects, detours, etc., are also used for more permanent applications. These materials include aluminum strips that are painted and glass-sphere reflectorized on one side and provided with an adhesive on the other. Such tapes conform to the macrotexture of the pavement and allow the macrotexture to show through.

Two-component, chemically set materials are being used to a limited extent at present. New epoxy-polyester materials applied in the medium-thickness range of 0.75-1.5 mm (0.030-0.060 in) show promise for durable pavement-marking lines. The development of equipment for their application is needed before these materials will see widespread use.

MEASUREMENT OF FRICTION OF PAVEMENT-MARKING MATERIALS

Measurement of the skid resistance of marking materials by using full-scale methods on actual installations is not possible with conventional skid-testing equipment. However, by using the Pennsylvania Transportation Institute's Pavement Friction Tester, a standard locked-wheel tester that has very high response instrumentation, it is possible to obtain SNs on very short installations. Installations 30 cm wide by 6 m long (1x20 ft) were placed on a variety of pavements and measured at speeds that ranged from 50 to 80 km/h (30-50 miles/h). Test methods appropriate for small areas include texture profile measurements and British pendulum testing.

Also of interest, particularly at crosswalks of city streets, is the pedestrian slip resistance. To evaluate the safety from the pedestrian's standpoint, the static coefficient of friction is required (5). A device developed at the National Bureau of Standards (NBS), the NBS-Brungraber Portable Slip-Resistance Detector, is applicable to measurements on small samples and can be taken into the field for testing on actual installations.

TEST MATERIALS

Eleven different marking materials in a total of 39 formulations were evaluated in the laboratory and in the field. Material variables included in the study were

1. Type of marking material (such as paint, thermoplastic, cold applied),
2. Pigment color (white or yellow), and
3. Surface (beaded or unbeaded).

The different types of materials that were included in the study are summarized below.

Material	Code	Number of Formulations Studied
Conventional alkyd paint	AC	2
Conventional chlorinated rubber paint	CC	2
Alkyd quick-dry paint	AQ	2
Chlorinated rubber quick-dry paint	CQ	2
Alkyd paint with premixed glass beads	AP	2
Chlorinated rubber paint with premixed glass beads	CP	2
Hot-extruded thermoplastic	HE	5
Hot-sprayed thermoplastic	HS	3
Cold-applied plastic	CA	10
Temporary tapes	TT	5
Two-part epoxy-polyesters	TP	4

Emphasis was placed on the hot-sprayed, hot-extruded, and cold-applied materials because they are being used extensively, are long lived and, because of their thickness, have the greatest potential for producing low levels of friction.

Each material was evaluated both in the laboratory and in the field. Three field-test sites were used. Seventeen marking materials were applied in and out of the wheel tracks on PA-45, a six-year-old dense-graded asphalt pavement that has an SN_{40} [SN at 40 miles/h (64 km/h)] of approximately 40. Eight materials were placed in and out of the wheel tracks on PA-871, a four-year-old portland cement concrete pavement that was finished with longitudinal brooming. The SN_{40} for this pavement is approximately 55. The third test site was the Pennsylvania Transportation Research Facility test track, which has different surfaces that range in SN_{40} from 30 to 65. The surfaces include portland cement concrete, an open-graded friction course, a dense-graded asphalt concrete, and a surface coated with Jennite.

Commercial application equipment was used to apply the paints, the two-part epoxy-polyesters, and the hot-sprayed and hot-extruded thermoplastics. The cold-applied materials and the temporary tapes were simply pressed into place. The field-test stripes were 152 mm (6 in) wide by 6 m (20 ft) long. In order to make the laboratory samples as representative of the field samples as possible, the paint and the hot-sprayed and two-part materials were sprayed onto panels placed on the pavement just ahead of the field test stripes. This was not possible with the extruded materials, which were extruded in the laboratory by using material from the same production batches as those used in the field.

Four different laboratory panels were used to provide a variety of surface textures. Duplicate panels were prepared for each material tested. The majority of the panels were 16-gage galvanized steel plates 152 mm (6 in) long by 102 mm (4 in) wide. A limited number of panels were made in the laboratory with (a) broomed portland cement concrete, (b) coarse-textured asphalt concrete and, (c) fine-textured asphalt concrete. These surfaces were designed to simulate texture extremes that might be encountered in the field.

TESTING PROGRAM

The testing program was designed to provide information about the friction resistance and texture of the marking materials. Specific test procedures included

1. SN measurements (ASTM E274) at 48, 64, and 80 km/h (30, 40, and 50 miles/h) at all field sites;
2. Use of the NBS-Brungraber Portable Slip-Resistance Detector at all field sites and for all laboratory panels;
3. Microtexture and macrotexture profile measurements at selected field samples and for selected panels;
4. BPN (ASTM E303) at all field sites and for all laboratory panels; and
5. Atlas Twin-Arc Weatherometer exposure on selected laboratory panels followed by Brungraber, BPN, and texture measurements.

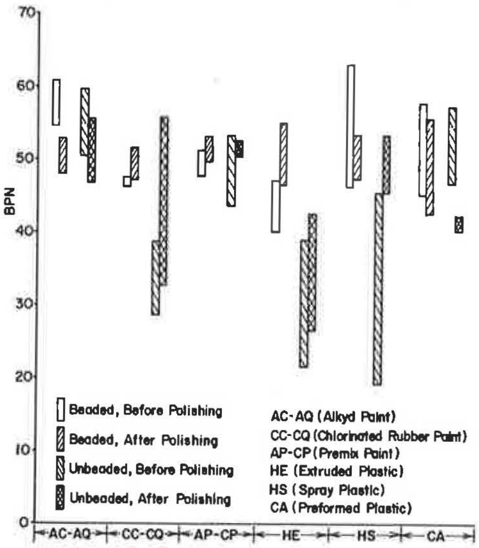
SN_{40} measurements were obtained on 14 different days in the fall of 1978 and the spring of 1979 in order to determine seasonal and daily variations in skid resistance. SN_{30}, SN_{40}, and SN_{50} data were obtained on several different days in order to establish PNGs for each of the field-test stripes. The field BPN, Brungraber, and texture data were taken in the fall of 1978 and the spring-summer of 1979.

In order to simulate wear in the field, all of the laboratory panels were subjected to polishing by using the Pennsylvania State University Reciprocating Pavement Polisher. Preliminary testing indicated that terminal polishing was obtained after 3000 passes with 30-μm silica grit. All the panels were subsequently tested by means of this polishing sequence. BPN and Brungraber data were obtained for all of the panels before and after 200 h of weatherometer exposure. After exposure the panels were polished and the BPN, Brungraber, and texture data obtained.

RESULTS

BPN data for the metal plates are shown in Figure 1. Each bar shown in the graph represents data from duplicate test panels. Because the yellow and white materials were not significantly different, data for the white and yellow formulations were combined. Therefore, each bar in Figure 1 represents data from a minimum of four panels. Five different materials are represented by the hot-extruded plastics (HE) data, 3 materials by the hot-sprayed plastics (HS) data, and 10 materials by the cold-applied (CA) data. The temporary tapes (TT) and CA materials are not shown because they are not readily classified as beaded or unbeaded.

Figure 1. BPN results for steel laboratory panels.



The BPN data for most of the beaded materials in Figure 1 are in the range of 50 ± 5, both before and after polishing. It is apparent that, when beaded materials are applied to a smooth substrate, the characteristics of the beads predominate and their characteristics are not affected by polishing. There is little to be learned about beaded marking materials when they are applied to a smooth substrate.

The data for the unbeaded surfaces in Figure 1 reflect the friction of the marking material itself. The low values before polishing shown by the chlorinated rubber are difficult to explain; however, this trend was observed in the field. Similarly, there is no adequate explanation for the increase in the BPN data for the various chlorinated rubber paints (CC and CQ) after polishing. The low BPN results for the HE are also surprising, but again similar trends were observed in the field. The increase in the BPN results after polishing for the HS is undoubtedly the result of exposing the filler in the HS material.

In summary, Figure 1 indicates that some marking materials, notably the chlorinated rubber and the HE, have inherently low BPN values. As long as the beads are not worn from the surface, polishing has little effect on the BPN of the different materials. Finally, if filler materials are exposed during laboratory polishing, an increase in skid resistance may result, as shown with the HS.

The results presented in Figure 2 allow a comparison of the effects of surface texture on the BPN of the different beaded marking materials. Once again, little variation is observed between the different materials; the average holds at about 50 ± 5 points. The large BPN values obtained for the CC, CQ, AP, and CP paints on the coarse asphalt concrete surfaces were expected because the paint was too thin to fill the texture of the surface. Two results that are hard to explain are the reduced BPN values (by 10 points) obtained on the fine asphalt concrete for the AC, AQ, CC, and CQ materials. One possible explanation is a partial dissolving of asphalt on the surface; this process reduces the friction of the paint. Otherwise the surfaces in Figure 2 reflect the friction of the glass beads (average BPN value of 50 ± 5).

Figure 2. BPN results for beaded materials on surfaces with different textures.

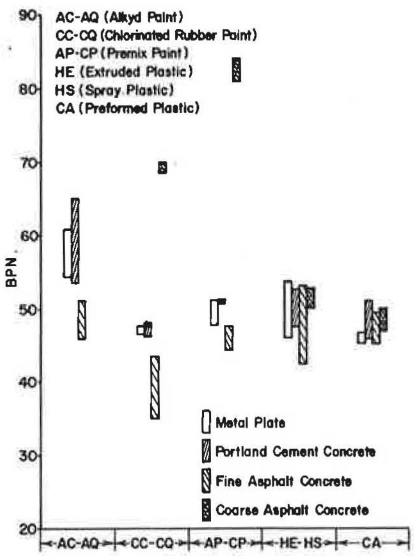


Figure 4. BPN results for panels subjected to weatherometer exposure.

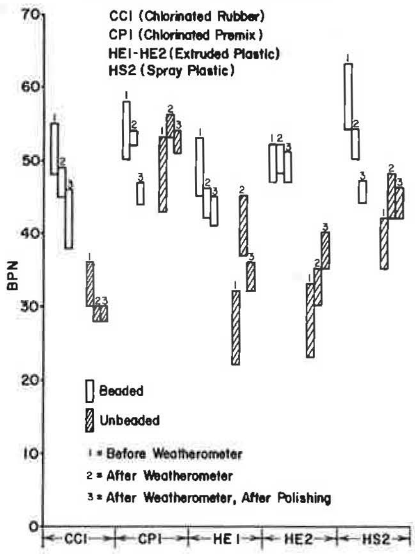


Figure 3. BPN results for unbeaded materials on surfaces with different textures.

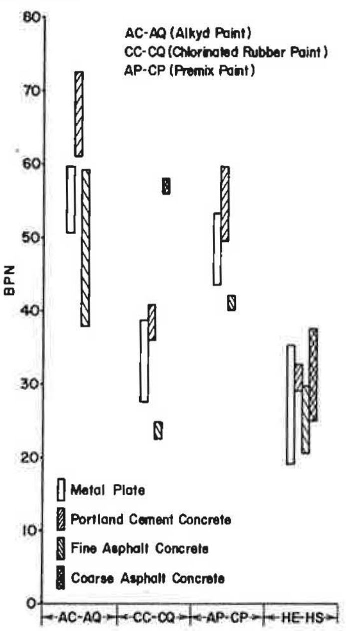
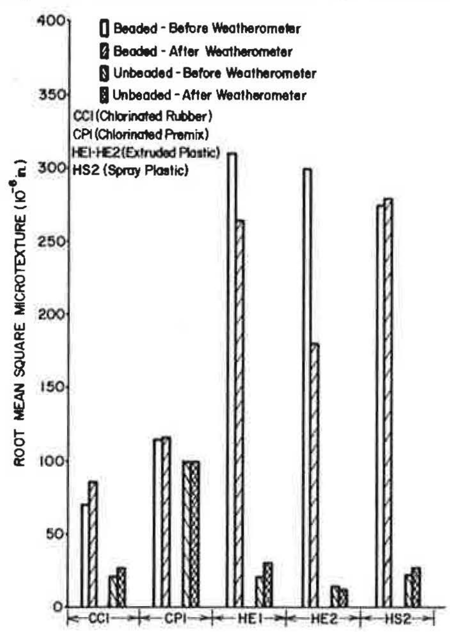


Figure 5. Texture for weatherometer samples before and after exposure.



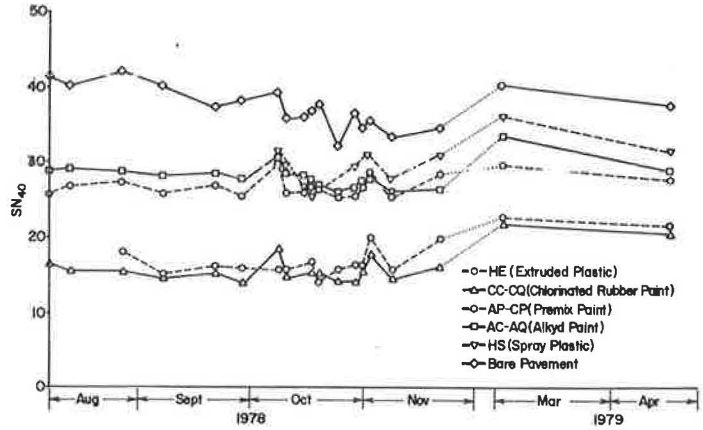
The BPN results presented in Figure 3 are for unbeaded materials applied to the four laboratory surfaces. These unbeaded surfaces show a greater variability than do the corresponding beaded surfaces of Figure 2. Texture effects are more pronounced for the paint materials; however, as expected, the thick thermoplastic materials are not affected by the texture of the surface. A decrease in BPN for the paints applied to the fine-textured asphalt-concrete surface is also apparent for the unbeaded surfaces.

The results presented in Figure 4 were obtained from steel panels after 250-h exposure in the Atlas Twin-Arc Weatherometer. BPN data were obtained before exposure, after exposure, and after exposure and polishing. Both beaded and unbeaded surfaces were tested. Visually, little change was obtained in the surfaces as a result of weathering. There was no observable loss of beads during exposure. Although some change occurred in the BPN results after exposure, the changes are not considered significant. Changes that resulted from polishing were approximately the same as for the unweathered plates after polishing. Only the HE formulation 1 (HE1) increased significantly after weathering, but the increase was largely lost after polishing. Based on the results shown in Figure 4, weatherometer data do not appear to be required in order to specify the friction of marking materials.

Microtexture data for the weatherometer samples, before and after exposure, are given in Figure 5. In general, little change in texture occurred during weathering, and the texture of the beaded surface was much higher than that of the unbeaded surfaces. Figure 5 also substantiates the hypothesis that weatherometer data are not required in order to specify the friction of marking materials.

The SN_{40} results shown in Figure 6 were

Figure 6. Daily variations in SN_{40} on PA-45.



obtained over a period of nine months that started just after the application of the marking materials. In this case the SN_{40} for the unmarked control surface averaged just under 40. The lowest SN_{40} values were obtained for the CC and CQ paints and, in spite of nine months' exposure, little improvement in skid resistance is shown. This result is very surprising in view of the fact that those paints did not obscure the texture of the pavement and that during the nine-month period there was considerable wearing away of the paint. In fact, the skid resistance of the CC and CQ paints was almost identical to that of the HE material.

The skid resistances of the AP, CP, AC, AQ, and HS materials were all less than that of the control surface. None of the marking materials approached the unmarked control surface even after nine months' exposure, indicating that reduced skid resistance is a long-term effect even for the relatively thin paints.

Both seasonal and daily variations were observed for the marking materials, as indicated in Figure 6. The seasonal trends for the marking materials appear to follow the same trend as the control surface; however, the daily trends appear somewhat mixed. Time of testing does appear to be an important consideration for marking materials and is probably associated with changes in local weather conditions.

Further research is needed to explain the wide variations in skid resistance among the various materials. Such explanations would aid in the more important task of developing marking materials that are more skid resistant.

Based on the observations made during the research study, it can be concluded that

1. Different marking materials have different characteristics and this can affect skid resistance;
2. Friction of beaded surfaces is determined primarily by the beads, even for relatively thin materials such as paints;
3. Reductions in skid resistance, even for the relatively thin paints, are not confined to the time period just after application but may last over a relatively long period in spite of considerable surface wear;
4. Accelerated exposure testing is not helpful in specifying the friction of marking materials;
5. Effects of daily and seasonal variations in the skid resistance of marking materials must be accounted for in making skid-resistance measurements; and
6. Certain marking materials, because of their low skid resistance, may be a safety hazard if applied over large areas, such as gore areas.

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