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Foreword

This Record will be of interest primarily to researchers in asphalts and asphalt mixtures and to materials engineers and others responsible for specifying asphalt mixtures for highways, city streets, and airports.

Lottman et al. used test data from moisture damage test sections of NCHRP Projects 4-8(3)/1 and 4-9(4) in two relative life mechanistic models to predict wet performance lives. Core properties obtained during a 10-year period were used to evaluate in situ wet lives of test sections. Comparisons indicate that predicted and in situ lives are similar.

Ishai and Nesichi discuss the development of a moisture damage sensitivity test for bituminous paving mixtures under hot and humid climatic conditions. Suggestions for further research are outlined.

Parker evaluated stress pedestal, boil, and indirect tensile tests for assessing the stripping potential of asphalt concrete mixtures. The tests were applied to mixtures of five aggregate combinations, asphalt from two sources, and three antistripping agents.

Al-Ohaly and Terrel discuss possible mechanisms by which improvements in adhesion may occur when mixtures are exposed to microwave energy and report results of resilient modulus and split tension tests conducted on mixtures prepared in the laboratory using a convection oven and a kitchen-type microwave oven.

Yoon and Tarrer conducted a laboratory investigation to relate some measurable properties of aggregate to the stripping propensity of a mix of aggregate and asphalt cement. Each aggregate was characterized in terms of its physical properties and its chemical and electrochemical surface properties. Stripping propensity was determined by using a boiling water test.

Coplantz and Newcomb compared four asphalt concrete water sensitivity test methods by ranking the relative resistance to water-induced damage of a variety of field-prepared mixtures obtained during construction.

Guin et al. report on an asphalt binder material, produced by catalytic hydroliquefaction of coal, that was functionally equivalent to petroleum asphalt. This synthetic coal-based asphalt, with approximately 3 percent polymer additive, met all ASTM and AASHTO specifications for asphalt paving cements, with the exception of the optional thin-film oven test weight loss specification.

Jennings discusses 20 test sections constructed in south-central Montana to compare the performance of asphalts from the state's four refineries alone and with several additives, including hydrated lime, fly ash, and an antistripping agent. Sections that contain carbon black, ChemCrete, an 85-100 grade asphalt, and a blend designed by the high-performance gel permeation chromatography model to resist cracking are also covered.

Thenoux et al. present an evaluation of the Corbett-Swarbrick procedure for separation of asphalt into four generic fractions and some of the solutions the research team adopted to major difficulties encountered with implementation of the test.

Glover et al. used gel permeation chromatography, the Corbett analysis procedure (ASTM D 4124-82), and the Heithaus test for component compatibility to characterize asphalts and to investigate the relationships between their chemical properties and physical properties and performance.

Thenoux et al. present the results of a study in which physical and fractional properties of original and recovered asphalts were obtained. Relationships between fractional components and physical properties were examined in detail. Relationships between fractional components and temperature susceptibility were also examined.

Schilling and Schreuders describe improved rapid-setting cationic and anionic emulsifiers, based on condensation products of polymeric ethylene amines with polyfunctional fatty acids, for bituminous emulsions suitable for slurry seals.

Pederson and Schuller present an evaluation of the use of emulsified asphalt in a new process called microsurfacing, which was developed in Germany and first used in the United States in late 1980. Natural latex rubber is incorporated into the asphalt emulsion and mixed with aggregate and other additives in a traveling pug mill.

Takallou and Hicks describe a rubber-modified asphalt mixture used in the United States. Mix ingredients and typical properties are presented, and the last section of the paper contains guidelines for use of rubber-modified asphalt mixtures in cold, moderate, and hot environments.

Krater et al. report on test strips of pavement, which contained seven combinations of a polyolefin and a styrene-butadiene rubber, laid in five states. Data were collected from preconstruction testing on laboratory-compacted samples and from pavement condition surveys of each site. Data on construction techniques were recorded, and postconstruction data from both field-mixed and laboratory-compacted samples and cores were collected.

Kim and Burati discuss the probability of failure of a structure as a function of load effect and resistance of the structure. Load effect in a pavement structure is defined as the radial stress induced by traffic loading; resistance is defined as layer strength.

Siddiqui et al. identify key equipment-related factors associated with inconsistencies in Marshall test results obtained using different compaction and testing equipment.

Kim et al. evaluate the mix design process used by the Oregon State Highway Division. Aggregate from four different sources was used. Creep test results indicate that current mix design criteria are probably inadequate for producing mixtures capable of withstanding high tire pressures and for identifying potentially highly deformable mixtures.

Baladi et al. show that indirect tensile tests can be used to infer the structural properties of compacted asphalt mixes without the need to use other expensive test apparatus. In another paper, these authors discuss improved relationships between some fundamental mechanical properties and mix variables and demonstrate that mix properties of asphalt mixes can be predicted from a knowledge of several parameters of compacted asphalt mix, magnitude of applied load, and test temperature. A new indirect tensile test apparatus was designed, fabricated, and tested.

Aunan et al. determined which laboratory compaction procedure best reproduces the pore size distribution of field-rolled bituminous sand mix surface courses.

Gemayel and Mamlouk conducted a detailed laboratory study of the effects of different mix components on several engineering properties of open-graded asphalt mixtures. Properties studied included density, air voids, Marshall stability and flow, resilient modulus, tensile strength, and permeability. Properties of the open-graded mixture were compared with those of dense-graded mixtures and open-graded cores from an experimental test section of porous pavement.

Mamlouk and Sarofim provide a rational understanding, based on principles of strength of materials, of the moduli commonly used for structural evaluation of asphalt mixtures. Young's, shear, bulk, complex, dynamic, double-punch, resilient, and Shell nomograph moduli are discussed, and their assumptions and limitations are evaluated.

Khedaywi investigated the effect of oil shale ash on properties of bituminous concrete. Oil shale ash was added to bituminous mixtures in different ratios, and Marshall stability and immersion and resilient modulus tests were performed.

Methods of Predicting and Controlling Moisture Damage in Asphalt Concrete

ROBERT P. LOTTMAN, LARRY J. WHITE, AND DOUGLAS J. FRITH

The basis for using mechanical property ratios to predict moisture sensitivity in asphalt concrete is discussed. Also discussed are (a) physical property ratios, which depict specific types of pavement distress such as fatigue cracking and are calculated from the wet accelerated conditioned and dry mechanical properties, and (b) mathematical models to predict field-developed wet performance life, which are based on relative life computational methods and incorporate the physical property ratios. Initial test data from the moisture damage test sections of NCHRP Projects 4-8(3)/1 and 4-8(4) are used in two relative life mechanistic models developed at the University of Idaho to predict wet performance lives. Periodic core properties obtained during a 10-year period are used to evaluate in situ wet lives of the test sections. Comparisons indicate that predicted lives and in situ lives are similar. Implications are that additional built-in complexity in moisture damage models may not be needed in the near future. Control of moisture damage is more reliably achieved through the application of both indirect tensile strength and resilient modulus cutoff ratios, which are readily calculated from prediction models. These ratios are dependent on strength and modulus test data, pavement location, and performance requirements. There is reason to think that a high reliability of zero moisture damage can be achieved in the field when cutoff ratios are exceeded.

At least one-half of the state highway agencies in the United States are experiencing moisture damage in asphalt concrete pavements. The agencies have established research projects to evaluate both the extent of the damage and laboratory methods to predict the associated moisture sensitivity of asphalt concrete mixtures before paving. In addition, National Cooperative Highway Program (NCHRP) Project 4-8(4), a 10-year field evaluation of moisture damage in test sections constructed by six U.S. state highway agencies, ended in November 1986 and completed the field study of NCHRP Project 4-8(3)/1, which began after the 1971 laboratory phase [Project 4-8(3)] (*1–4*). Therefore it is timely to relate the highlights of this and current research to the technical process required for a solution to the moisture damage problem.

Technological applications based on laboratory tests and field observations are needed to eliminate life-robbing distress in pavements caused by moisture damage. The intention is to provide a focus for these applications. Moisture sensitivity ratios (i.e., moisture damage ratios) for asphalt concrete mixtures are defined and discussed. These ratios are calculated from indirect tensile strength and resilient modulus laboratory tests and form the basis of all methods currently used to evaluate moisture damage. The authors' opinion is that mathematically based field models, which predict pavement wet performance life and determine laboratory cutoff ratios, will be routinely used to control moisture damage. Because this is a new technology, the physical concepts of current models are discussed. Pavement wet life predictions and evaluation based on periodic core tests of the NCHRP projects are presented to demonstrate results from two field models.

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RATIOS

Definitions of ratios and a summary of their applications are presented in this section.

Minimum Ratios

A given asphalt concrete mixture is represented by test specimens that possess realistic dry pavement characteristics. Specimens that match initial pavement cores are fabricated according to properly simulated field aging and compaction methods. When the specimens are subjected to accelerated moisture conditions (wet) in the laboratory, they should ideally have one wet/dry ratio corresponding to each mechanical property required. This ratio is desirably a minimum ratio, one that is descriptive of the basic, realistic "maximum" moisture sensitivity of a particular asphalt-aggregate combination, aged and compacted for the average field condition. This is to say that the minimum ratio is location independent, and thus it is a specific mixture characteristic. The practicality of this is that only one, or possibly two, laboratory accelerated conditioning method needs to be employed. This will eliminate the need to solve, at the local level, the difficult laboratory testing problem of finding a moisture conditioning scheme that matches the magnitude of the ratio to the climate and other characteristics of the pavement location.

Use of the minimum ratio in a pavement wet life mathematical model will solve the location-specific problem. The correct sensitivity of the model to climatic and other location-specific conditions is easier to develop and adjust than are laboratory accelerated conditioning methods. For example, minimum ratios obtained from the laboratory accelerated conditioning method are not always reached in the field in mild climates. An acceptable model should also demonstrate this.

Moisture Sensitivity

The mechanical property ratio (or physical property ratio), wet/dry, is inversely proportional to the moisture sensitivity of a mixture. The lower the ratio, the higher the sensitivity. This is a comparative approach. Many daily decisions are based on the comparative approach, so the ratio for a given mixture has valid application.

However, difficulty arises when the performance of two mixtures, each having a different ratio and a different dry mechanical property, needs to be compared. The source of difficulty is the current inability to know the difference in predicted field performance of the mixtures as reflected by their dry mechanical properties. The range of dry indirect tensile strength (dry ITS) of all asphalt paving mixtures in the United States is at least 40 to 180 psi at 55°F, yet each mixture meets some agency's minimum stability requirements and is deemed satisfactory. Relating pavement life to dry ITS (and other dry mechanical properties) is a pavement design problem that involves the interaction of mixture properties. It requires an absolute solution rather than a comparative one. When this problem is solved, moisture sensitivity (i.e., ratio) will be performance documented, and the need for complete comparison of different mixtures will be satisfied as well.

An illustration of the comparative approach using ratios follows. Suppose two mixtures are being compared; one is a control (or reference) mixture and the other is an additive- or modifier-treated mixture. Assume their ITS values are

- Control: dry ITS = 100, wet ITS = 60
- Treated: dry ITS = 85, wet ITS = 75

The indirect tensile strength ratios (TSRs), wet/dry, are control TSR = 0.60 and treated TSR = 0.88.

Clearly the treated mixture's moisture sensitivity is less than that of the control mixture because its TSR is greater. If there exists no definite evidence that the control mixture's dry field life, based on its dry ITS of 100, is better than that of the treated mixture, based on its dry ITS of 85, then the treated mix is a much better choice.

However, if evidence does exist that field life is proportional to dry ITS, then the ratio must be reflective of the control dry ITS. In this case, a combined TSR is calculated and it is equal to (Treated wet ITS/Control dry ITS) = 75/100 = 0.75 or, in a basic form, (Treated dry ITS/Control dry ITS) × (Treated wet ITS/Treated dry ITS) = 85/100 × 75/85 = 0.75. In the basic form, the first term is the "modifier effect" ratio and the second term is the "moisture sensitivity" ratio. The combined TSR of 0.75 is greater than the control mixture's TSR of 0.60, but it is less than the treated mixture TSR of 0.88. Thus this treated mixture has less overall performance advantage when described by the combined TSR of 0.75 instead of the individual TSR of 0.88.

Suppose the mixture is treated differently and, as a consequence, develops higher ITS values (e.g., dry ITS = 120 and wet ITS = 106). This mixture's TSR is 0.88, the same as that of the previous treated mixture. However, its combined TSR is 106/100 = 1.06, which is much better than the previous combined TSR of 0.75. A conclusion to be drawn from the combined TSR comparison is that this treated mixture's predicted wet performance is not only superior to that of the dry control mixture, it is also better than that of the previous treated mixture with the lower dry ITS.

Thus the comparative basis from which a ratio is calculated and used should be accompanied by knowledge of the difference in field performance of mixtures with different dry mechanical properties: does the increase of a mechanical property really improve toughness or durability?

The lack of specific answers to this question does not necessarily rule out using the dry properties of the control mixture as a reference. The combined TSR or combined resilient modulus ratio (MrR), or both, may be adequate for comparison of moisture sensitivity of two mixtures.

In addition, the use and philosophy of ratios can be built on to develop comparative (i.e., relative) wet-to-dry performance life prediction models. An interim step is the development of physical property ratios.

Physical Property Ratios

Physical property ratios, as defined here, characterize an asphalt concrete's working stress-strain moisture resistance to a specific field distress such as fatigue cracking or wheelpath rutting. These ratios are calculated from combinations of basic mechanical properties and are required in models that predict field performance life.

Currently it is practical to use ITS and resilient modulus (Mr) as basic mechanical properties and to determine moisture sensitivity from their ratios (TSR and MrR, respectively). It is expected that the moisture sensitivity depicted by the physical property ratios will not be equal to TSR or MrR for a given asphalt concrete mixture. However, it will be shown later that the mechanical properties ITS and Mr and their ratios remain basic mixture properties required for the control of moisture damage.

Physical property ratios can be used as individual ratios or as combined ratios. The physical property combined ratio appears to be more valid than the mechanical property combined ratio. Examples of physical property ratios follow.

Fatigue Life Ratio

Fatigue life ratio (FLR) is related to asphalt concrete cohesive life evidenced by the onset of wheel load–associated cracking in the asphalt concrete pavement layer. It is proportional to resistance to fatigue cracking and is developed from the mechanics of materials relationship for the relative position of the wet and dry strain fatigue strength lines, the wet and dry pavement layer strains, and their intersections. The relative positions of the two lines are predicted using the correlations of dry reference strain [defined as (2 dry ITS)/(Dry Mr)] and of strain-shifted toughness ratio (defined as TSR^2/MrR^2) at repetitions equal to 100,000 (5), and by functions of wet and dry Mr. The relationships were developed at the University of Idaho from laboratory tests that produced data on fatigue strength, ITS, and Mr for wet and dry conditions. The FLR equation is

$$FLR = [(2 \text{ dry } ITS/\text{dry } Mr)]^{-\text{wet } k + \text{dry } k} \times (TSR^2/MrR^2)^{-\text{wet } k} \times (\text{wet } \varepsilon^{-\text{wet } k})/(\text{dry } \varepsilon^{-\text{dry } k})$$

where k = the inverse of the slope of log strain (ε) vs log repetitions fatigue strength line, and is predicted by $k = -1.4 \times 10^{-3}\ Mr^{0.573}$, and ε = the tensile bending strain due to wheel loads and is predicted for average condition by $\varepsilon = 1.53 \times 10^{-3}\ Mr^{-0.187}$.

Dry and wet values of Mr are substituted in the equations to calculate the corresponding dry and wet values of k and ε.

A numerical example of FLR for the control mixture using 55°F test data follows:

dry ITS = 100, wet ITS = 60, TSR = 0.60 and
dry Mr = 757,600, wet Mr = 426,370, MrR = 0.56.

Thus,

dry $k = -3.391$, wet $k = -2.439$ and
dry $\varepsilon = 122 \times 10^{-6}$, wet $\varepsilon = 136 \times 10^{-6}$.

Substituting into the FLR equation,

$$FLR = 0.50$$

The control mixture's FLR can be greater or less than its TSR and MrR, depending on the ITS-Mr relationship. In this case, FLR is less, indicating that somewhat more moisture sensitivity exists by reference to fatigue life than by reference to either tensile strength or resilient modulus.

Examination of the FLR equation shows that FLR is maximized for a given TSR provided that MrR remains less than TSR.

Toughness Ratio

Toughness ratio (TR) relates asphalt concrete resistance to crack propagation in the pavement layer after the end of its cohesive life (i.e., fatigue life). When wet asphalt concrete has a lower crack propagation resistance than does dry asphalt concrete, time to terminal serviceability in the field will decrease.

Mathematically TR is a ratio between the proportionality of the wet to the dry areas under the failure stress-strain lines. It is approximated by the equation

$$TR = TSR^2/MrR$$

The control mixture's example values of TSR = 0.60 and MrR = 0.56 give TR = 0.64. It should be noted that this physical property ratio is somewhat greater than either TSR or MrR when TSR is greater than MrR. This indicates that the crack propagation process reflected by TR is usually less moisture sensitive than indicated by either TSR or MrR.

Wheelpath Ratio

Wheelpath ratio (WPR) relates the asphalt concrete's resistance to wheelpath rutting (permanent deformation) and in general is proportional to MrR. Although the WPR is not precisely defined at the present time, it will consist of wet and dry modulus-related values that will properly define the moisture sensitivity of permanent deformation. For example, WPR might consist of the characteristics of permanent deformation related both to adhesive loss (stripping) and to cohesion loss (asphalt binder softening), the effects of which on wheelpath rutting can be unequal. Percentage of stripping as well as wet and dry Mr are required to calculate adhesion and cohesion change (*5*). These values, in turn, would be incorporated into the WPR.

The mathematical field model for predicting wheelpath deformation with WPR will be a different model than the one that uses FLR and TR for fatigue cracking, although it might share some common computational methods.

Minimum Moisture Damage and Cutoff Ratios

Ratios such as TSR and MrR are presently calculated from mean ITS and Mr test values, but the probability that the ratios are lower than those calculated is proportional to the standard deviations of their test values. Tunnicliff and Root (*6*) applied mean and standard deviation test values to determine if mean values of additive-treated mixtures are statistically different, hence better, than those of corresponding untreated mixtures. This implies that ratios such as TSR will require statistical definition to better describe and control moisture sensitivity. Large standard deviations will require larger TSR when TSR is calculated from mean values. In the future, the direct or equivalent use of standard deviation (as well as mean values) associated with ITS and Mr will be used with field wet life prediction models to assess and provide the specified reliability needed to achieve minimum or zero moisture damage.

The physical property ratios (e.g., FLR and TR) are equal to 1 when the mechanical property ratios (e.g., TSR and MrR) are equal to 1. This indicates that zero moisture damage will result when both TSR and MrR are known and are verified to be equal to 1. In addition, the reliability of not exceeding a minimum, specified level of moisture damage using TSR and MrR together as cutoff ratios appears to be greater than that of using either TSR or MrR as a cutoff ratio. Therefore it appears that the control of properties for minimum or zero moisture damage requires application of both TSR and MrR. TSR and MrR need not be equal to 1 to achieve a specific minimum moisture damage, but they should be high values.

It is customary to allow mixtures to be used for paving when their mechanical property ratios are less than 1 but greater than a minimum ratio, for example 0.70. The 0.70 ratio is then called the cutoff ratio. The reason cutoff ratios are less than 1 appears to be that, at average reliability, the field conditions and variables associated with reaching the TSR cutoff in time account for a probable minimum loss of pavement life.

When physical property ratios developed from mechanical properties are used instead of the mechanical property ratios, intuitions are supplemented by explicit requirements to take advantage of the apparently improved predictability. One such requirement is to limit the extent of moisture damage in the field or, in other words, to specify the maximum percentage loss of pavement performance life as a result of moisture damage. This appears to average 10 percent currently (*4*). This figure is used with the field prediction model to calculate the cutoff ratio or ratios in terms of the mechanical property ratios of TSR and MrR. The calculated cutoff ratios are not always the same number (e.g., 0.70); instead they can be as high as 0.95 and sometimes below 0.70 because of differences in the maximum specified loss of pavement life, in the mechanical properties of asphalt concrete mixtures, and in pavement location factors (7).

High FLR and TR are required for good fatigue cracking wet life and are best achieved when the TSR cutoff is greater than the MrR cutoff (e.g., TSR cutoff = 0.75 and MrR cutoff = 0.70). TSR and MrR cutoffs are predicted for the six test sections of NCHRP Projects 4-8(3)/1 and 4-8(4) using a prediction model that incorporates physical property ratios of FLR and TR. They are discussed in a later section of the paper.

PREDICTION OF WET PERFORMANCE LIFE OF PAVEMENT

The translation of the laboratory minimum ratio or ratios to wet performance life is best visualized and accomplished by use of mathematical models. Basic concepts of the mechanics of the model should be understood and the correlation equations for constants should be reevaluated periodically using valid field or laboratory data as they become available.

All wet life prediction models consist of at least the following parts:

1. Field time change (e.g., reduction) of the physical property ratio or ratios from 1 at time equals zero (i.e., all dry) to the minimum ratio or ratios years later when wetness and thermal cycles maximize;
2. Application of a technically based method that relates predicted field ratio or ratios from Item 1 to the asphalt concrete pavement layer's wet performance life in repetitions of traffic loads; and
3. Translation of the ratio-life repetitions from Item 2 to obtain the wet performance life of the pavement layer in years.

Wet life is compared with the assumed reference or standard-designed all-dry life to determine if the proposed mixture will provide the required life when it becomes wet in the field. Reference to this information will minimize the additional life-cycle cost associated with this loss of life. Wet life prediction models can be readily used for calculating cutoff ratios. The cutoff ratios are referred to in the laboratory during mixture design analysis. Thus wet life prediction models are well suited for mixture evaluation before construction.

Unacceptably low wet life or corresponding low laboratory ratios (i.e., below cutoff ratios) require a change of mixture constituents; reduction of voids, if possible; or treatment by inclusion of additives that retain adhesion and cohesion in the mixture. Wet life prediction models should be realistically sensitive to changes in mixture variables and to additive treatments. The physical property ratios show this sensitivity because they depend on the magnitude of the ITS and Mr values or their ratios that, in turn, are sensitive to mixture variables and additives.

Brief descriptions of wet life prediction models follow:

1. Absolute life: Existing pavement design equations may be used. The equations, developed for "all dry life," are used with the appropriate wet damaged asphalt concrete property. The magnitude of wet life is an absolute value that is independent of dry life. Its accuracy is dependent on the design method and the moisture sensitivity characterization used in the method.
2. Relative life: Wet life is calculated and its magnitude is dependent on an assumed or precalculated dry life. The change of life relationship that is developed for the method is readily based on working stress-strain mechanics such as the relative (wet/dry) relationships of fatigue strength, stress, and strain in the pavement layer. The mechanics approach appears to have acceptable prediction accuracy when applied to the relative life method instead of the absolute life method. The stipulated condition of relative life methods is that wet life is related to dry life and only to the specific asphalt concrete pavement layer. Comparative decisions are based on this condition.

Absolute Life

The application of the AASHTO asphalt concrete pavement design life equation is an example of calculating absolute life. The AASHTO structural number's layer coefficient for the asphalt concrete due to moisture sensitivity is calculated by use of a ratio. A brief description of models that use TSR follows:

Time Reduction of Ratio

A method described in 1982 (7) incorporates two steps (or moisture stages) of the TSR. Figure 1 shows the two stages. The first stage is represented as dry with the ratio of the pavement layer equal to 1, and the second stage, which begins when the field ratio has decreased halfway to the laboratory minimum ratio, is represented by the minimum TSR associated with laboratory accelerated conditioning. Dry stage time can be adjusted to specific locations.

A more detailed method developed by Nesichi and Ishai at Technion University (8) incorporates three moisture stages: dry, saturated, and accelerated conditioned, which are represented by corresponding laboratory TSR-values. The TSR-values are predicted to decrease with time through the moisture stages using a rate constant established by experience or by correlation with the serviceability loss relationship in the AASHTO equation. The rate constant appears to be adjustable to represent different locations.

Relating Ratios to Wet Life Repetitions

Miner's rule (cumulative damage) is applied in the 1982 method (7). The AASHTO equation is used to calculate basic dry and wet lives in 18-kip single-axle equivalents for the two moisture stages represented by TSR equal to 1 and by TSR equal to laboratory minimum. The retained cohesion in the asphalt concrete layer during the wet stage is calculated using the minimum TSR. The resulting wet stage layer coefficient is used to calculate a wet stage structural number, from which the basic wet fatigue life is calculated. The specific life in the wet stage is calculated and is added to the life in the dry stage to equal the dry-wet life, or "wet" life. The rate of cumulative damage in the wet stage is greater than in the dry stage when minimum TSR is less than 1; therefore the dry-wet life would be less than the dry life.

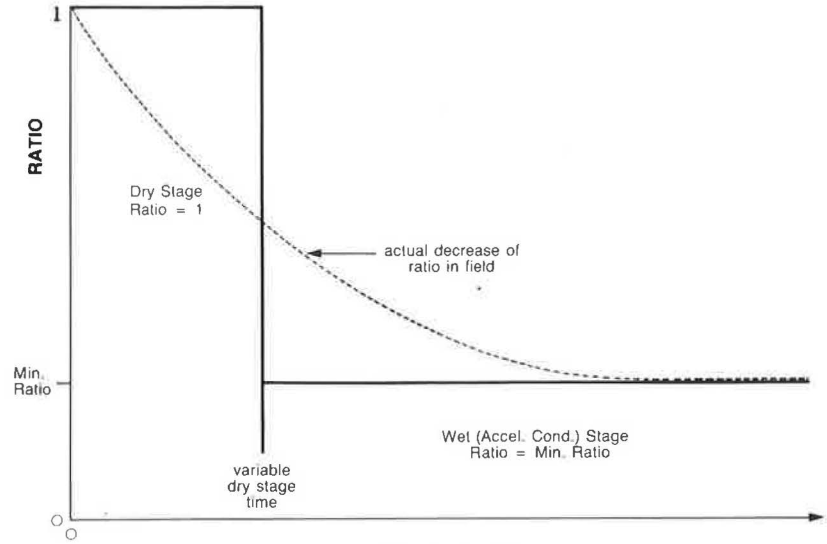

FIGURE 1 Two-stage decrease of ratio in field.

The Technion method (*8*) calls for the calculation of three pavement lives from the AASHTO equation to represent each stage after the length of the dry and saturation stages has been determined. Establishment of the TSR for each stage is readily adjusted to be compatible with the rate constant and loss of serviceability calculated from the AASHTO equation. The asphalt concrete layer coefficients for these stages are calculated using TSRs as measures of retained cohesion. The dry-wet life is the sum of the lives from each stage and can be compared with the dry life using a TSR equal to 1.

Repetitions to Years

Asphalt concrete life is mechanistically associated with repetitions, but life in years is used for decision making. This requires an estimate of the traffic rate. The life in years is usually stipulated as a design constraint to satisfy a planned field performance period at a given reliability. It is interpreted to be a dry life. The required repetitions of dry life are thereby established from the traffic rate constant and the required dry life in years. The traffic rate is the same when the pavement is wet and is undergoing moisture damage. Dry-wet or wet life repetitions calculated from the AASHTO equation are translated to wet life years by using the traffic rate. This approach applies to both absolute life and relative life methods.

Relative Life

Two models developed at the University of Idaho are described briefly to illustrate the relative life method. Both models reflect fatigue cracking distress and use physical property ratios instead of TSR or MrR alone. They are on computer software and predictions are available in minutes.

The Asphalt Concrete Moisture Damage Analysis System (ACMODAS) was developed in 1984 to meet the need to relate laboratory ratios to field wet life and to calculate cutoff ratios. The ACMODAS model was evaluated and applied by Busching et al. in 1985 in a South Carolina study of moisture damage assessment of aggregates and mixtures (*9*). Applications of the model are being evaluated at the materials and research facilities of several state highway agencies and at several additive-modifier companies.

An alternate version (1987) of the model, ACMODAS 2, incorporates the same objectives but has additional parts that reflect refinements in simulating the physical process of moisture damage.

Time Reduction of Ratio

A two-stage reduction of ratio FLR is incorporated in ACMODAS and is shown in Figure 1.

In contrast, a continuous function depicting the exponential decrease of ratios FLR and TR with time by a rate constant is incorporated in the alternate model, ACMODAS 2. It is shown in Figure 2. It has the form: Ratio = min.Ratio + $(1 - \text{min.Ratio})^{-KT}$, where min.Ratio is a physical property ratio (FLR and TR), K is a rate constant, and T is time in the field. In addition, a partial increase (i.e., recovery) of Ratio is specified to represent an annual, warm drying-out period. This is accomplished by superimposing a sine wave on the continuous function. The cyclic recovery is Ratio + 0.15, which corresponds to a sine amplitude of 0.075. A maximum recovery to Ratio = 1 may occur in the first year or so, especially if K is low. The predicted ITS and Mr mechanical properties in the FLR calculated at the time of onset of cracking (i.e., completion of cohesive life) are used to calculate the TR. The TR reduces with time by the same rate constant used with the FLR.

The models are field adjusted to a specific location. In ACMODAS this is done by varying the dry stage time and by using a regional factor, which is a multiplier of the wet strain in the FLR equation. The usual procedure is to set the dry stage time equal to 4 years and vary the regional factor. High regional factors are associated with a large number of annual freeze-thaw cycles. Adjustment in ACMODAS 2 is made by varying the continuous function rate constant. The rate constant is proportional to the number of annual freeze-thaw cycles and

24-hr cool-warm cycles. A high rate constant is associated with the combination of 150 freeze-thaw cycles and 150 cool-warm cycles per year. The effective moisture damage differences between the two types of cycles are recognized in guidelines for estimating the rate constant.

Figure 3 shows the influence of rate constants on predicted wet life using the ACMODAS 2 model. The lowest rate constant (.20) corresponds to a mild climate; the highest constant (.85) corresponds to a severe thermal cycle climate (e.g., 150 freeze-thaw and 150 24-hr cool-warm cycles per year).

Relating Ratios to Wet Life Predictions

Both models use life in years directly in their computational methods. Traffic rate can be applied to obtain a dry design life beforehand, and this dry life in years becomes the reference with which corresponding wet life is compared.

ACMODAS employs the cumulative damage method for each of the two moisture stages. The relative wet life is calculated assuming that the cumulative damage equals 100 percent for the sum of the two stages. A similar cumulative damage calculation is used in the absolute life method with the AASHTO equation (7).

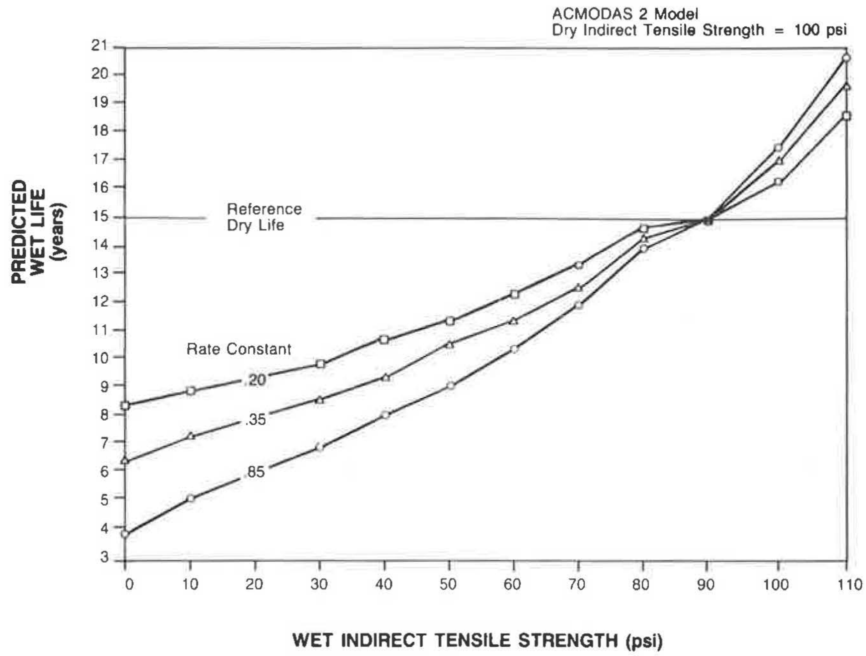

FIGURE 2 Continuous decrease of ratio in field with annual partial recovery.

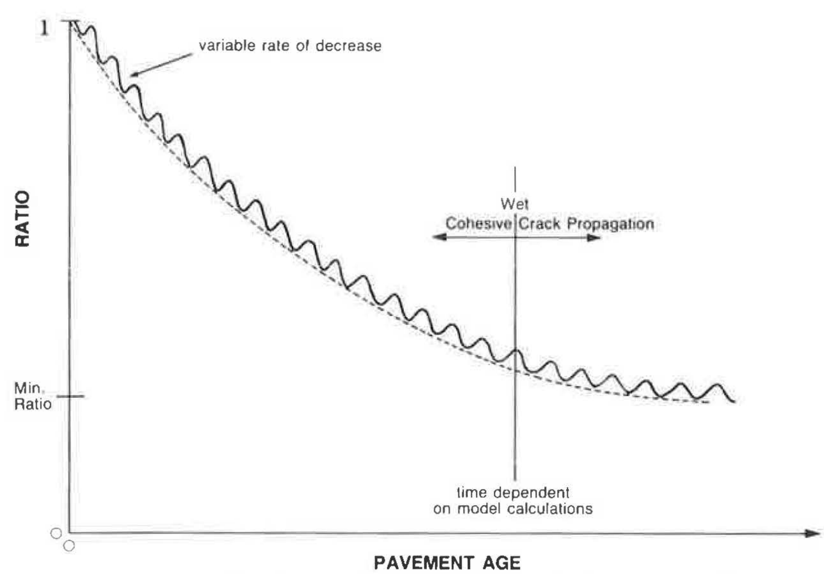

FIGURE 3 Effect of rate constant for ratio decrease in field on predicted wet life.

ACMODAS 2 employs the cumulative damage method for fatigue life (or cohesive life) and for crack propagation life using FLR and TR, respectively, on a continuing time basis. Time increments of damage are summed until no fatigue life remains. This is called wet cohesive life. Thereafter a similar incremental procedure is performed for crack propagation life. Predicted relative wet life therefore equals cohesive life plus crack propagation life.

The use of basic dry fatigue lives in the cumulative damage method of ACMODAS 2 is more complex. The basic dry fatigue lives are based on the present serviceability index (PSI) versus time (T) curve, which has the form PSI = initial PSI − AT^B. A and B are constants and are calculated to fit the PSI curve conditions of the reference dry life (e.g., 15 years), initial serviceability (e.g., 4.6), terminal serviceability (e.g., 2.5), and the specified percentage drop of PSI at the end of the dry cohesive life before crack propagation (e.g., 25 percent). The basic dry cohesive life used in cumulative damage calculations remains constant at two-thirds of the reference dry life. However, the basic dry crack propagation lives used for their respective time increments are not constant but decrease with increasing crack propagation time; they are calculated through slopes obtained from the equation of the PSI versus T curve in the crack propagation region. In Figure 4 predicted wet life relative to dry life is illustrated by the PSI versus T curve.

Comparison of Wet Life Predictions

It can be seen from the preceding discussion that the ACMODAS 2 model is more complex than the ACMODAS model. Also, the wet lives calculated from the same ITS and Mr data are not the same. Nevertheless, in many instances it appears that the wet lives are usually close enough for purposes of general agreement. For example, if the previously applied numerical ITS and Mr data for the control mixture are used (e.g., TSR = 0.60), the wet life from ACMODAS is 10.5 years and from ACMODAS 2 it is 10.8 years, when a 15-year reference dry life and average location (climatic) conditions are considered.

Given in Table 1 are the predicted relative wet lives for the six test sections evaluated in NCHRP Projects 4-8(3)/1 and 4-8(4). Wet life predictions for the Idaho (ID) and Montana (MT) asphalt concrete in the test sections are close to or greater than the 15-year reference dry life. This is primarily due to the high TSR and favorable ITS-Mr relationship used in the physical property ratios. General agreement of the models on predicted wet life ranking can be observed for the test sections.

There appears to exist a threshold of model complexity which it is not practical to exceed. There are two apparent reasons for this:

- The overall functionality between laboratory ratios and pavement life may be self-limiting in spite of the modeling of moisture damage to more sophisticated and complex levels and
- The accuracy of verifying predicted relative wet life in the field may not be sufficient to finely tune more complex models to idealistic levels.

In Situ Wet Life Predictions

Cores taken periodically from existing pavements can be tested in dry and saturated conditions. Accelerated conditioning is not used. Field moisture damage will be manifested in the saturated condition, and wet/dry periodic ratios are calculated. Relating these ratios to a computational method used in the models will provide a prediction of wet performance life for an existing pavement. For instance, the periodic core test data of the six state highway agency test sections in NCHRP Projects 4-8(3)/1 and 4-8(4) were applied in this manner to the ACMODAS and ACMODAS 2 computational methods. The wet lives of the test sections are given in Table 2. These lives are the probable in

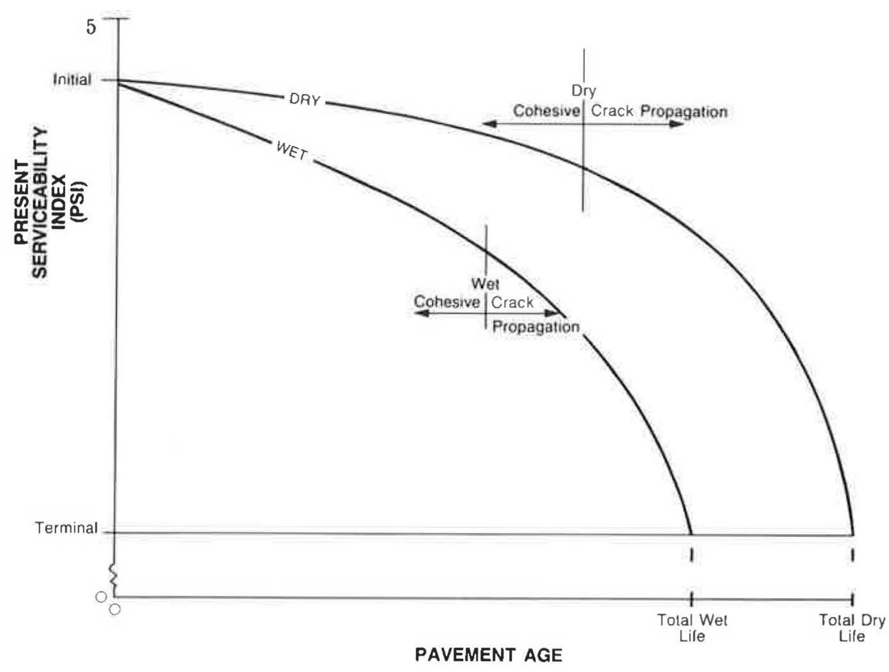

FIGURE 4 A predicted wet life serviceability curve relative to a dry life curve.

TABLE 1 COMPARISON OF PREDICTED WET LIVES FOR ASPHALT CONCRETE BY ACMODAS MODELS

Test Section Designation (state highway agency)	Initial ITS Data at 55°F			Wet Life Prediction in Years (dry life = 15 yr)					
					ACMODAS 2				
	Dry	Wet	TSR	ACMODAS	Cohesive	+	Crack Propagation	=	Total
ID	52	47	0.90	16.2	10.3		5.0		15.3
MT	47	40	0.85	15.7	10.0		4.9		14.9
VA	47	24	0.51	11.3	8.0		3.6		11.6
CO	36	16	0.44	11.3	8.1		3.4		11.5
AZ	97	39	0.40	7.7	6.2		3.2		9.4
GA	104	0	0	4.0	4.7		1.6		6.3

situ wet performance lives for the lower asphalt concrete layer in the pavement and appear to be reasonable on the basis of the 10-year pavement evaluation by the highway agencies. Comparison of the lives indicates that the results from the two models are again in general agreement. Another comparison indicates, however, that the three longer-lived test sections as predicted and listed in Table 1 are now in different order in Table 2 based on the periodic core evaluations. Certainly, one of the reasons for this difference is not being able to predict all incidental changes that occur in the field from the practical application of a single laboratory test. For instance, the "asphalt-fines" in the lower layer of asphalt concrete in the Idaho section appeared to be partly eroded after 5 years, giving rise to a TSR that is lower than the predicted minimum TSR developed from the accelerated conditioning of freeze plus 24-hr 140°F water soak. Maybe the application of several freeze plus warm water soak cycles would have produced the lower TSR by causing this erosion.

TABLE 2 COMPARISON OF PROBABLE IN SITU WET LIVES FOR ASPHALT CONCRETE USING PERIODIC CORE PROPERTIES

Test Section Designation (state highway agency)	Wet Life Prediction in Years (dry life = 15 yr)					
		ACMODAS 2				
	ACMODAS	Cohesive	+	Crack Propagation	=	Total
MT	16.9	10.8		4.9		15.7
VA	12.7	9.9		3.9		13.8
ID	11.5	8.5		4.1		12.6
CO	11.0	7.9		3.6		11.5
AZ	8.5	5.8		3.2		9.0
GA	7.6	5.7		3.1		8.8

Constructing a graph is a helpful method for comparing probable in situ moisture damage using the relative approach. On the graph, mechanical and physical property (periodic) ratios are plotted relative to reference dry life damage as a function of time. To accomplish this, the percentage remaining life versus time line is drawn to depict the probable load-associated life reduction of the dry asphalt concrete. It is a reference dry life line. Then ratios from cores are calculated, multiplied by the asphalt concrete's percentage remaining dry life, and plotted at the time corresponding to the core drilling. This is repeated periodically and wet life lines are drawn through the plotted points. A wet life line below the reference dry life line reflects moisture damage.

Figures 5 and 6 show examples of the wet life lines obtained from the periodic core ratios of the Virginia and Colorado test sections. Lines are shown for the ratios of TSR, MrR, FLR, and TR. Both test sections have moisture damage after 5 years. Notice that moisture damage characterized by MrR tends to be the highest and that characterized by FLR and TR tends to be the lowest.

If equal weighting is assigned to each of the four ratios, the broad band of wet life lines for the ratios of the Virginia test section in Figure 5 implies large variance and less certainty about the exact extent of moisture damage. The average retained life appears to be 20 percent at 10 years. The predicted wet life information from Tables 1 and 2 as well as the observation of 35 percent stripping in the 10-year cores also indicate the occurrence of moisture damage in the Virginia test section.

In contrast, it can be observed in Figure 6 that a much narrower band of wet life lines exists for the ratios of the Colorado test section; this implies a lower variability for the occurrence of in situ moisture damage after 5 years. Here, retained life of 20 percent at 10 years is more reliable. The 10-year cores from the Colorado test section show "severe" stripping (about 50 percent), and the predicted wet life information from Tables 1 and 2 verifies the occurrence of higher moisture damage (e.g., Table 2 gives an in situ life of around 11 years, about 2 years less than the Virginia in situ life).

PREDICTING CUTOFF RATIOS

Both absolute life and relative life prediction models can be used to calculate cutoff ratios that are applied during the design and analysis of a mixture in the laboratory. Cutoff ratios for TSR are calculated from the AASHTO equation and the two-moisture stage model and were presented previously by Lottman (7). The results indicate that the TSR cutoff is not a constant; it is dependent on pavement location, percentage of allowable reduction of life, and dry mechanical properties. Severe climatic conditions (e.g., many freeze-thaw cycles) and low percentage of allowable reduction of life (e.g., 5 percent) will increase the cutoff ratio. Similar results are indicated for relative life models.

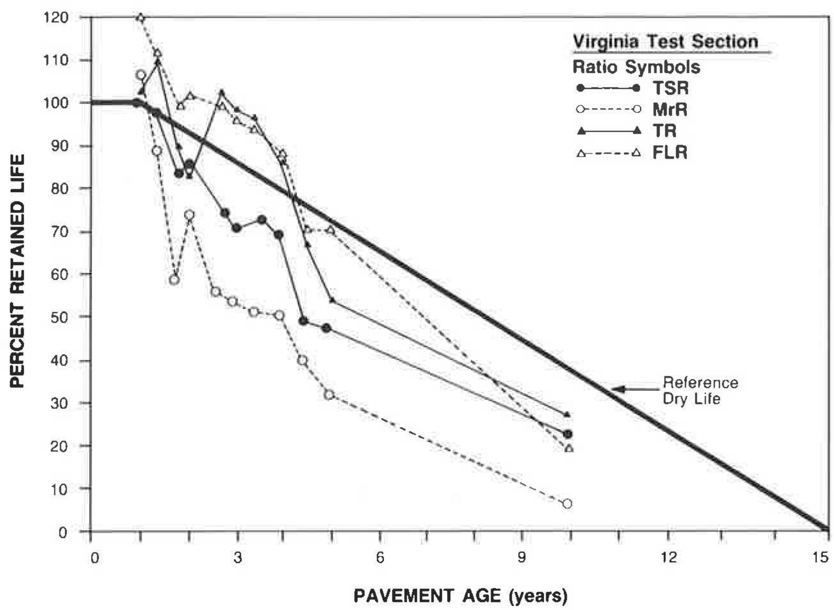

FIGURE 5 Relative wet life lines of four ratios for the Virginia moisture damage test section.

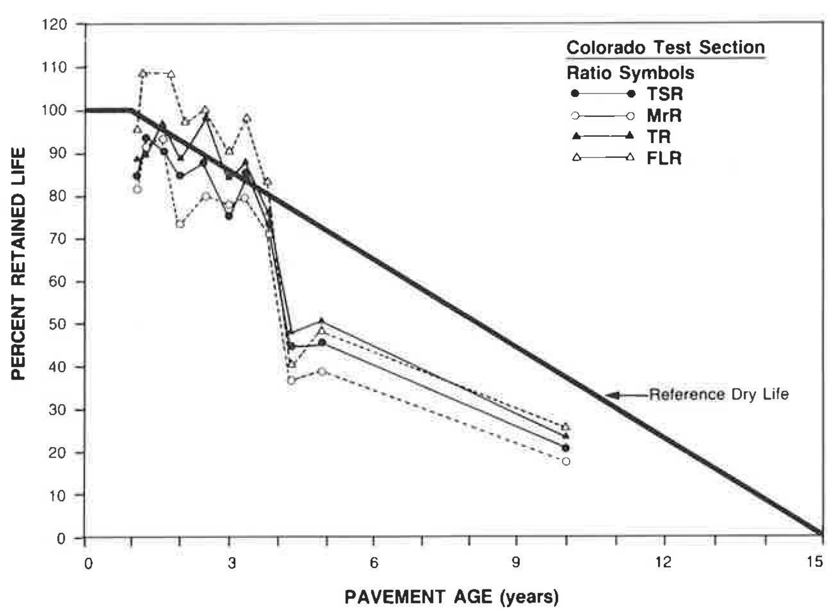

FIGURE 6 Relative wet life lines of four ratios for the Colorado moisture damage test section.

When the relative life models of ACMODAS and ACMODAS 2 are used with the previously applied numerical examples of the control mixture's ITS and Mr data, the respective TSR cutoffs are calculated to be 0.75 and 0.78. [These ratios are for average pavement location (i.e., climatic conditions) and for a 10 percent allowable reduction of life.] Because the control mixture's TSR of 0.60 is less than the TSR cutoff, the mixture is unsatisfactory (i.e., too much moisture sensitivity) and the mixture must be redesigned or an effective additive must be used. The TSR of the reconstituted or treated "new" mixture must be compared with the TSR cutoff to ensure that it is at least equal to the cutoff ratio. The TSR cutoff required for the new mixture might be different from the previous cutoff ratio if the treated mixture's dry mechanical properties are different.

TSR cutoffs are calculated from the ACMODAS 2 prediction model with average climatic conditions (rate constant = 0.35) for the six NCHRP 4-8(3) and 4-8(4) test sections. Initial field TSRs are listed next to the calculated TSR cutoffs in Table 3. The four lowest ranked sections (VA, CO, AZ, and GA) have initial TSRs less than the TSR cutoff and would have required mixture redesign or use of additives, or both. These moisture sensitive mixtures were purposely paved without redesign or treatment to satisfy the objective of moisture damage research in the NCHRP project.

TABLE 3 INDIRECT TENSILE STRENGTH AND RESILIENT MODULUS CUTOFF RATIOS FOR 10 PERCENT ALLOWABLE LIFE REDUCTION BY ACMODAS 2 MODEL

Test Section Designation (state highway agency)	Initial TSR	TSR Cutoff	Initial MrR	MrR Cutoff
ID	0.90	0.70	0.84	0.66
MT	0.85	0.68	0.80	0.64
VA	0.51	0.68	0.48	0.64
CO	0.44	0.67	0.41	0.63
AZ	0.40	0.76	0.38	0.71
GA	0	0.76	0	0.71

A TSR cutoff of 0.70 appears to be a current national average used in routine mixture testing. It is interesting to note that 0.70 appears to be about a model-predicted average for the NCHRP test sections (Table 3).

An advantage of prediction models is that MrR cutoff as well as TSR cutoff can be predicted readily. MrR cutoffs from the ACMODAS 2 model are listed in Table 3 for the NCHRP test sections. The four lowest ranked test sections have initial MrRs that are less than the MrR cutoff, which is a problem here because their TSRs are less than the TSR cutoff. For these specific mixtures, it appears that remedies to increase TSR to TSR cutoff would also increase MrR at the same time. However, performance goals might not be reached if MrR increased more rapidly than TSR when using some treatments. Therefore it is important to make sure not only that both TSR and MrR meet the respective cutoff ratios, but also that TSR remains greater than MrR.

DEVELOPING SUPERIOR PERFORMANCE

Current procedures deal with moisture sensitivity as a disadvantage; that is, there is moisture damage because of decrease in mixture adhesion and cohesion. Thus the effort is to keep the loss of pavement performance life to a minimum.

On the other hand, there appears to be an optimistic side to moisture sensitivity. The possibility exists for achieving improved mechanical properties in the wet stage. All asphalt concrete will contain moisture; therefore it will be advantageous to purposely make use of moisture to gain equivalent performance life by altering the wet ITS-Mr relationship through the use of chemical modification of asphalt or aggregate. The specific advantages of the improvement would be to increase the reliability of developing zero moisture damage in the field by increasing the wet life beyond dry life at average reliability. If 90 percent reliability of a specific reference dry life is stipulated, the achievement of an equal reliability of zero moisture damage may also be required.

Blends of several generic antistripping additives with modest concentrations of some polymeric modifiers can develop a combined FLR as high as 2.0, which should provide a high reliability of zero moisture damage. There are also blends that are not effective, giving combined FLR of less than 1.0. The high FLR requires adhesion loss to be negligible and cohesion gain to be greater for ITS than for Mr after accelerated conditioning.

The required TSR and MrR cutoffs corresponding to the high FLR (and TR) will also be higher ratios. The 90 percent reliability of zero moisture damage was evaluated on a limited basis in the ACMODAS 2 model using an average standard deviation of 10 percent and mean values of wet and dry ITS and Mr for treated moisture sensitive mixtures. What appears to be required is a TSR in the range of from 1.15 to 1.20 with an MrR of about 85 percent of the TSR for ratios calculated from mean values.

The predictability and cost-effectiveness of chemical compounds and blends that develop the proper gains of mechanical properties for a specific reliability of zero moisture damage are topics of future research on chemical modification and statistical application.

SUMMARY

Identification and application of moisture damage ratios currently are based on mechanical properties calculated from the indirect tensile strength (ITS) and resilient modulus (Mr) tests. These tests are becoming commonplace and will be used in laboratory procedures for the analysis of asphalt concrete mixtures.

Laboratory moisture conditioning (accelerated conditioning) that produces minimum mechanical property ratios for a specific mixture is recommended. The use of a prediction model corrected for the specific field environment (e.g., climate) is suggested for technical efficiency.

Physical property ratios are made up of ITS- and Mr-values and reflect a mixture's working resistance to a specific type of field damage. They are used in mathematical prediction models. In turn, the models are used to predict the wet performance life and laboratory cutoff ratios for ITS (TSR) and for Mr (MrR). Two physical property ratios currently used in the University of Idaho models are fatigue life ratio (FLR) for the onset of fatigue cracking and toughness ratio (TR) for crack propagation.

Absolute life and relative life models have a different basis of computations and comparisons. Relative life models appear suitable for incorporating mechanistic methods because the models use comparative calculations to a known reference performance. All models employ a field time decrease of ratio, from 1 at zero time to a laboratory minimum ratio at a later time when environmental conditions produce the moisture damage corresponding to the accelerated conditioning. The rate of decrease of ratio is changed according to climatic conditions, resulting in a significant change in the number of years for ratio decrease and for wet performance life.

Applications of ratios and prediction models to NCHRP field data indicate that there are advantages to using physical property ratios and models. Although the level of model complexity may be limited by current practical applications and verification skills, implications are that the ratios required for achieving the necessary reliability of obtaining zero moisture damage are best determined through the calculation of statistically equivalent wet life increases from prediction models.

It is advantageous to know and use the cutoff ratios calculated from a prediction model because the ratios are dependent on the specific mixture's ITS and Mr test values as well as pavement location data. TSR cutoff ratios of about 0.70 on average are used currently in the United States to limit field moisture damage. Prediction models indicate that 0.70 is, in general, associated with not more than 10 percent loss of dry performance life at average reliability. A TSR about equal to 1 and an MrR slightly less than the TSR appear to be required for a higher reliability of zero moisture damage. Even higher reliabilities (e.g., 94 percent) of zero moisture damage require the TSR of treated mixtures to be greater than 1 (with corresponding MrR less than TSR). This superior performance might be achievable with specific blends of antistripping additives and polymeric modifiers that not only stop adhesion loss but also promote cohesion gain through the correct buildup of mechanical property ratios.

ACKNOWLEDGMENT

Although the applications of physical property ratios and prediction models herein are developed by the authors, a broad base of effort has been invested in the testing, evaluating, and reporting of laboratory and field data that provide ideas for future application. Credit for this is given to all who are engaged in research on prevention of moisture damage in asphalt concrete. Special recognition is given to the highway agencies of Arizona, Colorado, Georgia, Idaho, Montana, and Virginia and the FHWA who participated in NCHRP Field Projects 4-8(3)/1 and 4-8(4). The National Science Foundation's Industry/University Cooperative Project ECE-8411271 at the University of Idaho with Carstab Products is providing the opportunity to evaluate combinations of generically based additives and modifiers in moisture sensitive mixtures.

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Laboratory Evaluation of Moisture Damage to Bituminous Paving Mixtures by Long-Term Hot Immersion

ILAN ISHAI AND SIMON NESICHI

This paper deals with the development of a moisture damage (MD) sensitivity test for bituminous paving mixtures under hot and humid climatic conditions. The concept and test are based on characterizing the moisture damage sensitivity by long-term durability curves that express the variation of retained strength with hot immersion (60°C) time as long as 14 days. The original version of the test was modified by characterizing MD sensitivity using the retained strength value at 6 days of hot water immersion. Both versions of the test have been proven to be superior to the traditional 1-day Marshall immersion test in detecting sensitive mixtures. The modified version is a practical substitute for its former version that was based on longer immersion periods (up to 14 days) and a greater number of sample sets (five versus three needed by the current procedure). The development of the test was supported by laboratory and initial field experience, as well as by theoretical considerations of the behavior of mixtures during long immersion periods. Suggestions for needed further research are outlined at the end of the paper.

The damaging effects of moisture on the physical properties and mechanical behavior of bituminous paving mixtures have been known for many years (*1*). To overcome the problem, many laboratory tests were developed, all attempting to evaluate mixtures' sensitivity to moisture damage (MD). The most popular and practical of these tests appear to be the immersion-mechanical tests, which measure changes in mechanical behavior of compacted samples caused by exposure to moisture (*2*). Typically, the results of these tests are reported in terms of percentage retained strength. Various tests of this type have been developed (*3–8*).

For many years, the Marshall immersion test (*4*, pp. 29–30) was the only MD sensitivity test in use in Israel. Indeed, this test is still included in various national specifications with quality criteria of 60 to 75 percent retained stability values for roads and highway pavements and 75 percent for airfield pavements.

Recently, doubts about the test's ability to detect sensitive mixtures have arisen. In some cases, bituminous mixtures with hydrophilic aggregate, which complied with the specification's criteria, stripped in the field shortly after placement (*9*). Doubts about the quality of the test were also raised by other writers (*10*) and in personal communications. Therefore a research project, one of the aims of which was to develop a better test, was conducted at the Transportation Research Institute of the Technion, Haifa (*11*). The purpose of this paper is to describe this research and its main findings with respect to the laboratory evaluation of moisture damage to bituminous paving mixtures.

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DEVELOPMENT OF TEST

Basic Requirements

The basic requirements for developing a laboratory MD sensitivity test can be summarized as follows (*11, 12*):

1. It must be fairly rapid and simple,
2. Its reproducibility must be fairly high,
3. It must bring out differences sought in a distinctive manner,
4. It should have a sound analytical basis,
5. The needed testing time and equipment should enable commercial laboratories to include the test as a part of routine procedures for design of job-mix formulas,
6. The exposure to moisture should cause damage similar in amount and type to that occurring in the field, and
7. Acceptance and rejection criteria should be based on correlation with field experience.

An effort was made to develop and adopt an MD test that, for local conditions, had been proven to be superior to the Marshall immersion test. This test is the Durability Index (DI) Test, which had originally been developed for research purposes (*13*) but was proven to be successful in field use as well (*9*).

Original Version of Durability Index

Because the original version of the DI test has already been described in detail (*9, 13*), only a short review of its main features is given here. The test is based on subjecting five identical sets of Marshall specimens to hot water immersion (at 60°C) for up to 14 days. Each set is tested after a different immersion period (0, 1, 4, 7, and 14 days), and the retained strength values versus immersion time graphically describe a durability curve (Figure 1). The strength parameter usually used is the Marshall stability value; however, other strength parameters, such as resilient modulus (*13*), have also been used.

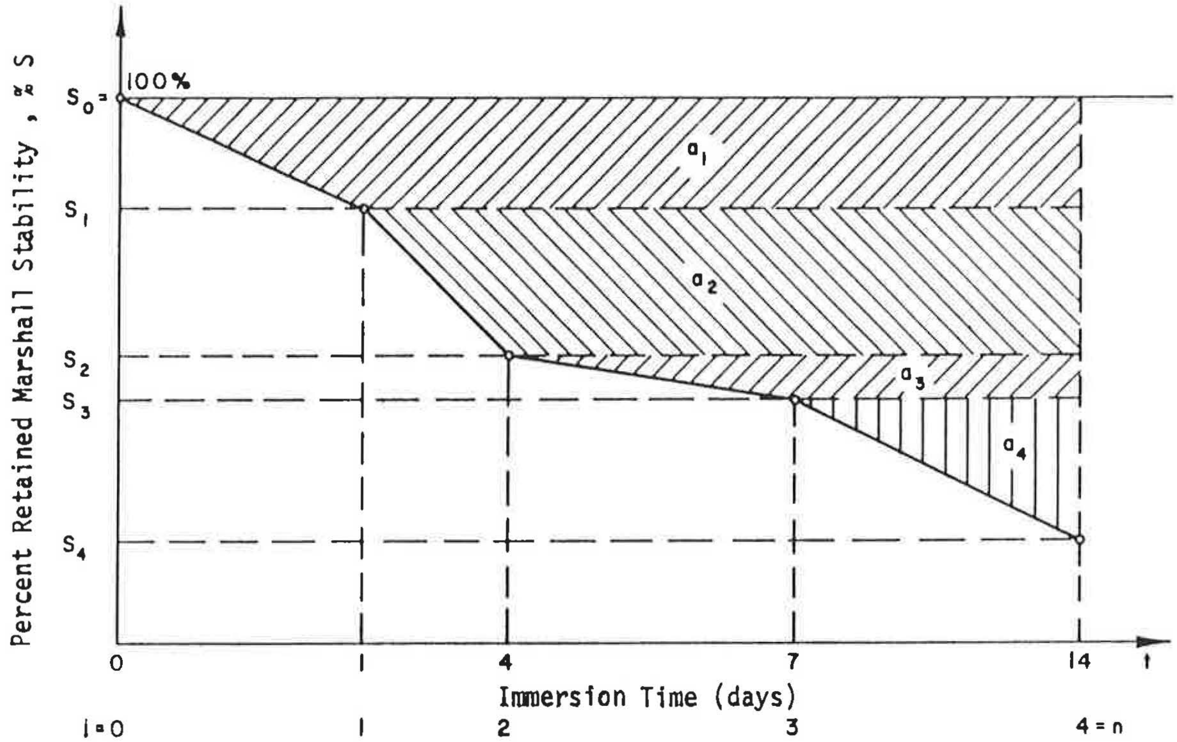

FIGURE 1 Schematic description of durability curves with parameters that define durability indices (*13*).

The MD sensitivity of the mixture tested is reported in terms of **Durability Index (DI) or Equivalent Retained Strength** (ERS). The DI is defined as the average strength loss area enclosed between the durability curve and the line $s_0 = 100$ percent. Based on Figure 1, DI is expressed as

$$DI = (1/t_n) \sum_{i=1}^{n} a_i = (1/2t_n) \sum_{i=0}^{n-1} (s_i - s_{i+1}) \times [2t_n - (t_i + t_{i+1})] \qquad (1)$$

where

s_{i+1} = percent retained strength at time t_{i+1},
s_i = percent retained strength at time t_i, and
t_i, t_{i+1} = immersion periods (from beginning of test).

It should be noted that the area increments (a_i) are defined and partitioned horizontally because they express the relative contribution of the immersion period increments to the total loss in strength. In this respect, the relative weight of the early time increments is much higher than that of later ones.

The DI expresses an equivalent 1-day strength loss. Positive values of DI indicate strength loss, negative ones strength gain. By definition, DI < 100. Consequently, it is possible to express the percentage 1-day ERS as

$$ERS = (100 - DI) \qquad (2)$$

It should be noted that although only five immersion periods are used to describe a 14-day period, later studies (*11*) revealed that no changes in ERS values resulted when more incremental testing periods (up to almost daily strength tests) were added (Figure 2). Experience (*9, 13*) has proven that the test complies fairly well with Requirements 2, 3, and 4 mentioned previously. No field data yet exist to verify its compliance with Requirements 6 and 7, although hot immersion appears to be more logical for Israeli conditions than, say, freeze-thaw cycles given the hot and humid climate. Experience has proven the applicability of the Marshall test for reliable detection of MD sensitivity. This test was also adopted because the method and testing equipment are commonly used locally in bituminous paving technology. However, the long immersion period and the comparatively large number of samples involved are not compatible with Requirements 1 and 5. Therefore a research effort was undertaken to improve these limitations.

Development of Modified Version of DI

Review of numerous durability curves based on retained Marshall stability values derived under the DI testing procedure reveals an interesting phenomenon between 4 and 7 days of immersion. When the values of retained strength versus bitumen content are plotted, it turns out that the curve representing the equivalent strength loss (DI) is usually located in the zone confined by the curves representing 4 and 7 days of immersion. This brought up the idea that the whole durability curve may be replaced by the 5- or 6-day retained strength value (Figure 3).

The first attempt to verify this assumption was made by subjecting three mixtures to moisture immersion periods as required in the original version, with an additional test after 5 days of immersion. The mixtures were composed of identical basalt aggregate (Table 1) and 60- to 70-pen bitumen. However, the variations in filler type and percentage of

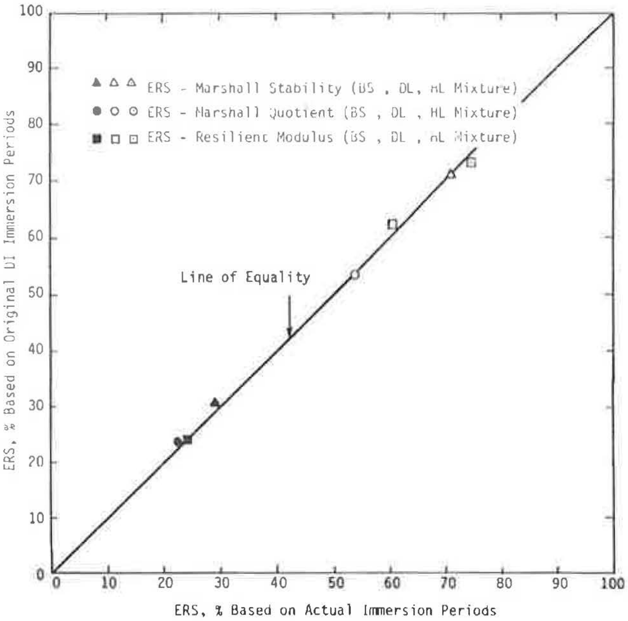

FIGURE 2 Equivalent retained strength values for different immersion periods versus values based on original DI procedure (*11*).

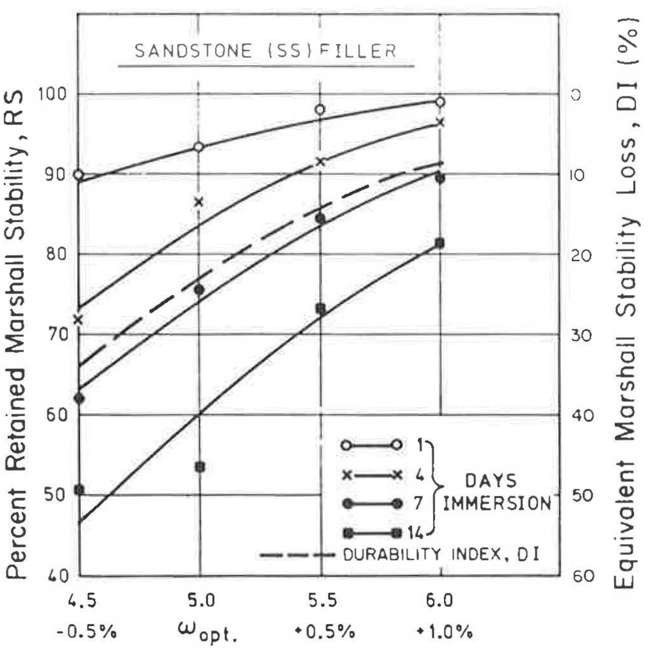

FIGURE 3 Effect of bitumen content on durability potential for different periods of immersion: dolomite aggregate, sandstone filler (*13*).

permeable voids in the mixture (Table 2) provided the needed variability in MD resistance. Retained strength values as required by the original DI test version, as well as after 5 days of immersion, were calculated for three mechanical parameters (Marshall stability, Marshall quotient, and resilient modulus) and are shown in Figure 4. Because all of the values in Figure 4 fell to the right of the line of equality, it became obvious that a 5-day immersion period was not enough; therefore the 6-day value was tried. At first, theoretical 6-day retained strength

TABLE 1 PROPERTIES OF BASALT AGGREGATE USED IN RESEARCH

Property	Value
Specific gravity	
Bulk	
+No. 4	2.72
−No. 4	2.65
Apparent	
+No. 4	3.0
−No. 4	2.86
Water absorption (%)	
+No. 4	3.75
−No. 4	4.0
Sand equivalent (%)	63.5
Los Angeles abrasion, 3/8 in.–1/2 in. (%)	25
Bitumen absorption (%)	1.5
Gradation: percentage passing sieve	
1/2 in.	100
3/8 in.	82
No. 4	60
No. 10	42
No. 40	23
No. 80	15
No. 200	6.5

values, computed from durability curves derived in various studies (*9, 11, 13*) by assuming a linear drop in retained strength between 4- or 5-day and 7-day immersion periods, were compared with original DI test values. This time, agreement was excellent with a coefficient of variation of 0.99 (*11*) for the 36 mixtures studied, as shown in Figure 5.

Given this high degree of correlation, it was decided to form a data bank of various types of mixtures, for which durability testing based on the original version of the DI test, as well as the modified, shortened 6-day immersion test procedures, would be conducted. The experience gained with various types of mixtures (conventional hot mixtures, recycled hot mixtures,

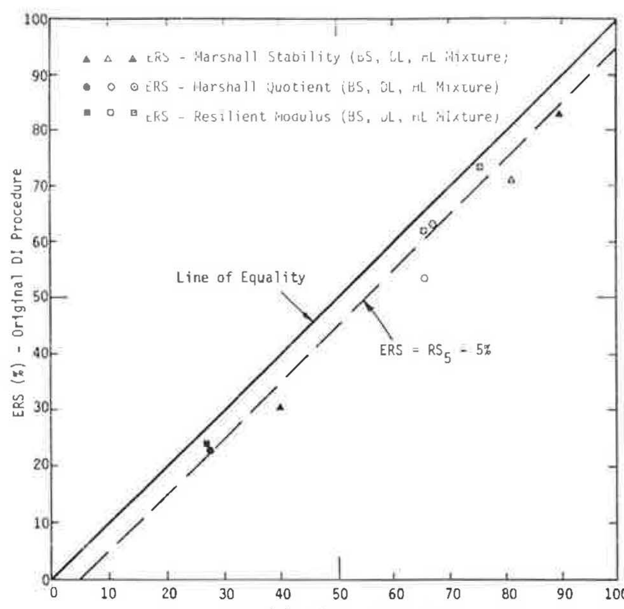

FIGURE 4 Equivalent retained strength values based on original DI procedure versus retained strength values after 5 days of immersion.

TABLE 2 PROPERTIES OF MIXTURES USED (*11*)

Filler Designation	Filler Type	ω_{opt}^{a} (%)	ω_{act}^{a} (%)	Mixture Density (kg/m^3)	Marshall Stability (lb)	Resilient Modulus at 25°C (kg/m^2)	Tensile Strength (psi)	Flow (1/100 in.)	Marshall Quotient (lb/in. × 10^{-2})	Air Voids (%)	Bitumen Saturation (%)	VMA (%)	Permeable Voids[b] (%)
BS	Basalt	6.0	5.5	2394	2,217	37 812	114.5	15.3	114.9	6.83	54.71	15.08	4.8
HL	Hydrated lime	7.0	6.5	2362	1,933	34 580	91.9	18.3	105.6	6.02	64.45	16.93	3.9
DL	Dolomite	5.0	5.0	2454	3,072	32 912	126.9	18.1	169.7	4.65	63.09	12.59	2.8

[a]Percentages of total mixture weight, ω_{opt}, ω_{act} – optimum and actual bitumen contents, respectively.
[b]Lottman's procedures (*5*).

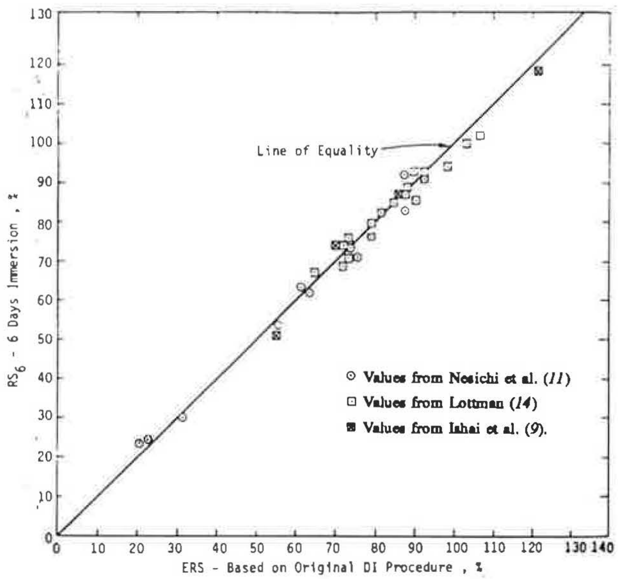

FIGURE 5 Retained strength values after 6 days of immersion versus retained strength values based on original DI procedure.

60- to 70-pen bitumen binder, HFMS-2h hot emulsion binder, PPA binder, etc.) (Table 3) was quite successful, and it was therefore decided to replace the ERS value of the original DI test with one 6-day immersion value. Practically, that means reducing the number of testing sets from five to three (0, 1, and 6 days of immersion), thus reducing the total immersion period from 14 to 6 days. The 1-day retained strength value is needed for control purposes. The switch to the shorter version of the test is supported not only by experience but also by theoretical considerations.

Modified DI Test—Theoretical Considerations

One of the interesting features of the durability curve that characterizes the behavior of mixtures under the first version of the DI test is that it can be described by the mathematical function of the form

$$S_T = S_0 e^{-KT} \tag{3}$$

where

S_0 = the initial strength value (at zero days of immersion) in strength or stability units;

TABLE 3 COMPARISON OF ERS AND RS_6 FOR VARIOUS MIXTURES

Description of Mixture	ERS (%)	RS_6 (%)	Reference
Dense-graded asphalt concrete, 5.5% 60-70 bitumen, dolomite aggregate	54	51	18
Dense-graded asphalt concrete, 7% HFMS-2h hot emulsion, dolomite aggregate	62	63	18
Dense-graded recycled mix, 2% HFMS-2h hot emulsion	75	74	18
Dense-graded asphalt concrete, PPA/VTB (43%/57%) bitumen, basalt aggregate	69	64	19
Dense-graded asphalt concrete, PPA/EXT (83%/17%) bitumen, basalt aggregate	53	48	19
Dense-graded asphalt concrete, PPA/EXT (85%/15%) bitumen, basalt aggregate	67	64	19
Dense-graded asphalt concrete, PPA/EXT (83%/17%) bitumen, dolomite aggregate	83	80	19
Dense-graded asphalt concrete 5% 60-70 bitumen, Israeli Specification 52 gradation	71	69	Unpublished data, 1987
Dense-graded asphalt concrete, 4.5% 60-70 bitumen, French LCPC gradation	86	85	Unpublished data, 1987

S_T = the strength value after T days of immersion, and

K = a constant that dictates the MD buildup rate.

In terms of relative (retained) strength values, S is replaced by RS, and the mathematical function is simplified to

$$RS_T(\%) = 100e^{-KT} \tag{4}$$

It is interesting to note that the same function was found to be applicable to other MD sensitivity tests, such as cyclic TSR tests (*14, 15*) and cyclic water vacuum saturation followed by compressive strength testing (*16*). Another inverse exponential function was used earlier by the authors to quantify reduced asphaltic pavement life from MD (*17*).

Although this function is typical of most paving mixtures, two extreme exceptions do exist: (a) stabilized or treated mixtures (e.g., where hydrated lime filler replaces at least part of the original one), which appear to strengthen during moisture exposure, and (b) extra-sensitive mixtures, which may disintegrate in less than 1 day of immersion (e.g., where glass bead filler replaces the original one). Examples of these extreme types of behavior are detailed elsewhere (*13*).

That the durability curve can be described by Equation 4 is of great practical importance for the DI test in its modified form because it provides a mathematical justification for the replacement of the original DI version by the modified one. Consider, for instance, a typical durability curve (Figure 6), which deteriorates according to Equation 4. In that case, the ERS can be computed as

$$ERS = 100\% - (1/14)\left[100 \times 14 - 100\left(\int_0^{14} e^{-KT}\, dT\right)\right] = 100 - DI \qquad (5)$$

This continuous equation is equivalent to the discrete form of Equations 1 and 2.

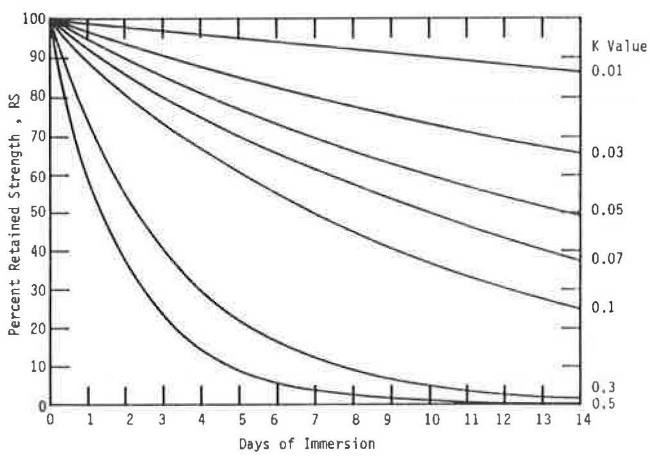

FIGURE 6 Various theoretical durability curves.

As mentioned earlier, and shown in Figure 5, calculations based on strength drops measured daily in immersed samples versus calculations based on the original DI test revealed an excellent match between durability indices in both procedures. Therefore the use of Equation 5 is justified. Moreover, RS values after 6 days of immersion can also be calculated according to

$$RS_6 = 100e^{-K6} \qquad (6)$$

The sensitivity of moisture damage behavior can be demonstrated by comparing RS results for the various K-values (Table 4 and Figure 6). This comparison proves that good agreement exists between results obtained by the original and the modified DI procedures. The large deviations at RS < 25 percent (on the order of 8 to 10 percent) are not important because the samples are in a state of severe failure.

TABLE 4 VALUES OF DI, ERS, RS_6, AND RS_{14} AS A FUNCTION OF K

K	DI (%)	ERS (%)	RS_6 (%)	RS_{14} (%)
0.01	6.68	93.31	94.17	86.9
0.03	18.34	81.65	83.50	65.7
0.05	28.08	71.91	74.08	49.6
0.07	36.26	63.74	65.70	37.5
0.10	46.18	53.82	54.90	24.6
0.30	76.55	23.45	16.50	1.5
0.50	85.73	14.27	4.97	0.1

Equation 4 can shed new light on the traditional Marshall immersion test and criteria. In many countries, the acceptance and rejection limiting criterion in this test is $RS_1 = 75$ percent. Based on Equation 4, this is equivalent to ERS $\cong RS_6 \cong 18$ percent. Israeli experience suggests that this value is far below the minimum acceptable value of ERS (or RS_6), which is about 50 percent (equivalent to $RS_1 \cong 89$ percent in the traditional test).

Assuming that the reduction in the mechanical strength in the DI test reflects moisture damage development under local conditions (hot, humid, and rainy climate), raising the immersion Marshall stability criterion RS by about 14 percent would mean multiplying the long-term equivalent retained stability criterion by a factor of about 2.8.

The ERS or RS_6 criterion is better than the 1-day (RS_1) criterion because this criterion reflects the long-term durability behavior of the mixture and the stabilization of the moisture damage development function (durability curve). At RS_1 the curve is quite steep and sensitive whereas at RS_6 (which is equivalent to ERS) the function is stabilized and flattened.

Practical Application of the Modified DI Test

The following are suggested steps for the practical application of the modified DI test:

1. Prepare three sets of identical Marshall samples or equivalent (at least two sample per set).
2. Test the sets after 0, 1, and 6 days of immersion for their Marshall stability (or other strength test).
3. Compute retained strength values after 1 and 6 days of immersion.
4. Using the expression

$$K = [\ln(RS/100)/T] \qquad (7)$$

calculate K-values for the data given in the following table.

T (days)	*RS (%)*	*Designation of K*
1	RS_1	K_1
6	RS_6	K_6
6	$RS_6 + 5$	K_6^+
6	$RS_6 - 5$	K_6^-

5. Check that the following condition exists:

$$K_6^+ < K_1 < K_6^-$$

This condition is needed to assure that the retained strengths at 1 and 6 days are compatible. If this condition is not satisfied, repeat the test.

6. Report the MD sensitivity of the mixture as the value of retained strength after 6 days of immersion (RS_6).
7. If the mixture is extra-sensitive to moisture damage ($RS_1 \rightarrow 0$) skip stages 4–6 and report its sensitivity to MD as RS_1.

8. If the mixture is treated or stabilized ($RS_1 > 100\%$, $RS_6 > 95\%$) report its MD sensitivity as its RS_6-value and skip stages 4–7.

SUMMARY, CONCLUSIONS, AND SUGGESTIONS FOR FURTHER RESEARCH

This paper has dealt with the development of a moisture damage (MD) sensitivity test for bituminous paving mixtures under hot and humid climatic conditions. The concept and test are based on characterizing the moisture damage sensitivity by long-term durability curves that express the variation of retained strength with hot immersion (60°C) times of up to 14 days. The original version of the test was modified by characterizing the MD sensitivity using the retained strength value at 6 days of hot water immersion.

Both versions of the test have been proven to be superior to the traditional 1-day Marshall immersion test in detecting sensitive mixtures. The modified version is a practical substitute for the former one, which was based on longer immersion periods (up to 14 days) and a greater number of sample sets (five versus three needed by the current procedure). The development of the test was supported by laboratory and initial field experience, as well as by theoretical considerations of behavior of mixtures during long immersion periods.

Further research is needed, however, to improve various aspects of the test, such as

1. Replacing the destructive Marshall test by more fundamental types (e.g., resilient modulus, indirect tensile), which may also result in a reduction in the number of samples needed.
2. Shortening the needed immersion period (e.g., by adding a preliminary vacuum saturation phase). This addition may accelerate MD development, and thus the same amount of MD might be caused in shorter periods.
3. Developing statistical quality control standards to use as acceptance and rejection criteria.
4. Carrying out a Lottman-type (*5*) field study to correlate these criteria with field experience.

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Evaluation of Tests To Assess Stripping Potential of Asphalt Concrete Mixtures

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Stress pedestal, boil, and indirect tensile tests were evaluated for assessing the stripping potential of asphalt concrete mixtures. The tests were applied to surface and base-binder mixtures that included five aggregate combinations, asphalt cement from two sources, and three antistripping agents. The field performance of mixes with the five aggregate combinations ranged from good to poor, and the asphalt cements and antistripping agents are representative of those used in Alabama. The boil and indirect tensile tests were most promising, although neither they nor the stress pedestal test accurately predicted the expected performance of all mixes. They did, however, produce consistent, although at times apparently incorrect, predictions. There was also reasonably good correlation between boil test and indirect tensile test values, which improves their credibility as predictors of stripping when applied to specific mixes. Use of the tests for general evaluation of material sources is not recommended; they must be applied to specific material combinations. Variability in aggregate drying, gradation, and asphalt content may be an important factor affecting stripping potential. Additional testing to establish the influence of these factors is recommended.

The destructive influence of moisture in asphalt concrete has been extensively investigated. Numerous test procedures have been developed and are continually evolving to evaluate asphalt concrete mixtures and possible remedies that could reduce stripping. Materials alone present thousands of variables that may influence stripping (*1*). Kennedy et al. (*2*) in 1983 investigated the stripping potential of selected materials from Texas using three different tests. Stuart (*3*) in 1986 investigated the stripping potential of material selected from several states with several techniques. These two independent studies, and many more, have concluded that there is no generally applicable way to reliably evaluate the water susceptibility of proposed aggregate-asphalt combinations.

The quantification of stripping potential during material selection and mixture design has remained difficult. Conventional specifications for asphalt mixtures do not totally evaluate the asphalt-aggregate bond (*4*). A possible approach that would consider stripping with other mixture requirements could be achieved by following the conventional design method first, then evaluating the proposed mixture by conducting moisture susceptibility tests.

Because of this problem, this study was initiated in 1984 to develop or recommend, or both, a test procedure to assess the stripping potential of Alabama asphalt concrete mixtures and to study the effectiveness of antistripping additives. The study incorporated five aggregate combinations (two limestone and three gravel), asphalt cement from two sources, and three antistripping agents representative of materials used in Alabama. The stripping potential of mixtures designed with the Marshall method was evaluated with boil tests, stress pedestal tests, and indirect tensile tests.

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DEVELOPMENT OF TEST PROGRAM

The testing program was initiated by evaluating the boiling and stress pedestal tests on surface course mixes with the five aggregate combinations and asphalt cement from two sources. The results of these tests were generally better than expected and showed lack of strong correlation with field performance. Indirect tensile tests were then run on asphalt cement from one source. These test results showed similar correlation with field performance.

Further review revealed that coarser and leaner base-binder mixtures may be more vulnerable to stripping than are finer and richer surface mixtures. Field experience reinforced this belief. Cores taken from pavements constructed with some of the aggregates under study showed that stripping generally began and was concentrated in bottom layers of pavements. A significant exception to this has been stripping in surface mixes that have been overlaid. At this stage, the decision was made to evaluate base-binder mixtures and to eliminate the stress pedestal test. The remaining tests were performed with all five aggregate combinations and asphalt cement from two sources. The effectiveness of antistripping agents was studied at all stages of the testing program.

MATERIALS

Properties of the component materials and mixes have been previously described (*5*) and will only be summarized here.

Asphalt Cement

Asphalt cements were obtained from two sources and were labeled AC1 and AC2. Both were viscosity grade AC-20 meeting Alabama Highway Department specifications. The manufacturers mix crude from various sources, but at the time of sampling the majority of the crude oil was from the Gulf of Mexico.

Aggregate

Aggregate combinations were selected, after consultation with Alabama Highway Department central laboratory and division personnel, to provide a range of field performance from good to poor. The characterization of an aggregate combination is subjective and based on experience of field personnel with asphalt-aggregate mixes containing the aggregate. The characterization is, therefore, general in nature rather than specific and relates to the potential for stripping rather than to the performance of a particular mix. A reasonable characterization would be that a mix of "A" materials would be less likely to strip than a mix of "B" materials. As results presented later will verify, factors other than aggregate composition (gradation and asphalt content) influence test results.

Five typical aggregate combinations of from three to five individual aggregates each were selected and arbitrarily labeled A through E. Aggregates were combined to produce mixes that met either surface or base-binder course specifications. Therefore, for each aggregate combination, there will be a surface mix and a base-binder mix. The gradations and design asphalt contents were those obtained by the Marshall mix design procedure.

Combination A

These are basically limestone mixes that have good reported performance with few signs of pavement distress attributable to stripping. Surface Mix A contains 85 percent crushed limestone and 15 percent natural sand and has an asphalt content of 5.5 percent. It has been used primarily for shoulder paving and leveling. Base-Binder Mix A contains 100 percent crushed limestone and has an asphalt content of 4.25 percent. The limestone is dense (specific gravity ≈ 2.8) dolomitic material with an absorption of about 1 percent.

Combination B

These are basically gravel mixes with variable reported performance. Before the use of antistripping additives, stripping damage was severe. Antistripping additives have improved performance; however, some stripping problems are still reported. Both surface and base-binder mixes contain 10 percent limestone screenings and 90 percent siliceous sand and gravel. The surface mix has an asphalt content of 7.5 percent and the base-binder mix 4.5 percent. The gravel and sand are from the same source and are described as "cherty" materials (specific gravity ≈ 2.5) with relatively high absorption (3 percent). The surface mix contains crushed gravel and the base-binder mix contains uncrushed gravel.

Combination C

These are siliceous gravel mixes with moderate reported performance. Even before the use of antistripping additives, only minor stripping problems were reported. Both the surface and the base-binder mixes contain 15 percent fine sand and 85 percent coarse sand and gravel from a primary source. Asphalt contents are 6.25 and 4.55 percent for the surface and base-binder mixes, respectively. The coarse sand and gravel are predominantly sound quartz and quartzite materials (specific gravity ≈ 2.6) with relatively low absorption (1 percent).

Combination D

These are siliceous gravel mixes with poor reported stripping performance. The use of antistripping additives has improved performance, but gravels from this region of the state continue to be regarded as particularly susceptible to water damage. The mixes contain 10 and 15 percent fine sand and 90 and 85 percent washed sand and gravel from a primary source. Asphalt contents are 6.25 and 4.9 percent for the surface and base-binder mixes, respectively. The washed sand is primarily sound quartz, but the coarser particles tend to be similar to the gravel. The gravel is a highly variable cherty material (specific gravity ≈ 2.5) including light and porous particles. Absorption is relatively high at about 2.7 percent.

Combination E

These are basically limestone mixes with good reported stripping performance. Both the surface and the base-binder mixes contain 10 percent natural sand and 90 percent crushed limestone from a primary source. Asphalt contents are 5.5 and 4.15 percent for the surface and base-binder mixes, respectively. The limestone has a relatively high calcium carbonate content (approximately 90 percent), a specific gravity of about 2.6, and absorption of about 1 percent.

Antistripping Additives

Three antistripping additives were used: hydrated lime and two proprietary chemical agents. The hydrated lime (HL) is high calcium and was applied at a rate of 1 percent by weight of aggregate. One proprietary liquid agent, labeled BA, is a metalloamine (or polyamine) with a recommended dosage rate of 0.5 percent by weight of asphalt cement. The second proprietary liquid agent, labeled KB, is an amidoamine with a recommended dosage rate of 0.5 to 1 percent by weight of asphalt cement.

TEST PROCEDURES

Aggregates were combined according to the job mix formulas, and sieved on eight sieves to produce portions with particle sizes ranging from passing $1^1/_2$ in. to No. 200. Required aggregate from each portion was then combined to meet required gradations. After this stage, sample preparation and testing were dependent on the type of test.

Indirect Tensile Test

Samples were prepared in accordance with ASTM D 1559. Mixing and compaction temperatures were selected on the basis of asphalt cement viscosity. Compaction levels were varied to meet the 6 to 8 percent voids requirement. Two testing procedures were used (Table 1).

Boil Test

Samples for boil test were prepared and tested in accordance with ASTM D 3675 except for the following variations:

TABLE 1 INDIRECT TENSILE TEST PROCEDURES

Treatment	Procedure 1	Procedure 2
Mix aging	No aging	15 hr at 140°F
Compacted specimen curing	No curing	24 hr at room temperature
Initial saturation (%)	60–80	60–80
Freezing	No freezing	15 hr at 0 ± 4°F
Soaking	24 hr at 140°F 3 hr at 77°F	24 hr at 140°F 3 hr at 77°F
Age of specimen at testing (days)	2	4
Voids range (%)	6–8	6–8
Loading strips (width in in.)	1/2	1/2
Rate of loading (in./min)	2	2
Testing temperature (°F)	77	77
Similar procedure	Tunnicliff and Root (*1*)	Modified Lottman (*6*)

1. Boiling time was 10 min,
2. Samples were stirred three times during boiling, and
3. Specimens were cooled to room temperature before the water was drained.

Details of the test procedure are given elsewhere (*7*).

Stress Pedestal Test

The test procedure is an adaptation of a test proposed by the Laramie Energy Technology Center (*8*). The test was performed according to procedures recommended by Kennedy et al. (*9*).

DISCUSSION OF TEST RESULTS

Indirect Tensile Test

Results from indirect tensile tests are summarized in Table 2 and plotted in Figures 1 and 2. The following can be inferred from these data:

1. Test Procedure 2 is more severe than Procedure 1 (except for Aggregate Combination B). This may be the result of differences in aging of the mixture, curing of the compacted specimen, and specimen conditioning. The aging and curing for Procedure 2 may enhance adhesion and result in a higher mechanical strength. The cycle of freezing for Procedure 2 may result in larger loss of strength than does only soaking in Procedure 1. In general, the larger strength loss more than compensates for the strength increase due to aging and curing. However, Aggregate Combination B has porous cherty gravel of high absorption. The aging and curing may have increased asphalt absorption and adhesion enough to offset the more detrimental effects of freezing. The net result is that Procedure 2 gave higher strength ratios for this particular aggregate combination.

2. From Figure 2 it appears that mixtures with AC2 are somewhat less susceptible to water damage than are those with AC1, although the differences are not large. Standard physical tests indicate no dramatic differences in asphalt cement properties, and retained strength differences are thought to be the result of asphalt-aggregate interaction.

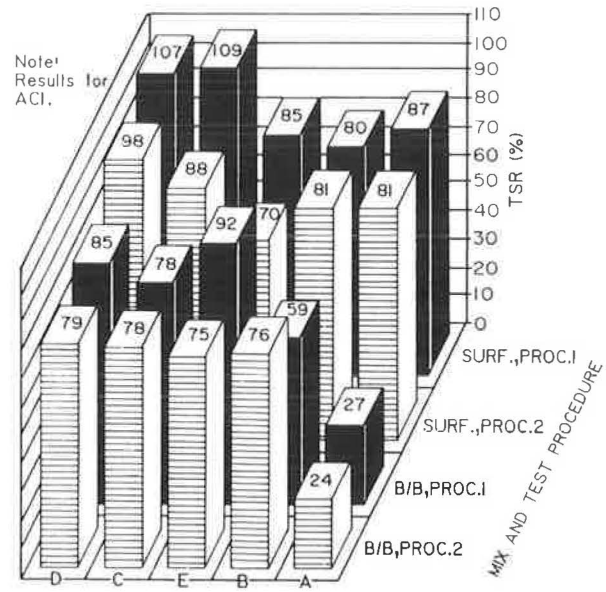

FIGURE 1 Effect of test procedure and type of mixtures on TSR.

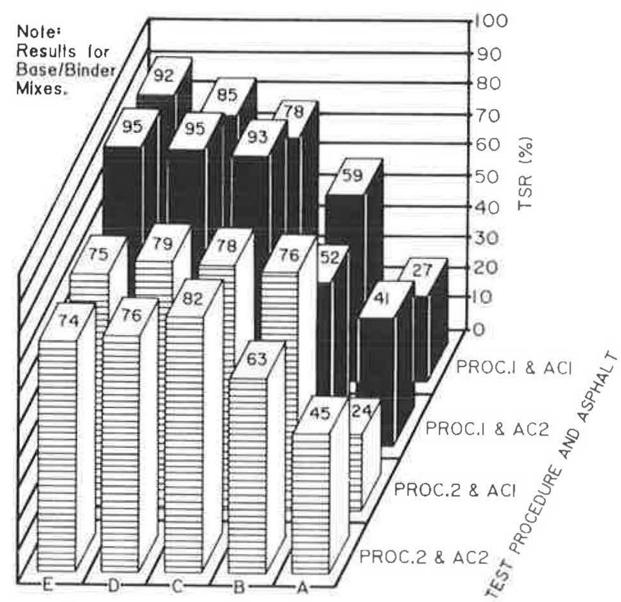

FIGURE 2 Effect of test procedure and source of asphalt cement on TSR.

3. Figure 1 shows that base-binder mixtures are more vulnerable to water damage than are surface mixtures. Speculation is that the differences in tensile strength ratios (TSRs) are due to the coarser gradation and lower asphalt content of the base-binder mixtures. Asphalt content of base-binder mixtures is compensated for somewhat by coarser gradation, but generally these mixtures are "leaner" than surface mixtures. Gradation will also affect the nature of the voids in a mix. Although void content was controlled at 6 to 8 percent, coarser gradation will produce fewer but larger voids. These larger voids will permit easier access to water and, thus, increase the potential for stripping. This phenomenon was apparent during vacuum saturation. During the trial-and-error attempts to achieve 60 to

TABLE 2 INDIRECT TENSILE TEST RESULTS, NO ADDITIVES

		Procedure 1[a]				Procedure 2[a]			
Aggregate Combination	Asphalt Content (%)	Initial Voids (%)	Final Saturation (%)	TSR (%)	SMR (%)	Initial Voids (%)	Final Saturation (%)	TSR (%)	SMR (%)
Asphalt Cement 1									
A									
Surface	5.5	6.5	89	87	–	6.2	91	81	58
B/B	4.25	7.4	94	27	11	7.7	89	24	10
E									
Surface	5.5	6.4	82	85	59	7.4	96	70	56
B/B	4.15	7.0	88	92	90	6.8	88	75	47
C									
Surface	6.25	7.3	80	109	–	6.9	82	88	72
B/B	4.55	6.9	89	78	45	6.7	85	78	55
B									
Surface	7.5	7.0	101	80	–	6.2	101	81	53
B/B	4.5	6.6	100+	59	30	6.4	100+	76	60
D									
Surface	6.25	7.4	88	107	76	7.5	93	98	71
B/B	4.9	6.6	97	85	47	6.6	98	79	44
Asphalt Cement 2									
A, B/B	4.25	6.7	100+	41	17	6.4	100+	45	23
E, B/B	4.15	6.5	85	95	83	6.9	88	74	43
C, B/B	4.55	7.2	93	93	68	6.7	83	82	68
B, B/B	4.5	7.7	100+	52	29	7.2	100+	63	55
D, B/B	4.9	6.9	95	95	54	7.2	100	76	46

NOTE: B/B = base-binder.
[a]Average of at least three specimens.

80 percent saturation, less intense partial vacuums and much smaller times were required with base-binder mixtures.

Table 3 gives a summary of a three-way analysis of variance to determine the overall effect of test parameters on tensile strength and TSR for different mixtures. In this table, the aggregate combinations are grouped into three subgroups in relation to reported field performance as follows:

- Nonstripping mixtures: Aggregate Combinations A and E,
- Stripping mixtures: Aggregate Combinations B and D, and
- Variable mixture: Aggregate Combination C.

Analysis of variance was conducted on all mixtures as well as on stripping and nonstripping subgroups at the 5 percent level of significance. Table 3 can be interpreted as follows:

1. Aggregate mineralogy is the dominant factor affecting tensile strength and TSR.
2. Test procedure and source of asphalt cement affect both conditioned and unconditioned strengths with a resulting insignificant effect on TSR.
3. There is no relationship between the combined effect of test parameters and the individual effect of each parameter. For example, if the source of asphalt produces a significant effect and the type of aggregate also produces a significant effect, the combined effect may or may not be significant.

The effectiveness of antistripping agents was studied by using them to improve mixtures that had low TSR. In accordance with this criterion, antistripping agents were used in base-binder mixtures for Aggregate Combinations A and B only.

Figure 3 shows the sensitivity of the tested mixtures to antistripping agents. It can be seen from the figure that the agents increased TSR values above the 70 to 80 percent range only for Aggregate Combination B. A possible reason is that the agents were formulated for siliceous material not limestone. However, an extenuating circumstance is the lower TSR values for Combination A without additives. The percentage increases in TSR are similar for both combinations.

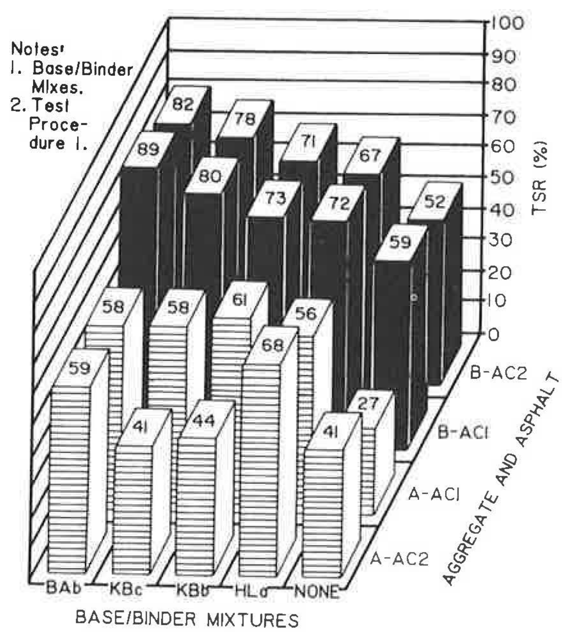

FIGURE 3 Effect of additives on TSR for Aggregate Combinations A and B.

TABLE 3 EFFECT OF TEST PARAMETERS (significance table)

Test Variable	Type of Mixture	Test Parameters: Asphalt Cement Source (A)	Test Procedure (T)	Aggregate Type (M)	Combinations of Test Parameters: AT	AM	TM	ATM
TSR	All	N	N	S	N	S	S	–
	Nonstrip	N	N	S	N	N	N	–
	Strip	N	N	S	N	N	N	–
Unconditioned strength	All	S	S	S	S	N	N	S
	Nonstrip	S	N	N	N	N	N	N
	Strip	S	S	S	N	N	N	N
Conditioned strength	All	S	S	S	N	S	S	S
	Nonstrip	S	N	S	N	N	S	S
	Strip	N	S	N	S	N	N	S

NOTE: S = significant at 5 percent level (α = .05), N = not significant at 5 percent level (α = .05), and dashes = not tested.

The Duncan multiple range test, at the 5 percent significance level, was used to rank the additives according to their improvement of TSR. For Limestone Mix A, all of the additives fell at the same rank. For Gravel Mix B, at the 5 percent level of significance, antistripping agents BA and KB at 1 percent dosage were the first ranked, KB at 0.5 percent dosage was the second, and the hydrated lime was third.

Boil Test

Boil test results are tabulated in Table 4. Values without antistripping additives are plotted in Figure 4. Comparison of coating retention for AC1 and AC2 indicates little difference. Coating retention differences for surface and base-binder mixtures can be noted in Figure 4. Base-binder Mixtures A and B retain much less asphalt after boiling than do surface mixtures. The differences are much less pronounced for Aggregate Combinations C, D, and E, which is consistent with indirect tensile test results.

The effect of additives on coating retention was also investigated for Aggregate Combinations A and B. The data in Table 4 indicate greater improvement in coating retention for liquid agents than for hydrated lime. When lime is used, a white powdery coating (assumed to be due to unbound lime) often results. This tends to reduce the luster and intensity of the black coating, which in turn reduces perceived coating retention and, therefore, the rating.

This observation suggests that the boiling test may not adequately judge the effectiveness of lime as an antistripping agent. Hazlett (*6*) has also suggested that the boil test more favorably evaluates liquid antistripping agents. The data in Table 4 also indicate that the antistripping agents improved coating retention more for Aggregate Combination B than for Aggregate Combination A. This is consistent with the indirect tensile test results.

Stress Pedestal Test

A limited study was performed with surface mix aggregate proportions only. Asphalt cement from both sources and three antistripping agents were used in testing the five aggregate combinations.

Test results are given in Table 5. No significant difference between AC1 and AC2 could be detected. Lime increased cycles for cracking above 25 [suggested limit in Kennedy et al. (*9*)] for Aggregate Combinations A, B, and C. With the exception of Agent BA at 0.5 percent dosage, liquid antistripping agents did not increase cycles to cracking. Unlike the boil test, the stress pedestal test favorably evaluates lime.

TABLE 4 BOIL TEST RESULTS WITH AND WITHOUT ADDITIVES

Aggregate Combination	Mix Type	Percentage of Asphalt Coating Retained: Asphalt Cement 1 with Antistripping Agent: None	HL[a]	BA[b]	KB[b]	KB[c]	Asphalt Cement 2 with Antistripping Agent: None	HL[a]	BA[b]	KB[b]	KB[c]
A	Surface	70	80	95	95	–	70	90	85	80	–
	Base	25	50	60	60	–	35	50	60	50	55
E	Surface	95	–	–	–	–	90	–	–	–	–
	Base	90	–	–	–	–	90	–	–	–	–
C	Surface	95	–	–	–	–	75	–	–	–	–
	Base	80	–	–	–	–	85	–	–	–	–
B	Surface	55	70	90	60	85	60	75	90	50	80
	Base	25	65	95	70	80	35	55	90	75	80
D	Surface	95	–	–	–	–	95	–	–	–	–
	Base	90	–	–	–	–	95	–	–	–	–

[a]1 percent hydrated lime (based on aggregate weight).
[b]0.5 percent antistripping agent (based on asphalt weight).
[c]1 percent antistripping agent (based on asphalt weight).

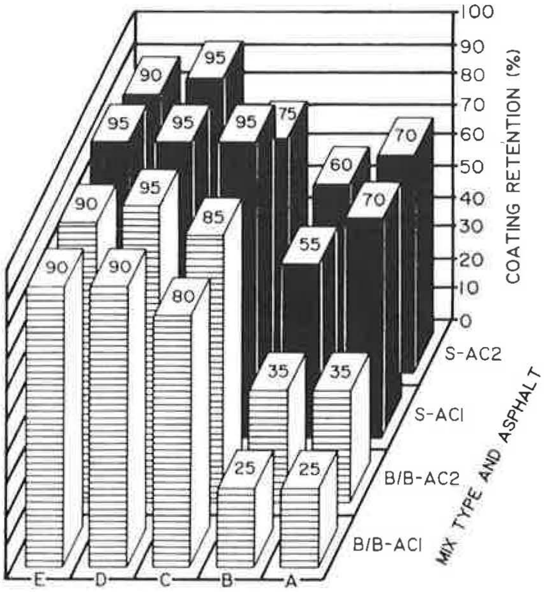

FIGURE 4 Effect of type of mixture and source of asphalt on coating retention.

Comparison of Boil and Indirect Tensile Tests

Both the indirect tensile and the boiling tests express the moisture damage to the mix as a ratio. A correlation of these results is shown in Figure 5 in which TSR for Test Procedures 1 and 2 are considered separately. Both least-squares linear regression equations fall to the right of the line of equality, indicating that the TSR percentage is greater than the coating retention percentage. The coefficients of determination indicate a much stronger correlation with Procedure 1. The freezing in Procedure 2 may introduce additional variability. The positive nature of the correlations, combined with the reasonably strong coefficient of determination for Procedure 1, indicates that both tests are, at least in part, measuring similar phenomena.

The test results are plotted in Figures 6 and 7 for surface and base-binder mixtures, respectively. If 70 percent TSR and 90 percent coating retention are used as criteria for separating stripping and nonstripping, Figure 6 indicates that both tests correctly characterize Mixture E as a nonstripper and incorrectly characterize Mixture D as a stripper. Moreover, the indirect tensile test results indicate that all of the surface mixtures are nonstrippers, and the boiling test predicts only Mixtures A and B as strippers.

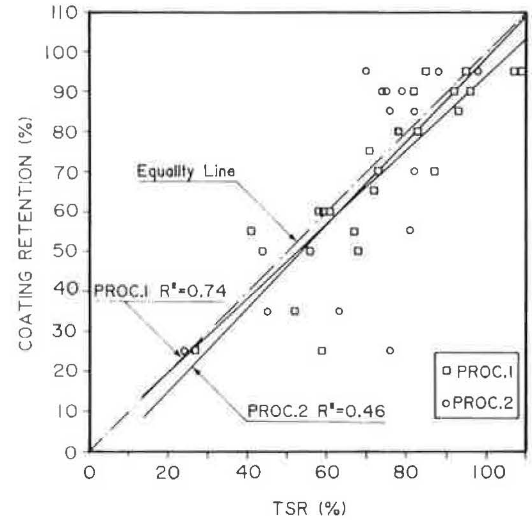

FIGURE 5 Relationship between indirect tensile and boil test results.

Figure 7 shows that, for base-binder mixtures, a strong correlation exists between the boiling and indirect tensile tests. Both tests correctly characterize Mixture E as a nonstripper and incorrectly characterize Mixture D as a nonstripper. Both tests also incorrectly characterize Mixture A as a stripper and correctly characterize Mixture B as a stripper. The boiling test indicates that Mix C is a stripper, and the indirect tensile test indicates that Mix C is a nonstripper.

MATERIAL EVALUATION

The five aggregate combinations, the asphalt cement from two sources, and the two tensile test procedures were evaluated using indirect tensile and boiling test results. Table 6 gives the ranking of the five aggregate combinations based on tensile strengths and stripping resistance as indicated by TSR and coating retention. Each mean value of strength, TSR, and coating retention for a specific type of mix, source of asphalt cement, and type of test procedure was assigned a number of points from one to five. For example, the lowest mean unconditioned strength value for Aggregate Combinations A through E for each test procedure was given one point, the second lowest was given two points, and so on until the highest value was given five points. The number of points for each aggregate combination in each case was totaled. The aggregate combination that had the highest total points was given the highest rank (i.e., one).

TABLE 5 RESULTS OF STRESS PEDESTAL TEST ON SURFACE MIXTURES

Aggregate Combination	Asphalt Cement 1 with Antistripping Agent					Asphalt Cement 2 with Antistripping Agent				
	None	HL[a]	BA[b]	KB[b]	KB[c]	None	HL[a]	BA[b]	KB[b]	KB[c]
A	15	–	–	–	–	13	>25	>25	11	–
E	>25	–	–	–	–	>25	–	–	–	–
C	15	–	–	–	–	18	>25	13	14	–
B	16	>25	13	9	9	17	>25	17	6	7
D	>25	–	–	–	–	>25	–	–	–	–

[a]1 percent hydrated lime (based on aggregate weight).
[b]0.5 percent antistripping agent (based on asphalt weight).
[c]1 percent antistripping agent (based on asphalt weight).

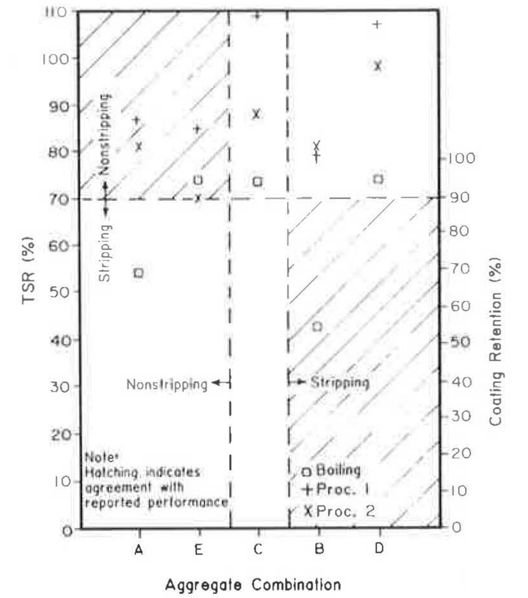

FIGURE 6 Comparison of boil and indirect tensile test results for surface mixtures.

It can be seen from the table that, in terms of unconditioned strength, Mix E has the highest rank and Mix D has the lowest. After conditioning, the table indicates that Mix C has the highest strength and that Mix A has the lowest.

The data in the table indicate that the indirect tensile and the boiling tests are well correlated in ranking the material in terms of stripping resistance as indicated by TSR and coating retention. The ranking indicates that Mixtures D and E are more resistant to water damage than are Mixtures A and B; Mixture C is in the middle. This is consistent with reported field performance for Mixtures B, C, and E but inconsistent for A and D.

Evaluation of test procedures and asphalt cement sources was based on mean values. Table 7 gives mean values for tensile strength, TSR, and coating retention. AC2 consistently produced higher strength, TSR, and coating retention than did AC1. Although the differences are not large, because of their consistency it may be inferred that mixes with AC2 would

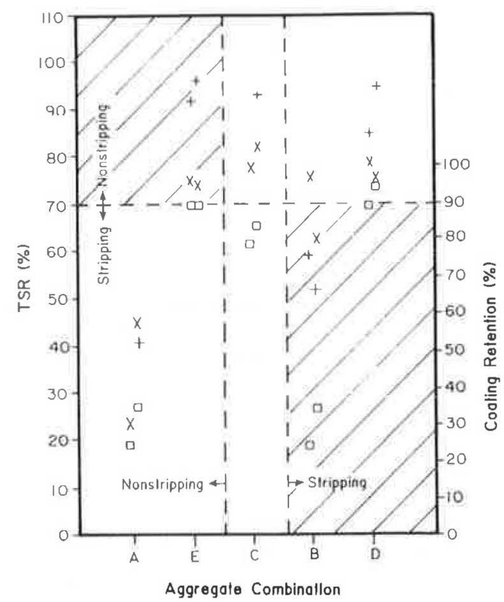

FIGURE 7 Comparison of boil and indirect tensile test results for base-binder mixtures.

TABLE 6 RANKING OF AGGREGATE COMBINATIONS

	Aggregate Combination	Points Based on Means[a]: Surface AC1 Procedure 1	Surface AC1 Procedure 2	Base-Binder AC1 Procedure 1	Base-Binder AC1 Procedure 2	Base-Binder AC2 Procedure 1	Base-Binder AC2 Procedure 2	Total	Rank[b]
Unconditioned strength	A	4	3	5	2	4	4	22	2
	B	2	1	3	4	2.5[c]	5	17.5	3
	C	1	4	2	5	2.5[c]	2	16.5	4
	D	3	2	1	1	1	1	9	5
	E	5	5	4	3	5	3	25	1
Conditioned strength	A	3	3	1	1	1	1	10	5
	B	1	1	2	4	2	3	13	4
	C	2	5	4	5	4	5	25	1
	D	4	4	3	2	3	2	18	3
	E	5	2	5	3	5	4	24	2
TSR	A	3	2.5[c]	1	1	1	1	9.5	5
	B	1	2.5[c]	2	2.5[c]	2	2	12	4
	C	4.5[c]	4	3	4	3	5	23.5	2
	D	4.5[c]	5	4	5	4.5[c]	4	27	1
	E	2	1	5	2.5[c]	4.5[c]	3	18	3
Coating retention	A	2		1.5[c]		1.5[c]		5	4
	B	1		1.5[c]		1.5[c]		4	5
	C	3		3		3		9	3
	D	5		4.5[c]		4		13.5	1.5
	E	4		4.5[c]		5		13.5	1.5

[a]Points = 1 (lowest mean) to 5 (highest mean).
[b]Rank = 1 (highest total) to 5 (lowest total).
[c]Equal means.

TABLE 7 EVALUATION OF TEST PROCEDURES AND ASPHALT CEMENT FROM TWO SOURCES

	Mean Unconditioned Strength (psi)	Mean Conditioned Strength (psi)	Mean TSR (%)	Mean Coating Retention (%)
AC1	142	103	72	72
AC2	146	109	78	73
Procedure 1	133	104	70	–
Procedure 2	153	111	73	–

have somewhat greater resistance to detrimental effects of moisture.

Finally, the data in the table indicate that Test Procedure 2 produces higher strengths but lower TSRs than does Procedure 1. This is as expected, because aging of the mix and curing of the specimens increase strengths in Procedure 2. However, freezing in Procedure 2 produces larger strength reductions that more than compensate for the larger unconditioned strength. This results in lower TSRs for Test Procedure 2.

Numerous factors including aggregate composition, asphalt content, and gradation influence boil and indirect tensile test results. These, combined with the subjective nature of the characterization of the stripping propensity of the mixes, result in the not totally unexpected poor correlation with test results. This should not be interpreted as an invalidation of the test procedures or performance. Rather it indicates that additional refinement of test procedures and applications to specific mixes will be necessary to improve test result–performance correlation.

An extension of this research will provide the data for improving these correlations. Material and mix samples will be taken during construction and subjected to laboratory tests to study the effects of incomplete drying and segregation (gradation). Differences between complete laboratory drying and incomplete field drying may be primary contributors to lack of correlation for porous gravels such as those in Combination D. Cores will be taken immediately after construction and periodically thereafter to study the effects of compaction (voids) and to develop mix-specific performance data.

CONCLUSIONS

Measurement of moisture damage is a complex problem that is sensitive to discrepancies between laboratory and field conditions. Ideal testing and handling of materials, which can be achieved in the laboratory, can hardly be achieved in the field. On the other hand, field environmental conditions can only be approximately simulated in the laboratory. Conclusions include the following observations:

1. A pass-fail criterion, according to which all reported moisture-susceptible mixtures fail and all reported moisture-resistant mixtures pass, could not be developed for any of the tests evaluated.
2. The tests correctly categorized Aggregate Combinations B, C, and E as reported by field performance, but they did not correctly characterize Aggregate Combination A as a nonstripper or D as a stripper.

This weak correlation with field performance, coupled with the strong correlation between test results, led to the following conclusions:

1. The tests may not be valid indicators of stripping, or the subjective reported field performance may not be valid for specific mixes.
2. Variability in gradation, asphalt content, drying, mixing, and compaction may significantly affect stripping potential. Standard laboratory tests on samples with carefully controlled gradation and asphalt content, according to mix design, may not be sufficiently severe. Field sampling and testing should be conducted to establish the influence of construction variability on stripping potential and to establish correlations between laboratory tests and specific mix performance.

Given the assumption that the tests are valid indicators of stripping, the material tested can be described as follows:

1. Limestone Aggregate Combinations A and E have different stripping potential although they possess high tensile strength. Base-binder mixes with Aggregate Combination A have much higher stripping potential than do similar E mixes.
2. Cherty gravel Aggregate Combinations B and D also have different stripping potential; Mix D has lower strength but a higher retained ratio than Mix B.
3. Aggregate Combination C possesses moderate stripping potential.
4. Base-binder mixtures are more susceptible to moisture damage than are surface mixtures made up of the same constituents. This is attributed to differences in asphalt content (film thickness) and the nature of the voids resulting from differences in gradation.
5. AC2 is somewhat more resistant to moisture damage than is AC1.
6. The tests measure improvements when antistripping agents are added. The stress pedestal test assesses the effect of lime favorably, but the boil test assesses its effects unfavorably.

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Effect of Microwave Heating on Adhesion and Moisture Damage of Asphalt Mixtures

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Microwave energy has been demonstrated to be capable of heating pavements rapidly, uniformly, and to depths of up to 5 in. without overheating the surface. Microwave treatment of asphalt mixtures is believed to have the potential of improving asphalt adhesion to aggregate. Presented in this paper is the work that was carried out to investigate this aspect of microwave heating in two ways. First, possible mechanisms by which adhesion improvement may occur when mixtures are exposed to microwave energy are discussed. Second, results of resilient modulus and split tension tests conducted on mixtures that were prepared in the laboratory using a convection oven and a kitchen-type microwave oven are reported. The study involved preparing three groups of mixtures: plain, virgin with an antistripping additive, and artificially aged materials. Test results indicate that microwave energy treatment of asphalt mixtures improves their adhesion and their resistance to water damage.

Using microwave energy to heat and dry pavement materials was thought of more than 20 years ago (*1*). In the last 5 years, however, commercial applications have become of interest to private parties and government agencies in the United States. The FHWA and some state departments of transportation (2), the U.S. Air Force (*3*), and the U.S. Department of Energy (*4*) are some of the agencies that have sponsored early work on microwave heating applications for asphalt and portland cement concrete pavements. Prototype equipment has been built and several field trials have been carried out in the United States and Europe to demonstrate the feasibility of microwave heating of pavement beds. A system that uses microwave energy to heat reclaimed materials for recycling is already in operation in Texas and California (*5*). Development of more efficient and adaptable equipment for the production of various pavement materials and for road construction and maintenance is in progress.

Microwave energy was initially envisioned for rapid, uniform heating of asphalt pavement roads and materials. The benefit of deep heating by microwave without a significant difference in the temperatures of the surface and the bottom of the pavement was also recognized (*6–8*). However, other benefits of microwave heating have also been claimed. A lower aging rate for asphalt cement as a result of faster heating compared with conventional methods has been reported (*9, 10*). Possible improvement in the adhesion of asphalt to aggregate and a consequent increase in water damage resistance have also been reported (*11, 12*).

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Although not the subject of this paper, the successful implementation of commercial microwave equipment is emerging. The economic advantages of microwaves to heat paving materials in place or using an over-the-road technique appear to be positive for special applications that are limited only by imagination. Savings of 30 to 40 percent over currently used hot-mix recycling methods appear to be possible. Special applications might include pothole repair, longitudinal joint heating and repair, warm asphalt emulsion mixtures, and the like. The purpose of this paper and previous work by the authors (*11, 12*) is to explore possible problems with microwave heating as well as explore its many potential benefits.

Details and results of research to evaluate the effect of microwave heating on the adhesion of asphalt cement to aggregate and on the resistance of asphalt mixtures to water damage such as stripping are presented. The preliminary results of this effort were briefly reported earlier (*11*) and are part of a larger study (*12*).

MICROWAVE HEATING CONCEPTS

When microwaves pass through a material, the material is subjected to an alternating electromagnetic field that changes millions of times per second. If the material is electrically neutral (has no electric charge) microwaves will pass through it as if it were not there. Carbon tetrachloride, benzene, paraffin wax, and carbon dioxide are examples of microwave-transparent materials. However, when a material is not electrically neutral, its dipolar molecules, which carry a pair of closely spaced charges equal in magnitude but opposite in sign, tend to act like microscopic magnets in the presence of microwaves and attempt to line up (polarize) with the field. Maximum polarization occurs when all dipoles align with the applied field. Polarization is not restricted to dipolar molecules; any relative displacement of positive and negative charges within the material is considered a form of polarization. Other forms of polarization include electronic; atomic; and, in the case of two adjoining materials, interfacial polarization (*13*). Total polarization is the sum of all of these. Molecules usually fail to keep up with the rapid changes in the direction of the field because some forces, such as viscosity or solidity of the surrounding medium, restrict their movement and because of the effect of simultaneous movements of molecules. In trying to overcome these forces, microwave energy is converted to heat.

Materials differ in their response to microwave energy. Some, such as water and aggregate, heat quite well, but others, such as Teflon and asphalt, exhibit poor response. The penetration of microwaves will be infinite in perfectly transparent substances, zero in reflective materials such as metals, and a finite value in other absorptive materials. The efficiency of a material in absorbing microwave energy, which affects the rise of temperature and penetration of microwaves into the material, may be described by its dielectric properties. The variables that are of interest are the dielectric constant of the material (*e'*), the dielectric loss factor (*e"*), and the dissipation factor or loss tangent of the material (tan *d*). The dielectric constant (*e'*) influences the amount of energy that can be stored in the material in the form of an electric field. The dielectric loss factor (*e"*) indicates how much of that energy a material can dissipate in the form of heat. The loss tangent is equal to *e"*/*e'*. The dielectric constant and loss tangent values have been tabulated for many materials at different frequencies and temperatures (*13, 14*).

The dielectric properties of asphalt cement and aggregates at microwave frequencies are quite low because of the viscosity of the asphalt cement and the lattice forces in the aggregate that hinder the orientation of polar molecules (*15*). Although aggregates possess low dielectric properties, it was found that they are the components that generate heat when asphalt mixtures are subjected to microwaves. Most aggregates respond well to microwaves; only a 14 percent variation in microwaves absorption of these materials has been reported (*6*). Success in heating aggregate is attributed to its low specific heat of 0.2 (*6*); its metallic mineral content (*11, 12*); and, most important, absorbed moisture (*2, 11, 12*). The dielectric properties of some types of aggregates, soils, rocks, and minerals are available (*13, 16, 17*). Dielectric properties of asphalt cement and some types of aggregate are given in Table 1.

MECHANISMS OF IMPROVING ADHESION BY MICROWAVE ENERGY

When pavement material is exposed to microwave energy, aggregate will generate heat and transfer it to the asphalt cement. As long as microwaves are applied, heat generation will continue. The addition of asphalt cement, although it will absorb some of the heat, will not stop the temperature of the mix from rising. The environment (i.e., the air temperature) around the self-heating material is cooler than the material itself. In recycled mixtures, the asphalt cement film on aggregate might be melted, redeposited, and even impregnate permeable voids in the particles as a result of continuous heating as shown in Figure 1a. Thus chances for improved adhesion of asphalt cement to aggregate and resistance to water-stripping action might be increased.

The polarization effect of microwaves could also contribute to improvement in the adhesion of asphalt cement. Polarization would be responsible for orienting dipolar molecules within one material, and interfacial polarization would be responsible for bringing opposite charges on adjoining surfaces to accumulate along the interface as shown in Figure 1b and c. Randomness in orientation of polar molecules in asphalt cement also might be reduced for higher cohesion and shear resistance, if enough energy were available to overcome the viscosity of the asphalt cement.

Polar antistripping agents that are used to promote adhesion of asphalt to aggregate could benefit from the polarization effect of microwaves. Positively charged (cationic) antistripping agents migrate to and are adsorbed by the aggregate surface, lowering its affinity for water and increasing its affinity for oil. This preferential modification of aggregate surface charge favors asphalt cement over water, resulting in stronger adhesion and water-stripping resistance. The degree of success of these agents depends on the concentration of the surfactant used, the efficiency of migration, and the force or strength of the adsorbing bond. The addition of these materials to asphalt cement in hot mixing is believed to be inefficient (*18*). Migration of the agent to the aggregate interface is hindered by the increase in asphalt viscosity on cooling. In typical hot mixtures, unless they are stored for a long time (12 hr), only approximately 30 to 40 percent of the original concentration of antistripping agent performs in the intended manner (*18*). Microwave energy could be used to speed the migration of agents by forced polarization action as shown in Figure 1d. The least microwaves can do is to reduce the randomness in the

TABLE 1 DIELECTRIC PROPERTIES OF ASPHALT PAVEMENT MATERIALS

Material	Temp. °C	freq. MHz	e'	tan d 10^{-4}	Ref.
Water	25	300	77.5	160	[13]
	25	3000	76.7	1570	[13]
Asphalt cement	26	3000	2.5	11	[13]
AC 60/70 Esso	20	2450	2.43	--	[16]
AC 40/50 Shell	20	2450	2.52	--	[16]
AC 180/220	20	2450	2.45	--	[16]
Aggregate -Diorite	20	2450	5.6-7	178 - 357	[16]
Asphalt concrete					
with diorite	20	2450	5.8	344	[16]
with limestone	20	2450	6.7	149	[16]
with quartzite	20	2450	4.0	62	[16]

orientation of charged molecules at the interface of the aggregate surface between agent and aggregate on one side and agent and asphalt cement on the other. The viscosity of the asphalt at which polarization is optimum is a key factor in taking advantage of this phenomenon. Interfacial polarization may not be as strong at microwave frequencies as it is at radio frequencies. However, the total polarization could have a favorable result. On the other hand, if poorly compatible aggregate and binder are treated with microwave energy, mismatching and a double layer may occur, resulting in a weaker bond or complete debonding.

In summary, fast heating by microwaves in conjunction with its polarization effect should reduce asphalt cement aging and improve the bonding of asphalt to aggregate, as well as resistance to water action. Furthermore, the increase in the polarity of the binder would increase its ability to polarize under the applied field, yielding stronger bonding.

EXPERIMENTAL PROCEDURES

The relative effect of microwave heating on asphalt adhesion to aggregate and resulting resistance to the weakening action of water was evaluated by Lottman's procedure for predicting moisture damage to asphalt mixtures (*19*) in conjunction with the diametral resilient modulus test as described by Schmidt (*20*) as well as the diametral split tensile strength.

Three groups of mixtures were tested: (a) virgin plain mixtures, (b) virgin mixtures with an added polar antistripping agent, and (c) recycled mixtures.

To represent existing and anticipated pavement microwave heating systems, several heating methods were used in preparing specimens from the three groups of mixtures. Conventional heating, microwave heating alone, and the latter in combination with conventional heating were the three basic heating methods. In this laboratory study, a kitchen-type microwave oven was used. These devices operate at 2450 MHz, whereas pavement heating equipment uses magnetrons operating at 915 MHz.

Materials

- Aggregate: Crushed glacial gravel, dense graded, conforming to Washington State Department of Transportation (WSDOT) Class B specification,

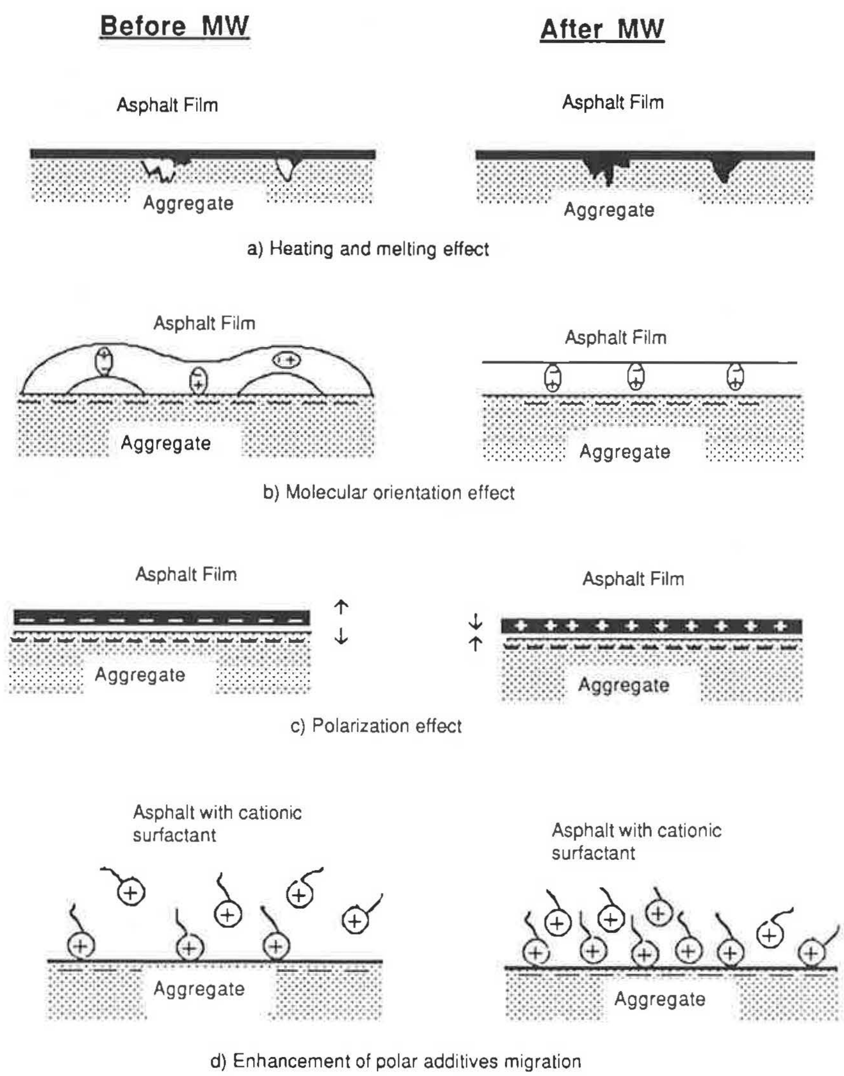


FIGURE 1 Mechanisms of asphalt adhesion improvement with microwave energy treatment.

- Asphalt cement: AR-4000W from Chevron,
- Recycling agent: RA-275 from Pester, and
- Antistripping agent: tallow tetramine from Exxon (Tomah).

Mixtures

- Virgin plain mixtures: These mixtures were prepared with 5.5 percent asphalt cement. Each sample was made by adding 70 g of asphalt to 1200 g of aggregate.
- Virgin mixtures with tallow tetramine: These mixtures were similar to plain mixtures except that tallow tetramine was added to hot asphalt in the amount of 0.5 percent of asphalt weight.
- Recycled mixtures: These mixtures were prepared from materials that were initially similar to plain mixtures. However, asphalt cement content was reduced to 4 percent of total weight. Artificial aging was accomplished by heating loose samples of mixture in a forced-draft oven at 240°F for 24 hr. The recycling agent was added to aged mixtures in the amount of 2 percent of total weight.

Preparation of Specimens

Specimens were fabricated according to the Marshall procedure (ASTM D-1159). Specimens were compacted by applying 50 blows on each side. The heating methods were as follows:

1. Conventional heating: A convection oven was used to heat asphalt cement, aggregate, and recycled asphalt pavement (RAP) to the mixing temperature of 300°F. RAP materials were heating for 2 hr in the convection oven before the addition of the recycling agent. Mixtures prepared by this method of heating were used to produce control specimens for the three groups of mixtures, and the method was intended to simulate conventional mixing in the plant.
2. Microwave heating: A kitchen-type microwave oven was used to heat plain aggregate for virgin plain mixtures and for mixtures with tallow tetramine in addition to RAP from 75°F to about 300°F in 10 min. The asphalt cement and recycling agent had to be heated in the conventional oven because they did not heat by microwaves. Moisture content of aggregate ranged from 0.3 to 0.5 percent. Aggregate was heated on a ceramic plate on top of a turntable in the microwave oven for 7 min. This was followed immediately by 3 min of heating in the microwave oven without the turntable, in order to get the center of the sample as hot as its edges. Preliminary trials showed that the use of the turntable resulted in heating the edges more than the center of the sample. This method was intended to simulate a plant in which only the aggregate is heated by microwaves.
3. Conventional oven plus "zapping" in microwave oven: After hot mixing of virgin plain mixtures and RAP with a recycling agent, the entire mix was treated for 2 min in the microwave oven. In the case of virgin mixtures with the tallow tetramine, three zapping periods were applied: 0.5, 2, and 5 min. This method can be used to simulate postmixing treatment with microwaves at a conventional hot-mix plant.
4. Conventional plus microwave oven supplemental recycle: This method of heating was restricted to recycled mixtures. RAP was heated in the convection oven for 2 hr at 250°F, followed by 4 min in the microwave oven to bring the mixture temperature to 300°F before the recycling agent was added. This technique was used to simulate one of the commercial microwave plants (*5*) that preheats RAP with hot gases, then finishes heating with microwaves.

In the recycled mixtures group, control mixtures, made with virgin and aged materials with no recycling agent heated in the convection oven, were also prepared. All recycled mixtures were subjected to a curing period of 20 min at 280°F in the convection oven after mixing with the recycling agent and before compacting. Materials that were zapped in the microwave oven for 2 min were cured for only 18 min.

Testing

Because the resilient modulus (M_R) test is nondestructive, each sample was used to determine dry M_R; M_R after water conditioning; and, finally, failure by split tensile strength (S_t) test. The extent of stripping was evaluated by visual inspection of failed specimens.

The diametral resilient modulus test was conducted at 74°F before and after Lottman conditioning. Applied load was 100 lb at a frequency of 20 cycles per minute and loading duration of 0.1 sec. Each specimen was tested twice by rotating it 90 degrees and taking the average of 5 readings after 50 conditioning pulses. Before dry testing, specimens were stored in a temperature control chamber overnight. In the case of tests after moisture conditioning, specimens were placed in a water bath at 74°F for at least 2 hr. The split tension tests were also performed at 74°F with a loading rate of 2 in./min. These tests were made only on dry specimens, and no data were available on retained tensile strength after moisture conditioning because comparison specimens were not made.

TEST RESULTS AND EVALUATION

Summaries of test results for all mixtures are given in Tables 2–4. The resilient moduli (M_R) before and after one cycle of freeze thaw conditioning are shown in Figures 2–4. The percentages of retained M_R-values after conditioning are shown in Figures 5–7. The results of the diametral split tensile strength test (S_t) after conditioning are shown in Figures 8–10.

Virgin Mixtures

Results show that zapped mixtures had a higher M_R before and after conditioning and higher S_t than conventionally prepared mixtures. Zapped mixtures were the only material that was entirely treated with microwaves. The heat generated from within aggregate particles coated with asphalt cement might have enhanced the coating and adhesion of asphalt, to which the increase in M_R and S_t might be attributed. It is also possible that heat from zapping caused the asphalt to flow and satisfy aligned charges on the aggregate particles' surface and even to flow into permeable voids. The failure surfaces after the split tension test showed no signs of stripping and compared closely with those of conventional mixtures.

Mixtures prepared from microwave-heated aggregate show the second highest averaged dry M_R. However, the actual data

TABLE 2 SUMMARY OF TEST RESULTS FOR VIRGIN MIXTURES

Specimen no.	Density p/cf	Voids %	M_R Dry 1,000 psi	Avg. 1,000 psi	M_R Cond. 1,000 psi	Avg 1,000 psi	% M_R Retained	Avg. % M_R Retained	S_t psi	Avg. S_t psi
CC1	148.2	3.71	239.26		207.98		86.9		73.4	
CC2	148.2	3.70	227.67		185.55		81.5		99.03	
CC3	148.2	3.74	227.91	231.61	197.32	196.95	86.5	85	106.4	92.9
MW1	147.5	3.98	248.92		169.19		67.9		70.5	
MW2	148.1	3.67	234.95		169.08		71.9		82.88	
MW3	149.0	2.99	278.16	254.01	206.96	181.74	74.4	71.5	89.5	81
CZMW1	148.0	3.79	265.27		211.04		79.5		111.5	
CZMW2	148.2	3.40	259.00		213.63		82.4		111.1	
CZMW3	148.5	3.80	263.6	262.62	234.72	219.8	89.0	83.7	119.4	114
Test temp. (F)			73°		73°				73°	
Loading rate			20 pulse/min		20 pulse/min				2 in/min	
Loading duration			0.1 sec.		0.1 sec.					

CC: Conventional oven heating MW: Microwave oven heating
CZMW: Conventional heating plus zapping in microwave oven

TABLE 3 SUMMARY OF TEST RESULTS FOR VIRGIN MIXTURES WITH ANTISTRIPPING AGENT

Specimen no.	Density p/cf	Voids %	M_R Dry 1,000 psi	Avg. 1,000 psi	M_R Cond. 1,000 psi	Avg. 1,000 psi	% M_R Retained	Avg. % M_R Retained	S_t psi	Avg. S_t psi
CC-T 1	146.70	4.45	214.50		186.60		87.00		99.70	
CC-T 2	146.84	4.23	195.70		175.60		89.70		106.20	
CC-T 3	147.51	3.97	205.50	205.20	202.50	188.23	98.50	91.70	110.30	105.40
MW-T1	147.66	3.93	249.05		211.30		84.80		114.10	
MW-T2	147.28	4.24	237.50		183.90		77.40		110.00	
MW-T3	146.99	4.31	219.30	235.30	188.00	194.40	85.70	82.60	116.40	113.50
CZMW(0.5)-T1	145.62	5.32	201.80		203.30		100.70		105.10	
CZMW(0.5)-T2	146.95	4.50	229.20	215.50	241.80	222.50	105.50	103.20	114.90	110.00
CZMW(2)-T1	145.25	3.79	215.40		209.90		97.40		114.20	
CZMW(2)-T2	147.13	4.18	222.80		222.20		99.70		117.00	
CZMW(2)-T3	147.16	4.33	225.20	221.10	233.00	221.70	103.50	100.30	116.00	115.70
CZMW(5)-T1	147.77	3.94	264.20		260.50		98.60		132.70	
CZMW(5)-T2	148.44	3.48	258.80	261.50	256.78	258.64	99.20	98.90	131.60	132.20
Test Temp. °F			73°		73°				73°	
Loading Rate			20 pulse/min		20 pulse/min				2 in/min	
Load Duration			0.1 Sec.		0.1 Sec.					

CC-T: Conventional oven heating
MW-T: Microwave oven heating
CZMW-T(n): Conventional heating plus zapping in microwave oven. Number in parentheses indicates zapping time.

appear to be widely scattered. One of the points is high, causing the mean value to be to the high side. However, after conditioning, M_R suffered about 29 percent loss, which made M_R after conditioning even lower than that of conventionally prepared mixtures. The split tensile test results correspond very closely with M_R results after conditioning. The uncertainty of uniform heating of aggregate with microwaves and the presence of aggregate particles that were not sufficiently hot when hot asphalt was added and mixed did not facilitate good coating and adhesion. Steps that were taken to ensure uniform temperature through the aggregate mix probably were not adequate. The scattering of points in Figure 2 indicates that uniform heating was not achieved. In addition to probable defects in coating, the alignment of charges on the aggregate surface that were not satisfied by asphalt cement may have increased the affinity of aggregate to water. Examination of the failure surface after the split tension test revealed severe stripping.

Mixtures with Antistripping Agent

Test results show that mixtures made with microwave-heated aggregate had a higher dry M_R. This is believed to be due to the improved orientation of aggregate surface charges. Again, lack of uniform microwave heating is considered the major reason for the greater loss of strength after water conditioning. However, M_R and S_t after the freeze-thaw cycle are slightly higher than for conventional mixing.

In the case of zapped mixtures, M_R before and after the freeze-thaw cycle, percentage of M_R retained, and S_t are higher than those of conventionally prepared mixtures. In this case the entire mixture, including the antistripping agent that had been incorporated in asphalt cement, was exposed to microwave radiation. The test values were in direct proportion to zapping time. The increase in test values of zapped mixtures is thought to have been caused by the mechanisms described earlier. First,

TABLE 4 SUMMARY OF TEST RESULTS FOR RECYCLED MIXTURES

Specimen no.	Density p/cf	Voids %	M_R Dry 1,000 psi	Avg. 1,000 psi	M_R Cond. 1,000 psi	Avg. 1,000 psi	% M_R Retained	Avg. % M_R Retained	S_t psi	Avg. S_t psi
VM1	142.99	8.49	264.30		103.10		39.00		44.20	
VM2	142.47	8.79	268.30	266.30	91.80	97.50	34.20	36.60	42.10	43.20
AM1	140.67	10.00	1018.50		660.30		64.80		143.30	
AM2	139.70	10.30	1072.70		750.60		69.90		153.30	
AM3	140.10	10.18	1006.30	1032.50	772.90	727.93	76.80	70.50	137.30	144.63
CC-R1	144.66	4.80	664.30		579.30		87.20		195.50	
CC-R2	143.99	5.10	665.50		603.60		90.70		190.10	
CC-R3	145.18	4.56	587.30	639.00	600.30	594.40	102.20	93.02	197.20	194.27
MW-R1	145.60	4.30	595.50		588.20		98.70		206.70	
MW-R2	146.00	4.00	616.50		587.00		95.20		205.90	
MW-R3	145.48	4.30	534.10	582.00	580.50	585.23	108.60	100.56	207.20	206.60
CZMW-R1	145.70	4.10	694.30		775.69		111.70		223.50	
CZMW-R2	144.90	4.78	645.70		656.00		101.60		208.70	
CZMW-R3	145.50	4.26	579.79	639.93	675.50	702.40	116.50	109.76	207.10	213.10
CSMW-R1	145.28	4.27	533.50		479.20		89.80		181.60	
CSMW-R2	145.10	4.42	665.20		597.20		89.80		203.10	
CSMW-R3	144.95	4.27	589.30	596.00	591.90	556.10	100.30	93.30	208.80	197.80
Test Temp. (F)			74°		74°				74°	
Loading Rate			20 pulse/min		20 pulse/min				2 in/min.	
Loading Duration			0.1 sec.		0.1 sec.					

VM: Virgin materials. AM: Aged materials. CC-R: Conventional recycle. MW-R: Microwave recycle.
CZMW-R: Conventional plus microwave zapping recycle. CSMW-R: Conventional plus microwave supplemental recycle.

CC: Conventional oven heating
MW: Microwave oven heating
CZMW: Conventional heating plus zapping in microwave oven

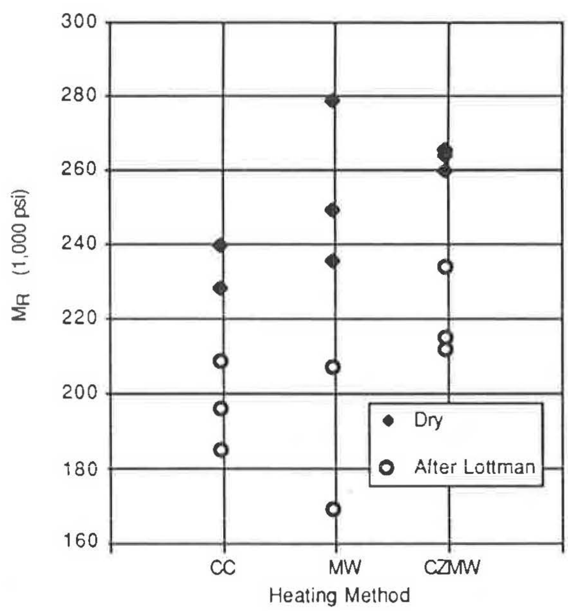


FIGURE 2 Effect of heating method on resilient modulus of virgin asphalt mixtures.

microwave heating might have facilitated uniform coating. Second, the polarization effect of microwaves improved polar molecule orientation and then increased the rate of migration of the antistripping agent toward the aggregate interface. Finally, the extra heating and mixing of these materials aged and stiffened them more. However, the extra heating did not appear to have great impact on stiffness at short zapping times. For example, in the case of zapping for 0.5 min, the temperature actually decreased, which resulted in greater air voids. Yet all test values (i.e., M_R before and after the freeze-thaw cycle, percentage of M_R retained, and S_t) are higher than those for conventional mixtures. At a longer zapping time of 5 min, during which the temperature increases by an average of 41°F, the extra heating effect might be more significant.

Recycled Mixtures

Results show that there was no major difference among dry M_R of all recycled mixtures subjected to the different heating methods.

Unlike that of fresh mixtures, dry M_Rs for both conventionally recycled and zapped recycled mixtures are almost identical. However, the M_R of zapped mixtures after Lottman's conditioning exhibited a slight increase for all samples tested. It is true that the number of samples may not be large enough to make an inference about this behavior, but zapped mixtures were exposed to microwave energy after the addition and mixing of the recycling agent. It is possible that some structuring or polarization of the asphalt cement and the recycling agent had taken place and that, in the presence of water and exposure to a temperature of 140°F for 24 hr, aging or strengthening resulted. If that is the case, the selection of a recycling agent that is compatible with microwave treatment ought to be of concern when designing microwave-heated recycled mixtures. Split tensile strength is shown to be the highest for zapped mixtures (about 10 percent higher than that of conventionally recycled mixtures); split tensile strength of

CC: Conventional oven heating

MW: Microwave oven heating

CZMW (0.5): Conventional heating plus zapping in microwave oven for 0.5 minutes

CZMW (2): Conventional heating plus zapping in microwave oven for 2 minutes

CZMW (5): Conventional heating plus zapping in microwave oven for 5 minutes

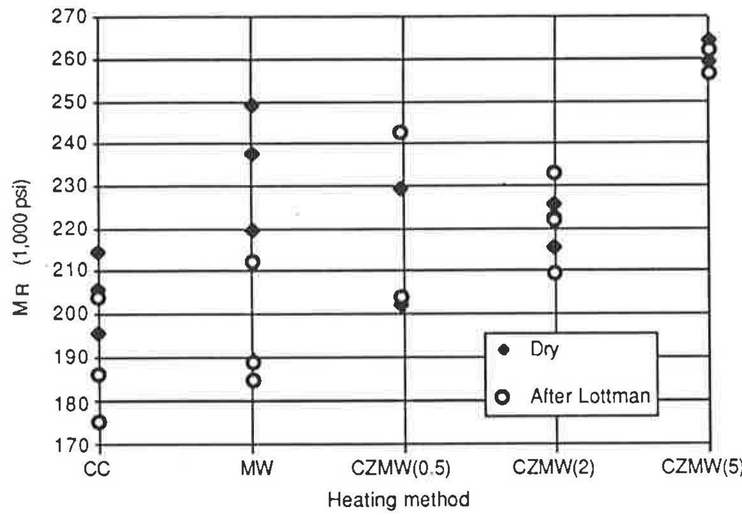


FIGURE 3 Effect of heating method on resilient modulus of virgin mixtures with antistripping additive.

VM: Virgin materials

AM: Aged materials

CC-R: Conventional recycle

MW-R: Microwave recycle

CZMW-R: Conventional plus microwave zapping recycle

CSMW-R: Conventional plus microwave supplemental recycle

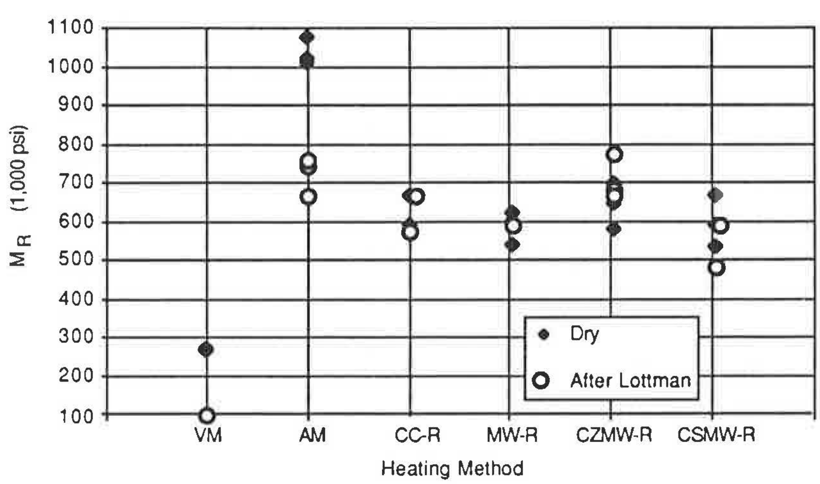
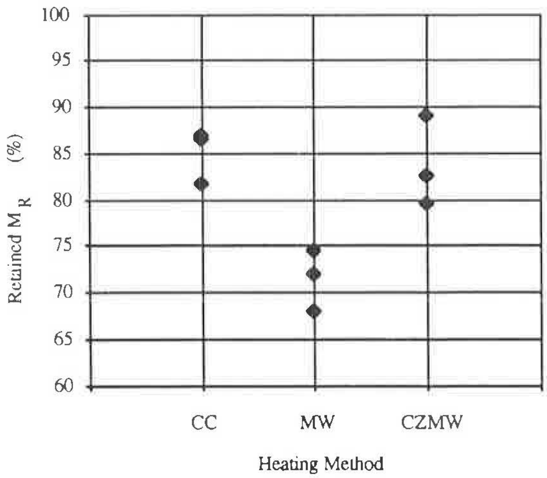


FIGURE 4 Effect of heating method on resilient modulus of recycled mixtures.



CC: Conventional oven heating
MW: Microwave oven heating
CZMW: Conventional heating plus zapping in microwave oven

FIGURE 5 Effect of heating method on percentage of resilient modulus retained for virgin mixtures after Lottman conditioning.

CC: Conventional oven heating
MW: Microwave oven heating
CZMW (0.5): Conventional heating plus zapping in microwave oven for 0.5 minutes
CZMW (2): Conventional heating plus zapping in microwave oven for 2 minutes
CZMW (5): Conventional heating plus zapping in microwave oven for 5 minutes

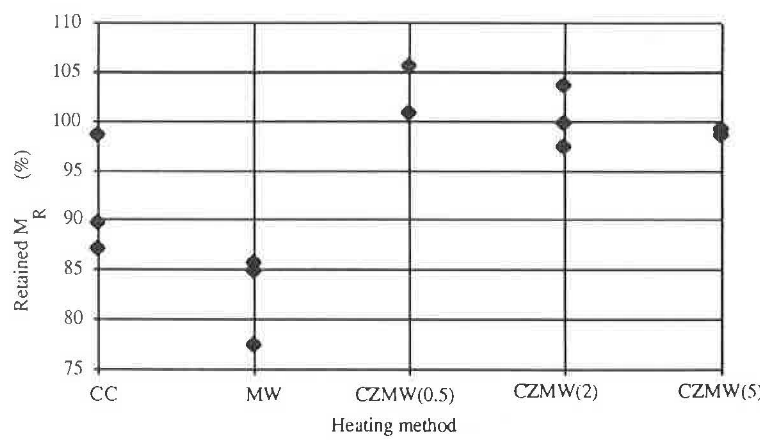
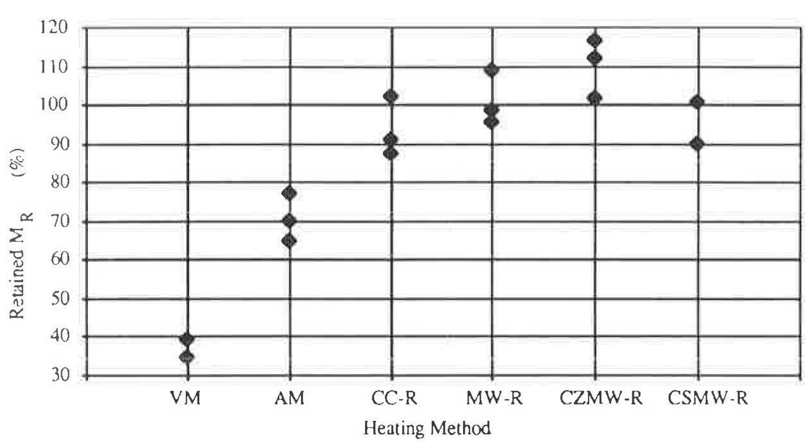


FIGURE 6 Effect of heating method on percentage of resilient modulus retained for virgin mixtures with antistripping agent after Lottman conditioning.



VM: Virgin materials
AM: Aged materials
CC-R: Conventional recycle
MW-R: Microwave recycle
CZMW-R: Conventional plus microwave zapping recycle
CSMW-R: Conventional plus microwave supplemental recycle

FIGURE 7 Effect of heating method on percentage of resilient modulus retained for recycled mixtures after Lottman conditioning.

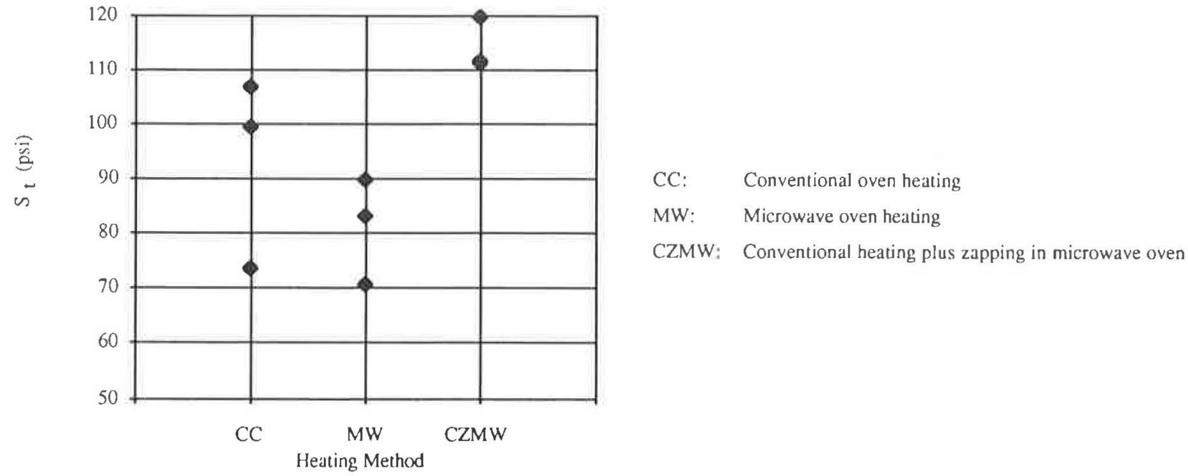


FIGURE 8 Effect of heating method on split tensile strength of virgin mixtures after Lottman conditioning.

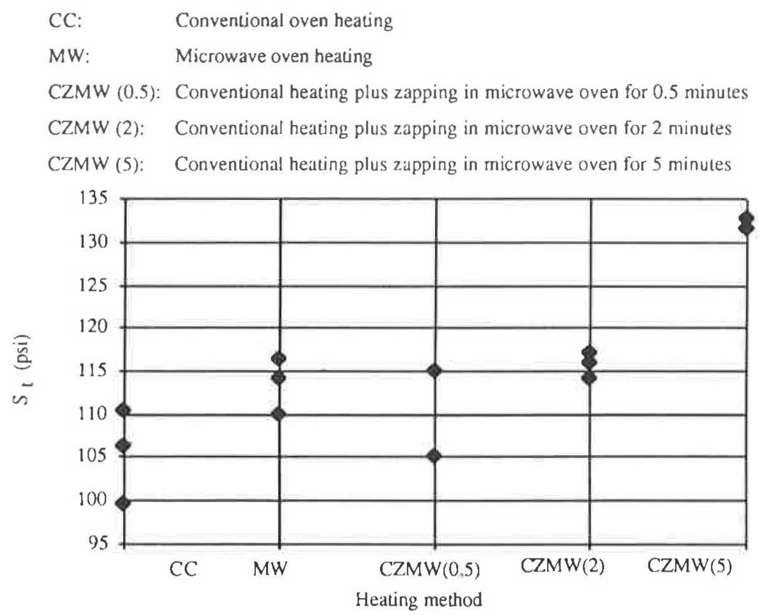


FIGURE 9 Effect of heating method on split tensile strength of virgin mixtures with antistripping additive after Lottman conditioning.

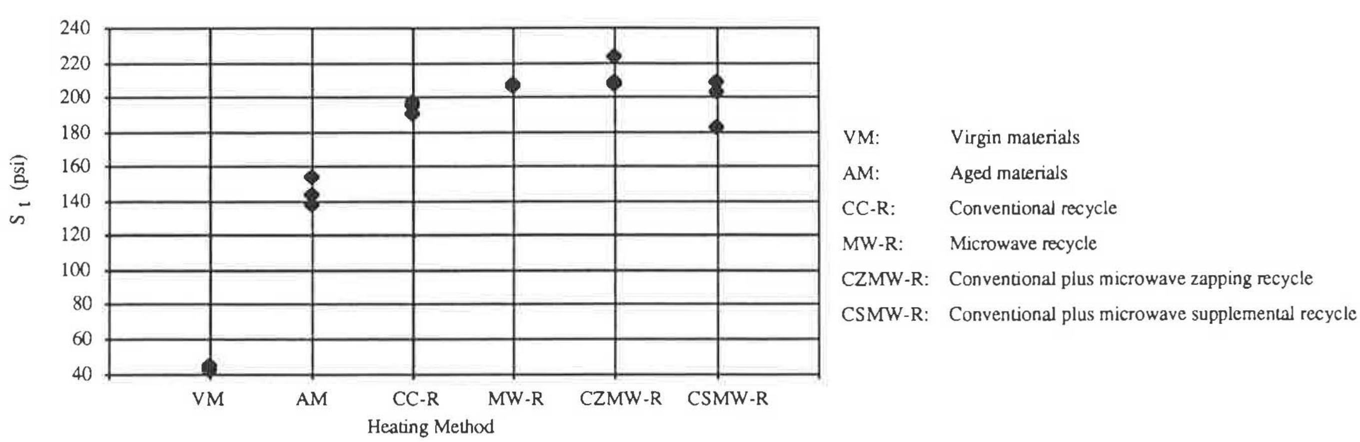


FIGURE 10 Effect of heating method on split tensile strength of recycled mixtures after Lottman conditioning.

microwave-heated mixtures is only 7 percent higher. Examination of samples after split tensile strength tests reveals that none of the recycled mixtures suffered any stripping.

CONCLUSIONS

On the basis of the results of this study, it appears that two conclusions are warranted:

- The zapping of hot asphalt mixtures, virgin or recycled, in the microwave oven has resulted in higher M_R and S_t indicating an improvement in asphalt bonding to aggregate. The resistance to stripping of zapped mixtures is as good as or better than that of conventionally heated mixtures.
- However, heating of plain aggregate in the microwave oven alone does not produce a uniform temperature. Consequently, adhesion of asphalt to aggregate may not be uniform, and stripping and reduction in strength may occur even with the use of polar antistripping additives.

These findings appear to support the hypothesized mechanisms of microwave treatment's role in improving the adhesion of asphalt to aggregate.

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Effect of Aggregate Properties on Stripping

HYON H. YOON AND ARTHUR R. TARRER

A laboratory investigation was conducted to relate some measurable aggregate properties to the stripping propensity of a mix of aggregate and asphalt cement. Several different types of aggregates were used in this study; each was characterized in terms of its physical properties, such as pore volume and surface area, and its chemical and electrochemical surface properties. Stripping propensity was determined by using a boiling water test. Under certain conditions, aggregates that have a relatively high surface electrical potential and those that impart a high pH value to water in contact with their surface were found to exhibit a high susceptibility to stripping. These results were interpreted in terms of electrochemical properties of the aggregate surface. It was also observed that the effectiveness of some types of antistripping additives was sensitive to the pH of water in contact with the aggregate and that curing the asphalt-aggregate mixture and precoating the aggregate improved stripping resistance considerably.

Stripping of asphalt films from aggregate surfaces occurs when there is a loss of adhesion between the aggregate surface and the asphalt cement. This is primarily due to the action of water or moisture. The resulting deterioration can be a serious problem, which causes loss of integrity of the asphalt concrete and subsequent failures that require early and costly maintenance. The primary factors that affect stripping in an asphalt paving mixture include the physical and chemical properties of both the aggregate and the asphalt cement (such as surface characteristics and mineral composition of the aggregate and chemical composition, surface tension, and viscosity of the asphalt). Construction methods and external environmental factors also contribute to stripping. Stripping is, therefore, a complex phenomenon influenced by various factors.

Over the years, a great deal of basic and applied research has been conducted to determine the nature of adhesion and stripping in asphalt paving mixtures (*1–9*). Several major theories of stripping have been detailed and are summarized by Majidzadeh and Sanders (*10*) and Taylor and Khosla (*9*). The most widely accepted mechanisms of stripping are detachment, displacement, spontaneous emulsification, and pore pressure.

The rationale for these stripping mechanisms is provided by the mechanical, thermodynamic, or interfacial energy or chemical concepts of adhesion and loss of adhesion, or both. The mechanical concept that explains the stripping mechanisms suggests that the bond strength between the asphalt and the aggregate surface is dependent on a mechanical interlock developed by the penetration of the asphalt into pores and cracks on the surface of the aggregate particle (*1*). The thermodynamic or surface energy concept involves the wetting behavior of asphalt at the asphalt-aggregate-water-air interface (*1, 7, 11–14*). The degree to which stripping occurs depends on the interfacial free energy relationships at the aggregate-asphalt-water-air interface. The chemical concept involves adsorption of asphalt on aggregate surfaces and chemical reactions between the adsorbed asphalt compounds and the constituents of the aggregate surface (*7, 8, 15*). Herein, water solubility of the asphalt-aggregate bond is the main factor affecting stripping.

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It is generally thought that the primary mechanism responsible for stripping involves the displacement by water of an asphalt cement film from the aggregate surface. The actual role of water in stripping is, however, still not entirely understood. Also, it has been difficult to relate quantitatively stripping potential to materials selection and mixture design parameters.

The purpose of this research work was to study the nature of stripping in asphalt paving mixtures and to determine the importance of the physical and chemical properties of the aggregate in the mix.

Table 1 gives a summary of the physicochemical properties of asphalt, aggregate, and the asphalt- aggregate mixture that might influence stripping, according to different mechanistic stripping theories. Although all of these properties are involved, this research was primarily concerned with aggregate properties such as surface area, porosity, and chemical and electrochemical properties.

Several different types of aggregates were selected for this study, and physical properties, such as surface area and pore volume, were measured. Changes in the pH of water after the addition of aggregate and in the surface charge of aggregates in water were also measured. These measurements gave some information on the chemical properties of the aggregate surface. For each aggregate characterized (in terms of its physical and chemical properties) and evaluated, the boiling water test was performed to determine the stripping propensity of the aggregate when coated with a given asphalt cement (*16*). If the asphalt separated from the aggregate surface during this test, it was said to have stripped, and it was concluded that the asphalt-aggregate bond was weak and had a high stripping propensity.

On the basis of the results of this experimental evaluation and the published findings of other researchers, a mechanism by which stripping might occur was proposed. The observed relationships between the properties of an aggregate and its stripping potential were found to be consistent with the proposed stripping mechanism.

TABLE 1 PHYSICOCHEMICAL PROPERTIES OF ASPHALT, AGGREGATE, AND MIXTURE INFLUENCING STRIPPING

Material	Physicochemical Property
Asphalt	Viscosity Surface tension Volatility Relative fraction polar constituents Phenol group concentrations Carboxylic group concentrations Amine group concentrations
Aggregate	Size and shape Pore volume and size Surface area Chemical constituents at surface Acidity and alkalinity Adsorption site surface density Surface charge or polarity
Asphalt-aggregate mixture	Pore space fraction filled with asphalt Asphalt adsorption ratio Chemical constituents of adsorbed asphalt

EXPERIMENTAL WORK

Materials

Asphalt

An AC-20 asphalt cement supplied by the Alabama Highway Department was used in this study.

Aggregates

Five aggregates used in Alabama road construction were supplied by the Alabama Highway Department. These aggregates (granite, limestone, dolomite, quartz gravel, and chert gravel) were crushed and screened to 3/8-in. to No. 4 mesh size.

Antistripping Additives

Three different antistripping additives were used in this study. Two of the additives (Additives 1 and 2) were commercial antistripping additives, and one of the additives was a hydrated lime. In each case, the additive was mixed with asphalt cement by first placing the additive in a glass beaker and then pouring the heated asphalt cement into the beaker. The mixture was then stirred with a hot spatula to ensure thorough mixing.

Procedure

Boiling Water Test

After having been soaked in distilled water for 24 hr and towel dried, 100 g of saturated aggregate was placed in a stainless steel bowl and kept in an oven maintained at 300°F for 1 hr. Next, 5.5 g of asphalt cement, with or without an antistripping additive, was heated at 275°F for 10 min and then poured onto the preheated aggregate. The asphalt content was found to have only an insignificant effect on the boiling water test result when it was above the minimum amount required to completely coat the aggregate (3 to 4 g). The asphalt and aggregate were mixed using a hot spatula for 2 min and then placed in an oven maintained at 300°F for 10 min. After it had cooled to room temperature, the completely coated mixture was placed in boiling water (250 mL in a 400-mL beaker) over a hot plate. The water was allowed to boil slowly for 10 min, during which time it was stirred using a glass rod for 10 sec at 4-min intervals. The mixture was then kept in the water while it cooled to room temperature. After cooling, the water was drained from the beaker, and the mixture was placed on a paper towel and allowed to dry.

The amount of resultant stripping was determined visually and reported in terms of the observed percentage of asphalt coating retained on the aggregate. A rating board was developed that had 10 intervals representing from 0 to 100 percent of retained coating in order to standardize, as much as possible, the visual evaluation (*16*).

Pore Volume and Surface Area

The pore volume and surface area of each aggregate (3/8-in. to No. 4 mesh size) were measured using a mercury porosimeter (Autoscan-33 Porosimeter, QuantaChrome Co.). The surface area of an aggregate was calculated from a cumulative pore volume curve measured using the porosimeter and assuming that the aggregate had cylindrical pores open at each end. According to the Washburn equation (*17*), the surface area is

$$S = (1/\gamma \cos \theta) \int_0^V P dV \tag{1}$$

where P is compression pressure, V is pore volume, γ is the surface tension of mercury, and θ is the contact angle of mercury over the solid sample. Using the commonly accepted values of $\gamma = 480$ erg/cm and $\theta = 140$ degrees, surface area was calculated by graphically integrating the cumulative pore volume curve (*17*).

pH Measurements on Aggregate Powder

Powdered aggregate (10 g) was added to distilled water (100 mL) in a beaker and the pH of the water was measured using a pH meter with a glass/calomel electrode. The mixture was stirred with a magnetic stirrer bar, and the pH was recorded over time.

The aggregate powders for this experiment were obtained by first crushing and grinding the aggregates and then screening them to the No. 60 to No. 80 mesh size range in order to increase the surface area.

Surface Charge Measurements

The zeta potentials of the powdered aggregates in water were measured with a Lazer Zee Meter (Model 501, Pen Kem, Inc.). This meter measured electrophoretic mobility to determine the zeta potential of the aggregate particles in water. The sample for this measurement was a mixture of 10 g of aggregate powder (less than 325 mesh size) in 100 mL of distilled water.

DISCUSSION OF RESULTS

Five different aggregates were characterized by measuring their pore volumes and surface areas, the pH values of contacting water, and surface charges in water. The experimental results were evaluated with respect to the effect of (a) pore volume and surface area, (b) pH value of contacting water, and (c) surface charge of aggregate on the stripping propensity of the aggregate as measured by the boiling water test. The results are discussed in the following subsections.

Pore Volume and Surface Area

Five aggregates (granite, limestone, dolomite, chert gravel, and quartz gravel) were tested for surface area and porosity, and their stripping propensity was determined by the boiling water test (Table 2). A low pore volume or surface area suggests a smooth, crystalline surface with low surface roughness. On the basis of purely mechanical considerations—that is, the requirement for large areas of interfacial contact and surface roughness to have good adhesion and interlock—the low pore volume and surface area of granite should imply the existence of low adhesive bond strength with the asphalt cement and high moisture susceptibility.

TABLE 2 PORE VOLUMES AND SURFACE AREAS OF AGGREGATES AND THEIR STRIPPING PROPENSITIES AS DETERMINED BY THE BOILING WATER TEST

Aggregate	Pore Volume ($cm^3/g \times 10^3$)	Surface Area (m^2/g)	Average Pore Size ($mm \times 10^4$)	Percentage Coating After Boiling
Granite	3.2	0.116	1.10	10
Dolomite	6.5	0.586	0.44	35
Chert gravel	23.0	2.09	4.40	55
Quartz gravel	5.4	0.052	4.15	65
Limestone	6.2	0.079	3.14	90

NOTE: Average pore size = 4 [(Pore volume)/(Surface area)].

A comparison of the effect of different pore sizes was provided by dolomite and limestone (Table 2). Dolomite and limestone were used for this comparison because of the similarity of their chemical and electrochemical properties. Although dolomite had a higher surface area than did limestone, it had a higher stripping propensity because of its smaller pore size. It should be noted that, even though the dolomite had nearly the same pore volume as the limestone, it had about seven times as much surface area, which meant that the dolomite had a smaller pore size. In Table 2, the average pore size for each of these aggregates was estimated by assuming that the pores had a cylindrical shape.

One possible reason for the observed adverse effect of pore size on the stripping propensity of the dolomite and limestone aggregates is that, when an asphalt cement coats a rough surface that has fine pores, air is trapped and the asphalt can hardly penetrate the fine pores. Consequently, only a fraction of the aggregate's apparent surface area might actually be in contact with the asphalt cement. In general, the depth of penetration of the asphalt depends on the size of the pore, as well as the viscosity and surface tension of the asphalt.

Overall, a correlation between the physical properties of an aggregate and its stripping propensity could not be established. The chemical properties of the aggregates considered varied significantly, and it was thought that this had an overriding effect. As the data in Table 2 indicate, although limestone and crushed quartz gravel had nearly the same physical properties (pore volume and surface area), limestone had a better stripping resistance. The chemical properties of the limestone were much different from those of the quartz gravel, as will be discussed later.

pH of Contacting Water

In 1960 Hughes et al. (*3*) reported that the adhesion between asphalt and aggregate in the presence of water became weakened when the pH of the buffer solution was increased from 7.0 to 9.0. Scott (*7*) also reported the same observation for the effect of pH on stripping. In this study, some insight into the effect of pH changes was developed by considering the chemical and electrochemical properties of the aggregate surfaces.

Figure 1 shows the changes in pH values caused by the addition of several different aggregate powders to water. Similar results have been reported for different aggregates by Scott (*7*). Apparently (Figure 1), limestone and dolomite, which are known to be basic aggregates, caused the pH of the water to rise to a relatively high value. Also, granite, which is known to be acidic, reacted with water, leading to a gradual increase in the pH of the system. The silicate lattice of the granite surface reacted with water to impart excess hydroxyl ions as follows:

$$-\overset{|}{\underset{|}{Si}}-O-Na + H_2O \rightarrow -\overset{|}{\underset{|}{Si}}OH + Na^+ + OH^- \quad (2)$$

Equation 2 illustrates a typical hydrolytic reaction of the salt of a weak acid.

To assess the sensitivity of stripping to changes in the pH of water in contact with the aggregate surface, a series of boiling water tests was performed using water of different pH values. The pH of the water was modified by adding HCl or NaOH solution. As shown in Figures 2 and 3, changes in the pH of the water had significant effects on stripping. Stripping became

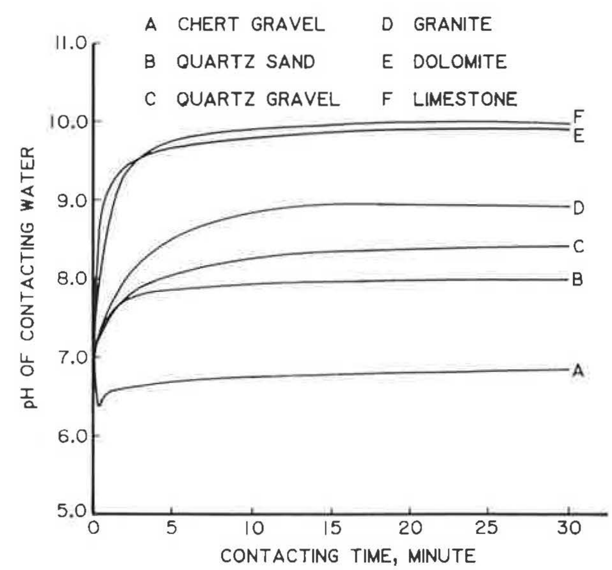

FIGURE 1 Changes in pH of water in which aggregates were immersed.

more severe as the pH value increased from 2.0 to 7.0 for the granite aggregate (Figure 2) and from 3.0 to 13.0 for the crushed chert gravel (Figure 3). This result indicates that a significant change in the pH of water in contact with the aggregate surface could cause stripping to occur.

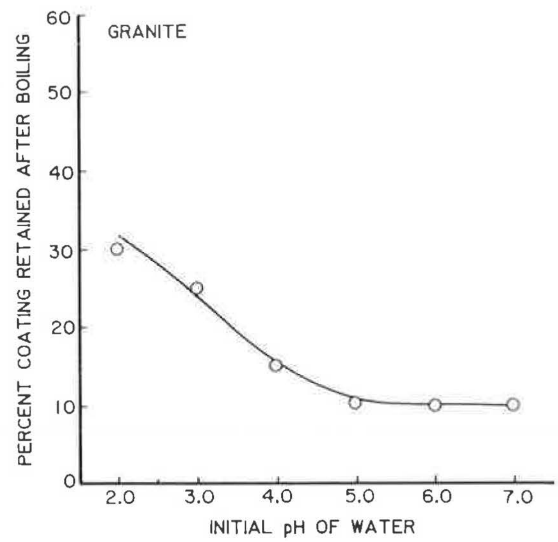

FIGURE 2 Effect of pH of water used in the boiling water test on granite.

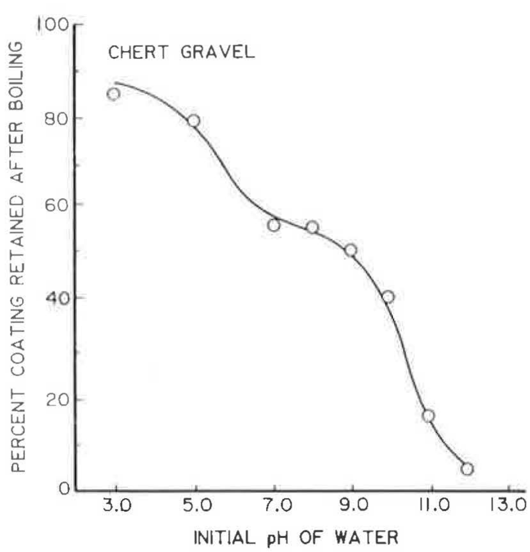

FIGURE 3 Effect of pH of water used in the boiling water test on chert gravel.

When an aggregate is being coated with asphalt, the aggregate selectively adsorbs some components of the asphalt, such as the more polar species of the asphalt, and hydrogen bonds or salt links are formed. The types and quantities of adsorbed components are thought to play an important role in adhesion and stripping (*8*). The presence of ketones and phenolics is thought, for example, to improve stripping resistance whereas carboxylic acids, anhydrides, and 2-quinolenes are thought to increase stripping sensitivity because of the high water susceptibility of their bonds with aggregate surfaces.

The water susceptibility of the hydrogen bonds (or some other dipole-dipole attractive bonds) and the salt links (i.e. the ionic bonds) between the adsorbed asphalt components and the aggregate surface would increase as the pH of the water present at the aggregate surface was increased. For this reason, stripping damage might be expected to occur with an aggregate that causes an increase (to a relatively high value) in the pH of any water present at the asphalt-aggregate interface.

The data shown in Figure 4 appear to agree with this suggestion. Granite, which imparted a high pH to contacting water, had a higher stripping propensity than did either crushed chert gravel or crushed quartz gravel, both of which imparted a lower pH to the contacting water. The pH values shown in Figure 4 were obtained by measuring the pH of the water used in the boiling water test of each aggregate after cooling.

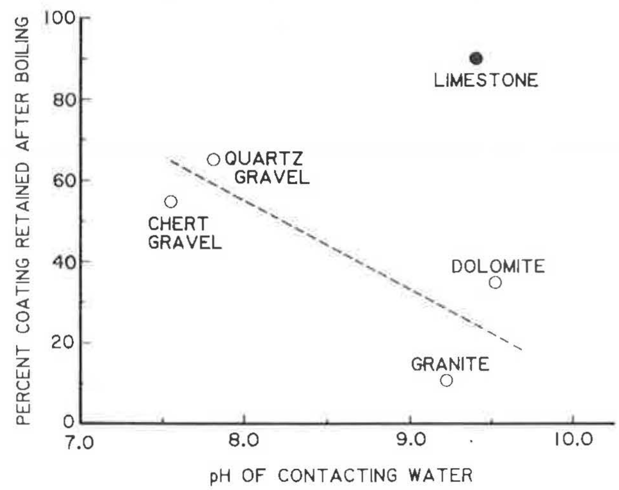

FIGURE 4 Comparison of pH of contacting water and stripping propensity as determined by the boiling water test.

Aggregate surface properties, other than pH changes, must also be considered. The different types of metal ions involved in the interaction between the asphalt and the aggregate can also play an important role in stripping. As shown in Figure 4, for example, although limestone imparted a relatively high pH to contacting water, it had a high stripping resistance. Alkaline earth metals in limestone associate strongly with asphalt components such as carboxylic acids to form alkaline earth salts, and the bonds formed are not dissociated easily in water even at a high pH; that is, in this case the adsorption is strong because of the insolubility of the alkaline earth salts formed between the limestone and the bitumen acids.

Surface Potential

The responses of stripping propensity to differences in the surface electrical charge of the aggregates are shown in Figure 5. The interfacial activity occurring between charged surfaces of the mineral aggregates and asphalt cements can be of fundamental importance to the stripping of asphalt from aggregate (*18*). That is, the surface charge of the aggregate can be as important as are specific chemical interactions; mineral aggregates possess distinctive polarities or electrochemical properties.

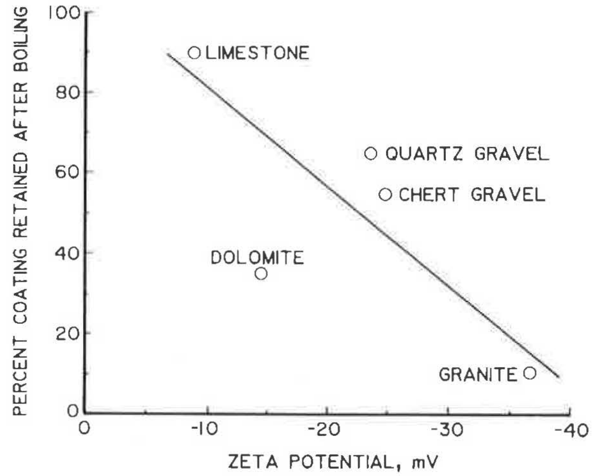

FIGURE 5 Comparison of aggregate surface potential and stripping propensity as determined by the boiling water test.

The functional groups of an asphalt that are adsorbed on an aggregate surface come mainly from the acid fraction of the asphalt. For instance, one of the acid molecules is represented by carboxylic acids (R-COOH). In the presence of water, the acid molecules are separated into two ions, the carboxylate anion ($R\text{-}COO^-$) and the proton (H^+), causing the asphalt surface to have a negative polarity at the interface. The increase in pH of water present at the aggregate surface increases the extent of dissociation of the acid molecules.

The aggregates with water present are negatively charged to varying degrees (Figure 5). As a result, a repulsive force develops between the negatively charged aggregate surface and the negatively charged asphalt surface at the interface, which causes the separation of the asphalt from the aggregate surfaces (stripping) (*7*). Solid surfaces in contact with water usually acquire charges through chemical reactions at the solid surface and adsorption of complex ions from the solution (*19*). For instance, metal oxide surfaces in water are hydrolyzed to form hydroxyl groups,

$$\begin{array}{ccc} \overset{\displaystyle O}{\diagup \quad \diagdown} & & \overset{H\quad\; H}{\overset{O\quad\; O}{|\quad\;\; |}} \\ -M-O-M- & + H_2O \leftrightarrow & -M-O-M- \\ & & |\quad\;\; | \end{array} \tag{3}$$

which, subsequently, dissociate according to the reactions

$$-M-OH\ (s) + H_2O \leftrightarrow -M-O^-\ (s) + H_3O^+\ (aq) \tag{4}$$

$$-M-OH\ (s) + H_2O \leftrightarrow -M-OH_2^+\ (s) + OH^-\ (aq) \tag{5}$$

to generate surface charges. The extent to which each of Reactions 4 and 5 proceeds depends on the pH of the solution and the type of mineral. A high pH value of the water in contact with the mineral surface would therefore cause the surface to be negatively charged.

The intensity of the repulsion developed between the asphalt and the aggregate depends on the surface charge of both the asphalt and the aggregate. As shown in Figure 5, granite, which had a high stripping propensity, possessed a relatively high surface potential whereas limestone, which had a high stripping resistance, had a relatively low surface potential (as determined by zeta potential measurements). The general observed trend was that the aggregates that had a relatively high surface potential in water were more susceptible to stripping.

From the foregoing discussion, it appears that, although physical properties of the aggregate are important, chemical and electrochemical properties of aggregate surfaces play an even more important role in stripping.

It should be noted that the values of pH and surface charge indicated in this study are only true for the sample tested because these values will be changed with variation of the mineral source and its aging history.

Antistripping Additives

Antistripping additives are substances added to asphalt cements to promote adhesion of the asphalt cement to the aggregate surface and thus to improve resistance to stripping. When antistripping additives were incorporated into the asphalt cement, the effectiveness of each additive varied considerably as the pH of the contacting water was changed, as shown in Figure 6. Here, the pH of water used in the boiling water test (contacting water) was changed by adding HCl or NaOH solution. Granite was used as a standard aggregate in this set of experiments. Some additives lost their effectiveness as the pH of the contacting water increased; the extent of effectiveness lost varied for each additive.

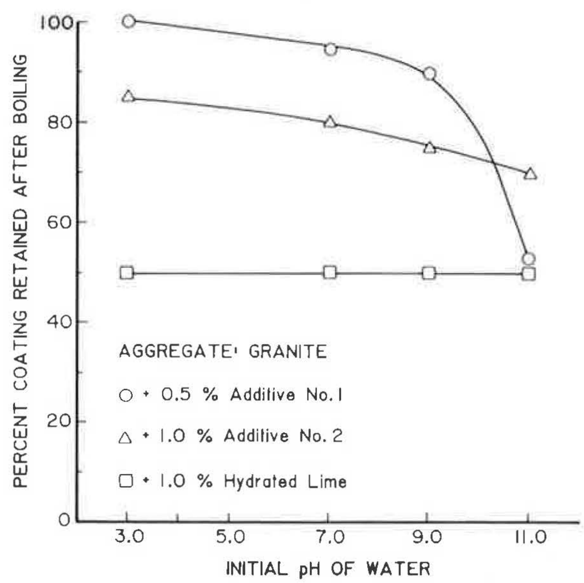

FIGURE 6 Effect of pH of water used in the boiling water test when using additives.

The effectiveness of Additive 1 was more sensitive to the pH of the contacting water than was the effectiveness of either of the other two additives. Additive 1 was a type of polyamine. This implied that, when the pH of the water was increased to a high value, the adsorption bond between amine-type additives and aggregate surfaces was weakened. As a result, water could more easily displace the asphalt from the aggregate surface. On the other hand, the performance of hydrated lime remained constant, independent of the pH of the contacting water. The effectiveness of hydrated lime apparently was not as susceptible to environmental changes as was that of the polyamine-type additive.

It was found that by storing the asphalt-aggregate mixture for a few hours at 300°F (i.e., curing the mix), the effectiveness of some additives improved considerably even at a high pH value of contacting water (Figure 7). The formation of a film of polymerized asphalt (coke) on the aggregate surface was thought to be the primary cause of the observed curing effect. Some evidence in support of this postulation is provided in Figure 8. A granite was precoated with phenanthrene, which is known to have a high coking propensity. The result was an accelerated curing effect; that is, there was a faster response in stripping resistance to curing. It should also be noted that an aggregate surface can be conditioned by precoating with coke-forming compounds (e.g., phenanthrene, coal liquids) to enhance stripping resistance. More work related to curing mixtures to enhance their stripping resistance is to be published soon.

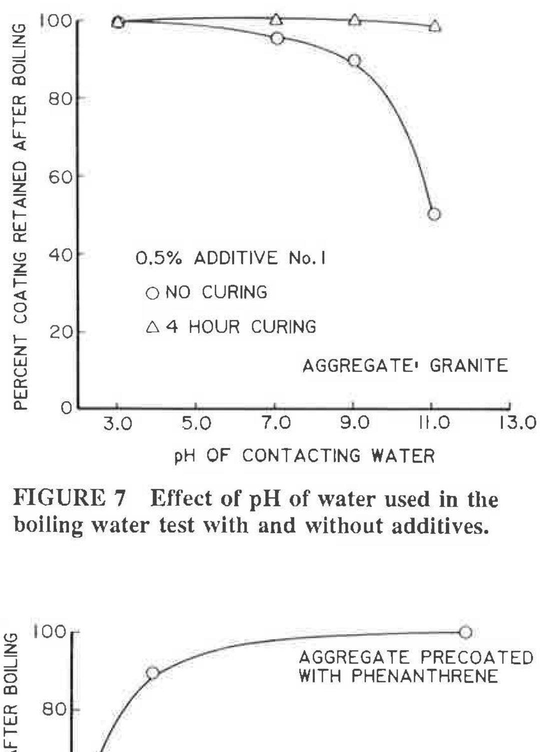

FIGURE 7 Effect of pH of water used in the boiling water test with and without additives.

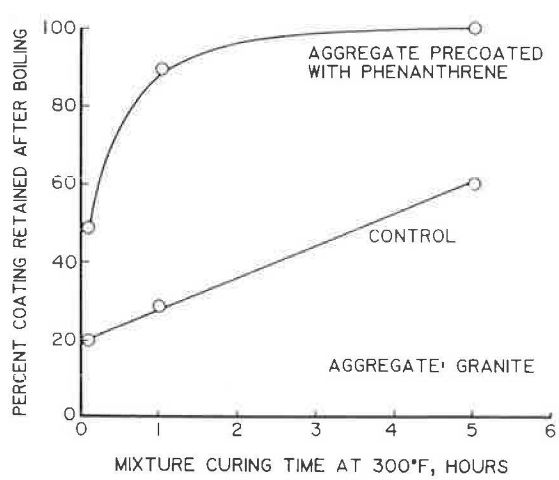

FIGURE 8 Acceleration of curing effect on stripping resistance by a high coking propensity additive, phenanthrene.

SUMMARY

Physicochemical properties of the asphalt, the aggregate, and the asphalt-aggregate mixture that might influence stripping are summarized in Table 1. It was found that, although the physical properties of an aggregate affected stripping, there was no strong correlation between the physical properties of the aggregate, such as pore volume and surface area, and the stripping propensity of the aggregate. Chemical and electrochemical properties of the aggregate surface in the presence of water were observed to have a significant effect on stripping.

All mineral aggregates tested in this study had distinctive pH values when immersed in water and had distinctive electrochemical surface properties that could be measured in terms of their zeta potential. It was found that aggregates that had relatively high surface potentials in water or that imparted relatively high pH values to water, or both, were more susceptible to stripping.

These results were thought to be due in part to the existence of electrical repulsive forces between the asphalt and the aggregate surface. When the pH of the contacting water was high, the negative surface potential of the asphalt in contact with the aggregate was also high because of the dissociation of acidic components in asphalt. Most aggregates would have a negative charge of varying intensity when in contact with water. The electrical repulsion between the two negatively charged surfaces, that is, the asphalt and the aggregate surface, can cause the asphalt to separate (or strip) from the aggregate surface.

It was found that the effectiveness of some additives could be sensitive to the pH of water in contact with aggregate. Storing the asphalt-aggregate mixture for a few hours at 300°F caused the effectiveness of some additives to improve considerably. The curing rate was accelerated when the aggregate was precoated with phenanthrene or coal liquids.

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Water Sensitivity Test Methods for Asphalt Concrete Mixtures: A Laboratory Comparison

JOHN S. COPLANTZ AND DAVID E. NEWCOMB

This study provides a comparison of four asphalt concrete water sensitivity (stripping) test methods by ranking the relative resistance to water-induced damage of a variety of field-prepared mixtures obtained during construction. Each test method evaluates water sensitivity by determining resilient modulus or indirect tensile strength, or both, of a compacted specimen before and after moisture conditioning. Conditioning of the samples is performed by vacuum saturation to predetermined levels and, in some cases, freeze-thaw cycles. One test method consists of vacuum saturation only. Another adds a single freeze-thaw cycle to vacuum saturation. The third method is a repeat of the second method, but at a lower saturation level. Finally, the fourth test method is an extension of the third, involving additional freeze-thaw cycles. Test results indicated that stripping damage did not occur in specimens subjected to vacuum saturation only. Freeze-thaw cycles caused damage to the specimens. Higher saturation levels resulted in increased damage to the specimens, as expected, but the rank of relative water sensitivity of the mixtures was found to be nearly the same. Laboratory performance after seven freeze-thaw cycles varied with aggregate and asphalt characteristics and could not be predicted using performance data from one freeze-thaw cycle only.

Many flexible pavements have suffered from an increased rate of damage to the asphalt concrete layer due to the effects of water. The damage is the result of a lack of cohesion within the mixture caused by a loss of bond strength between asphalt cement and aggregate. This moisture damage mechanism is sometimes referred to as stripping.

Many different laboratory test methods are used by state highway agencies to quantify the water sensitivity of a mixture and to estimate improvements in field performance of water sensitive mixtures that may be realized by the use of antistripping additives. Experts agree that the best test method to use is one in which the laboratory moisture damage mechanism closely simulates that which occurs in the field. They also state that the method used to measure the effects of moisture damage should be some type of fatigue, resilient modulus, or tensile test (*1–5*). In addition, the test should be run on the actual aggregate and asphalt cement to be used in the roadway and should be severe yet sensitive enough that the effect of the amount and kind of antistripping additive can be identified (*4, 5*).

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Four water sensitivity test methods currently in use were evaluated by testing a variety of asphalt mixtures. Each test method involved submersion and vacuum saturation of a compacted bituminous specimen in water. Three of the four test methods incorporated freeze-thaw cycling after saturation. Resilient modulus and indirect tensile strength were obtained before and after vacuum saturation or freeze-thaw conditioning, or both. The study was intended to evaluate the test methods, not the properties of the mixtures. The test methods are quite similar in nature: they differ in time and amount of exposure to saturation and to freeze-thaw cycling. Past experience indicates that test methods that involve exposure to saturation only are not severe enough to predict stripping characteristics. Test methods incorporating exposure to saturation and freeze-thaw cycling have received much more acceptance from researchers as indicators of stripping potential. However, the amount of time required to perform these more extensive tests is generally unpopular with most highway agencies. This study was undertaken to determine whether the freeze-thaw cycle can be eliminated and whether the standard vacuum saturation plus one freeze-thaw cycle test can predict multiple freeze-thaw behavior.

PROJECTS EVALUATED

Dense-graded, field-prepared mixtures were obtained from various locations throughout northern Nevada. The mixtures were sampled from construction operations during the summer of 1986. All mixtures were of Type 2 specification and were in accordance with the Nevada Department of Transportation (NDOT) standard specifications (*6*). A list of these requirements is given in Table 1. Table 2 is a listing of all of the projects that were evaluated for the study. Project locations, traffic data, and general climatic data are also given. Many of the projects were overlays of existing sections. These projects were chosen from a larger data base that will be used for field evaluation studies. Each mixture selected for this study was known to be water sensitive. Projects were selected to provide an overall range of material types as well as environmental conditions.

MATERIALS

The materials used in the mixtures evaluated in this study are given in Table 3.

TABLE 1 STANDARD SPECIFICATIONS FOR TYPE 2 AGGREGATES AND DENSE–GRADED ASPHALT CONCRETE MIXTURES

Sieve Size	Percentage Passing by Weight
1 in.	100
$^3/_4$ in.	90–100
$^1/_2$ in.	
$^3/_8$ in.	63–85
No. 4	45–63
No. 10	30–44
No. 16	
No. 40	16–24
No. 200	3–9

NOTE: Liquid limit = 35 percent max; plasticity index = 6 percent max; fractured faces = 50 percent min; and Los Angles abrasion = 45 percent max. For asphalt concrete, stability = 35 min and air voids = 3 to 6 percent.

TABLE 2 PROJECTS EVALUATED FOR STUDY

Mixture	Location	Air Freeze-Thaw Cycles	Climate
A	US-50, Churchill Co.	154	Moderate
B	5th Street, Carson City	176	Moderate
C	US-395 business, Washoe Co.	154	Moderate
D	US-395 business, Washoe Co.	154	Moderate
E	IR-580, Washoe Co.	154	Moderate
F	IR-80, Elko Co.	165	Severe
G	IR-80, Elko Co.	230	Severe
H	US-95, Clark Co.	69	Mild to moderate

TABLE 3 MATERIALS USED IN MIXTURES

Mixture	Asphalt Type	Asphalt Content (%)	Lime Content (%)
A	AR-4000	6.5	1.5
B	AR-4000	6.5	1.5
C	AR-4000	7.0	1.5
D	AR-4000	7.0	1.5
E	AR-4000	6.0	1.5
F	AC-10	6.5	1.5
G	AR-8000	6.5	1.5
H	AR-8000	–	No lime

NOTE: Dash indicates data not available.

Asphalts

Projects A through D used AR-4000 asphalt cement as the binder. Project E was constructed using an AC-10, and Projects F and G were constructed with AR-8000 as the binder.

Aggregates

A large majority of the aggregate sources throughout the state of Nevada show a tendency to be water sensitive. To combat the damaging effects of water, all mixtures except Mix H contained hydrated lime as an antistripping material. For mixtures containing lime, aggregates on the cold feed belt were sprayed with water to prewet the surface. The lime was then added in powdered form. Mixing of the lime and aggregate was accomplished either by a series of riffles between the cold feed belts or by pugmill. The lime-aggregate mixture was then carried to the drum mixer by an additional cold feed belt.

SAMPLE PREPARATION

Dense-graded asphalt concrete mixtures were evaluated for the study. The mixtures were sampled in the field in a loose state from behind the laydown machine. The samples were placed in plastic concrete cylinder molds and transported to the laboratory. On arrival, the mixtures were split into representative sample sizes suitable for testing. The samples were then reheated to a compaction temperature of 230°F and compacted into standard specimens 4 in. in diameter by 2.5 in. high by the Hveem method. The compactive effort was adjusted to provide air void levels in the 7 to 9 percent range. This was done to ensure that the laboratory-compacted specimens would closely resemble expected field conditions with respect to air void levels. After compaction, the samples were allowed to cool overnight to 77°F. Testing of the samples began approximately 24 hr after compaction.

TEST METHODS

Conventional quality control tests were performed by NDOT. The tests were performed in accordance with standardized AASHTO (*7*) and NDOT (*8*) procedures. These data are not reported in this paper; however, all mixtures conformed to specifications. Resilient modulus, indirect tensile strength, air void and saturation measurement, and water sensitivity tests were performed by the University of Nevada-Reno Construction Materials Laboratory. These test methods are briefly discussed.

Resilient Modulus

The resilient modulus (Mr) of the test specimens was determined at 77°F in accordance with ASTM D 4123 (*9*). Control samples were tested in the dry condition, and samples subject to vacuum saturation or freeze-thaw conditioning, or both, were tested under saturated-surface-dry (SSD) conditions.

Indirect Tensile Strength

Indirect tensile strength, or split tension test (St), results were obtained at 77°F by using the loading procedure described by Lottman (*10*). However, a 2.0-in./min deformation rate was used until sample failure occurred. Control samples were also tested in the dry condition, and samples subjected to vacuum saturation or freeze-thaw cycling, or both, were tested under SSD conditions.

Water Sensitivity Tests

Four different methods of testing water sensitivity were evaluated. Each test method required specimens to be compacted to an air void level near that of field conditions (7 to 9 percent). Each method called for vacuum saturation of the samples with water. Three of the four test methods used freeze-thaw cycles to further condition the specimens. Measurements of resilient modulus and indirect tensile strength were determined for conditioned and unconditioned specimens. Measurements on conditioned specimens were taken at SSD conditions and compared with the test results of the unconditioned samples. The first three test methods are essentially slightly modified versions of those used by Lottman (*10*). The last method is a modification of that used by Scherocman et al. (*5*). Detailed descriptions of each test method follow. The testing sequence is shown in Figure 1.

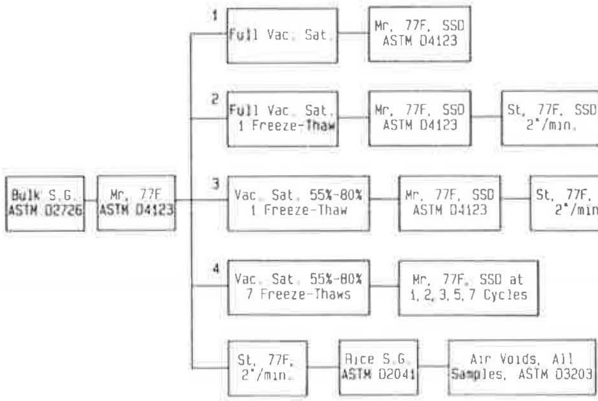

FIGURE 1 Testing sequence used in study.

1. Test Method 1 consisted of vacuum saturation at a level of 24 in. of Hg for 30 min. Saturation levels were not determined, but, on the basis of previous experiments, they were estimated to be near 100 percent. Resilient modulus measurements were taken before and after conditioning.

2. Test Method 2 consisted of vacuum saturation to the same level and duration used in Method 1. The samples were wrapped in plastic and frozen at −20°F for 15 hr. The frozen specimens were unwrapped and submerged in 140°F water for 24 hr, then submerged in 77°F water for approximately 2 hr. Resilient modulus and indirect tensile strength measurements were taken before and after conditioning.

3. Test Method 3 used a vacuum level and duration corresponding to a saturation level of from 55 to 80 percent. The saturation level was determined by the procedure described by Tunnicliff and Root (*4*). It was found that a vacuum level of 14 in. of Hg for 5 min followed by 30 min under atmospheric conditions produced saturation levels within this range. The samples were then subjected to the freeze-thaw cycle used in Test Method 2. Resilient modulus and indirect tensile strength measurements were taken before and after conditioning.

4. Test Method 4 used the same vacuum level and duration as Test Method 3 to provide saturation levels between 55 and 80 percent. The samples were then run through a series of multiple freeze-thaw cycles, each of which was the same as that used in Test Method 2. Resilient modulus measurements were obtained after one, two, three, five, and seven cycles and compared with the test results of the unconditioned samples.

TEST RESULTS

Measurements of resilient modulus, indirect tensile strength, and air voids were recorded for each mixture before conditioning. Results are given in Table 4. Values of resilient modulus ranged from 228 to 999 ksi. Indirect tensile strength test values ranged from 82 to 238 psi. As expected, mixtures prepared with AR-8000 asphalt cement have higher stiffness values. Note that air void levels were held between 6.8 and 9.1 percent to simulate the range of air void levels expected in the field.

TABLE 4 AVERAGE TEST RESULTS BEFORE CONDITIONING AND SATURATION LEVEL FOR TEST METHOD 3 AFTER SATURATION BY VACUUM

Mixture	Resilient Modulus (ksi)	Indirect Tensile Strength (psi)	Air Voids (%)	Percentage Saturation (Method 3)
A	419	147	8.2	71
B	519	135	7.3	93
C	505	129	7.7	76
D	406	–	9.1	74
E	404	94	7.5	78
F	269	82	9.0	75
G	999	238	7.0	88
H	228	–	6.8	70

NOTE: Dashes indicate no data available.

Resilient modulus and indirect tensile strength measurements taken after Test Methods 1–3 are shown in Figure 2. Results from Test Method 1 actually show higher resilient moduli than those obtained from unconditioned specimens. Lower resilient modulus and indirect tensile strength values were observed for Test Method 2 than for Test Method 3. Measurements of resilient modulus for Test Method 4 are given in Table 5 and shown in Figure 3. In general, for all mixtures, the resilient modulus decreases with an increasing number of freeze-thaw cycles.

Retained resilient modulus and indirect tensile strength ratios after Test Methods 1–3 are shown in Figure 4. Each ratio was calculated by dividing the conditioned value by the original value and multiplying by 100 percent. Test Method 1 ratios were the greatest of the three. Ratios for Test Method 2 were lower than for Test Method 3. Resilient modulus ratios for Test Method 4 are given in Table 6 and shown in Figure 5. The ratios tend to decrease as the number of freeze-thaw cycles increases. This type of behavior is to be expected

TABLE 5 RESILIENT MODULUS BEFORE AND AFTER VACUUM SATURATION AND MULTIPLE FREEZE-THAW CYCLE CONDITIONING

	Resilient Modulus at 77°F (ksi)					
Mixture	Original	1 Cycle	2 Cycles	3 Cycles	5 Cycles	7 Cycles
A	489	324	273	189	120	68
B	572	318	230	166	112	72
C	506	277	146	107	50	28
D	427	305	281	211	151	86
E	440	209	201	190	173	117
F	265	123	89	63	47	30
G	950	477	490	475	448	394
H	271	112	55	33	16	9

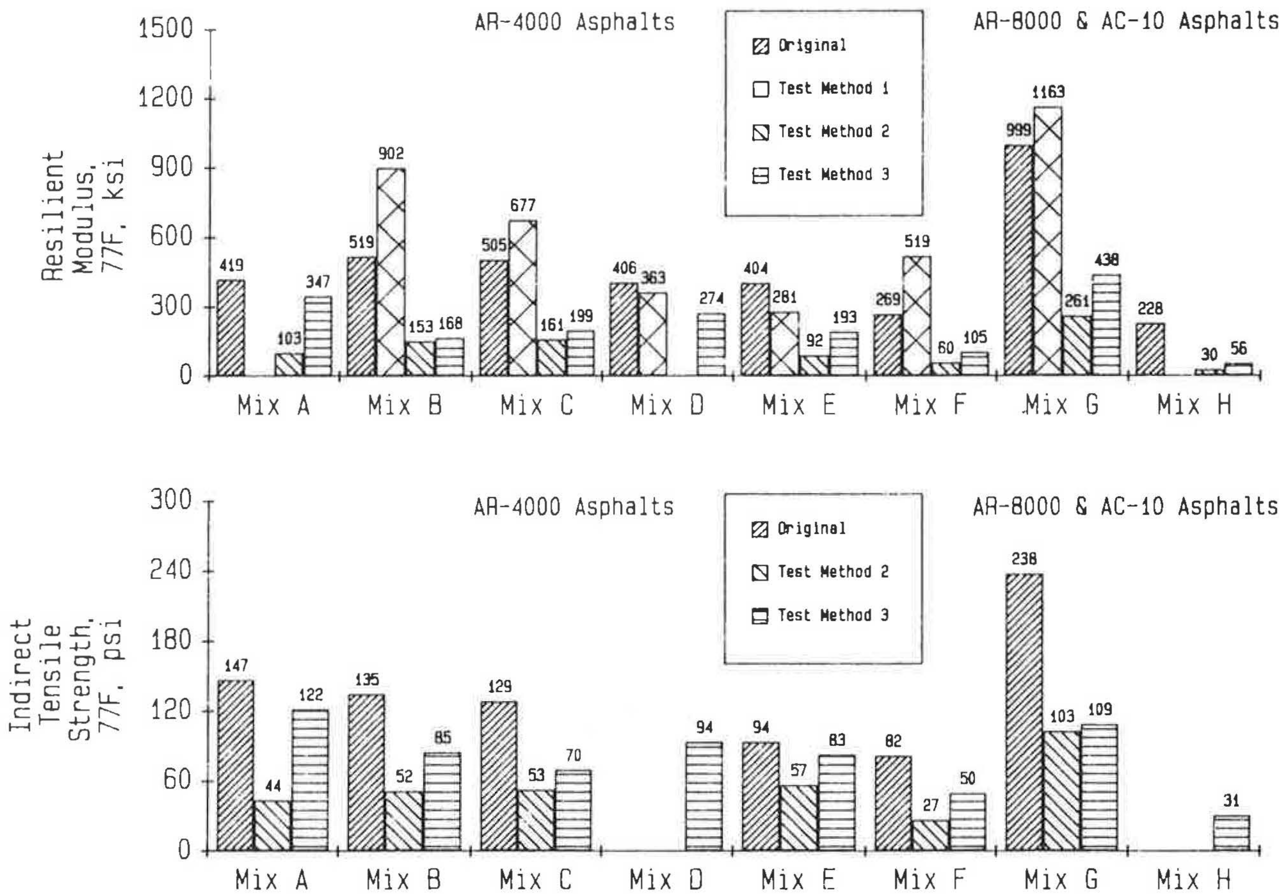

FIGURE 2 Resilient modulus and indirect tensile strength before and after vacuum saturation and freeze-thaw conditioning.

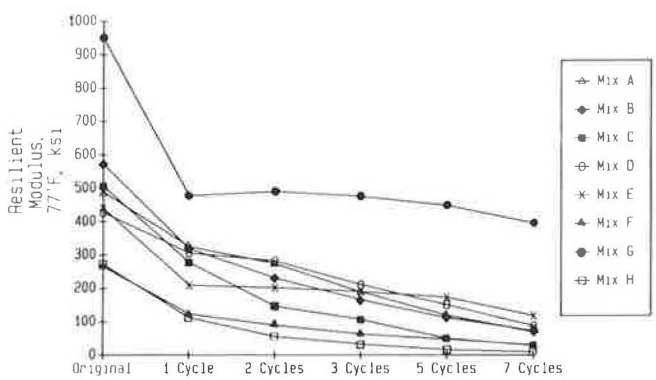

FIGURE 3 Resilient modulus versus freeze-thaw cycles.

when mixtures are repeatedly subjected to severe freeze-thaw cycles.

DISCUSSION OF TEST RESULTS

Recalling that the intent of this paper is to provide a comparison of test methods and not mixture properties, it can be seen from Figure 2 that, in terms of damage to the specimens, Test Method 1 proved to be the least severe. This method consisted of vacuum saturation only; no freeze-thaw cycles were used. A majority of the test results indicated that increases in resilient modulus took place after vacuum saturation. This type of behavior is not uncommon (*10*). Difficulty in controlling the temperature of the vacuum saturation bath may also have led to an increase in measured strength. Nevertheless, these results indicate that vacuum saturation without freeze-thaw cycling may not be severe enough to damage the mixtures. The performance of these materials after a freeze-thaw cycle indicates that they are water sensitive. However, vacuum saturation alone did not appear to initiate a stripping mechanism.

Data from Test Methods 2 through 4, each involving the use of freeze-thaw cycles, indicated that a substantial amount of damage occurred to each mixture. This would tend to favor the use of freeze-thaw cycles in a water sensitivity test method for wet-freeze regions. Comparisons of resilient modulus and indirect tensile strength values after freeze-thaw conditioning (Figure 2) indicate that the severity of damage is greater for Test Method 2 than for Test Method 3. This trend is shown for all mixes tested regardless of material type. Recall that saturation levels for Test Method 2 were near 100 percent whereas levels for Test Method 3 were held between 60 and 85 percent (Table 4).

A plot of retained resilient modulus ratios comparing Test Methods 2 and 3 is shown in Figure 6. Likewise, Figure 6 shows a plot of retained indirect tensile strength ratios for the two test methods. The data for the resilient modulus ratios indicate that there may be a relationship between ratios for the

TABLE 6 RATIOS OF RETAINED RESILIENT MODULUS AFTER VACUUM SATURATION AND MULTIPLE FREEZE-THAW CYCLE CONDITIONING

Mixture	Saturation (%)	Air Voids (%)	Resilient Modulus Ratios (%) After 1 Cycle	2 Cycles	3 Cycles	5 Cycles	7 Cycles
A	77	8.0	66	56	39	25	14
B	85	6.8	56	40	29	20	13
C	75	7.6	55	29	21	10	5
D	77	8.5	71	66	49	35	20
E	81	8.0	47	46	43	39	27
F	65	8.9	46	34	24	18	11
G	82	7.3	50	52	50	47	41
H	72	7.2	41	20	12	6	3

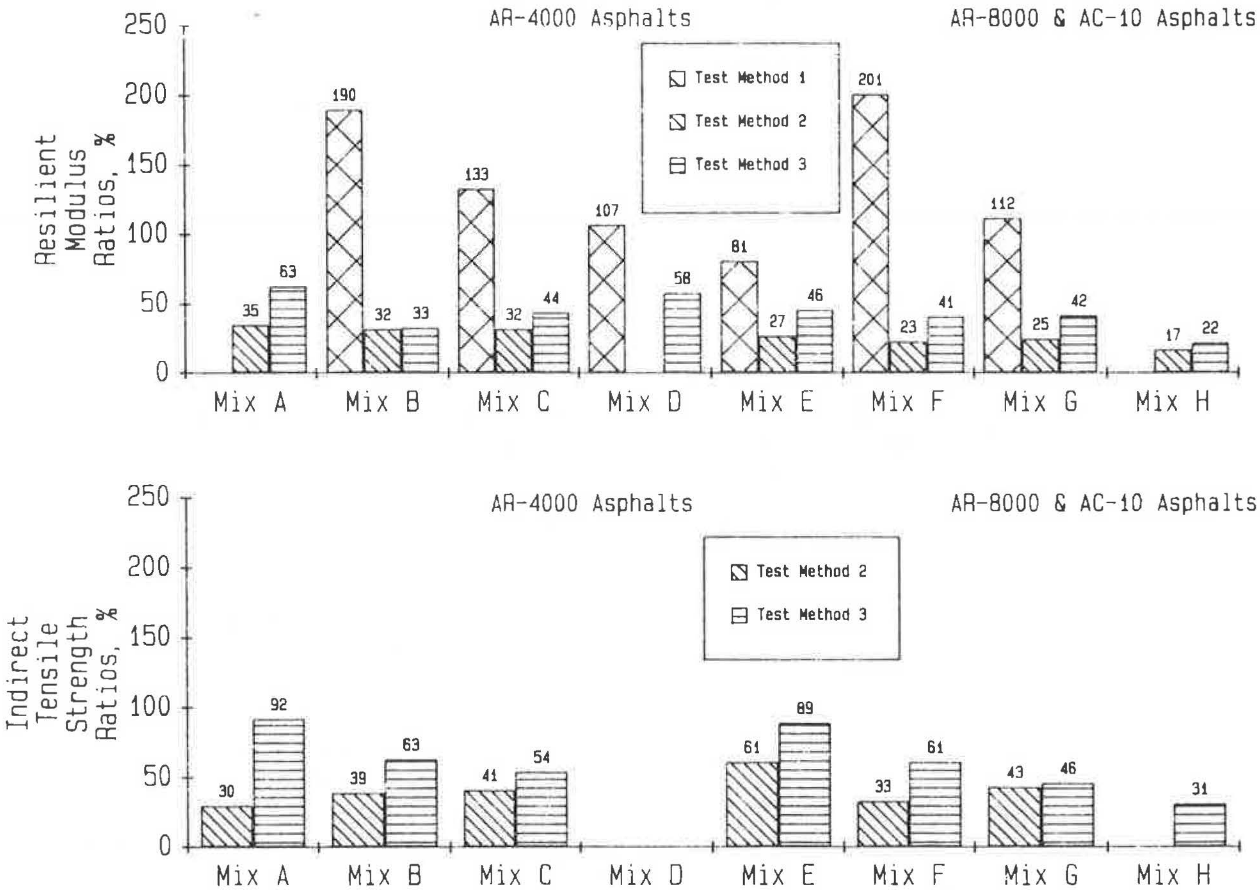

FIGURE 4 Retained resilient modulus ratios and indirect tensile strength ratios before and after vacuum saturation and freeze-thaw conditioning.

two test methods. The indirect tensile strength ratios are much more scattered. Some of the data scatter may be due to the range of saturation levels used for Test Method 3. The indirect tension ratios were calculated using two subsets of samples, one for original values and one for conditioned values. The resilient modulus ratios were calculated from the same subset of samples. The inherent variation in properties between the two indirect tension subsets could also explain the wider data scatter. Because of the small data base available, a regression analysis to determine the relationship (if any) between the two test methods was not performed.

Test Method 4 was a repeat of Test Method 3 with the addition of more freeze-thaw cycles. As expected, additional freeze-thaw cycles resulted in lower resilient moduli (Table 5 and Figure 3) and also lower retained resilient moduli (Table 6 and Figure 5). The relationship between resilient modulus or retained resilient modulus and the number of freeze-thaw cycles is not a constant. Generally, as the number of freeze-thaw cycles increases, the resilient modulus of the mixture decreases. Each mixture has its own unique curve, indicating that behavior through multiple freeze-thaw cycles is a characteristic of mixture composition. These concepts are shown in Figure 3.

As the data in Table 7 indicate, the sensitivity of each mixture to water-induced damage was given a relative ranking based on the retained resilient modulus ratios from each test

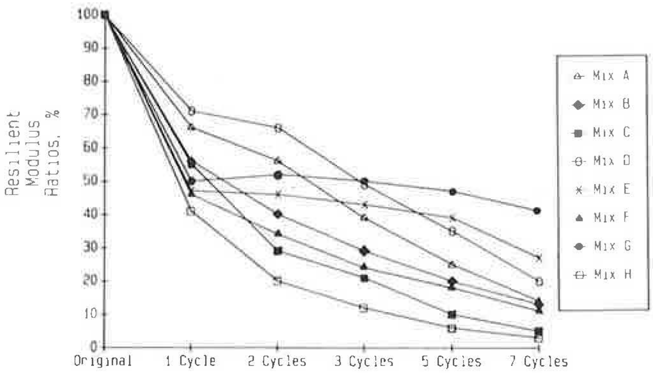

FIGURE 5 Retained resilient modulus ratios versus freeze-thaw cycles.

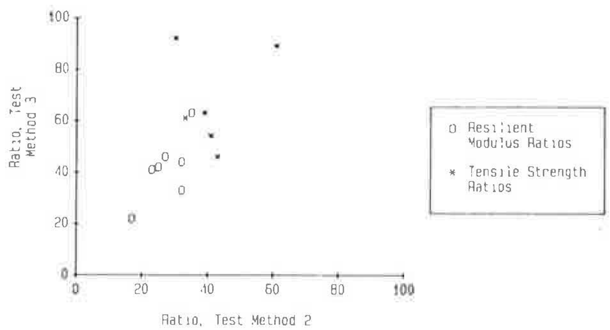

FIGURE 6 Plot of retained resilient modulus ratios and tensile strength ratios—Test Method 3 versus Test Method 2.

method. It can be seen that Test Method 1 clearly does not provide an adequate indication of performance when freeze-thaw cycles occur. The differences in sensitivity ranking between Test Method 1 and Test Methods 2 and 3 are considerable. For Test Method 1, Mix F ranked highest in resistance to water-induced damage. After freeze-thaw cycles were added, its resistance to water damage decreased to sixth highest. Likewise, Mix E was ranked sixth highest in resistance to water-induced damage by Test Method 1. After seven freeze-thaw cycles, its resistance to water damage was ranked as second highest, a jump of four positions. Damage rankings for Test Methods 2 and 3 are in considerable agreement with the exception of Mix B. Additional freeze-thaw cycles, however, affected the outcome somewhat. Comparing results of Test Method 3 with those after three and seven cycles of Test Method 4 indicates that Mix B and Mix G increase in relative resistance to water-induced damage, and Mix A and Mix C experience decreased resistance. It appears that the additional cycles may be needed to accurately determine long-term performance of these mixes in high freeze-thaw regions.

CONCLUSIONS

For the materials and mixtures involved in this study, the conclusions stated herein are applicable. Generating accurate and useful water sensitivity test data requires time. Results indicate that mixtures subjected only to vacuum saturation may not show evidence of stripping potential, but the same mixtures may readily strip with the addition of one or more freeze-thaw cycles. A freeze-thaw cycle adds 40 hr to the duration of the test but is required to accurately determine a given mixture's sensitivity to water. This level appears to be adequate for most work involving acceptance of material sources. Higher saturation levels will generally increase the amount of stripping that occurs in any given mixture. A controlled level of saturation may provide better control on the amount of water entering the permeable voids of the sample and reduce the chances of possible swelling of the sample as the result of oversaturation. Swelling of this type may affect the internal void structure of the sample and allow damage to occur that might not occur under field conditions.

The amount of damage to a mixture caused by stripping varies with the number of freeze-thaw cycles. In general, the amount of water-induced damage increases with additional freeze-thaw cycles. Because the change in water sensitivity of a mixture between one freeze-thaw cycle and seven freeze-thaw cycles is a characteristic of the mixture, retained resilient modulus ratios for seven freeze-thaw cycles cannot be accurately predicted from a test method that uses only one freeze-thaw cycle.

When the feasibility of using a material is being investigated, one freeze-thaw cycle should be sufficient in most cases. This test can determine if there are problems with the material that could cause stripping-related distress early in pavement life. To determine the long-term effect of stripping on field performance, multiple freeze-thaw cycles may need to be used. Multiple cycles may indicate that additional stripping can occur after only one freeze-thaw cycle. The true stripping potential of a mixture may be masked somewhat by test methods that use only one cycle.

TABLE 7 RELATIVE RANK OF SENSITIVITY TO WATER DAMAGE FOR EACH MIXTURE

Method 1	Method 2	Method 3	Method 4 After 3 Cycles	Method 4 After 7 Cycles
Mix F	Mix A	Mix A	Mix G	Mix G
Mix B	Mix B	Mix D	Mix D	Mix E
Mix C	Mix C	Mix E	Mix E	Mix D
Mix G	Mix E	Mix C	Mix A	Mix A
Mix D	Mix G	Mix G	Mix B	Mix B
Mix E	Mix F	Mix F	Mix F	Mix F
Not tested	Mix H	Mix B	Mix C	Mix C
Mix A	Not tested	Mix H	Mix H	Mix H
Mix H	Mix D			

NOTE: Mixtures are ranked highest to lowest in water-induced damage based on retained resilient modulus values.

ACKNOWLEDGMENT

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Coal-Based Synthetic Asphalt: Oxidative Aging and Testing of Compacted Bituminous Mixtures

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An asphalt binder material, functionally equivalent to petroleum asphalt, has been produced by catalytic hydroliquefaction of coal. The synthetic coal-based asphalt, with approximately 3 percent polymer additive, meets all ASTM and AASHTO specifications for asphalt paving cements, with the exception of the optional thin-film oven test weight loss specification. Age hardening of the coal-based asphalt is primarily due to air oxidation rather than volatilization or polymerization. Marshall stability and immersion-compression testing show that the polymer-modified, coal-based asphalt produces compacted mixtures with good moisture resistance and high compressive strength compared with those of control samples prepared with petroleum asphalt of the same viscosity grade.

In the United States, highway construction and maintenance consume 25 million to 30 million tons (22.7 million to 27.3 million metric tons) of asphalt each year. In hydrocarbon value, this consumption is approximately equivalent to 400,000 barrels (63 588 m^3) per day of crude oil. Thus considerable savings in U.S. petroleum consumption could be attained by replacing asphalt with a functionally equivalent substitute. In light of the current oil glut, there is little economic motivation for commercial production of synthetic asphalts. However, considering worldwide political uncertainties, which could quickly affect oil supplies, and the increasing U.S. consumption and declining U.S. production rates, it appears to be useful to continue a limited research effort along these lines and to document findings for future contingencies. Development of such technology also increases the potential value of the vast U.S. coal resources.

Since 1974 many processes for converting coal to liquid and gaseous fuels have been developed. It is possible that similar technology could be used to produce a product functionally equivalent to petroleum-based asphalt (i.e., coal-based asphalt). Thus the overall objective of this project was to investigate, in the laboratory, the application of current liquefaction technology to the production of a synthetic asphalt product from coal. This is a formidable objective because of the inherently differing properties of coal and petroleum. Because asphalt specifications have been developed from petroleum experience, possible difficulties in producing a material that meets these specifications from a chemically different raw material (i.e., coal) can be expected.

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Coal tar pitches and oils derived from carbonization processes have been used to some extent for various road paving and coating applications (*1–4*). The production of most of these materials did not involve high-temperature, high-pressure, catalytic hydrogenation typical of current direct coal liquefaction technology. However, Calkins and Silver (*5*) and later Hoffman (*6*) did use coal liquefaction technology at high severity to produce a coal-based asphalt-type product. In general, this product had higher viscosity temperature susceptibility (VTS) and oxidative hardening rates than did corresponding petroleum asphalt.

Previous work (*1*) has shown that synthetic asphalt cements with acceptable aging indices and VTS properties can be produced from coal by severe hydrotreatment followed by addition of approximately 3 percent by weight styrene-butadiene copolymer. The purpose of the present paper is to report additional results regarding oxidative hardening of the coal-based asphalt and results of testing compacted bituminous mixtures prepared with the coal-based asphalt for compressive strength and water damage susceptibility.

EXPERIMENTAL MATERIALS AND PROCEDURES

Materials

An Illinois No. 6 bituminous coal was catalytically hydrogenated in a batch autoclave in the presence of a heavy coal-derived V1067 recycle solvent and a $NiMo/Al_2O_3$ catalyst. The V1067 solvent was produced by the two-stage catalytic coal liquefaction process at Wilsonville, Alabama, in Run 245 and has a viscosity of 97 poises at 60°C. Complete elemental analysis and asphalt testing properties for the V1067 solvent, and analysis of the other materials, are available elsewhere (*1*). The polymer modifier used throughout was a random, linear copolymer of styrene and butadiene (45 percent styrene, density 0.965 g/cm^3) purchased from Aldrich Chemical Company.

Reaction Procedures

The base reaction conditions for the production of the coal-based asphalt are given in Table 1. Liquid and solid reaction products were diluted with trichloroethylene (TCE) and altered

TABLE 1 BASE REACTION CONDITIONS FOR COAL-BASED ASPHALT REACTIONS

Factor	Value
Temperature (°C)	425
Pressure (psig)	2,500
Reaction time (hr)	2
Agitation rate (rpm)	2,000
Catalyst	Shell 324 $NiMo/Al_2O_3$, 1/32-in. extrudates
Coal type	Illinois No. 6 (bituminous)
Solvent	V1067
Solvent-to-coal ratio	2 to 1
Feed-to-catalyst ratio	10 to 1

NOTE: 1 psi = 6.89 kPa, 1 hr = 3,600 sec, 1 in. = 2.54 cm, and psig = pounds per square inch, gauge.

to remove ash and insoluble organic matter (IOM). Sufficient styrene-butadiene copolymer dissolved in TCE was added to the liquid product after filtration to obtain approximately 3 percent by weight polymer in the final asphalt product after distillation. The coal-based asphalt product was then prepared by distillation of the liquid at 1 mm Hg to obtain a specified viscosity-graded asphalt. Complete details of these procedures are available elsewhere (*1*). No polymer modifier was used in control experiments performed with petroleum-based AC-20.

Testing Procedures

The properties of the distillation residue were evaluated to assess its potential as a synthetic asphalt. The following procedures were used: absolute viscosity at 60°C (ASTM D 2171), kinematic viscosity at 135°C (ASTM D 2170), penetration at 25°C (ASTM D 5), ductility at 25°C (ASTM D 113), solubility in TCE (ASTM D 2042), and thin-film oven test (TFOT) (ASTM D 1754). Testing of compacted bituminous mixtures was performed using the Marshall testing method (AASHTO T 245) and the immersion-compression (IC) test (AASHTO T 165 and T 167) to assess compressive strength and water damage potential. More specific information on moisture damage could be obtained by AASHTO T 283; however, this test was not used.

RESULTS AND DISCUSSION

Aging

Previous results indicated that coal-based asphalt cements tended to age harden faster than their petroleum counterparts (*1*). In this earlier work the major variable was the (unmeasured) partial pressure of oxygen present during the aging time at elevated temperature in the TFOT. Because the primary difference in the experiments was the amount of oxygen present, this, not simply loss of volatiles or internal polymerization, appeared to be the major factor in the hardening of the coal-derived material in the TFOT, although the absence of the latter two processes was not conclusively shown.

To further examine this hypothesis and explore in more detail the mechanism of age hardening of coal-based asphalt, a series of additional experiments was performed under more closely controlled conditions using a sample of heavy coal-derived V1067 solvent that had been distilled to produce a residue with a viscosity of 2145 poises (215 Pa-sec) at 60°C, corresponding to an AC-20 grade asphalt. No polymer modifier was added to this material. The purpose of these experiments was to attempt to separate the processes of oxidation, polymerization, and volatiles loss, all of which could contribute to age hardening of the coal-based asphalt.

To facilitate the aging study in a more closely controlled atmosphere, TFOT conditions specified in ASTM D 1754 were slightly modified: The cylindrical flat-bottomed pan containing the asphalt sample was placed in a capped 1-gal stationary vessel inside the TFO. The vessel was equipped with a gas inlet and outlet tube. The vessel contents then were maintained under flowing pure nitrogen or air preheated to the oven temperature of 163°C. This controlled atmosphere flowed over the sample at a rate of 1 standard cubic foot per hour (2.83×10^{-2} m^3/hr). The sample was not rotated.

In addition to the flowing air and nitrogen experiments, some experiments were carried out in a closed stainless steel batch reactor (tubing bomb) like that described by Brooks et al. (*7*). The purpose of these closed tubing bomb experiments was to examine the influence of internal polymerization and condensation reactions, by themselves, on the age-hardening process. Because this was a completely closed system, no loss of volatiles was possible.

As the data given in Table 2 indicate, in a flowing air or nitrogen atmosphere, the viscosity increased with increasing aging time. Generally, two samples were tested under each aging condition. There is some variation in the results; this was most probably due to the formation of a film on the surface of the samples, which made it difficult to obtain a uniform representative sample. However, the large differences among the different aging conditions are significant. For the 5-hr run, the viscosity of the sample aged in air is about 70 times that of the sample aged in nitrogen. These results confirm that oxygen is the major factor in the age hardening of coal-based asphalt.

For control purposes, age hardening of a commercial AC-20 grade petroleum asphalt was also determined under the same conditions. With an initial viscosity of 2209 poises (221 Pa-sec) for the petroleum AC-20, the viscosity after 5 hr under flowing nitrogen and air increased to 2740 poises (274 Pa-sec) and 14 119 poises (1412 Pa-sec), respectively. The petroleum asphalt experienced a weight loss of 0.045 percent in nitrogen and a weight gain of 0.045 percent in air. The age hardening experienced in the flowing nitrogen is primarily a result of volatiles loss for both the AC-20 control and the coal-based asphalt. The results in the batch reactor (tubing bomb) under nitrogen at 163°C (Table 2) indicate that, in the absence of volatiles loss and oxidation, there is no significant age hardening.

The viscosity of coal-based asphalt remained essentially unchanged even after 15 hr at reaction temperature. The viscosity of the AC-20 control was 2161 poises (216 Pa-sec) after 5 hr in the tubing bomb. These tubing bomb results indicate that condensation (polymerization) reactions alone (in the absence of oxygen) are not a significant factor in the short-term aging of coal-based asphalt.

Coal mineral matter is known to have a catalytic effect in various chemical reactions. The specific ash sample used here was not tested for catalytic activity; however, in past work, it was found that essentially all tested coal ashes, especially those containing iron, possess some activity (*8*). To examine a

TABLE 2 AGE HARDENING OF A COAL-BASED ASPHALT IN CONTROLLED ATMOSPHERE EXPERIMENTS

	Flowing Nitrogen[b]		Flowing Air[b]		Tubing Bomb[d], Nitrogen Atmosphere,
Aging Time[a] (hr)	Viscosity[c] (poises)	Weight Loss (%)	Viscosity (poises)	Weight Loss (%)	Viscosity (poises)
1	2850	0.85	4889	0.61	1975
	2562	0.58	4641	0.62	2121
3	3992	2.20	32 238	1.74	2077
	3652	1.76	25 998	1.76	2060
5	5308	3.19	422 604[e]	2.90	1900
	6050	3.11	243 540	2.54	
15	16 941	6.63	—[f]	8.18	2023
	13 245	6.38			2058

NOTE: 1 poise = 0.1 Pa-sec.
[a]Aging conditions: temperature = 163°C, sample amount = 25 g.
[b]Flow rate = 1 standard cubic foot per hour (8×10^{-6} m^3/sec).
[c]Viscosity of sample before aging = 2145 ± 47 poises; all viscosities are measured at 60°C in poises.
[d]Agitation rate = 500 cycles/min (52.2 rad/sec) (2 steel balls added); N_2 pressure at room temperature = 100 psi (689 kPa).
[e]Large difference in values caused by extreme hardening and film formation on sample surface.
[f]Viscosity too high to measure.

possible catalytic effect of coal ash on the aging process, 6.6 percent by weight of coal ash was added to a sample of the coal-based asphalt and it was subjected to flowing air at 163°C. The presence of ash in the coal-based asphalt increased the initial viscosity before the test from 2145 poises (215 Pa-sec) to 2715 poises (272 Pa-sec). After aging 3 hr in air, the viscosity was 27 584 poises (2758 Pa-sec), and a weight loss of 1.38 percent was noted.

Comparison with the corresponding results after 3 hr in air (Table 2) shows no significant catalytic effect of ash on aging. Additional studies conducted with the residue from previous coal liquefaction experiments, which contained coal mineral matter and heavy, highly carbonaceous organic material commonly called insoluble organic matter (IOM), also showed no effect on aging.

To examine the effect of water vapor (present in asphalt hot-mix plants) on the aging process, 3-hr aging experiments were conducted at 163°C using flowing air saturated with water at room temperature. A marginal decrease of approximately 15 percent in aging was observed when the flowing air contained water vapor compared with air without water vapor. It is possible that this effect is caused by the decrease in oxygen partial pressure in the gas mixture due to added moisture.

The use of antioxidant compounds, usually at 1 to 4 percent by weight, has been investigated in an attempt to reduce the age hardening of petroleum asphalts. Oxidative aging is generally thought to proceed by a free radical chain reaction, initiated by light or heat, or both, and propagated by peroxide radicals. The antioxidants are thought to inhibit this process by either (a) terminating the free radicals carrying the chain or (b) reacting with peroxide radical precursors to form inactive species. In particular, lead dialkyldithiocarbonate (LDADC) has been thought to function by the second mechanism, depending to some extent on the particular asphalt used (*9, 10*). LDADC in the form of Vanlube™ 100 (R. T. Vanderbilt Co., Inc., New York) was tested with the same coal-based asphalt (Table 2). Two percent by weight LDADC was added to two samples of coal-based asphalt, and they were then exposed to flowing air at 163°C. The presence of LDADC decreased the average initial sample viscosity from 2145 poises (215 Pa-sec) to 1747 poises (175 Pa-sec). After aging for 3 hr in air the average viscosity was 4548 poises (455 Pa-sec), and a weight loss of 2.1 percent was noted. Comparison with corresponding results after 3 hr in air without LDADC (Table 2) shows that the antioxidant reduced the amount of hardening, although the weight loss is still high. A similar effect was found by Haxo and White (*9*), who also made a detailed study of the volatility of the antioxidant and found this effect to be small at 60°C. It should be noted that the reduction in age hardening could have been partly due to a thin film formed on the asphalt surface during this test; however, further testing of this additive, such as in a rolling TFOT where film formation would not be a factor, does appear to be warranted.

A final set of measurements was made to examine chemical changes caused by aging in the coal-based asphalt samples (Table 2). A solvent fractionation technique was used with the following fractions (*11*):

- Oil—pentane solubles,
- Asphaltenes—pentane insolubles and benzene solubles,
- Preasphaltenes—benzene insolubles and methylene chloride/methanol (90/10) solubles, and
- IOM—methylene chloride/methanol (90/10) insoluables (ash free).

As the data given in Table 3 indicate, the oil and IOM in the original coal-based asphalt are 68.8 and 3.5 percent, respectively. For the sample aged in air for 15 hr, oil decreased to 57.5 percent and IOM increased to 14.0 percent. Thus it appears that, during age hardening of coal-based asphalt, the lighter fractions are oxidized to produce heavier, less soluble fractions. The product distribution of samples aged in nitrogen

TABLE 3 PRODUCT SOLUBILITY ANALYSIS OF COAL-BASED ASPHALT DURING AGING

	Original	Aged in Flowing Air for				Aged in N_2 (tubing bomb) for		
	(unaged)	1 hr	3 hr	5 hr	15 hr	1 hr	5 hr	15 hr
Oil	68.8	64.4	65.5	62.4	57.5	69.3	68.2	65.1
Asphaltenes	23.6	23.9	22.2	23.4	20.5	20.8	22.4	25.2
Preasphaltenes	4.1	4.3	4.0	5.1	8.0	6.5	5.6	6.0
IOM	3.5	7.4	8.3	9.1	14.0	3.4	3.8	3.7

NOTE: All values are in percentage by weight.

in the closed tubing bomb was not changed significantly, which again shows reaction with oxygen to be the primary cause of age hardening.

Testing of Bituminous Mixtures

The properties of bituminous mixtures prepared with the coal-based asphalt binder were evaluated by the Marshall testing method (AASHTO T 245) and by the immersion-compression test (AASHTO T 165 and T 167).

Marshall Stability Results

To produce sufficient specification-grade, coal-based asphalt for Marshall stability testing, two experiments (Runs CP 30 and 31) were performed in which Illinois No. 6 bituminous coal and V1067 solvent were catalytically hydrogenated in a 1-gal autoclave under the conditions given in Table 1. Experimental methods were the same as those used previously (*1*). When the temperature in the distillation flask reached 213°C at 1 mm Hg, an AC-20 grade coal-based asphalt was obtained (Table 4). After the two asphalt products from Runs CP 30 and 31 were collected, they were blended with approximately 3 percent by weight of copolymer, and the viscosity of the blended mixture was determined as indicated in Table 4. This blended mixture was used in the following testing.

To compare the resistance to plastic flow of coal-based asphalt mix with that of petroleum-based asphalt mix, both the coal-based asphalt (from Table 4) and Chevron AC-20 were sent to the Bureau of Materials and Testing of the Alabama Highway Department at Montgomery for Marshall stability testing according to AASHTO T 245. The aggregate mix used in this test, from Sharp Sand and Gravel Co., Tuskegee, Alabama, consisted of 15 percent pea gravel, 30 percent shot gravel, 40 percent coarse sand, and 15 percent fine sand. The sieve analysis for the job mix is given in Table 5 and is in accordance with the specification of the Alabama Highway Department. Each compacted mix (i.e., specimen) consisted of 67 g of asphalt and 1100 g of total aggregate; that is, it consisted of 5.74 percent asphalt and 94.26 percent aggregate by weight of total mix. Because of differences in specific gravities, equal weight percentages resulted in different volumes of binder being used in the tests (65.0 cm^3 versus 60.2 cm^3). As the data in Table 6 indicate, a larger volume percentage was obtained of the AC-20 than of the coal-based binder. Marshall testing was performed only at this single condition, and optimum binder contents were not determined for the two binders. Thus the conclusions obtained from the Marshall tests must be viewed with these limitations in mind.

TABLE 4 CHARACTERISTICS OF COAL-BASED ASPHALT USED IN MARSHALL TESTING

	Run		Blend of CP 30 and
	CP 30	CP 31	31
Product recovery (%)	97.36	97.37	
Coal conversion, maf (%)	84.59	84.98	
Asphalt/distillate ratio	1.43	1.40	
Distillation end point (°C at 1 mm Hg)	213	213	
Polymer modifier by weight (%)	3.31	3.34	
Asphalt properties			
Absolute viscosity at 60°C (poises)	1830	1897	1875
Kinematic viscosity at 135°C (cSt)	455	470	466
VTS	3.36	3.35	3.35

NOTE: 1 poise = 0.1 Pa-sec, 1 cSt = 10^{-6} m^2/sec; maf = moisture and ash free.

TABLE 5 SIEVE ANALYSIS OF AGGREGATE COMBINATION USED IN MARSHALL STABILITY TESTING

U.S. Sieve Series (square mesh type)	Percentage Passing by Weight	Section 411 Mix A Specification[a]
1/2 in.	100	100
3/8 in.	94	87–100
No. 4	75	60–85
No. 8	61	44–74
No. 50	15	10–32
No. 100	8	3–13
No. 200	4	1–5

NOTE: 1 in. = 2.54 cm. Aggregate mix composition: 15 percent pea gravel, 30 percent shot gravel, 40 percent coarse sand, and 15 percent fine sand.

[a]From *Standard Specifications for Highway Construction*, 1985 ed., State of Alabama Highway Department. This job mix is designed by the Marshall method to produce a minimum of 750-lb (3336-N) stability.

Table 6 gives a summary of the test results for Marshall stability testing. The average Marshall stability of the three coal-based asphalt specimens (after multiplication by a stability correlation ratio of 1.04) is 1,919 lb (8536 N); the average Marshall flow value of these three specimens is 9.9 (0.25 cm) in units of 1/100 in. (0.25 mm). The average Marshall stability for the three petroleum mix specimens, after multiplication by a stability correlation ratio of 1.04, is 1,389 lb (6178 N), which is lower than that of the coal-based asphalt mix. The average

TABLE 6 SUMMARY OF RESULTS FOR MARSHALL STABILITY TESTING

	Coal-Based Asphalt	Chevron AC-20
Asphalt Properties		
Absolute viscosity at 60°C (poises)	1875	2023
Kinematic viscosity at 135°C (cSt)	466	448
VTS	3.35	3.41
Specific gravity (25/25°C)	1.1127	1.0300
Marshall Test Results of Mixes		
Asphalt content, % by weight of aggregate (% of compacted bulk volume)	6.10 (12.4)	6.10 (13.4)
No. of blows at each end of test specimen	50	50
Mixing temperature (°C)	151.7	151.7
Compaction temperature (°C)	140.6	140.6
Test temperature (°C)	60	60
Bulk specific gravity of compacted mixture	2.257	2.262
Maximum specific gravity of mixture	2.435	2.411
Asphalt absorption (% by weight of aggregate)	0.49	0.47
Effective asphalt content (% by total weight of mixture)	5.29	5.31
Voids in mineral aggregate (% of bulk volume)	18.0	17.8
Air void content (% of total volume)	7.3	6.2
Marshall stability (N)	8536 (1,919 lbf)	6178 (1,389 lbf)
Marshall flow (0.25 mm)	10	8.8

NOTE: 1 N = 0.224 lbf; 1 cm = 0.4 in.; t°C = (t°F − 32)/1.8; 1 kPa = 0.145 psi.

Marshall flow value for these specimens is 8.8 (0.22 cm), which is also lower than that of the coal-based asphalt mix.

According to Marshall design criteria from the Asphalt Institute Manual Series 2, for medium traffic conditions, the Marshall stability should be at least 750 lb (3336 N), and the Marshall flow should be between 8 and 18 (*12*). Examination of the results in Table 6 shows that requirements are met by both compacted mixtures. It is notable that the coal-based asphalt mixture shows both greater Marshall stability and Marshall flow values, although the higher flow value for the coal-based asphalt could be viewed as a negative factor. In addition to meeting the previously mentioned criteria, a satisfactory paving mixture should have sufficient voids in the compacted mixture to allow for a slight amount of additional compaction under traffic loading without flushing, bleeding, and loss of stability; however, void content should be low enough to keep out harmful air and moisture.

The data in Table 6 indicate that the coal-based asphalt compacted mixture has slightly higher air voids than the Chevron AC-20 compacted mixture. This results because, as noted earlier, the specific gravity (25/25°C) of the coal-based asphalt is higher than that of Chevron AC-20, which results in less volume of coal-based asphalt in the mixture when equal binder weights are used. The air voids given in Table 6 are within the 3 to 11 percent specified for base or binder courses in Table II-2 of the AASHTO *Interim Guide for Design of Pavement Structures;* however, they are greater than the 5 percent recommended by the Asphalt Institute Marshall design criteria (*12*). Thus some adjustments in the mix would be necessary to produce a design that had all properties within recommended limits. Because of the limited scope of the present study, this procedure was not performed. However, similar procedures used in compressive strength testing (Tables 7 and 8) show that air voids can be varied by changing the asphalt binder content of the mix. Of course, other properties would need to remain within specifications.

TABLE 7 COMPRESSIVE STRENGTH FOR THE LIMESTONE MIX AT VARIOUS COAL-BASED ASPHALT CONTENTS

	Mixture			
	A	B	C	D
Coal-Based Asphalt Properties				
Absolute viscosity at 60°C (poises)	1805	1768	1856	1856
Kinematic viscosity at 135°C (cSt)	445	447	413	413
VTS	3.37	3.36	3.45	3.45
Specific gravity, 25/25°C	1.1127	1.1127	1.1127	1.1127
Polymer modifier (% by weight)	3.23	3.38	3.13	3.13
Test Results				
Asphalt content, % by weight of total mix (% by bulk volume of compacted mixture)	4.11 (8.60)	6.00 (12.8)	6.91 (14.7)	9.69 (20.4)
Bulk specific gravity of compacted mixture	2.330	2.378	2.368	2.344
Maximum specific gravity of mixture	2.580	2.515	2.464	2.396
Effective asphalt content (% by total weight of mixture)	3.96	5.85	6.76	9.54
Voids in mineral aggregate (VMA), % of bulk volume	17.94	17.91	19.04	22.26
Air void content (% of bulk volume)	9.69	5.45	3.90	2.17
Compressive strength at 25°C (psi)	374 ± 18	549 ± 22	445 ± 26	402 ± 20

NOTE: 1 psi = 6.8948 kPa.

Compressive Strength Testing

To compare compressive strength of coal-based synthetic asphalt and petroleum asphalt, immersion-compression testing was conducted (AASHTO I 165 and I 167). In these experiments, two aggregate mixtures, which are commonly used for highway construction in Alabama, were tested using synthetic coal-based asphalt and a control petroleum asphalt. One mixture consisted of No. 78 limestone (20 percent), No. 810 limestone (65 percent), and coarse sand (15 percent). The other mixture consisted of 1/2-in. (1.27-cm) crushed gravel (70 percent), 3/8-in. (0.95-cm) to dust limestone (10 percent), and coarse sand (20 percent). The sieve analysis for the limestone aggregate mix and the crushed gravel aggregate mix is given in Table 9. As a baseline, these aggregate mixtures and Chevron AC-20 grade petroleum asphalt, which contained no antistripping agent, were used. The coal-based asphalt used in these tests was prepared at conditions noted previously in Table 1 and included approximately 3 percent of the polymer additive described earlier.

TABLE 8 COMPRESSIVE STRENGTH FOR THE LIMESTONE MIX AT VARIOUS PETROLEUM ASPHALT CONTENTS

	Mixture				
	A	B	C	D	E
Asphalt content, % by weight of total mix (% by bulk volume of compacted mixture)	4.16 (9.40)	5.61 (12.9)	6.48 (14.9)	7.34 (16.8)	9.01 (20.4)
Bulk specific gravity of compacted mixture	2.327	2.369	2.364	2.354	2.337
Maximum specific gravity of compacted mixture	2.560	2.503	2.471	2.439	2.366
Effective asphalt content (% by total weight of mixture)	3.97	5.43	6.30	7.16	8.83
Voids in mineral aggregate (VMA), % of bulk volume	18.06	17.85	18.77	19.86	21.44
Air void content (% of bulk volume)	9.10	5.35	4.33	3.48	1.23
Compressive strength at 25°C (psi)	258 ± 8	318 ± 7	325 ± 12	322 ± 16	255 ± 13

NOTE: 1 psi = 6.8948 kPa. Properties of the petroleum asphalt: absolute viscosity at 60°C = 2023 poises, kinematic viscosity at 135°C = 448 cSt, VTS = 3.41, and specific gravity (25/25°C) = 1.0300.

Effect of Binder Content

Experiments were performed with both coal-based asphalt and the petroleum AC-20 control asphalt to evaluate the effect of binder content on the compressive strength of a compacted bituminous mixture. The specimens were prepared according to the procedures in ASTM D 1074 (AASHTO T 167) with various asphalt contents using the limestone aggregate described in Table 9. The specimens were cured for 24 hr at 60°C and then maintained at 25°C in an incubator for 24 hr. Testing was done in axial compression at a uniform rate of vertical deformation of 5.08 mm/min (0.2 in./min), and the compressive strengths were obtained at each asphalt content. In this work, all asphalt contents were based on total weight of compacted mixture. Experimental results obtained according to methods specified by the Asphalt Institute (*12*) are given in Tables 7 and 8. The properties of the coal-based asphalt in Table 8 vary slightly from column to column because it was prepared in individual 1-gal autoclave reactions. For the mix with 6 percent by weight asphalt, the maximum specific gravity was determined by vacuum saturation (ASTM D 2041). To calculate the maximum specific gravity of the other mixtures in Tables 7 and 8, it was assumed that the effective specific gravity was constant at various asphalt contents. Because asphalt absorption does not vary appreciably with variations in asphalt content, this assumption is acceptable (*12*). As the data in Table 7 indicate, the compacted mixture with 6 percent by weight coal-based asphalt gives the highest compressive strength, 549 psi. For the petroleum asphalt, 6.48 percent by weight asphalt content was near the optimum. The results given in Tables 8 and 9 indicate the existence of an optimum binder content for maximum compressive strength. Additional experiments would be needed to more accurately locate the optimum. For the remaining tests, a coal-based asphalt content of 6 percent by weight was used.

Effect of Temperature on Compressive Strength

As noted previously, the compressive strength at 25°C of compacted specimens prepared with coal-based asphalt is typically higher than that of the AC-20 control specimens. This trend was examined further at additional temperatures of 40°C and 60°C. To attain the test temperature, the 4- × 4-in. cylindrical specimens were stored in an oven for 4 hr at 40°C or

TABLE 9 AGGREGATES USED IN COMPRESSIVE STRENGTH AND IMMERSION COMPRESSION TESTING

U.S. Sieve	Percentage Passing
Sieve Analysis of Limestone Aggregate Mixture	
3/4 in.	100
1/2 in.	100
3/8 in.	96
No. 4	68
No. 8	46
No. 50	11
No. 100	8
No. 200	5
Sieve Analysis of Crushed Gravel Aggregate Mixture	
3/4 in.	100
1/2 in.	100
3/8 in.	97
No. 4	74
No. 8	57
No. 50	21
No. 100	12
No. 200	8

60°C and then tested immediately upon withdrawal. The coal-based asphalt used at 40°C and 60°C was prepared under the reaction conditions of Table 1 and had an absolute viscosity of 1805 poises at 60°C, a kinematic viscosity of 445 cSt at 135°C, a VTS of 3.37, and a polymer modifier content of 3.23 percent by weight. Equal volumes (105 cm^3) of the coal-based and petroleum asphalt binders were used. The results given in Table 10 and shown in Figure 1 indicate that the compressive strengths for the coal-based asphalt mix remained higher at the elevated temperatures for both the limestone and the crushed gravel mixes.

Water Damage Potential

A large problem in pavement durability is the stripping action of water on the asphalt aggregate mixture and the resultant loss of strength and pavement integrity (*13*). One measure of this susceptibility to water damage can be obtained by the immersion-compression test (AASHTO T 165 and T 167). The retained compressive strength of specimens is measured after

TABLE 10 COMPRESSIVE STRENGTH AT DIFFERENT TEMPERATURES

	Coal-Based Asphalt		Petroleum Asphalt (AC-20)	
	Limestone	Crushed Gravel	Limestone	Crushed Gravel
Binder in total mix (% by weight)	6.00	6.00	5.61	5.61
Volume binder in total mix (cm^3)	105	105	105	105
Compressive strength at 25°C (psi)	549 ± 22	473 ± 19	318 ± 7	249 ± 15
Compressive strength at 40°C (psi)	273 ± 16	217 ± 15	168 ± 10	140 ± 4
Compressive strength at 60°C (psi)	128 ± 8	116 ± 4	107 ± 6	86 ± 5

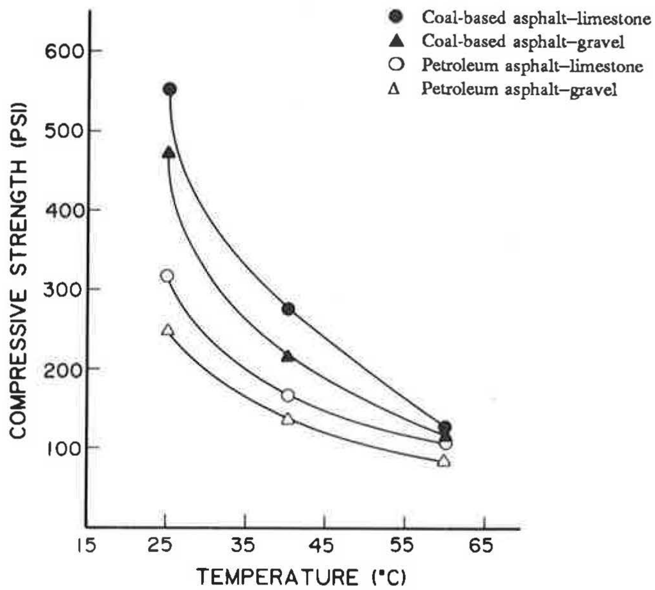

FIGURE 1 Compressive strength as a function of temperature.

immersion in water at 60°C for 24 hr and compared with the initial compressive strength. Tables 11 and 12 give the results of these tests for both the limestone and the gravel mixes with coal-based and petroleum asphalts. These results indicate superior performance of the coal-based asphalts in regard to susceptibility to water damage. For both aggregates, the coal-based mixes have higher percentages of retained strengths after immersion. Although the increased retained strength index of the coal-based asphalt for the limestone aggregate may not be highly significant, the increased retained strength for the gravel is quite significant. Many antistripping additives give less retained strength improvement than is shown by the difference between the coal-based asphalt and the AC-20.

Effect of Coal Solids

In commercial production of a coal-based asphalt, it would not be economical to remove the mineral matter and undissolved coal solids (IOM) from the asphalt product. This material most likely would be retained in the binder and would thus act as filler in the binder-aggregate mix. The effect that this type of filler material (i.e., coal mineral matter and insoluble organic matter) would have on the resulting properties of the bituminous concrete is not known, and thus some experiments were performed in this area.

Solids-free, coal-based asphalt in the AC-20 range was produced as usual. To this asphalt, sufficient coal liquefaction residue (67 percent ash, 37 percent IOM) obtained from coal liquefaction runs at the Advanced Coal Liquefaction Research and Development Facility, Wilsonville, Alabama, was added to produce a binder containing 29 percent by weight solids. This

TABLE 11 SUMMARY OF RESULTS FOR IMMERSION-COMPARSSION TEST FOR LIMESTONE MIX

	Coal-Based Asphalt Mix	Chevron AC-20 Mix
Asphalt Properties		
Absolute viscosity at 60°C) (poises)	1768	2023
Kinematic viscosity at 135°C (cSt)	447	448
VTS	3.36	3.41
Specific gravity (25/25°C)	1.1127	1.0300
Polymer modifier (% by weight)	3.38	0.00
Test Results		
Asphalt content, % by weight of total mix (% by volume of compacted mixture)	6.00 (12.8)	5.61 (12.9)
Bulk specific gravity of compacted mixture	2.378	2.369
Maximum specific gravity of mixture	2.515	2.503
Asphalt adsorption (% by weight of aggregate)	0.161	0.194
Effective asphalt content (% by total weight of mixture)	5.85	5.43
Voids in mineral aggregate (% of bulk volume)	17.91	17.85
Air void content (% of bulk volume)	5.45	5.35
Initial strength at 25°C (psi)	549	318
Retained strength at 25°C (psi)	481	222
Retained index, %	88	70

NOTE: 1 psi = 6.8948 kPa.

TABLE 12 SUMMARY OF RESULTS FOR IMMERSION-COMPRESSION TEST FOR CRUSHED GRAVEL MIX

	Coal-Based Asphalt Mix	Chevron AC-20 Mix
Asphalt content, % by weight of total mix (% by volume of compacted mixture)	6.00 (11.9)	5.61 (12.0)
Bulk specific gravity of compacted mixture	2.198	2.205
Maximum specific gravity of mixture	2.374	2.366
Asphalt absorption (% by weight of aggregate)	0.259	0.307
Effective asphalt content (% by total weight of mixture)	5.76	5.32
Voids in mineral aggregate (% of bulk volume)	18.78	18.19
Air void content (% of bulk volume)	7.41	6.80
Initial strength at 25°C (psi)	473	249
Retained strength at 25°C (psi)	397	144
Retained index (%)	84	58

NOTE: The asphalts described in Table 11 were used. 1 psi = 6.8948 kPa.

solids-containing binder was then used to prepare compacted specimens for the immersion-compression test. An amount of the −200 mesh solids in the limestone aggregate equal in weight to the coal-derived solids contained in the binder was removed before mixing so that the total weight of solids in the compacted specimens remained the same as in earlier tests. The results of tests with the coal-derived, solids-containing binder are given in Table 13. Comparison with the corresponding data in Table 12 shows that these results are not significantly different. Thus, on the basis of these admittedly limited data, it appears that coal mineral matter and IOM will not be detrimental to the properties of bituminous concrete prepared with coal-derived asphalt binder if the solid material is accounted for as aggregate fines. Although the coal-based asphalt binder contained 29 percent by weight coal-derived solids, because the binder content of a compacted specimen was only 6 percent by weight, the coal-derived solids were only 2.6 percent by weight of the total aggregate.

TABLE 13 IMMERSION-COMPRESSION TEST FOR THE COMPACTED MIXTURE OF LIMESTONE AGGREGATE AND SOLIDS-CONTAINING COAL-BASED ASPHALT

	Value
Properties of Solids-Free Coal-Based Asphalt	
Absolute viscosity at 60°C (poises)	1752
Kinematic viscosity at 135°C (cSt)	397
VTS	3.46
Specific gravity (25/25°C)	1.1127
Polymer modifier (% by weight)	3.13
Test Results	
Asphalt content (solids containing)	
% by weight of total mixture	7.74
% by volume of compacted mixture[a]	14.7
Asphalt content (solids free)	
% by weight of total mixture	6.00
% by volume of compacted mixture	12.14
Bulk specific gravity of compacted mixture	2.251
Maximum specific gravity of mixture	2.435
Asphalt absorption (% by weight of aggregate)	0.161
Effective asphalt content (% by total weight of mixture)	5.85
Voids in mineral aggregate (% of bulk volume)	22.29
Air void content (% of bulk volume)	7.56
Initial strength at 25°C[b] (psi)	583 ± 33
Retained strength at 25°C[b] (psi)	525 ± 24
Retained index (%)	90

NOTE: 1 psi = 6.8948 kPa.
[a]Based on assumed specific gravity of coal solids of 1.5.
[b]Value was obtained from average of the specimens.

CONCLUSIONS

A polymer-modified coal-based asphalt has been produced that is functionally equivalent to petroleum asphalt according to standard laboratory tests. The coal-based binder is more susceptible to age hardening than are petroleum binders. Age hardening of the coal-based synthetic asphalt binder is caused primarily by oxygen, not internal polymerization or volatilization. Antioxidant compounds indicate some promise for decreasing the age hardening of coal-based asphalt. Bituminous concrete prepared with polymer-modified coal-based asphalt has satisfactory Marshall test values, compressive strength, and moisture resistance, which generally exceed those of the same viscosity-grade petroleum asphalt. The presence of coal solids (e.g., ash and undissolved coal) has little effect on aging or compacted mixture properties, as long as it is allowed for as filler material in the mix.

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Predicting the Performance of Montana Test Sections by Physical and Chemical Testing

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Twenty test sections were constructed on a portion of Interstate 90 in south-central Montana. The objective was to compare the performance of asphalts from each of the state's four refineries in sections alone and with each of several additives including hydrated lime, fly ash, and an antistripping agent. Sections containing carbon black, ChemCrete, an 85-100 grade asphalt, and a blend designed by the high-performance gel permeation chromatography (HP-GPC) model to resist cracking were also constructed. Physical and chemical data available at the time of construction in 1983 are discussed in relation to the cracking and rutting performance evident after 4 years in service. The usefulness of these data for predicting performance is examined. In this data set, rutting is related to asphalt content, voids content, and possibly asphalt source. Cracking is related to asphalt source and, with one exception, to molecular size distribution from HP-GPC.

The Montana Department of Highways (MDOH) has frequently obtained quite different performances from materials that have met all existing specifications. Reasons for these differences in behavior have been difficult to pinpoint because of the variability inherent in field construction. In an effort to isolate factors that contribute to performance, MDOH constructed a series of test sections in which most factors were held constant and the key variables were asphalt source and type of additive.

Physical parameters normally obtained by the MDOH as well as some chemical parameters have been used to monitor the materials. Performance has been observed at regular intervals and clear-cut differences among the sections have now emerged. The rationale for the experimental sections is discussed in this paper, and details are given on the physical and chemical data available at the time of construction and their relation to performance 4 years later. The purpose of this approach is to find factors that might be used as predictive tools to the eventual end of reducing the chances of pavement failure.

EXPERIMENTAL DESIGN

Historically, virtually all of the paving asphalt used in Montana has been obtained from four refineries in the state. Three of these refineries are in the Billings area in south-central Montana; the fourth is in Great Falls in the north-central part of the state. No two of these refineries currently use the same crude blend, although parts of the crude slates of the Billings area refineries may overlap.

Two of the refineries use a propane deasphalting (PDA) process for all or part of their asphalt production. Asphalt may be made to grade from the PDA unit, or it may be blended to grade from a lower-penetration PDA product. The other manufacturers make asphalt to grade from the vacuum tower, although both may occasionally blend to achieve a desired grade.

Although the refining picture in the state is not as simple as might be hoped, and in spite of a few rather sharp changes that have occurred in either crude source or refining process, the products from each of the refineries were perceived to be remarkably stable before and during construction of the test sections. There was also a perception, based in part on asphalt analyses by high-performance gel permeation chromatography (HP-GPC) done in the laboratory (*1, 2*) and in part on field observations, that there were differences in performance, especially with respect to cracking, of the products from the four refineries.

Therefore it was desired to check the validity of the HP-GPC studies and the more casual observations by constructing test pavements in which the four asphalts could be compared under conditions as nearly identical as possible in regard to design, construction, subgrade, climate, and traffic. The MDOH occasionally uses certain additives or fillers in its paving mixes. Their long-term effects on the performance of pavement would also be tested under similar field conditions. In addition, MDOH wished to observe the behavior of pavements containing the commercial materials ChemCrete (California) and Microfil 8 (carbon black, Cabot Corp.) as well as a harder grade of asphalt (85-100 pen) the use of which was being proposed as an aid in the prevention of rutting. Finally, a section was designated for an asphalt mixture that would be blended to match the HP-GPC model for crack-resistant performance in Montana.

Twenty test sections were selected for construction. The components of the sections are given in Table 1. A site on Interstate 90 in the south-central plains of Montana was chosen. At that location a 3.2-mi segment of four-lane pavement was scheduled for all new construction. The landscape of gently rolling hills presented no extremes of grade or alignment and no apparent drainage problems. The subgrade was prepared as uniformly as possible over the length of the site. This site was long enough to permit the construction of 20 test sections, each 1,250 ft long, separated by 500-ft transition zones. Ten sections were planned for the eastbound lanes and 10 for the westbound

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TABLE 1 COMPONENTS OF TEST SECTIONS

	Additive			
Refiner	None	Lime	Fly Ash	Antistripping Agent
C[a]	1	2	4	3
B[b]	15	16	14	17
A[c]	6	8	5	7
D[d]	10	9	12	11

NOTE: All are 120-150 grade AC.
[a]Blend.
[b]200-300 with Microfil 8.
[c]200-300 with Chemcrete.
[d]85-100.

lanes. Fortunately, traffic loads are similar in both directions. Construction was begun in 1982; all paving was accomplished in June and July 1983.

CONSTRUCTION

Much of this section is based on work by Bruce (*3*). High-quality aggregate was available near the site. The gradation and physical characteristics of the aggregate met MDOH specifications, and the entire supply was stockpiled to ensure uniformity throughout construction.

Base construction consisted of 1.35 ft of compacted crushed (1.5-in.) base course and 0.20 ft of crushed top surfacing. The plant mix paving was placed in two lifts totaling 0.40 ft. Mixing was done in a drum dryer type of plant at 275°F. Dry additives (hydrated lime, fly ash, Microfil 8) were fed directly into the drum dryer where the first contact was with asphalt cement. Liquid additives (antistripping agent, ChemCrete) were added to the asphalt cement by means of an in-line feeder located just before the entrance to the mixer.

Great care was exercised by the contractor to ensure that each section was paved uniformly with the desired mix. Compaction was accomplished with vibratory and static steel rollers. No surface treatment was used. It should be noted that no serious problems were encountered during paving. The sections have been marked by signing on the right-of-way fence as well as by marks painted on the pavement.

Each test section received an individual mix design (Marshall method) in an effort to achieve constant stability and voids content in the pavement throughout the sections.

PERFORMANCE TO DATE

Observations of the test sections are made at least once a year by a laboratory team and on other occasions by personnel from MDOH. Cores are obtained annually for both physical and chemical testing as well. Observations include a count of all transverse cracks in each section, notation of other types of cracking, measurement of rutting by means of a string-line in both passing and driving lanes at intervals of 300 ft in each section, and notation of flushing and other forms of surface distress. To date only transverse cracking, rutting, and flushing are evident. No stripping of cores has been apparent.

During the winter of 1983–1984, differences in the test sections became evident, notably in the severe transverse cracking of Sections 5, 6, 7, 14, and 15. During the following summer, some of these cracks "healed" under the influence of traffic and the summer sun. Nevertheless, these sections remain some of the more severely cracked of the set.

Performance data gathered in March 1987 are given in Table 2 for rutting and in Table 3 for transverse cracking. Notice that Test Section 19, which was constructed with ChemCrete, does not appear in these tables. This section began to experience rutting in the driving lane soon after it was opened to traffic. The rutting was not uniform and had reportedly reached depths of 0.75 in. at some places when an overlay was applied to the driving lane for reasons of safety. To date, only two or three short random cracks can be found. These unusual problems with ChemCrete probably stem from several sources: the use of asphalt that was too soft (200-300 pen); over-rolling, which may have sealed the surface to access of oxygen; inadequate curing time; or possibly an improper combination of asphalt type and carrier for ChemCrete.

TABLE 2 RUTTING MEASUREMENTS, MARCH 1987

Section No.	Components (asphalt/additive)	Rut Depth (in.)	
10	D	0.16	
12	D/fly ash	0.16	
2	C/lime	0.22	
16	B/lime	0.22	≲ 0.25
20	200-300/Microfil 8	0.24	
1	C	0.25	
18	85-100	0.25	
9	D/lime	0.27	
4	C/fly ash	0.33	
11	D/antistrip	0.33	
17	B/antistrip	0.34	≲ 0.5
3	C/antistrip	0.38	
8	A/lime	0.39	
13	Blend	0.43	
5	A/fly ash	0.52	
14	B/fly ash	0.52	
7	A/antistrip	0.56	> 0.5
15	B	0.56	
6	A	0.62	

RESULTS AND DISCUSSION

At the time of construction, the following types of data were available for correlation with performance.

- Penetration grade of asphalt;
- Additive type, if used;
- Marshall stability, field;
- Marshall flow, field;
- Penetration, 77°F, recovered;
- Viscosity, 275°F, recovered;
- Ductility, 40°F, recovered;
- Percentage of voids in finished pavement;
- Percentage asphalt in finished pavement;
- Penetration-viscosity numbers, original asphalts;
- Corbett analyses, original asphalts;
- Asphalt source; and
- HP-GPC analyses.

In this portion of the paper, these factors will be discussed in turn.

TABLE 3 CRACK COUNTS, MARCH 1987

Section No.	Components (asphalt/additive)	Total Cracks
2	C/lime	0
13	Blend	0
1	C	1
3	C/antistrip	1
12	D/fly ash	2
20	200-300/Microfil 8	8
9	D/lime	7
4	C/fly ash	7
5	A/fly ash	12
6	A	18
10	D	19
16	B/lime	19
11	D/antistrip	23
15	B	23
8	A/lime	28
7	A/antistrip	29
17	B/antistrip	33
14	B/fly ash	36
18	85-100	>52

Penetration Grade

Sixteen of the 20 sections were constructed with 120-150 penetration grade asphalt cement (AC). With regard to penetration, the data given in Tables 2 and 3 reveal no uniformity of performance among these sections. Sections that contain 120-150 AC and no additive rank among the most severely and least severely rutted and also display wide differences in the extent of transverse cracking. Section 18, which contains an 85-100 pen asphalt from Refiner B, is more severely cracked and less severely rutted than the comparable section paved with 120-150 pen asphalt from the same source, indicating that penetration grade may be useful in predicting the relative performance of two grades from the same refinery or crude source.

Types of Additives

Tables 4 and 5 give information about the effects of additives on cracking and rutting. Looking first at the transverse cracking results (Table 4), it is evident that the effects of additives are dependent on both the type of additive and the refiner or crude source. For example, asphalt from Refiner C is robust except in the presence of 1.9 percent fly ash. In contrast, asphalt from

TABLE 4 EFFECT OF ADDITIVES ON TRANSVERSE CRACKING

Refiner	Without Additive	With Lime	With Fly Ash	With Antistripping Agent
C	1	0	7	1
D	19	7	2	23
B	23	19	36	33
A	18	28	12	29

NOTE: Total number of transverse cracks per test section (full-width plus one-lane cracks).

Refiner D, which is a poor performer without additive and with antistripping agent (Acra), has significantly improved performance with lime and fly ash. Even though asphalts from Refiners A and B are generally poor performers, there clearly are additive effects that may either help or detract from their performance without additives. The important point here is that the cracking performance of a given asphalt may be significantly altered in a beneficial manner by the proper choice of additive. Unfortunately, the rutting results (Table 5) are not so readily related to additive type because of differences in asphalt content and percentage of voids.

Physical Parameters

Values for various physical parameters normally measured by MDOH are given in Table 6. To evaluate possible correlations between the physical parameters (percentage asphalt, Marshall stability, Marshall flow, percentage voids, penetration, viscosity or ductility) and either rutting or cracking, simple linear regression analyses were performed. Correlations (r^2) for cracking versus these parameters were all less than 0.2 (for example, voids: $r^2 = 0.06$; percentage AC: $r^2 = 0.12$). Results were stronger for the correlation of rutting with the various parameters (Table 7):

- Rutting versus percentage voids: $r^2 = 0.54$,
- Percentage asphalt: $r^2 = 0.51$, and
- Ductility at 40°F: $r^2 = 0.25$.

It is interesting that the correlation of voids with rutting associates higher voids content with less rutting. This might not be surprising if the voids contents were quite small. However, the in-place voids in these sections range from 5.8 to 10.8 percent.

It is important to note that these correlations are derived from the data in Table 6 that were obtained from cores taken

TABLE 5 EFFECT OF ADDITIVES ON RUTTING

Refiner	Without Additive	With Lime	With Fly Ash	With Antistripping Agent
C	0.25[a] (6.2;8.9)[b]	0.22 (6.5;9.5)	0.33 (6.0;8.7)	0.38 (6.9;9.3)
D	0.16 (6.6;9.0)	0.27 (6.7;7.6)	0.16 (6.9;7.8)	0.33 (7.1;8.9)
B	0.56 (6.8;6.4)	0.22 (6.5;10.8)	0.52 (7.5;7.7)	0.34 (7.0;6.8)
A	0.62 (7.6;6.1)	0.39 (7.2;8.5)	0.52 (7.2;6.2)	0.56 (8.0;5.8)

[a]Rut depth in wheelpath, inches.
[b](% asphalt; % voids).

TABLE 6 PHYSICAL TEST DATA (upper lift)

Test Section	Components (asphalt/ additive)	AC (%)		Additive (%)		Marshall Stability, Field	Marshall Flow, Field	Voids In-Place (%)	Penetration, 77°F (recovered)	Viscosity, 275°F (cSt)	Ductility, 40°F (cm)
		Design	In-Place	Design	Constructed						
1	C	6.6	6.2			1,637	8	8.9	60	437	9
2	C/lime	6.2	6.5	1.5	1.5	1,876	9	9.5	47	441	7
3	C/antistrip	6.6	6.9	1.0	1.0	1,952	8	9.3	50	440	7
4	C/fly ash	6.2	6.0	1.5	1.9	1,762	8	8.7	63	410	10
5	A/fly ash	6.3	7.2	1.5	1.8	1,801	8	6.2	77	344	26
6	A	6.8	7.6			1,419	8	6.1	69	368	17
7	A/antistrip	6.8	8.0	1.0	0.9	1,502	9	5.8	75	340	24
8	A/lime	6.6	7.2	1.5	1.6	1,967	9	8.5	54	421	8
9	D/lime	6.3	6.7	1.5	1.5	1,874	8	7.6	64	463	7
10	D	6.6	6.6			1,661	9	9.0	78	378	6
11	D/antistrip	6.5	7.1	1.0	0.9	1,432	9	8.9	54	409	6
12	D/fly ash	6.3	6.9	1.5	1.9	1,727	8	7.8	54	437	6
13	Blend	6.5	6.8			1,638	7	8.5	89	263	39
14	B/fly ash	6.2	7.5	1.5	1.8	1,763	8	7.7	62	318	10
15	B	6.8	6.8			1,814	9	6.4	59	361	7
16	B/lime	6.2	6.5	1.5	1.6	1,602	9	10.8	61	328	8
17	B/antistrip	6.8	7.0	1.0	0.9	1,551	9	6.8	84	249	28
18	85/100	6.5	6.4			1,777	8	8.9	52	396	7
20	200-300 Microfil 8	6.2	6.9	1.0	0.9	1,576	8	9.5	57	317	7

from the pavement immediately after construction. The Marshall stability and flow data are from on-site testing, not from in-place cores. An earlier report by Bruce (*3*) contained a similar set of data that, in contrast, were taken from on-site testing (Marshall stability, flow, voids) and plant operation (percentage AC). The percentages of asphalt and voids from the on-site testing trailer did not correlate with the rutting measurements (percentage voids: $r^2 = 0.31$, percentage AC: $r^2 = 0.25$).

Penetration-Viscosity

Table 8 gives the penetration-viscosity numbers (PVNs) of both the original asphalts and the asphalts recovered from postconstruction cores along with the number of cracks and depth of ruts in the corresponding sections with no additives. Original asphalts with PVNs from −0.64 to −0.72 have shown both poor and excellent crack and rut resistance. Asphalt B with a PVN of −0.93 is both severely cracked and rutted. Unfortunately no PVN is available for the original blended asphalt, but similar blends made more recently have shown PVNs of approximately −0.9.

TABLE 7 RUTTING MEASUREMENTS, ASPHALT AND VOIDS

Rut Depth[a] (in.)	Asphalt[b] (%)	Voids[b] (%)	Section[c]
0.16	6.6	9.0	10
0.16	6.9	7.8	12
0.22	6.5	9.5	2
0.22	6.5	10.8	16
0.24	6.2	9.8	20
0.25	6.2	8.9	1
0.25	6.4	8.9	18
0.27	6.7	7.6	9
0.33	6.0	8.7	4
0.33	7.1	8.9	11
0.34	7.0	6.8	17
0.38	6.9	9.3	3
0.39	7.2	8.5	8
0.43	6.8	8.5	13
0.52	7.2	6.2	5
0.52	7.5	7.7	14
0.56	8.0	5.8	7
0.56	6.8	6.4	15
0.62	7.6	6.1	6

[a]Measurements taken 3/87.
[b]From cores of initial pavement.
[c]See Table 2, 3, or 6 for identification of sections.

TABLE 8 RELATIONSHIP OF PVN AND PERFORMANCE

PVN				
Original	Recovered	Asphalt	Cracks[a]	Ruts[a]
−0.65	−0.5	D	19	0.16
−0.71	−0.65	C	1	0.25
−0.72	−0.74	A	18	0.62
−0.93	−1.06	B	23	0.56
—[b]	−0.99	B	0	0.43

[a]In sections with no additive.
[b]Not available.

If the PVN of recovered asphalts is considered, there is again no relational trend with cracking, but there is a trend toward increased rutting with increased temperature susceptibility. The apparent influence of asphalt content on rutting cannot be ignored, however. Thus, in this small data set, PVN is not a good predictor of cracking but shows some trends with regard to rutting.

Corbett Analyses

Corbett fractionations were conducted on the original asphalts. The fraction percentages are given in Table 9 in relation to the number of cracks and depth of ruts (inches) in the corresponding test section without additives. In this small sampling, higher resistance to cracking in C may be associated with relatively lower values for naphthene aromatics and higher values for asphaltenes and saturates. Likewise, better rutting resistance of C and D may be associated with relatively higher amounts of asphaltenes. As pointed out earlier, the percentage of asphalt has a fairly strong and perhaps overriding influence on rutting relationships.

TABLE 9 RELATION OF CORBETT FRACTION TO PERFORMANCES

	Original Asphalt				
	ASP (%)	PA (%)	NA (%)	SAT (%)	Asphalt
Cracks					
1	18	28	34	20	C
18	12	29	46	13	A
19	16	25	42	17	D
23	11	27	46	16	B
Rutting (in.)					
0.16	16	25	42	17	D
0.25	18	28	34	20	C
0.56	11	27	46	16	B
0.62	12	29	46	13	A

NOTE: ASP = asphaltenes, PA = polar aromatics, NA = naphthene aromatics, and SAT = saturates.

Asphalt Source

Another major variable to be considered in these experimental sections was asphalt source. In Figure 1, the depth of ruts in inches is plotted against the components of the test section. The sections containing 120-150 AC from each refiner are grouped together. These plots show the variability in rutting resistance associated with the additives. Beyond that, however, it may be generalized that relative resistance to rutting increases in the order A < B << C < D. (Note that C and D are also associated with higher asphaltene content, as mentioned previously, and that the relationships are complicated by the influence of asphalt content.)

Included in this plot are the sections containing the blend, the 85-100 grade AC, and the 200-300 pen section with Microfil 8. The blend, which was designed to resist cracking, is not particularly resistant to rutting. The 85-100 asphalt is showing good resistance to rutting. However, it is noteworthy that its performance in this regard is no better than that of some of the sections constructed with 120-150 asphalt or even with

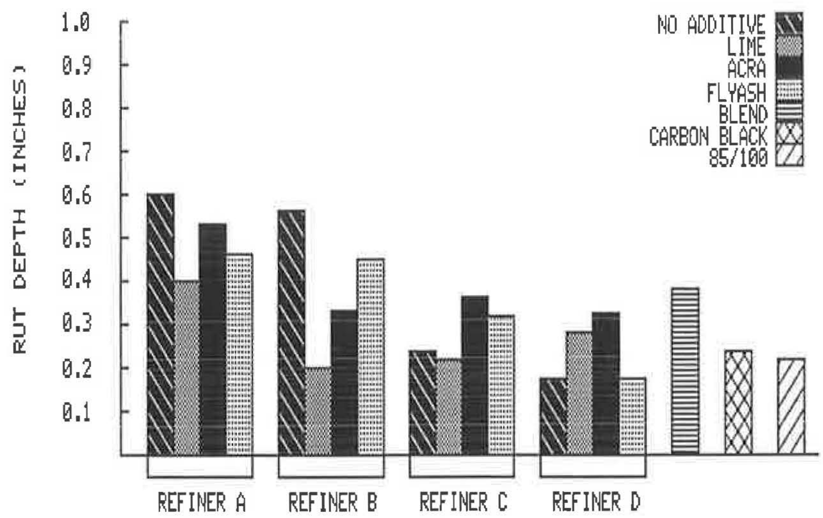

FIGURE 1 Rutting in test section pavements.

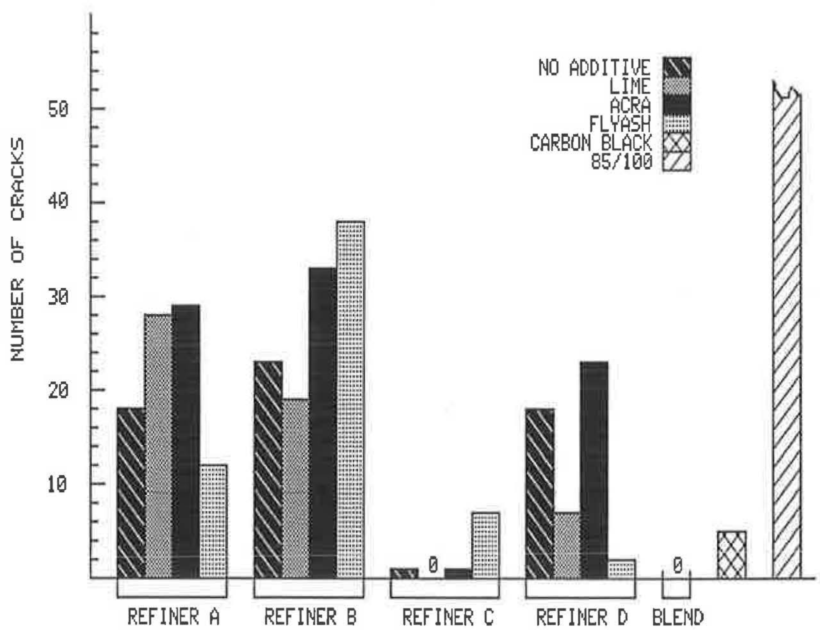

FIGURE 2 Cracking in test section pavements.

200-300 asphalt and Microfil 8. Moreover, its cracking performance is much worse than that of any of the other sections.

A similar plot of total number of transverse cracks versus components of the test section is shown in Figure 2. Again, the variability apparently introduced by additives is shown. In general, it may be seen that the resistance to cracking of the asphalts increases in the order B < A < D << C (C contains relatively lower concentrations of naphthene aromatics, as noted previously). The blend is resisting cracking, as expected, whereas severe cracking is evident in the 85-100 section. It should be noted that the source of the 85-100 and 200-300 grade asphalts is Refiner B and that the blend is composed of 75 percent B and 25 percent D, both of which are materials that tend to crack when used alone.

HP-GPC Analyses

Earlier work indicated that the ability of an asphalt to resist transverse cracking is related to its molecular size distribution as determined by HP-GPC (*1, 2*). More specifically, it has been shown that an excess of large molecular size (LMS) material above an optimum amount for a given climatic zone is associated with cracking. At the time of construction of the test sections, it was proposed that a virgin asphalt containing about 16 percent LMS would be likely to resist cracking in Montana. Indeed, the blend was designed to meet that criterion. In Figure 3, chromatograms of the virgin 120-150 asphalts and of the blend are compared. With the exception of Asphalt B, the theory that an excess of LMS material over the optimum amount is associated with cracking is supported.

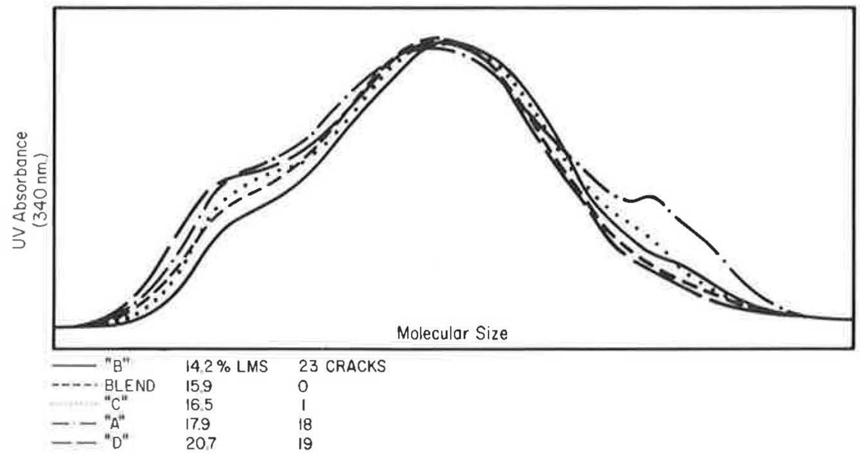

FIGURE 3 HP-GPC chromatograms for original asphalts.

Asphalt B is one of a small number of asphalts, found around the country by this research group, that are characterized by relatively high temperature susceptibility, relatively low asphaltene content, relatively low LMS content, and a tendency to crack severely quite early in their service lives. These particular asphalts are also products of propane deasphalting units. Thus interpretation of results with these asphalts should be guarded, especially regarding cracking.

Asphalt A, which shows a shoulder in the small molecular size (SMS) region of the chromatogram, can be seen from the traces to be rather similar to D in the LMS region. However, the LMS percentage is skewed to a lower relative amount by the presence of the SMS shoulder. This asphalt is also probably a product of propane deasphalting, and the shoulder apparently represents the blending stock. Its exact nature has not been determined. Both A and B are subject to early, severe cracking.

CONCLUSIONS

Use of the physical and chemical data available at the time of construction to predict performance after 4 years of service reveals that

1. Penetration grade may help to predict the relative performance of products from one refinery or crude source.
2. The effect of additives is both asphalt and additive dependent. The underlying chemical cause of this dependency is unknown. However, it may be extremely useful.
3. There is no apparent correlation of performance with Marshall stability, flow, penetration, and viscosity.
4. Although little correlation with ductility at 40°F is evident, asphalt content and voids content show good correlation with rutting. There is no correlation with cracking.
5. High PVN characterizes one of two asphalts that show early and severe cracking and one of three that have cracked to date, as well as one of two asphalts that have shown serious rutting tendencies.
6. Lower asphaltenes content from the Corbett separation characterizes the two asphalts that are rutting most severely. Resistance to cracking is associated with relatively lower concentrations of naphthene aromatics.
7. Within the 120-150 grade, both rutting and cracking differ with source. Performance is modified by additives.
8. Asphalts agreeing with the HP-GPC model have few if any cracks to date; those with more LMS than the model are demonstrating a tendency to crack and correlate well if Asphalt B, a known exception, is removed from consideration. Molecular size distributions vary with asphalt source.

ACKNOWLEDGMENTS

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Experiences with the Corbett-Swarbrick Procedure for Separation of Asphalt into Four Generic Fractions

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The Corbett-Swarbrick procedure for separation of asphalt into four generic fractions is evaluated. This procedure, currently accepted as an ASTM standard (ASTM D 4124-82), has been submitted to the ASTM committee for revision. The revised procedure involves considerable modifications to the existing standard. Oregon State University and the Oregon State Highway Division have implemented both procedures (the current ASTM standard and the revised procedure) and routinely used them in an ongoing research program. A number of difficulties have been experienced with both procedures. Some of these difficulties are related to interpretation of the standard, and some were due to the inexperience of the research team with the test procedures. The purpose of this paper is to present some of the ways the research team solved the major difficulties encountered with the implementation of the test. Several aspects of the test procedure are analyzed: method used for asphaltene precipitation, filtration, solvent concentration, and some problems related to the use of alumina and the chromatographic column.

Oregon State University (OSU) and the Oregon State Highway Division (OSHD) are involved in an ongoing program to monitor environmental effects on asphalt pavements. This involves the use of routine test procedures for asphalts and mixtures, laboratory aging procedures, and the implementation of a chemical test for asphalt fractionation. The Corbett-Swarbrick method of separating asphalt into four generic fractions was selected for implementation by both OSU and OSHD. Initially, the procedure documented in ASTM 4124-82, Separation of Asphalt into Four Fractions, was used. This procedure is referred to as Method A.

With the repetition of many tests, it was found that, even with improvements made to expedite the procedure, the test was still lengthy (2 days' work per test), expensive to run (Table 1), and relatively hazardous because of the large amount of solvents handled.

The standard test (ASTM D 4124-82) has been submitted for revision to ASTM Committee D04.47. The revised procedure involves considerable modifications to the existing standard. Although this new procedure (referred to as ASTM D 4124, Method B throughout this paper) is not yet a standard, it was decided to adopt it instead of the current standard procedure (Method A) originally described in ASTM D 4124-82.

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TABLE 1 COMPARISON OF METHOD A AND METHOD B

Item	Method A	Method B
Column length	1000 mm	500 mm
Column volume	754 cm^3	200 cm^3
Material cost, 1985 (alumina plus solvents)	\$40.00	\$15.00
Number of tests a day (one person)	0.5 test/day	2 tests/day
Other relative savings		Energy Nitrogen Laboratory space Asphalt sample

The implementation of both the present standard and the new short procedure created a number of difficulties. Some of these difficulties were related to interpretation of the standard, and some were due to the inexperience of the research team with such test procedures.

It was found by personal communications that a number of laboratories that have implemented the test procedure have had difficulties similar to those experienced by the OSU-OSHD team. Further, it appears that most researchers have deviated from the standard procedure and adopted various techniques to yield the required fractions.

The purpose of this paper is to present the major problems encountered by the OSU-OSHD team with the test and present some solutions that were developed.

CORBETT-SWARBRICK PROCEDURE

Currently accepted as ASTM D 4124, this procedure is essentially a selective adsorption-desorption column chromatographic technique (*1*) as shown in Figure 1. The asphaltenes are first separated from the other fractions by precipitation in a nonpolar paraffinic solvent (*n*-heptane). This removes the most polar and least soluble asphalt components so that further separation is possible of the remaining fraction known as petrolenes (maltenes). The remaining petrolene fraction is then adsorbed on a chromatographic column (alumina is used as the adsorbent phase) and sequentially desorbed with solvents of increasing polarity. The three fractions obtained from the petrolenes are saturates, naphthene aromatics (*n*-aromatics), and polar aromatics (*p*-aromatics).

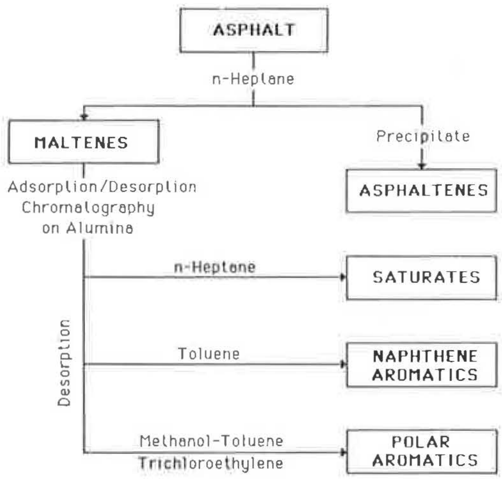

FIGURE 1 Corbett-Swarbrick scheme.

Some important aspects of the test may be summarized as follows:

- The fractionation scheme separates asphalt into less complex and more homogeneous fractions. The generic fractions are still complex mixtures of molecular groups, not well-defined chemical species.
- The precipitated asphaltene fraction in this method differs from the asphaltene fractions given by other methods because the precipitating solvent is *n*-heptane whereas other procedures use *n*-hexane and *n*-pentane.
- The method, which has been used in several research projects (*1–3*), presents one important advantage: it is considered nondestructive, and further separation or analysis can be done with the remaining fraction.
- The method is lengthy, as are most of the chemical composition analyses available for asphalt materials. Method A of this procedure, which is the present ASTM standard, is particularly lengthy; it takes approximately 2 days to complete one test. The short procedure (Method B) is relatively quick compared with the standard procedure (Method A); an experienced technician can usually perform two tests per day.

The problems encountered with the small column (Method B) are discussed in this paper. Method A, which uses a 1000-mm column, is essentially a large-scale test compared with Method B, which uses a 500-mm column. Thus, the problems could be considered similar in both cases.

The significant changes between Methods A and B are summarized in Table 1. The values shown represent experience at the ODOT and OSHD laboratories when both methods were used routinely.

IMPLEMENTATION

Asphaltene Precipitation

Four factors are to be considered during asphaltene precipitation for more uniformity in the standard procedure (*4*):

- Solvent concentration: Speight et al. (*5*) recommend that the asphalt/paraffin concentration be greater than 30 mL of solvent per gram of asphalt. The standard procedure uses 100 mL/g. This concentration was found to be satisfactory because it permits better stirring, and, provided this concentration is always used, no variations in asphaltene precipitated will occur.
- Stirring time: Speight et al. (*5*) recommend that this be greater than 8 hr. The standard recommends 30 min. The stirring time of 30 min has also been found to be insufficient for aged, recovered, and blown asphalts (*5*). The stirring device used could also influence the total amount of time required for asphaltene precipitation. For 2 to 3 g of asphalt (amount required in Method B) at the concentration of 100 mL/g, 2 to 3 hr of stirring with an air-powered device has been found to be sufficient for all asphalts used to date by the authors.
- Time of contact between asphalt and solvent: This includes stirring time plus settling time and should not be longer than 20 hr (*5*). If asphaltene precipitation is perfectly achieved during stirring, the authors believe that overnight settling should not be a requirement. Instead, other filtering devices could be used so that a quick separation of asphaltenes could be made as soon as precipitation is finished. The filtering apparatus used by the authors will be outlined in the next section.
- Temperature during precipitation: Use room temperature (*5*). The standard procedure recommends warming the asphalt in the flask before pouring the precipitating solvent. Also, the standard specifies that during stirring the solvent should be kept at a temperature near its boiling point (approximately 90°C). No heat application is recommended by the authors because of the direct effect that this has on the final asphaltene portion. It has been observed that warming the flask before and during stirring causes the amount of asphaltenes sticking to the glass to increase considerably. The asphaltenes sticking to the glass are not removable with *n*-heptane. However, asphaltenes adhering to the glass may be dissolved in toluene and recovered by solvent removal to improve the repeatability of the results.

Filtration Procedure

The filtration procedure described here does not correspond to the one given in the standard. The method described here is cheaper and more rapidly performed. Also, it has been observed that it yields the same proportion of asphaltenes.

The proposed procedure uses at least two filtration phases. The first phase is intended to collect the bulk of the precipitated asphaltenes immediately after stirring so that chances for the

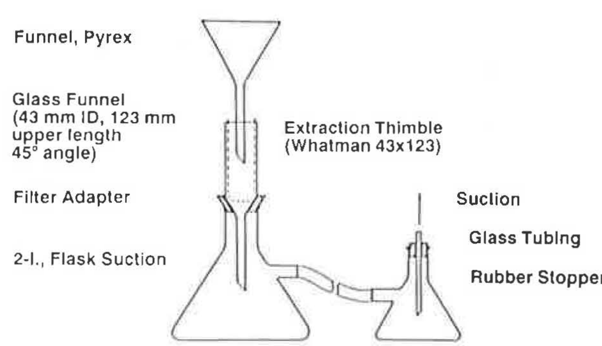

FIGURE 2 Filtration, first phase (slow procedure).

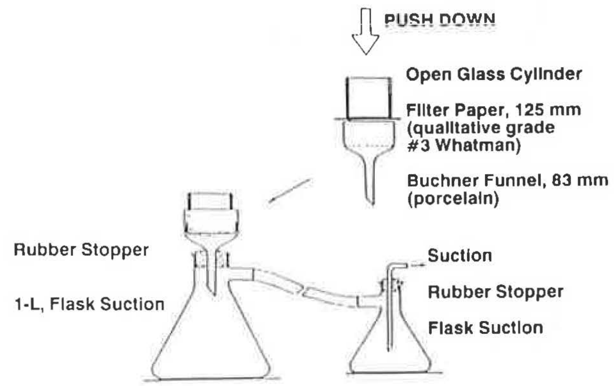

FIGURE 3 Filtration, first phase (quick procedure).

asphaltenes to stick to the glass are reduced. Also, there is no need to wait 12 hr for the settlement of the asphaltenes. Figures 2 and 3 show alternate first phase filtration methods. The second phase follows exactly the filtration procedure described in the original standard ASTM D 4124-82 (Figure 4). These procedures are described in more detail elsewhere (*4*).

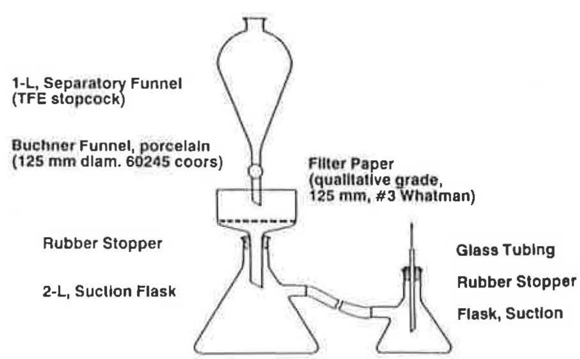

FIGURE 4 Filtration, second phase.

Removal of Residual Solvent from Asphaltene Fraction

The standard procedure calls for the use of an oven temperature of 104°C to remove the solvent from the recovered asphaltene fraction. It has been observed that asphaltenes have very unstable weight if heated at higher temperatures (above 60°C) or heated for longer periods. The authors recommend using 60°C until constant mass is achieved. However, through personal contact with H. Plancher at the Western Research Institute, Laramie, Wyoming, it was discovered that the instability of asphaltene weight is due to occluded resins, which may be removed by further washing of the asphaltenes with *n*-heptane.

Column and Alumina

Problems were encountered with the alumina used as the adsorption-desorption material in the chromatographic column. Alumina is specified in the standard as "F-20 chromatographic grade calcined at 413°C for 16 hours." The standard suggests that the alumina may be obtained directly from the manufacturer. However, it was found that the quality of the alumina varies among manufacturers and within the production lots of one manufacturer.

Table 2 gives results of a number of tests performed with one asphalt but with alumina samples of two different manufacturers (Manufacturers X and Y). Further, alumina samples from Manufacturer X were obtained from three different lot productions. The results given in Table 2 indicate that the alumina as it is received from the supplier does not comply with the specifications and has different adsorptive capacity. It should be noted that the alumina was not calcined before any of the tests the results of which are given in Table 2.

TABLE 2 COMPARISON OF ALUMINA OBTAINED FROM DIFFERENT MANUFACTURERS AND DIFFERENT LOT PRODUCTIONS

	Manufacturer			
	X			
	Lot Number			
Fraction	X1[a] (%)	X2[b] (%)	X3[c] (%)	Y[d] (%)
Asphaltenes	15.21	15.53	15.67	15.08
Saturates	23.24	15.84	10.65	12.77
Naphthene aromatics	45.10	44.02	38.21	40.91
Polar aromatics	15.61	22.85	35.08	29.91
Total	99.34	98.24	99.11	98.67

[a]Average of six tests.
[b]Average of three tests.
[c]Average of two tests.
[d]One test.

The explanation of this problem, which caused considerable delay in the test program, was that the adsorptive capacity of alumina is a function of moisture content, size, and surface area (*4*; *6*; *7*, p. 57). Size and surface area are controlled basically by the selection of an 80-200 mesh alumina. Moisture content is controlled by calcining the material at 413°C for 16 hr.

Although alumina is calcined before being packed in sealed bottles, the packing procedure probably is not carried out under vacuum conditions. Thus, during transportation and storage, the material can adsorb various amounts of water. The effect of this phenomenon is variable polarity in the alumina and erratic adsorption behavior. It is also possible that variations in particle size distribution (within specification) and impurities in alumina from different sources will influence the results of the tests.

The solution to this problem was to re-treat the alumina according to the specification given in the standard (413°C for 16 hr) and store it in a vacuum desiccator. Unfortunately, the standard is not clear in specifying such treatment as essential, which may cause other researchers to have the same problem.

The alumina from Manufacturers X and Y was re-treated and tested using another asphalt. The results, given in Table 3, indicate that there are no major variations among alumina from different sources if they are recalcined before the test is performed.

TABLE 3 COMPARISON OF ALUMINA OBTAINED FROM DIFFERENT MANUFACTURERS BEFORE AND AFTER RE-TREATMENT

	Manufacturer			
	X		Y	
	Re-treatment[a]		Re-treatment[b]	
Fraction	No (%)	Yes (%)	No (%)	Yes (%)
Asphaltenes	13.83	13.73	13.47	13.72
Saturates	16.76	11.35	14.14	11.26
Naphthene aromatics	44.42	24.65	36.29	24.24
Polar aromatics	23.01	49.30	36.05	49.07
Total	98.02	99.30	99.95	98.29

[a]Average of two tests.
[b]One test.

Another problem encountered was the filling of the column with alumina. The dry-pack method was preferred by the OSU-ODOT team (*8*, pp. 560–568) and found to be easily accomplished. Use of vibration during packing has a substantial effect on ease of packing.

Cutting Point and Elution Time

The total elution time was reduced by about one-third in this study by applying continuous vacuum to the column after recovering the saturates and the *n*-aromatic fractions at the specified elution rate of 5 mL/min. The procedure is briefly explained as follows: "After collecting the second fraction (*n*-aromatics), load the column with the last solvent, open widely valves A and B [Figure 5], and close valve C. Apply continuous vacuum to the column until trichloroethylene is recovered."

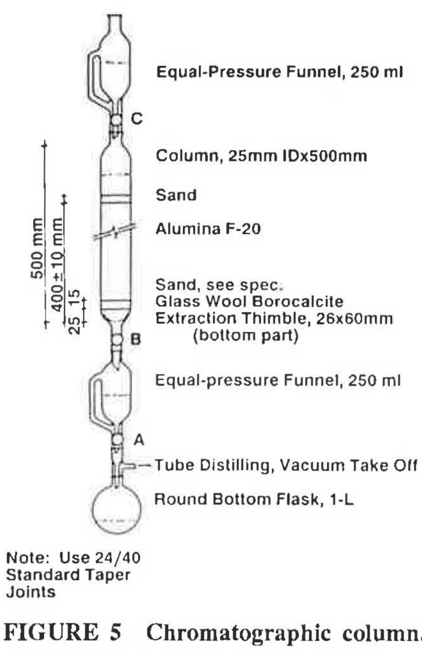

FIGURE 5 Chromatographic column.

The use of vacuum to extract the last fraction is also advantageous because it has been found (*4*) that all losses of the Corbett-Swarbrick procedure come from the *p*-aromatics that are retained in the alumina after elution in the column. The vacuum procedure appears to reduce these losses.

Fraction Concentration

The solution concentration of the petrolenes and their subsequent fractions is done most quickly and efficiently with the use of a rotovapor plus nitrogen. The standard procedure should be more explicit about concentration requirements because the use of a rotovapor presents considerable advantages in this test procedure:

- Materials are concentrated in a shorter time,
- There is less chance of overheating the concentrated solution, and
- There is no air pollution at all.

SUMMARY

Described in this paper are experiences in implementing the Corbett-Swarbrick procedure, currently ASTM standard method D 4124-82, for the fractionation of asphalt. A proposed procedure (Method B) was compared with the current standard method (Method A) and found to offer several advantages (Table 1), especially a decrease in testing time and a decrease in the cost per test.

The ASTM D 4124-82 procedure leaves room for users of the Corbett-Swarbrick method to deviate from the standard procedure, particularly those users who may be inexperienced with this type of test. It should be recognized that many state highway agencies, which may wish to adopt this test, may have experience only with the more common physical tests used for asphalt.

Some of the problems reported in this paper are related to the interpretation of the standard. However, some were due to the inexperience of the research team. One such problem was their failure to recognize that the alumina used in the chromatographic column must be recalcined before it is used, even if its container appears to be perfectly sealed. It is recommended that the standard include in the body of the text a statement regarding the need to recalcine the alumina before testing.

Other problems and recommendations for their solutions are summarized next in the hope that others will benefit from the experience gained by the authors.

For the asphaltene precipitation it is best not to warm the asphalt sample at any stage of the procedure; eliminate the warming of the flask before the test and eliminate the vapor bath to keep the solvent near its boiling point.

A quicker and simpler method is proposed that allows the filtration of the asphaltenes to be done immediately after asphaltene precipitation. This filtering procedure eliminates the necessity of settling the asphaltenes for longer periods, which, in the authors' experience, is not necessary if precipitation of the asphaltenes has been completely achieved during the stirring process. The other advantages of this method are fewer chances of having asphaltene losses, fewer chances of clogging the filtering device, and less time to perform the test.

When using either the large column (Method A) or the small column (Method B), one-third of the total time spent distilling the three fractions contained in the petrolenes could be reduced by applying vacuum to the chromatographic column after recovering the saturates and *n*-aromatics. This will also reduce losses of the *p*-aromatic fraction.

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Chemical Characterization of Asphalt Cement and Performance-Related Properties

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Gel permeation chromatography (GPC), the Corbett analysis procedure (ASTM D 4124-82), and the Heithaus test for component compatibility have been used to characterize asphalts and to investigate relationships among these chemical properties and physical properties and performance. The asphalts used in the present study were periodic samples of virgin asphalt from various suppliers in Texas, test pavement asphalts from three locations in Texas, and a group of 12 asphalts that had been rated according to pavement tenderness. GPC chromatograms, using toluene or tetrahydrofuran (THF) as the carrier solvent, show significant differences from one supplier to the next. Differences in chromatograms for a single supplier over a period of time may be quite significant. Furthermore, the differences between toluene and THF chromatograms are considerable and can likely be used as supplementary analyses. Finally, differences in viscosity grades (for a given refiner at a given time) frequently may be detected as differences in the shapes of the GPC chromatograms. For virgin asphalts from the Texas test sections, viscosity temperature susceptibility was found to correlate quite well with the molecular size fractions of the GPC chromatograms using either toluene or THF as a carrier solvent. Studies of pavement tenderness showed a relationship between component compatibility and tenderness and between the GPC large molecular size fractions and tenderness. In addition, asphaltene content was found to relate to component compatibility and to tenderness. Finally, as a cross correlation, the large molecular size region of the GPC chromatograms is directly related to the asphaltene content of asphalts with the exception of the Texas Diamond Shamrock asphalt.

Ultimately, the physical performance and durability of any material are determined at the molecular level. For pure component materials, molecular packing, structure, and interactions determine physical properties. In multicomponent materials these features are important, and interactions between differing molecular species play a significant role as well. These interactions can cause not just crystallinity or packing differences but also cohesive energy and compatibility effects manifested in phase behavior. Consequently, if the chemical nature and composition of a material can be adequately described, it ought to be possible to predict its physical properties, at least in principle.

One technique that has been used in numerous asphalt studies, and with some significant success, is gel permeation chromatography (GPC), also referred to as high-pressure GPC and size exclusion chromatography. This technique is used to separate the constituents of an asphalt, for the most part, on the basis of molecular size. Superimposed on the size exclusion separation, however, are solvent and gel interaction effects, so molecular functionality can play a significant role as well.

One of the earliest efforts at relating asphalt performance to GPC analyses was that by Bynum and Traxler (*1*). They determined GPC chromatograms on virgin asphalts and pavement cores from nine test sections in Texas. Rheological properties and vanadium concentrations were also determined. Considerable differences were noted among asphalts from different sources. Also the chromatograms obtained from asphalts from a given source often showed change with time. After 2 or 3 years, no consistent relation between molecular size and performance was evident, although the chromatograms of the poorer asphalts appeared to change more with time, and it was found that all of the inadequate asphalts contained a high percentage of vanadium.

More recently, Hattingh (*2*) used GPC and high-pressure liquid chromatography (HPLC) to study four South African road asphalts and reached conclusions on the large molecular size (LMS) fraction and its effect on tender mixes. Two roads were in good condition after 9 and 3 years, respectively, and two exhibited serious setting properties. The two "good" asphalts showed a much higher LMS fraction than the tender asphalts. Hattingh also used HPLC and a series of solvents to separate both whole asphalt and the maltenes into nine fractions following hexane precipitation of asphaltenes following extended thin-film oven tests (TFOT) on the asphalts used in the roads. He observed that the tender asphalts did not gain in large molecules with aging. This is the reverse of the situation with asphalts that tend to crack, and he thus concluded that too high an asphaltene or LMS percentage causes cracking and too low a percentage causes setting problems.

The most extensive studies of the use of GPC in predicting asphalt pavement performance have been those of Jennings and coworkers (*3–7*) in which they concluded that LMS can play a significant role in cracking. A study of 39 Montana roads of different ages built with asphalts of five penetration grades from all four of Montana's refiners led Jennings to conclude that cracking was related to the relative size of the LMS region of the GPC chromatograms (*3*). Conversely, he determined that low LMS is desirable to reduce cracking. He also concluded that asphaltene content as determined by heptane solubility was related in the same way to cracking.

Jennings and Pribanic (*6*) expanded the road study to include samples in 15 other states that have a wide range of climatic

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conditions. The nation was divided into zones of similar climate, and the condition of roads within each zone was compared on the basis of molecular size distribution. In general, in each zone there was an LMS level above which all of the roads were bad (again, with respect to cracking), and most of the good and excellent roads in each zone were those with a lower LMS. There was, however, a large difference between the LMS percentage that could be tolerated in warm zones and in very cold zones. Interestingly, several roads in Georgia with low LMS were exhibiting rutting.

Variation of GPC chromatograms with viscosity grades for the asphalts of Jennings's study of Montana highways was inconsistent. Some refineries showed identical GPC for the different viscosity grades; some showed consistent variation with grade; some showed random variation. Some old pavement exhibited GPC chromatograms that could not be derived from any of the asphalts now supplied in the state.

Adams and Holmgreen report GPC results for test pavements at three locations in Texas (*8*). There were ten asphalts and six or seven were used at each location. GPC chromatograms showed similarity in the common asphalts used at the different locations, and essentially no variation with viscosity grades, either AC-10 or 20, is reported. There was also no correlation found between the chromatograms of the virgin asphalts and tensile strength of the paving mixture.

In another study, Plummer and Zimmerman (*9*) evaluated asphalt cracking and its relation to molecular size distributions. Their study was based on 11 pavement samples in Michigan and 8 in Indiana. The Indiana results were more consistent than those of Michigan, but both showed more cracking with an increase in molecular size. Results from the Michigan roads showed that asphalts extracted immediately after construction were harder for the roads that eventually cracked (a mean penetration of 48 versus 62).

Still more recently, Zenewitz and Tran (*10*) reported a statistical evaluation of the pooled study of Jennings et al. that supports the earlier conclusion that cracking, LMS fraction, and climate are related. Although they observed no statistical correlation between LMS and cracking when all of the data were analyzed together, there was correlation within climatic zones. There was also correlation between LMS and climate. Colder climates tended to have asphalts that were lower in LMS than did warmer climates. This suggests that highway engineers and refiners have found that some asphalts work better than others in their region. In essence, it appears that a natural selection process has led highway districts to use the asphalts that have approximately the proper LMS range for the climate.

Although these studies highlight the efforts and successes at relating chemical properties of asphalt to physical properties and performance, they also point out that quantitative correlations of GPC chromatograms (as one example) with viscosity, cracking, or propensity for producing tender mixes have not been reported. Because of the complexity of asphalts and their variety of composition, such correlations will certainly be limited. Ultimately, it will take a number of chemical characterizations to accurately predict physical properties and performance.

Despite this inherent limitation, the objective of this work was to pursue quantitative relations among certain chemical characterizations of asphalts and their physical properties and performance. Specifically, this paper is a report on recent results on a number of aspects of GPC chromatograms and their relation to physical properties and performance of asphalt cement. GPC techniques involve using both toluene and tetrahydrofuran (THF) as carrier solvents. Reported are results on the variation of GPC chromatograms with supplier, with time, and with viscosity grade. Also included are results on the relation of chromatogram molecular size fractions to viscosity temperature susceptibility and to asphalt tenderness.

EXPERIMENTAL METHODS

The GPC technique has been reported previously (*11, 12*). The Heithaus peptization test was used for determining flocculation ratio (*13*). The Corbett analysis (ASTM D 4124-82) was used to obtain Corbett fractions; asphaltenes are reported in this paper (*14, 15*). Further details of these procedures are given as appropriate.

GPC Analysis

To prepare the asphalt samples for injection, the asphalt was weighed to 0.001-g accuracy and dissolved in distilled solvent measured to 0.1 mL to give a concentration of 7 percent by weight. The sample concentration was closely controlled because sample size has a definite impact on the results. After dissolution, the sample was filtered through a 0.45-micron filter to remove particles that might damage the column. Injecting a reference asphalt as a standard and known compounds, as described previously (*12*), was essential for monitoring column performance and obtaining reproducible data. Typical reproducibility that was obtained when proper procedures were followed is shown in Figure 1 for THF carrier solvent. Comparable reproducibility was obtained for toluene.

Two column configurations were used in this work according to the carrier solvent employed. When THF was used as a carrier, a 500-Å column followed by a 50-Å column was used. When toluene was used as the carrier solvent, only a 500-Å column was employed. The 50-Å column was omitted because of excessive fouling in the toluene system. The sample size injected in each case was 100 μL of the 7 percent by weight solution.

Heithaus Compatibility Test

The Heithaus peptization test (*13*) gives a measure of how well the asphaltenes are solubilized, or held in solution, by the resins and oils in the asphalts. The technique has been reported previously in the literature as a routine test procedure. In this work, an abbreviated version was used for determining the flocculation ratio. The flocculation ratio is determined from the amount of heptane required to initiate precipitation of asphaltenes in a toluene-asphalt solution. Starting with 1 g of asphalt and 1 mL of toluene, the flocculation ratio (FR) is calculated as

$$\mathrm{FR} = \text{Volume of toluene}/(\text{Volume of toluene} + \text{Volume of heptane})$$

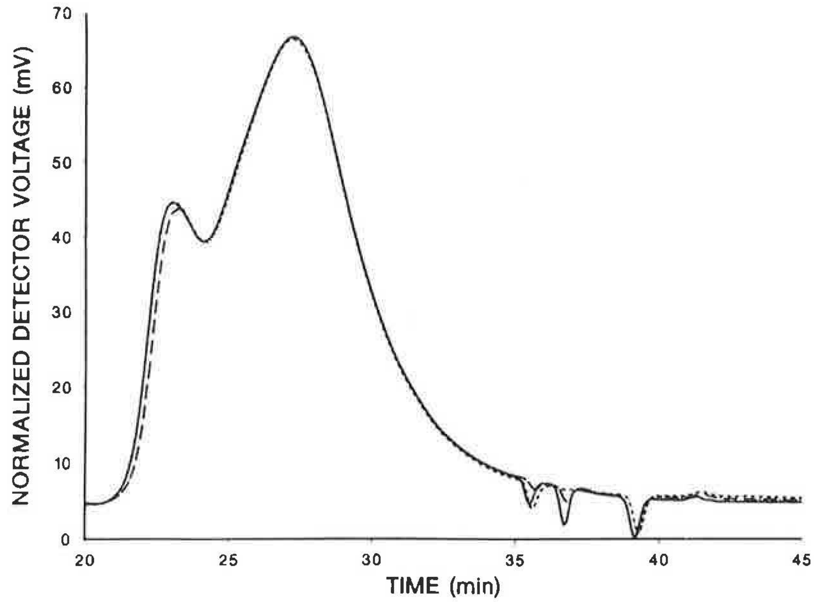

FIGURE 1 Reproducibility of three asphalt standard chromatograms in THF (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

Hence, the more heptane required to initiate precipitation (i.e., the better solubilized the asphaltenes are by the asphalt-toluene solution), the smaller the flocculation ratio, and a large flocculation ratio indicates poor compatibility.

RESULTS AND DISCUSSION

Chromatograms of Several Asphalt Suppliers

As has been the case in other studies, significant differences in the GPC chromatogram of asphalts provided by several suppliers were observed. Figure 2 shows chromatograms of virgin asphalts from six suppliers; five are AC-20 viscosity grades, and the other is an AC-10. These chromatograms are all taken using THF as the carrier solvent. In some respects, all of the chromatograms except that of the Diamond Shamrock asphalt are quite similar. First, all have an early peak or shoulder (an LMS fraction) eluting at about the same time. This is followed by a large, broad peak that again elutes at about the same time in each of these asphalts. The differences in the GPC chromatograms of these five asphalts are primarily differences in the relative sizes of these two features. The Cosden AC-20 has the smallest LMS region; it appears only as a shoulder on the broad major peak. For the other four similar asphalts, this region is better defined and somewhat better separated from the major peak. In terms of these features, these five asphalt chromatograms qualitatively are quite similar even though quantitatively they are readily distinguishable.

In contrast, the sixth asphalt, the Diamond Shamrock, is considerably different both qualitatively and quantitatively. Although it still exhibits an LMS peak and a broader later eluting peak, both of these differ from those of the other five

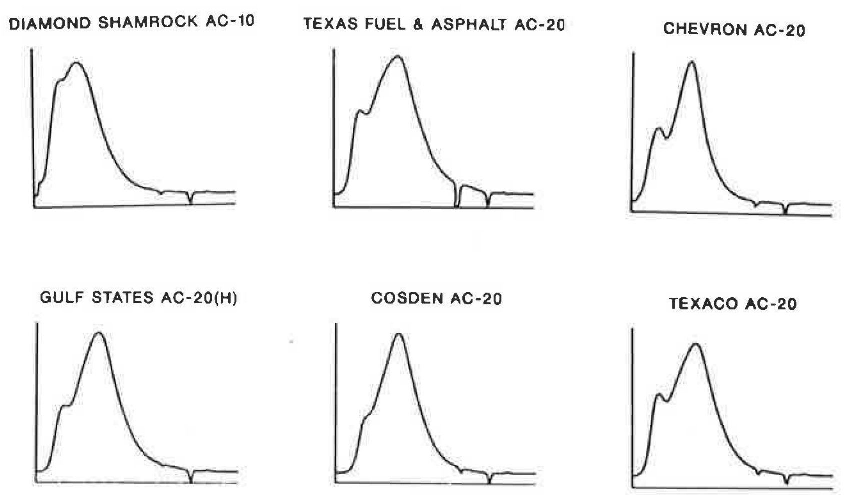

FIGURE 2 Representative chromatograms for several refiners in THF (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

asphalts in that each one elutes much earlier and, therefore, consists of, on average, larger molecular size material. Furthermore, the first peak is nearly as high as the second peak, and the overall spread of the Diamond Shamrock chromatogram is considerably narrower than that of the others. The Diamond Shamrock asphalt is different in other characteristics as well, and this makes it a unique and interesting asphalt for further study.

A comparison of several asphalts using toluene as the carrier solvent is shown in Figure 3. Again, striking differences among the different asphalts are seen to exist. As with THF, the average elution time for the Diamond Shamrock asphalt is less than it is for the others indicating, on average, a higher molecular size material. In toluene, however, it is not unusual to see a larger number of waves or peaks in the chromatogram than in THF. The Diamond Shamrock asphalt has three distinct peaks, and the MacMillan has four peaks or waves. The Cosden and the Exxon have fewer peaks (two and one, respectively). It may be that the toluene carrier solvent offers a better discriminating capability among various asphalts and, therefore, may have special value in asphalt analyses in spite of some problems that have been observed such as excessive fouling of the column or with desorption rates of the asphalt materials from the column (*12*).

Variability of Asphalt Supply with Time

Analyses of asphalts by GPC over a period of time have shown some significant differences in the chromatograms of single suppliers. Figures 4 and 5 show the variability of asphalts from

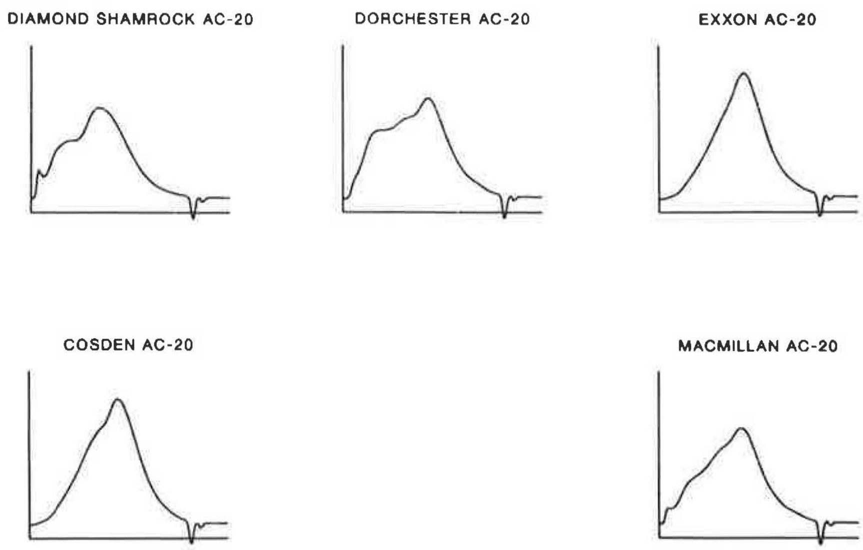

FIGURE 3 Representative chromatograms for several refiners in toluene (500-Å column; toluene: 100μL, 7 percent by weight).

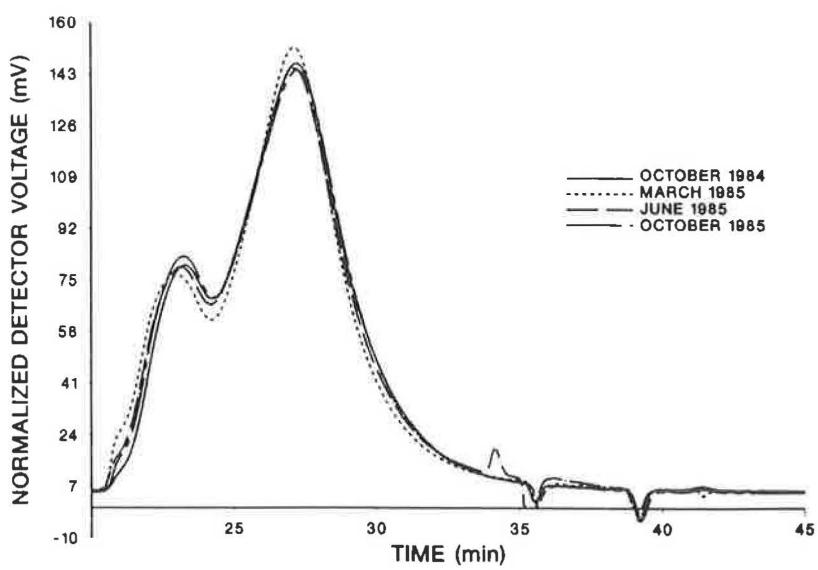

FIGURE 4 Variability of Chevron AC-20 over time (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

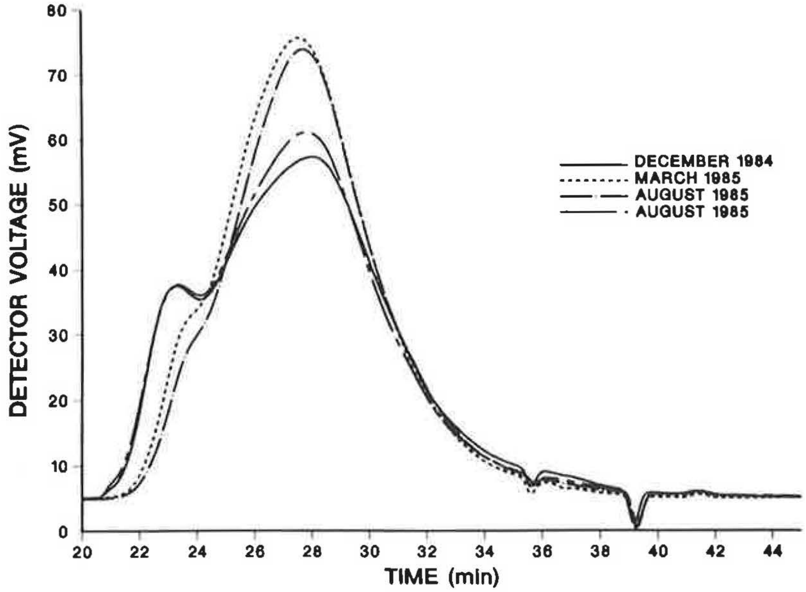

FIGURE 5 Variability of Gulf States AC-20, Houston over time (500-Å and 50-Å columns; THF: 100µL, 7 percent by weight).

two suppliers over a 1-year period. These are all AC-20 asphalts, and the chromatogram variability should be compared with the reproducibility shown in Figure 1. The Chevron asphalts, over this period of time, showed minimal variability whereas, in comparison, the Gulf States Houston exhibited some significant changes. It should be noted that the Gulf States Houston facility does not refine asphalts but is an intermediate supplier. Evidently, they had two different sources of asphalt during this time. Again, it should be pointed out that, in spite of these extreme differences shown by GPC, all four of these asphalts supplied by Gulf States in this figure, as well as those of Chevron in the preceding figure, are the same viscosity grade. Obviously, and not unexpectedly, asphalt viscosity is a highly nonunique function of composition. This provides considerable complication in relating different asphalt compositions to their physical properties and ultimately to their field performance.

GPC and Viscosity

As has been the case in previous research, inconsistent relations between GPC chromatograms and asphalt cement viscosity grade have been observed. In some cases, there are quite obvious and rational differences in the chromatograms for different grades; in others, the differences are minimal. In most cases, differences are fairly evident. Figure 6 shows chromatograms for virgin Witco AR 1000, 2000, and 4000 cements. The addition of a small molecular size cut to lower the viscosity is apparent. Figure 7 shows differences at the other (LMS) extreme for a Dorchester asphalt. A third asphalt (Texaco, Figure 8) shows minimal significant difference between AC-10 and 20 grades, whereas the AC-5 asphalt shows differences that exceed reproducibility. This last set of chromatograms emphasizes that, in spite of the obvious trends of the previous two examples, factors other than those detected by GPC, such as molecular interactions, play a significant role in establishing asphalt viscosity. This prevents a clean correlation between GPC chromatograms and viscosity.

GPC and Viscosity Temperature Susceptibility

In a previously reported study, ten asphalts from five suppliers were used at three different locations in Texas test pavements (*8*). For each test pavement, samples of virgin asphalts delivered to the construction site, hot mixes used in the test pavements, and 2- and 3-year cores from the test pavements were obtained. Standard viscosity tests were performed on the virgin asphalts as well as on asphalt extracted from the hot mixes and from the cores. The hot-mix samples were laboratory prepared and compacted. In addition, penetration was determined at 39.2°F and 77°F on the virgin asphalts, laboratory mixes, and cores. Data on the laboratory mixes and cores were obtained on asphalt extracted using trichloroethylene. The viscosities were determined at 77°F, 140°F, and 275°F. These data are reported by Adams and Holmgreen (*8*).

From these data, viscosity temperature susceptibility (VTS) using viscosities at 140°F and 275°F (*16*) and penetration index (PI) were calculated according to the equations.

$$VTS = [\log\log(100\eta_1) - \log\log(100\eta_2)]/(\log T_2 - \log T_1)$$

where η_1 and η_2 are the viscosities (poises) at temperatures T_1 and T_2 (kelvin), respectively, and

$$PI = (20 - 500A)/(1 + 50A)$$

where $A = [\log(\text{pen}_{T_1}) - \log(\text{pen}_{T_2})]/(T_1 - T_2)$ and pen_{T_1} and pen_{T_2} are the penetrations at the two temperatures.

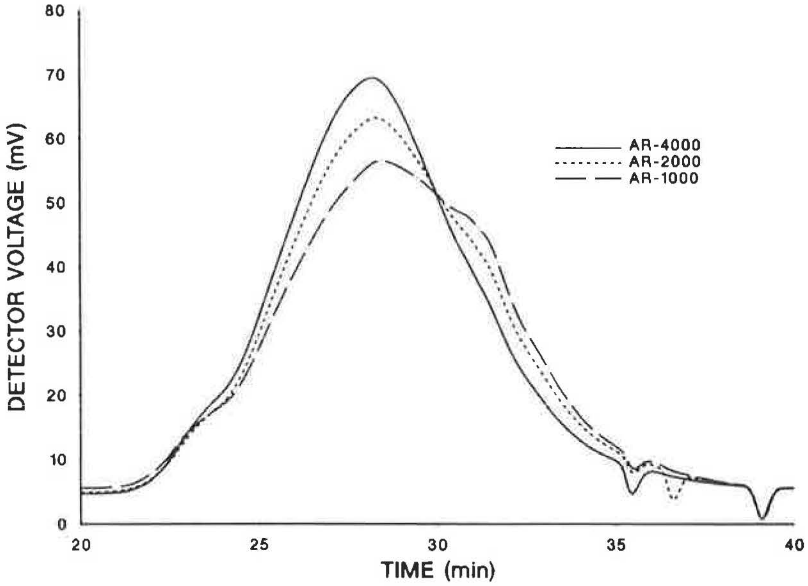

FIGURE 6 GPC comparison of three viscosity grades of a Witco asphalt (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

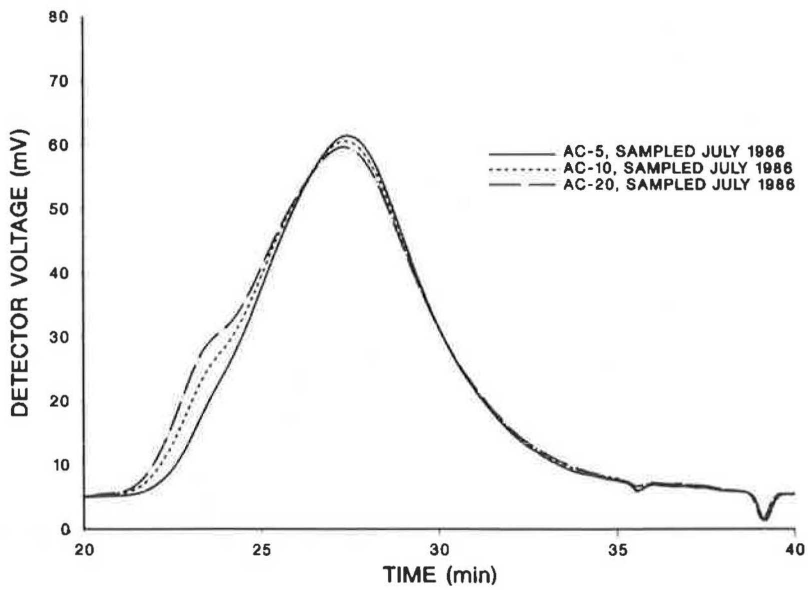

FIGURE 7 GPC comparison of viscosity grades of a Dorchester asphalt (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

For the purposes of correlating GPC chromatograms with these VTS and PI properties, the chromatograms have been divided into three equal time sections according to the procedures of Jennings and coworkers (*4, 5, 17*). For the THF analyses that used two columns in series, the chromatograms were divided into three 5-min sections between 20 and 35 min. For the toluene analyses, which used only a 500-Å column, the chromatograms were broken into three 4-min sections ranging from 10 to 22 min in retention time. Following Jennings, the terms large molecular size (LMS), medium molecular size (MMS), and small molecular size (SMS) are used for the three regions even though this interpretation is not exactly correct.

Using these calculated values, correlations of the viscosity temperature susceptibility and of the penetration index to the chromatogram molecular size distribution characteristics were checked. In each case, a multiple linear regression of the VTS or PI using two of the chromatogram areas as independent variables was attempted. Summaries of the fit parameters and correlation coefficients are given in Table 1. When all of the virgin, or tank, asphalt samples are lumped together, the THF GPC analyses correlate with viscosity temperature susceptibility with a correlation coefficient (r^2) of 0.88. This includes 18 asphalt samples. For the toluene analyses, using the 20 samples available in this data set, the correlation coefficient is

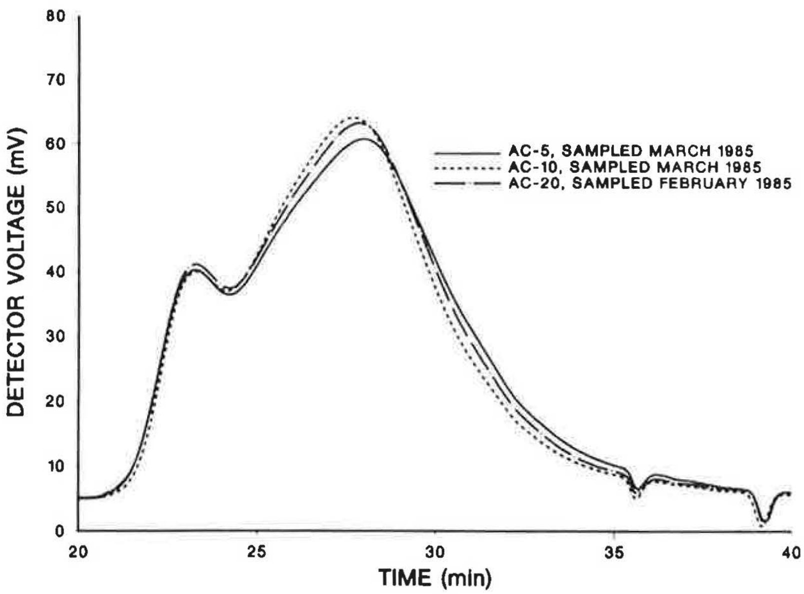

FIGURE 8 GPC comparison of three viscosity grades of a Texaco asphalt (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

TABLE 1 VTS CORRELATIONS OF DICKENS, DUMAS, AND LUFKIN TEST SITE ASPHALTS

Carrier Solvent	Data Set	*a*	*b*	*c*	r^2
THF	Tank—all grades (18 data points)	3.27	−0.00889	0.0275	0.88
	Tank—all AC-10 (8 data points)	2.48	0.00580	0.0543	0.95
	Tank—all AC-20 (10 data points)	3.73	−0.0174	0.0129	0.93
	Cores—all grades (25 data points)	0.854	0.0350	0.134	0.49
Toluene	Tank—all grades (20 data points)	2.97	−0.00763	0.0714	0.87
	Cores—all grades (17 data points)	2.56	0.00600	0.0930	0.49

NOTE: $VTS = a + b(LMS) + c(SMS)$.

essentially the same at 0.87. If all eight of the AC-10 asphalts are considered as a group, an r^2 of 0.95 is obtained, and if all of the AC-20 are grouped together (10 samples), the correlation coefficient is 0.93. Smaller subsets of the data probably are not warranted. Calculated VTS-values are shown versus experimental VTS-values for the THF analyses in Figure 9. A comparable plot was obtained for toluene analyses. Grouping the AC-10 and AC-20 asphalts is supported by the notion that VTS reflects changes in viscosity with temperature rather than the actual absolute values of viscosity for a given asphalt refiner or supplier. It may be reasonable to expect that both AC-10 and AC-20 grades will behave similarly with respect to these changes even though their viscosities may differ by a factor of 2. However, some improvement is observed when the AC-10 and the AC-20 asphalts are grouped separately.

Correlations of PI with LMS and SMS are not nearly as good as for VTS. For THF analyses a correlation coefficient of only 0.05 was obtained. For toluene the value was 0.59.

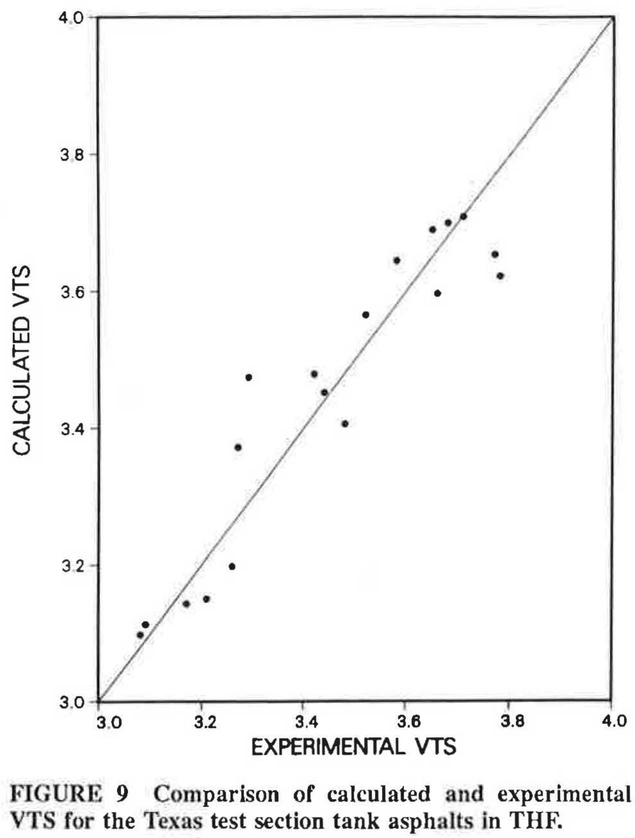

FIGURE 9 Comparison of calculated and experimental VTS for the Texas test section tank asphalts in THF.

No correlation of either VTS or PI with molecular size distribution was observed for the asphalts extracted from core samples. Using both THF and toluene analyses, the correlation coefficient for VTS for 25 cores was only 0.49. The correlation coefficients for PI, including all cores in the data set, were 0.12 for THF and 0.11 for toluene.

Asphalt Cement Tenderness and Component Compatibility

The word "peptize" means to cause to disperse in a medium or to bring into a colloidal solution. According to Heithaus (*13*), "Asphalts consist of a highly aromatic material (asphaltenes) of moderate molecular weight (ca. 2,000-1,000) dispersed or dissolved in a lower molecular weight medium (maltenes)." Considerable evidence indicates that the "apparent" high molecular weight often reported for asphaltenes is due to association of lower molecular weight molecules (*18*). Heithaus discussed the importance of the peptizability of the asphaltenes and of the peptizing power of the maltenes in determining the mutual solubility of these asphalt fractions. The state of peptization of the asphaltenes plays a major role in determining the rheological properties of an asphalt and the susceptibility of these properties to changes in asphalt composition and environmental factors. Heithaus suggests that in a neat asphalt there is probably no physical basis for a distinction between dissolved and dispersed phases. Altgelt and Harle (*19*) showed that the polarity of the so-called asphaltenes in petroleum asphalt greatly influenced their self-association and that the dispersibility of these asphaltenes in the asphalt was a dominant factor influencing asphalt viscosity.

The term "compatibility," as used herein, relates to mutual solubility of the molecular components in asphalt to produce homogeneity in the asphalt system and is thus a function of the state of peptization of the asphalt. By definition then, as highly associated molecular agglomerates in the asphalt become separated from their dispersing or solubilizing components, the system will have reduced component compatibility. An extreme example of poor compatibility might be a highly polar (heteroatom content or highly condensed aromatic ring systems) asphalt containing an excess of a low–molecular weight, nonpolar, paraffinic blending stock (*18*).

Heithaus (*13*) investigated asphaltene peptizability or degree of dispersion, the dispersion power of maltenes, and the state of peptization of asphalts. Little application of the state of peptization parameters applied by Heithaus appears in the literature. The California Highway Department, in extensive pavement performance studies (*20, 21*), indicates that the parameters used by Heithaus for the state of peptization of asphalt may be useful in predicting pavement durability. Researchers at the University of New Mexico (*22, 23*) have related Heithaus' results to compatibility of asphalt with modifiers. Good component compatibility is probably a necessary requirement for good pavement durability. Akzo Chemie America uses the test to measure emulsibility of asphalts (J. Dybalski, unpublished data).

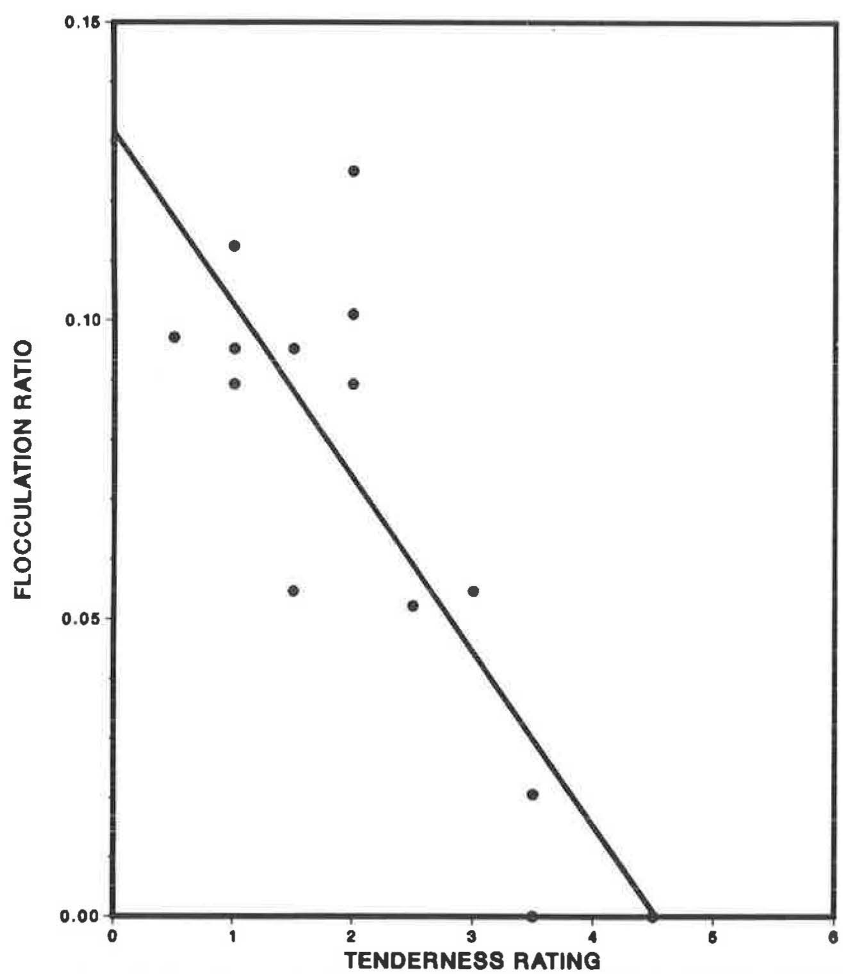

FIGURE 10 Virgin asphalt flocculation ratios and tenderness ratings.

A number of asphalts of various paving grades and from different sources were studied with respect to tenderness and possible correlation with chemical properties. In this study, pavement tenderness is based on user comments about the asphalt's general performance history with particular emphasis on construction and early pavement performance (*24*). The tenderness rating ranges from 0 to 5 with 0 indicating that tenderness or slow-setting problems are never associated with the asphalt and 5 indicating that these problems are always associated with the asphalt.

Figure 10 shows a reasonable correlation between the Heithaus peptization flocculation ratio and the subjective tenderness ratings. This correlation was expected because highly peptized asphalts had been previously associated with slow-setting mixtures (*25*).

GPC and Tenderness

It has been suggested previously that low values of the LMS fraction as determined by GPC are an indicator of tenderness (*2, 6*). The asphalts in Figure 10 have also been correlated using LMS, and the results are similar to those shown in Figure 10, except for one asphalt, a Diamond Shamrock AC-10, which is off the graph in LMS even though it sometimes produces tender mixtures.

Asphalt tenderness is compared with asphaltene content in Figure 11. In this plot, Diamond Shamrock (denoted by DS) fits quite well. With the exception of this one asphalt, all of the techniques plot about equally well with much of the scatter undoubtedly due to the subjectivity of the tenderness rating.

Basically, asphalts can be broken into two families with relation to tenderness. The tender asphalts are those with tenderness ratings above 2. These asphalts have low flocculation ratios, low LMS (with the exception of Diamond Shamrock asphalt), and low asphaltene content. The nontender asphalts have tenderness ratings of 2 or lower. Accordingly, they have high flocculation ratios, high LMS, and high asphaltene content. This implies that there are strong cross correlations between flocculation ratios, LMS, and asphaltene content. A plot showing the variation of LMS with asphaltene content is shown in Figure 12. With the exception of Diamond Shamrock, which is again off the graph, the correlation is very nearly within the accuracy of the data, so that, in most cases, when high LMS is found, it means high asphaltenes. Figure 13 shows the complete chromatograms of several asphalts spanning the range of tenderness rating. The presence of a significant amount of LMS materials in nontender asphalts and, conversely, the absence of a significant LMS peak for tender asphalts are evident.

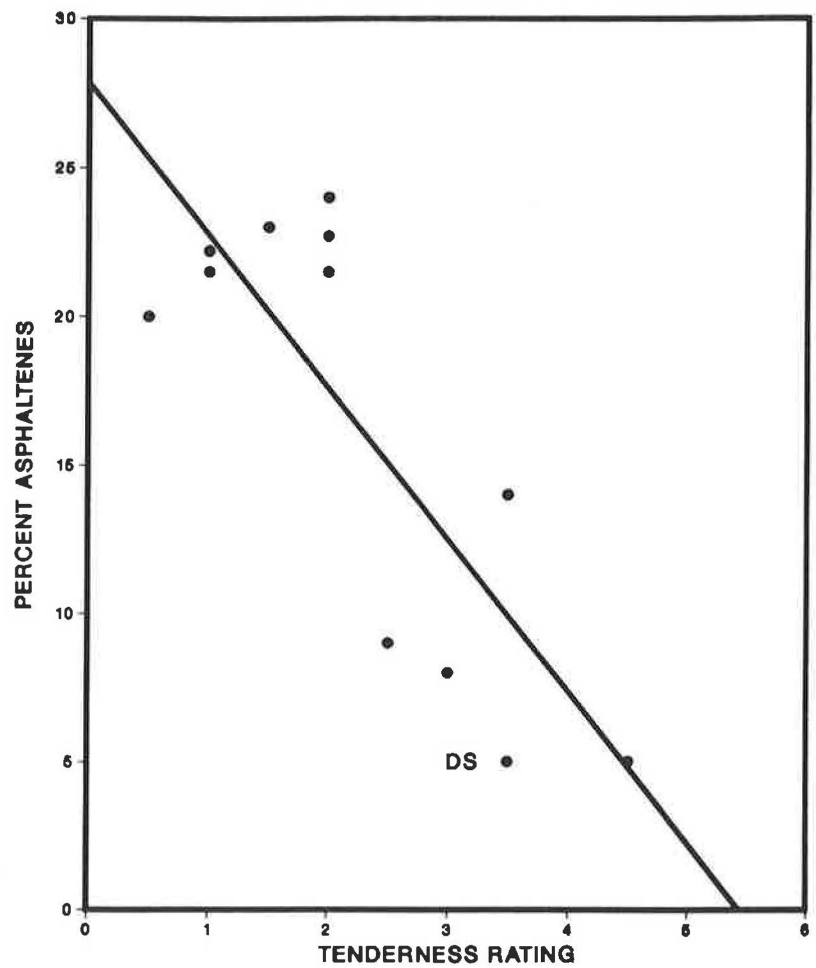

FIGURE 11 Corbett analysis of asphaltene fractions and pavement tenderness ratings.

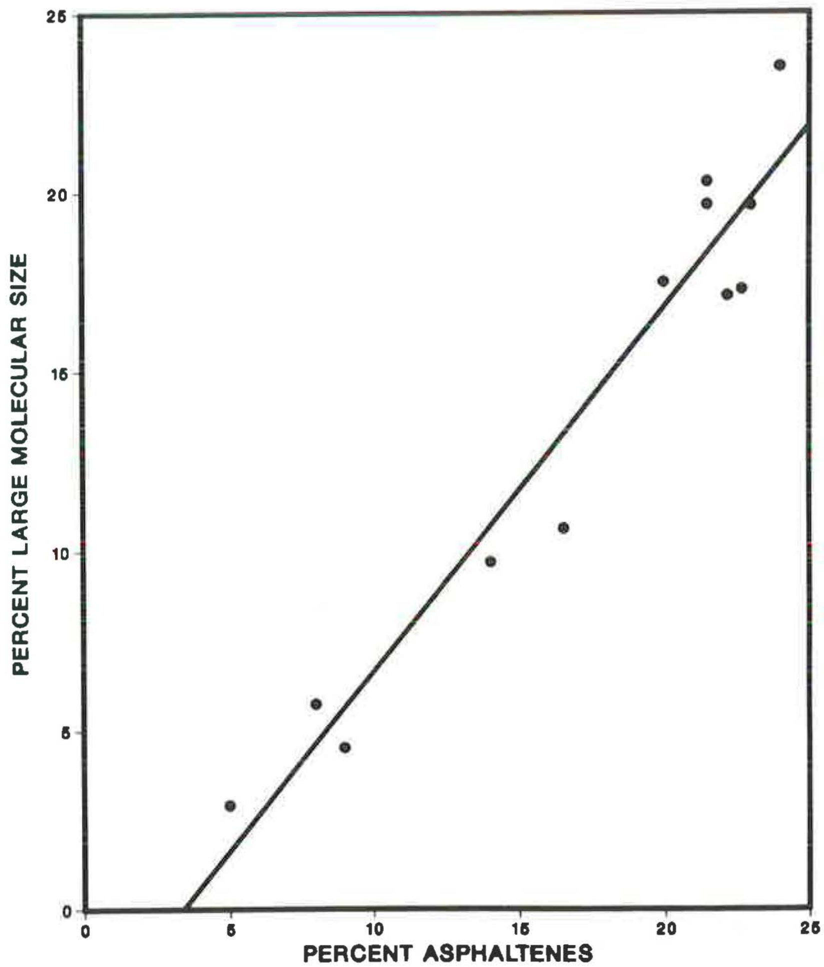

FIGURE 12 LMS fractions and asphaltene fractions.

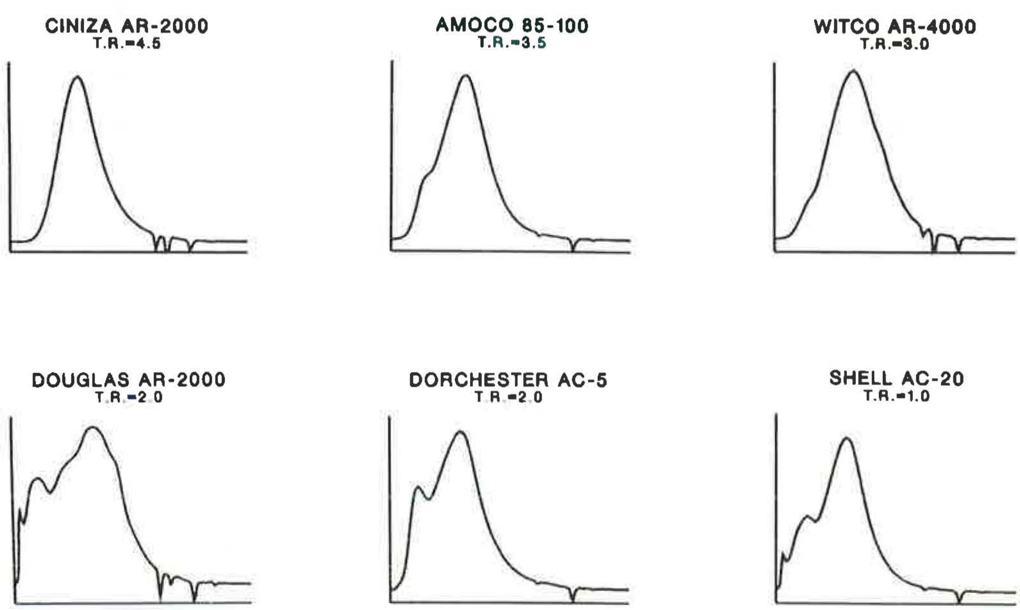

FIGURE 13 Comparison of GPC profiles and pavement tenderness ratings (500-Å and 50-Å columns; THF: 100μL, 7 percent by weight).

CONCLUSIONS

In this work, a number of asphalt properties and characteristics have been studied in light of gel permeation chromatography chromatograms. Results using both THF and toluene as carrier solvents, which give distinctively different chromatograms, were obtained. The chromatograms are quite distinctive with respect to asphalt supplier and there are readily observed differences over time in the chromatograms for selected suppliers. Usually, differences in viscosity grades for a given refiner at a given time are evident from the shapes of the GPC chromatograms.

Viscosity temperature susceptibility has been found to correlate quite well with the large and small molecular size fractions of the GPC chromatograms for virgin asphalts. Correlations of VTS for asphalts extracted from cores, however, were not found, and neither did the penetration index for either virgin or extracted asphalts correlate with the GPC areas.

Finally, the Heithaus flocculation ratio was found to relate to asphalt tenderness. Asphalts that often produce tender mixtures were also observed to have a smaller fraction of the GPC chromatogram area in the LMS region.

ACKNOWLEDGMENTS

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Evaluation of Physical and Fractional Properties of Asphalt and Their Interrelationship

G. THENOUX, C. A. BELL, AND J. E. WILSON

Presented in this paper are the results of a research study in which physical and fractional properties of original and recovered asphalts were obtained. Asphalts from eight projects in Oregon were evaluated. The relationships between the fractional components (obtained with the Corbett-Swarbrick procedure) and physical properties were examined in detail. The relationships between fractional components and temperature susceptibility parameters were also examined. Also presented are evaluations of four different asphalt extraction-recovery procedures and a pressure oxygen bomb device used for laboratory aging of asphalt.

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Also presented are evaluations of four different asphalt extraction-recovery procedures and of a pressure oxygen bomb device used for laboratory aging of asphalt.

The objectives of this paper are to

1. Present laboratory testing results for asphalt pavement materials used in the research project. These include results from tests on physical properties and chemical composition, as well as results from mathematical calculation of various property indices.
2. Evaluate possible relationships between physical properties and chemical composition and between property indices and chemical composition.
3. Evaluate the results obtained on recovered asphalt samples from four different extraction procedures.
4. Compare the aging of asphalt using Fraass samples in a pressure oxygen bomb (POB) with aging in the rolling thin film oven (RTFO).

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RESEARCH PROGRAM

The research program was organized in five different parts. Each part is described briefly.

Project Selection

Eight different highway projects in Oregon were selected to represent a range of performance and highway environments. Figure 1 shows the approximate location of the projects, and Table 1 gives a general description of the present condition of the highway segments under study.

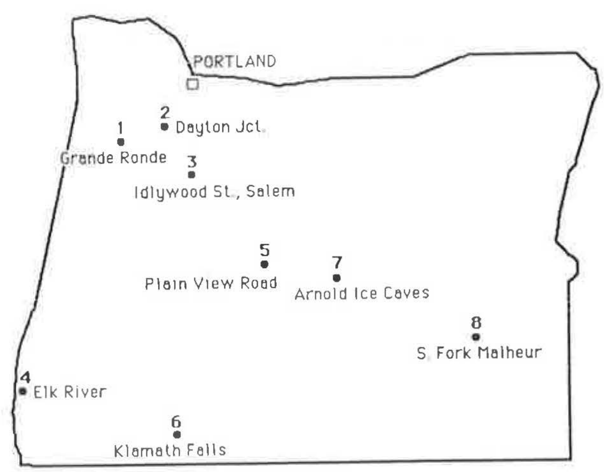

FIGURE 1 Locations of projects.

Core Sampling

Cores were taken from the travel lanes of each project and from shoulders in Projects 5 and 7 (Locations 5s and 7s, respectively, in Table 2). For Project 3, a city street (no shoulders), cores were also taken from a location away from the traffic path (Location 3a in Table 2).

The cores were cut in half: a top layer of approximately 1.5 to 2 in. and a bottom layer ranging from 2 to 4 in. Separate testing was performed on each of the two layers to differentiate environmental effects on the exposed and the unexposed part of the pavement. Table 2 gives detailed information on the top layers of the cores.

TABLE 1 PROJECT DESCRIPTION

Proj. #	Name and Location	Year of Construc.	ADT for 1985 (1)	Trucks % (1)	Rating Cond. for 1985 (2)	General Observations
1	Grande Ronde - Wallace Bridge, St.Hwy-18	1980	9500	12.5	very good	No significant cracking ravelling or shoving
2	Dayton - Lafayette Jct., St.Hwy-18	1980	4050	12.3	very good	No significant cracking ravelling or shoving
3	Idlywood Street, City of Salem	1974	n/a	n/a	good	No significant cracking ravelling or shoving
4	Elk River - North Port Orford, U.S.Hwy-101	1976	5100	13.9	fair	n/a
5	Plain View Rd. - Deschutes River, U.S.Hwy- 20	1980	3550	13.3	fair	5% raveled 5% cracked
6	Klamath Falls-Green Spring Jct., U.S.Hwy-97	1981	9600	36.8	good/fair	5% raveled 5% cracked
7	Arnold Ice Caves - Horse Ridge, U.S.Hwy- 20	1973	1350	14.8	fair	25% shoved 10% cracking
8	S. Fork Malheur - New Princeton, St.Hwy- 78	1974	190	5.0	poor	95% cracks, 5% spalling 5% ravelling

(1) Kim et al. (*3*).
(2) Unclassified publication of the Pavement Management Unit, OSHD, February 1985.

TABLE 2 CORES AND MIX PROPERTIES

Proj. (#)	Thickness (in)	Max. Sp. Grav.	Air Voids	A/C %	Asphalt Suplier	Mix Type	Mr (ksi)	Nf (1)
1	1.72	2.476	11.1	5.0	Chevron AR4000w	B-mix	862.00	80350
2	2.44	2.580	8.5	5.7	Chevron AR4000w	B-mix	1103.19	10005
3	1.91	2.459	11.8	5.9	Chevron	B-mix	771.87	276292
3a	-	-	-	-	AR4000	-	-	-
4	1.44	2.421	5.0	7.0	Douglas AR4000	B-mix	281.94	-
5	1.41	2.497	8.3	5.8	Chevron	B-mix	568.97	42480
5s	1.83	2.484	9.0	5.6	AR4000		703.78	129064
6	2.49	2.535	6.1	5.2	Witco AR2000	B-mix	1031.63	4112
7	1.55	2.444	4.3	6.7	Douglas	B-mix	243.30	295241
7s	1.92	2.434	4.3	6.9	120/150p		186.30	1876282
8	1.44	2.158	8.7	7.6	Shell AR2000	C-mix	621.94	87662

(1) Nf, calculated for 100 microstrains

Implementation of the Corbett-Swarbrick Procedure

This part of the project was reported extensively elsewhere (*1*). The Corbett-Swarbrick procedure (current ASTM D 4124) was submitted for revision by ASTM Committee D 04.47, and a new small-scale test was proposed. Although the proposed procedure is not yet an official standard, it was used in this study for the evaluation of asphalt composition. An early description was given by Corbett (*2*).

Laboratory Testing Program

Figure 2 shows a summary of the laboratory testing program. Four groups of tests were performed for all eight projects:

1. Physical properties of original samples and after the RTFO: Original properties of the asphalt were available from the date of construction, but, because the data were incomplete, these properties were measured again using original asphalt that had been stored in sealed cans at room temperature. The repetition of these tests also served to determine whether the stored asphalt underwent changes during the storage period.

Asphalt properties after RTFO were also available from the date of construction, but these data were also incomplete. The stored asphalt was artificially aged in the RTFO and tested for physical properties.

Table 3 gives the results obtained for original asphalts and after the RTFO along with the results already available from the date of construction.

2. Physical properties of core-recovered asphalt: Asphalt was extracted and recovered from the top and bottom layers of each core. The current procedure of the Oregon State Highway Division (OSHD) was used to obtain samples for all eight projects (Method A) (*4*). Three other methods were also used to extract and recover asphalt from Projects 3, 5, and 7. Figure 3 shows the general scheme of the extraction and recovery procedures used.

3. Asphalt chemical fractionation test results: The Corbett-Swarbrick procedure yields four distinct fractions: asphaltenes, saturates, naphthene aromatics, and polar aromatics. All of the results presented in the next section represent the average of two independent tests. Table 4 gives the standard deviation and range for each fraction obtained and the proposed criteria given by ASTM D 4124.

4. Fraass test results before and after POB aging test: This part of the research involved the use of Fraass samples, which were aged in a POB device for 2 and 5 days and subsequently tested for Fraass breaking point and fractional composition. The purpose of this part of the research was to assess the changes in fractional components of the asphalt after the POB aging test. The changes in composition were compared with

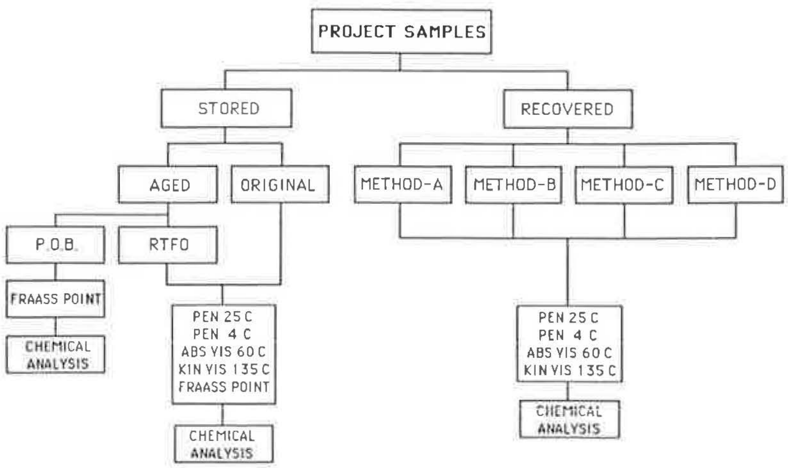

FIGURE 2 Laboratory research program.

TABLE 3 PHYSICAL PROPERTIES OF ORIGINAL AND RTFO SAMPLES

	Data Available from Date of Construction				Data Measured During 1985			
Sample	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)
1-Orig.	18	73	1552	352	23	72	1783	364
2-Orig.	18	73	1552	352	26	77	1613	352
3-Orig.	50	139	-	-	47	128	1169	353
4-Orig.	49	134	1110	340	48	128	1124	335
5-Orig.	20	80	1504	368	22	74	1577	345
6-Orig.	17	85	1052	201	17	88	1059	190
7-Orig.	46	140	762	236	31	128	768	244
8-Orig.	25	100	-	-	15	84	992	190
1-RTFO	-	39	4191	572	16	43	4216	545
2-RTFO	-	39	4191	572	16	44	3960	526
3-RTFO	-	66	4306	608	30	60	4592	665
4-RTFO	-	65	4344	633	32	65	4193	619
5-RTFO	-	46	3858	494	20	52	3858	513
6-RTFO	-	66	1876	255	15	66	1678	247
7-RTFO	-	-	2164	-	30	66	2524	393
8-RTFO	-	60	2051	260	14	54	2068	267

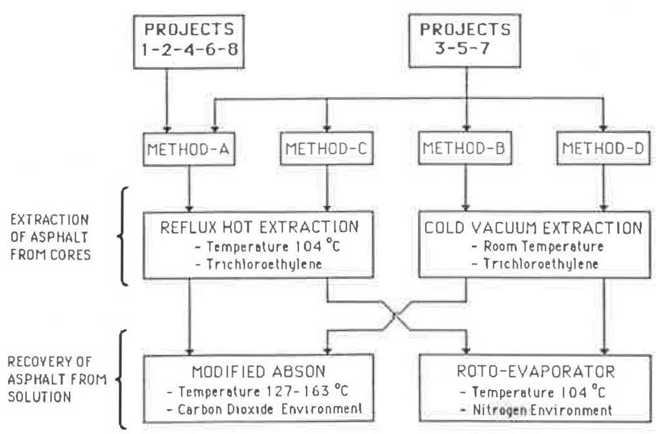

FIGURE 3 Asphalt extraction and recovery procedures.

changes in Fraass temperature and with changes after the RTFO. Only Projects 3, 5, and 7 were used for these tests. The use of the Fraass test sample for aging studies and its advantages are reported elsewhere (*5*, *6*). The characteristics of the POB device were reported by Kim et al. (*3*). The conditions for the test were as follows: 100 psi oxygen pressure, 60°C (140°F), and 2 and 5 days of aging.

Because the amount of materials obtained from the aged Fraass sample is relatively small, only one physical property was measured (Fraass brittle point), and the fractional analysis was run only once.

Asphalt Property Indices

To correlate chemical composition analysis with temperature susceptibility, the following indices were calculated:

- Penetration index (*PI*) (*7*, *8*):

$$PI = (20 - 50A)/(1 + 50A) \tag{1}$$

where

A = $[\log P(T1) - \log P(T2)]/(T1 - T2)$,
$T1$ = 25°C,
$T2$ = 60°C,
$P(T1)$ = penetration measured at 25°C, and
$P(T2)$ = penetration calculated at 60°C using the following relationship:

$$P(T2) = \{[-5.42 \log (V60/13{,}000)]/[8.5 + \log (V60/13{,}000)]\} - \log 800 \tag{2}$$

TABLE 4 REPEATABILITY OF RESULTS FOR INDIVIDUAL CHEMICAL FRACTIONS

Fractions	Actual Testing		ASTM Criteria			
	Single Operator		Single Operator		Multi Laboratory Precision	
	Standard Deviation	Range	Standard Deviation	Range	Standard Deviation	Range
Asphaltenes	0.40	1.1	0.32	0.9	0.95	2.7
Saturates	0.31	1.1	0.44	1.2	0.70	1.9
N-Aromatics	0.88	2.1	1.03	2.9	2.26	6.4
P-Aromatics	0.53	1.6	0.78	2.2	2.37	6.7

where *V*60 = absolute viscosity.

Large negative values of *PI* indicate greater temperature susceptibility. Typical asphalts have values between +2 and −2.

- Viscosity temperature susceptibility (*VTS*) (*9*):

$$VTS = [\log \log V(T2) - \log \log V(T1)]/(\log T1 - \log T2) \quad (3)$$

where

$T1$ = 333 K (60°C),
$T2$ = 408 K (135°F),
$V(T1)$ = absolute viscosity at 60°C in poises, and
$V(T2)$ = kinematic viscosity at 135°C in poises.

where 1 cSt * (0.95/100) ≈ 1 poise. Greater VTS indicates greater temperature susceptibility.

- Penetration viscosity number (*PVN*) (*10*):

$$PVN = [(4.258 - 0.7967 \log P25 - \log KVis)/(0.7951 - 0.1858 \log P25)] * (-1.5) \quad (4)$$

where *P*25 is penetration at 25°C, and *KVis* is kinematic viscosity at 135°C in centistokes. Lower *PVN* indicates greater temperature susceptibility.

- Penetration ratio (*PR*):

$$PR = (\text{Pen @ } 4°\text{C, 200 g, 60 sec})/(\text{Pen @ } 25°\text{C, 100 g, 5 sec}) \quad (5)$$

Lower *PR* indicates greater temperature susceptibility.

DISCUSSION OF RESULTS

Data from Date of Construction Versus Data from Stored Asphalt

Table 3 gives physical properties of original samples before and after the storage period. Some minor variations in physical properties were noticed. These are attributed to variation in testing over a period of 5 to 11 years, not to any aging that may have occurred during the storage period. By looking at changes in penetration at 25°C, and absolute and kinematic viscosity, it can be observed that there is no general trend in the changes undergone by each asphalt sample (i.e., some tests indicate hardening and others softening or no changes at all). The average variation was around 5 percent in both directions (hardening or softening).

Relationship Among Chemical Fractions

Six possible relationships among the four fractions obtained with the Corbett-Swarbrick analysis were studied. Figures 4–9 show these relationships and include results from original samples, RTFO, and recovered asphalt using Method A.

Initially, the relationships studied were to be based only on the data for original samples. Some trends were noticed in these data, but they were insufficient to use to extrapolate beyond the range shown by the type of asphalt used. To increase the range variation, the data from the RTFO aging test were included, and more clear trends were observed. Plotting the results of original samples together with the RTFO results caused the range of variation of chemical fractions to increase, and the trends found when plotting original results alone were improved.

Results from recovered asphalt, using Method A, cover an even larger range of asphalt fraction proportions. These results were also added to the analysis, but, as seen from Figures 4–9, a different trend is observed for the recovered asphalt than for the original plus RTFO samples.

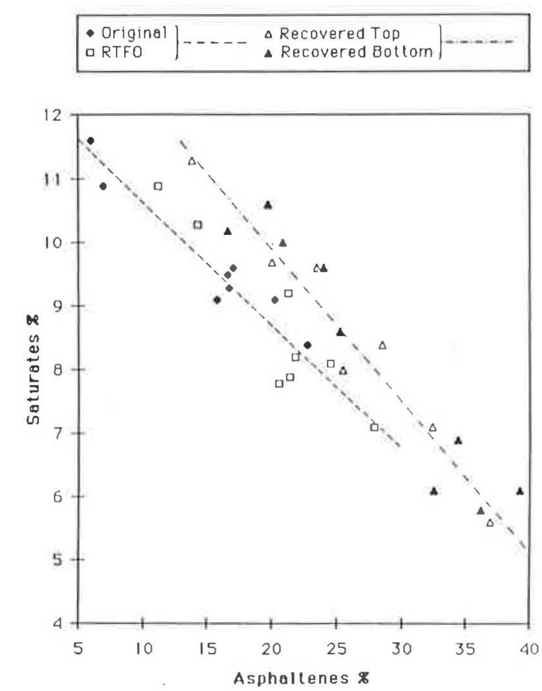

FIGURE 4 Saturates versus asphaltenes.

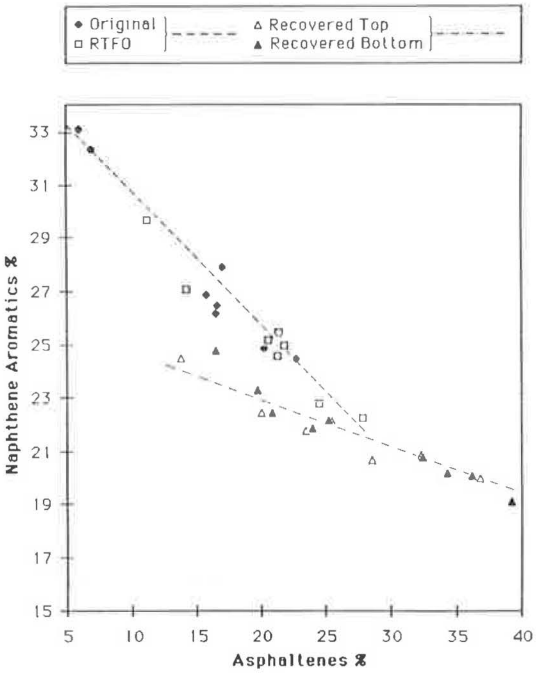

FIGURE 5 Naphthene aromatics versus asphaltenes.

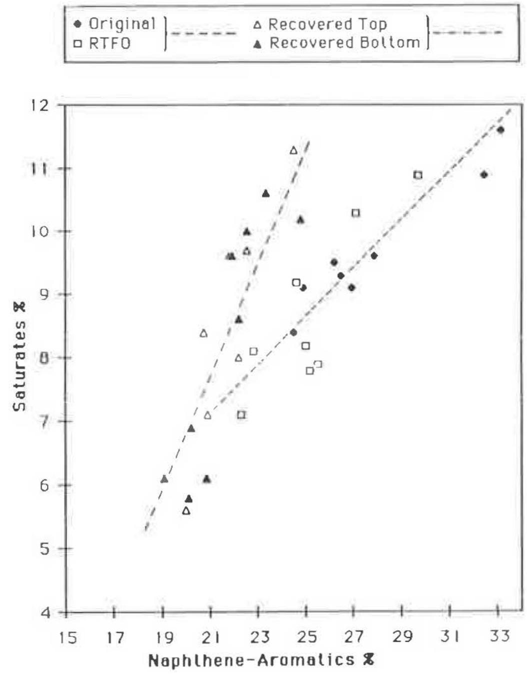

FIGURE 7 Saturates versus naphthene aromatics.

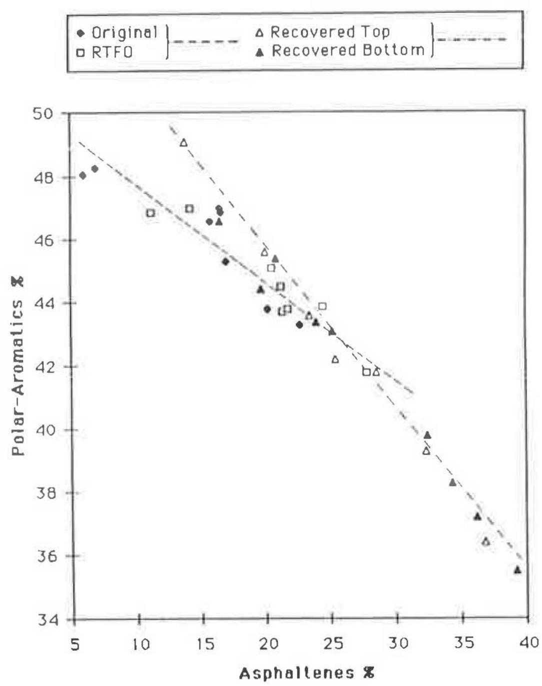

FIGURE 6 Polar aromatics versus asphaltenes.

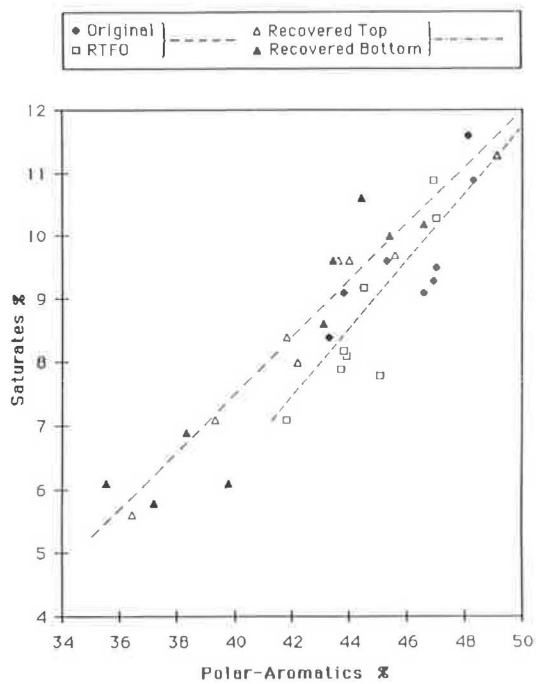

FIGURE 8 Saturates versus polar aromatics.

That recovered asphalt does not show the same relationships that were observed for original plus RTFO samples may be explained as follows:

- Recovered asphalt, after going through the extraction and recovery procedure, may be chemically altered and no longer represent the in-place asphalt.
- The RTFO–accelerated aging procedure does not duplicate the chemical changes undergone by asphalt under natural weathering. The RTFO was designed to simulate aging during construction, not field aging.

On the basis of just the results of this research study, it is difficult to determine how these two factors contribute to the differences in relationships observed in Figures 4–9.

For either the original plus RTFO or the recovered asphalt, relatively good linear relationships were observed between any two chemical fractions. Linear regression equations in which

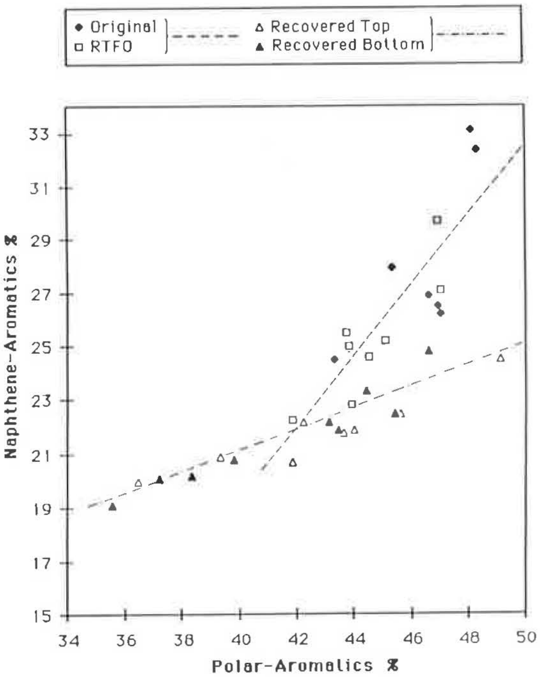

FIGURE 9 Naphthene aromatics versus polar aromatics.

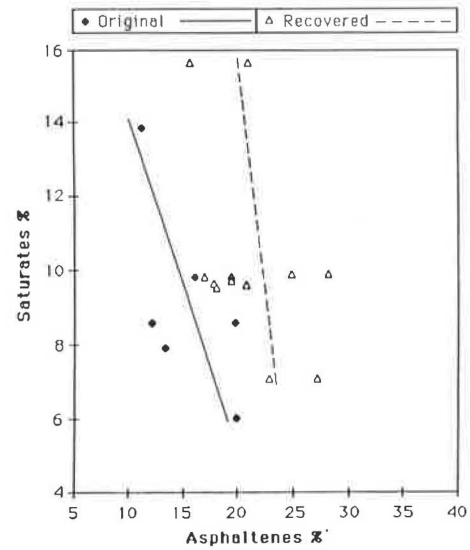

FIGURE 10 Saturates versus asphaltenes in the Michigan Road Test.

the proportion of one component is expressed as a function of the other are given in Table 5. It can be observed that the relatively good relationships (R^2 greater than 0.8) are those in which the asphaltenes are the independent variable and the saturates, naphthene aromatics, and polar aromatics are the dependent variables, respectively. Because the asphaltenes are the first components obtained during fractionation (before chromatographic analysis), if the percentage of asphaltenes is known the proportion of the other three fractions may be estimated.

The relationships given in Table 5 were obtained from a relatively small population (i.e., the original asphalt samples obtained from the eight projects were from five different suppliers, and three of the projects constructed in 1980 used asphalt from the same supplier). The relationships obtained for recovered asphalt may be considered representative of a larger sample because the recovered asphalt represents samples aged under 22 different environmental and traffic conditions (i.e., samples from eight different environments, from the road surface and bottom layer, from travel lanes, from shoulders, and so forth).

Comparison with Other Studies—Relationship Among Chemical Fractions

To determine whether the equations obtained (Table 4) can be extrapolated to other asphalts, results from the Michigan Road Test (*11*) were analyzed. This was the first large-scale program in which the Corbett-Swarbrick analysis was used to characterize asphalt. Figures 10–12 show selected relationships and that, although results from the Michigan Road Test deviate from the relationships developed in this study (Table 4), the general trends are similar. One reason for the deviation may be that the multilaboratory precision range is quite high (ASTM D 4124); these values are given in Table 3. A second reason for some of the large deviations could be that some asphalts may exhibit inexplicable anomalies during repeated trials of the test. This was true when original samples from Project 7 were used, and similar cases have been reported in the literature (*12*).

Recovered asphalt data (*11*) are closer to the trends given by the regression equations than are the original asphalt data. Nevertheless, some deviations were expected because the Abson asphalt recovery procedure used in the present research is a modified version of the original ASTM procedure used in the Michigan Road Test.

One of the objectives of the Michigan Road Test was to relate compositional changes to pavement durability (wear and

TABLE 5 RELATIONSHIP EQUATIONS FOR CHEMICAL FRACTIONS

	Relationship	Original + RTFO			Recover by Method-A	
		Linear Relation	R	Range (+ or -)	Linear Relation	R
1	Saturates vs Asphaltenes	%SA = 12.69-0.197%Asp	0.87	0.93	%SA = 14.49-0.231%Asp	0.89
2	N-Aromatics vs Asphaltenes	%NA = 35.24-0.489%Asp	0.95	2.64	%NA = 27.46-0.206%Asp	0.81
3	P-Aromatics vs Asphaltenes	%PA = 50.65-0.297%Asp	0.87	1.59	%PA = 55.46-0.506%Asp	0.97
4	Saturates vs N-Aromatics	%SA = - 0.78+0.376%NA	0.81	-	%SA = - 9.42+0.808%NA	0.58
5	Saturates vs P-Aromatics	%SA = - 16.63+0.569%PA	0.75	-	%SA = -10.39+0.446%PA	0.87
6	N-Aromatics vs P-Aromatics	%NA = - 34.35+1.341%PA	0.72	-	%NA = - 6.05+0.379%PA	0.72

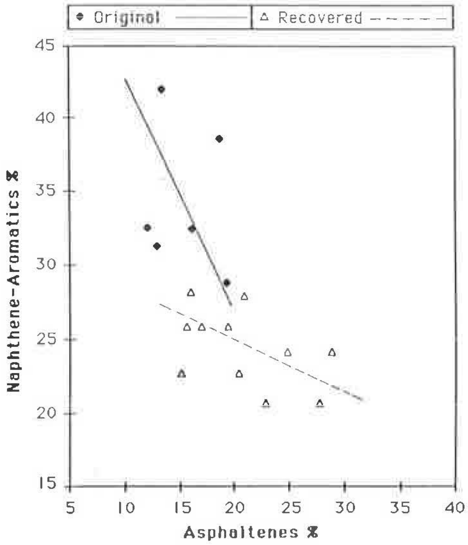

FIGURE 11 Naphthene aromatics versus asphaltenes in the Michigan Road Test.

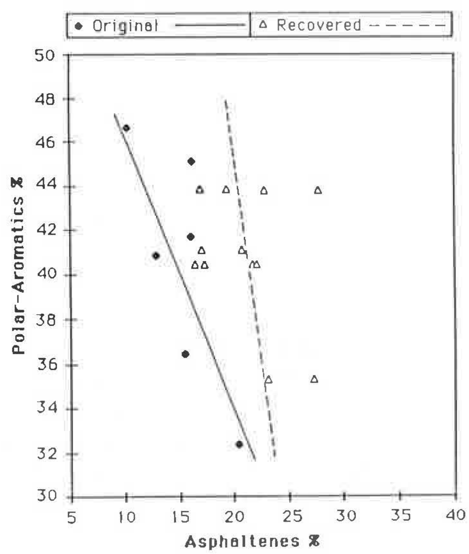

FIGURE 12 Polar aromatics versus asphaltenes in the Michigan Road Test.

weathering qualities). The test was conducted on a 6-mi test section where meticulous care was exercised in controlling mix and construction variables (e.g., aggregate gradation, binder content, temperatures, placing and compaction, and so forth). Although considered a well-controlled experiment, the Michigan test did not result in any definition of a "desirable" asphalt in terms of fractional composition.

Relationship Between Chemical Composition and Physical Properties

Four physical properties of all samples were measured: penetration at 40°C (ASTM D 5, AASHTO T49), penetration at

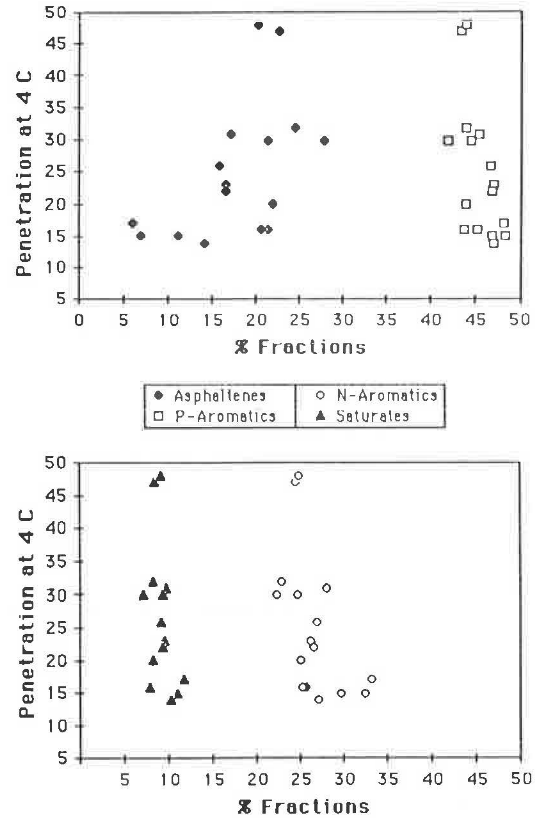

FIGURE 13 Penetration at 4°C versus chemical fractions: original and RTFO samples.

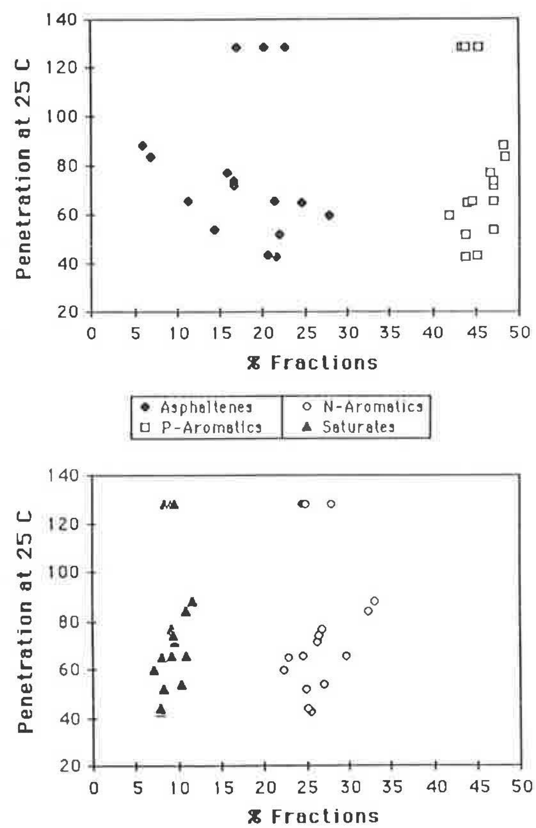

FIGURE 14 Penetration at 25°C versus chemical fractions: original and RTFO samples.

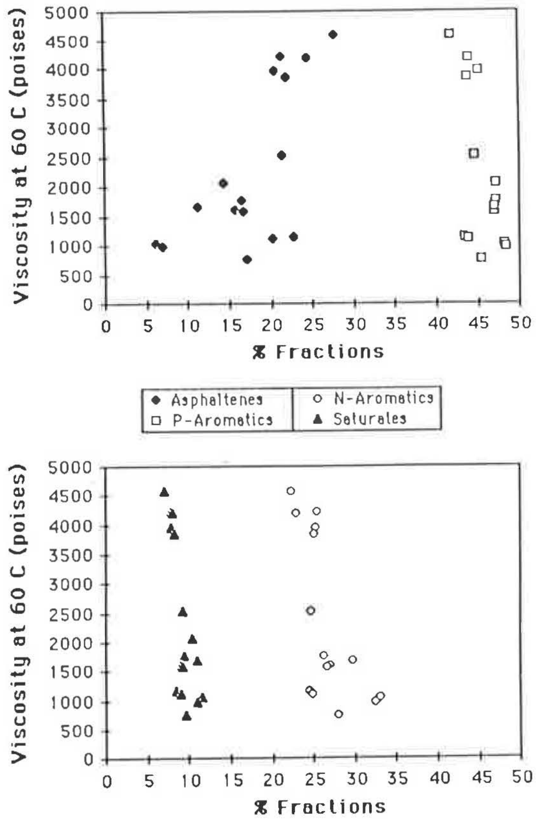

FIGURE 15 Viscosity at 60°C versus chemical fractions: original and RTFO samples.

25°C (ASTM D 5, AASHTO T49), absolute viscosity at 60°C (ASTM D 2171, AASHTO T202), and kinematic viscosity at 135°C (ASTM D 2170, AASHTO T201).

With the amount of data gathered during the research, it was possible to look for relationships between individual chemical fractions and each of the physical properties measured. The study of these relationships was done with two different groups: original samples combined with RTFO samples (Figures 13–16) and recovered asphalt using Method A (Figures 17–20).

Analysis indicates that the compositional profile of recovered asphalt differs from that of original or RTFO samples, or both (Figures 4–9). For this reason, the study of relationships between physical properties and chemical fractions should be treated separately.

Penetration at 4°C Versus Chemical Composition

Figure 13 shows the relationships for original and RTFO samples. Figure 17 shows the relationships for recovered asphalt. The approximate vertical distribution of the data points for saturates, naphthene aromatics, and polar aromatics indicates that penetration at 4°C is independent of the percentage of these three fractions or quite sensitive to changes in the percentage of any of these fractions. The variable distribution of the asphaltene fraction may indicate the following two effects:

1. The asphaltene fraction has an impact on penetration at 4°C, but the significant scatter in the data suggests that some other physicochemical property of the asphaltenes may be significant.
2. The test for penetration, in general, may not be sensitive enough, or, because of its empirical nature, the test may not measure the effect of other variables such as shear rate, shear stresses, and changes in volume (*13*). It is possible that both of these are true.

Penetration at 25°C Versus Chemical Composition

Figure 14 shows the relationships for original and RTFO samples. Figure 18 shows the relationships for recovered asphalt. The relationships between chemical fractions and penetration at 25°C were found to be similar to those for penetration at 4°C.

Although the relationships for penetration (at both 4°C and 25°C) versus chemical fractions look similar for both groups of samples (original plus RTFO and recovered asphalt), it was observed that the naphthene aromatics and the polar aromatics showed "opposite" behavior in both groups of samples (i.e., for original plus RTFO samples the naphthene aromatics show a larger variability than do the polar aromatics). For recovered asphalt larger variability was found for the polar aromatics than for the naphthene aromatics.

The observed phenomena indicate again that the recovered asphalt may not necessarily represent the material that was in

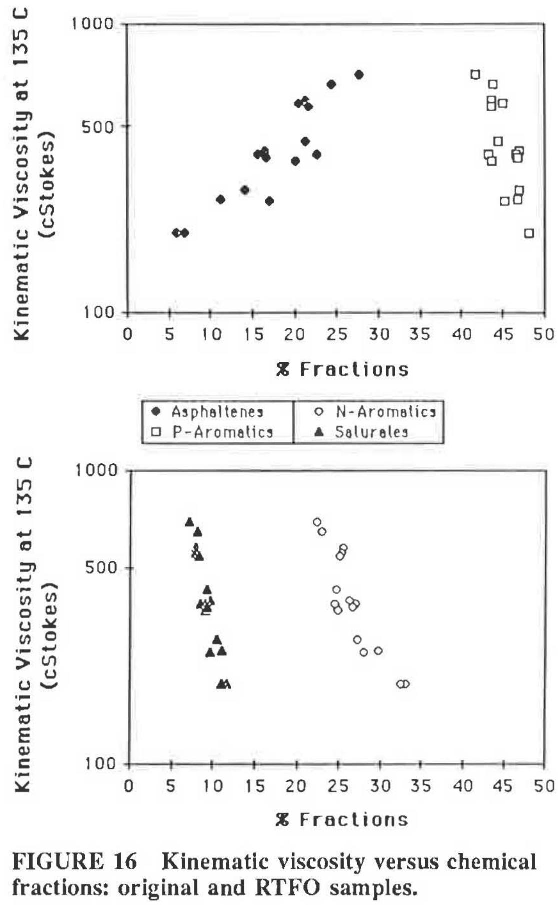

FIGURE 16 Kinematic viscosity versus chemical fractions: original and RTFO samples.

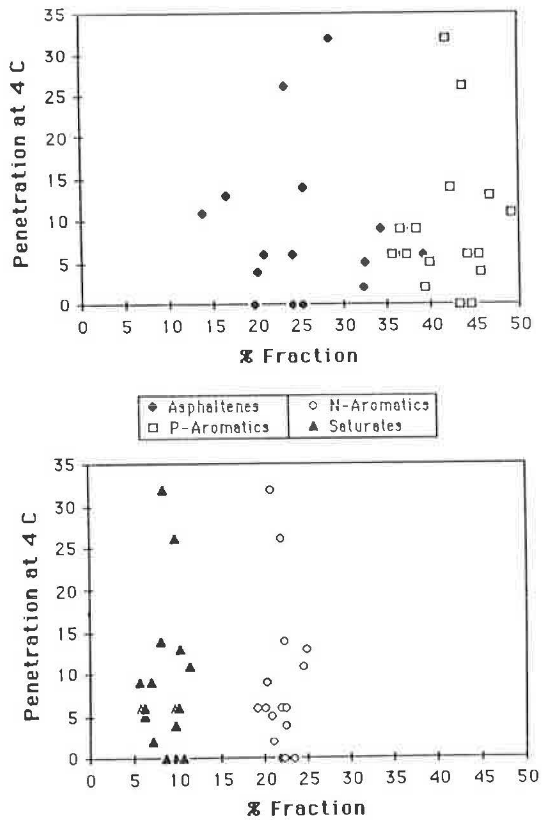

FIGURE 17 Penetration at 4°C versus chemical fractions: Recovery Method A.

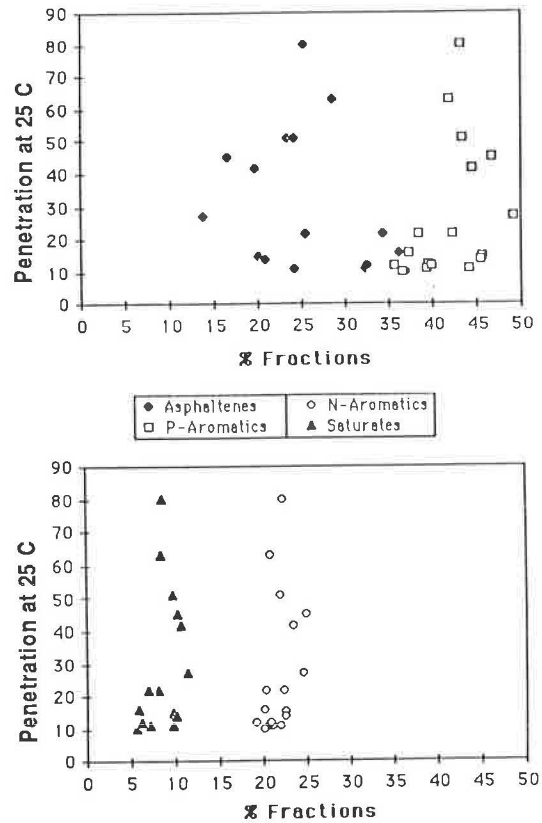

FIGURE 18 Penetration at 25°C versus chemical fractions: Recovery Method A.

place in the road. The same reasoning given when discussing relations among fractions may be applied here to partly explain these differences in behavior between recovered asphalt and original asphalt.

Absolute Viscosity at 60°C Versus Chemical Composition

Figure 15 shows the relationships for original and RTFO samples. Figure 19 shows the relationships for recovered asphalt. The relationships for viscosity at 60°C look quite similar to those observed for penetration, but there are more noticeable trends.

The relationship for asphaltenes is more pronounced than it is for the penetration tests. Viscosity at 60°C shows some type of dependency on the percentage concentration of the asphaltene fraction, but with large variability in the lower viscosity range. This variability may be attributed to the capillary viscometer because recording lower viscosity values manually may be subject to more imprecision than is present in the higher range of viscosity.

The general trend for the relationship between viscosity at 60°C and asphaltenes indicates that the higher the asphaltene content the higher the viscosity. For the other three fractions the relationship is opposite.

Comparison of results for original plus RTFO samples with those for recovered asphalt indicates that the relationships are similar. The results are also similar to those found with the penetration data (i.e., the naphthene aromatic fraction shows a larger variability for original samples than for recovered asphalt and vice versa for the polar-aromatics fraction.

Kinematic Viscosity Versus Chemical Composition

Figure 16 shows the relationships for original plus RTFO samples. Figure 20 shows the relationships for recovered asphalt. Both figures show the kinematic viscosity axis in logarithmic scale. The logarithmic viscosity at 135°C exhibits a good relationship with all four fractions for both original asphalt and recovered asphalt. The greater the percentage content of asphaltenes and the lower the percentage content of the other three fractions, the higher the viscosity at 135°C.

The reason for the better relationship between a physical flow property measured in the higher temperature range (viscosity at 135°C) and chemical fractions may be explained by the following extract from a paper by Petersen (*14*):

> At higher temperatures (Newtonian flow region) the polar interactions between molecules dominate in influencing the flow behavior and the effects of molecular shape or geometry are minimized. At lower temperatures, the kinetic energy of the molecules is lowered and the molecules tend to associate or agglomerate into immobilized entities with a more or less ordered spatial arrangement which is influenced by the geometry of the molecule and its polar functionality.

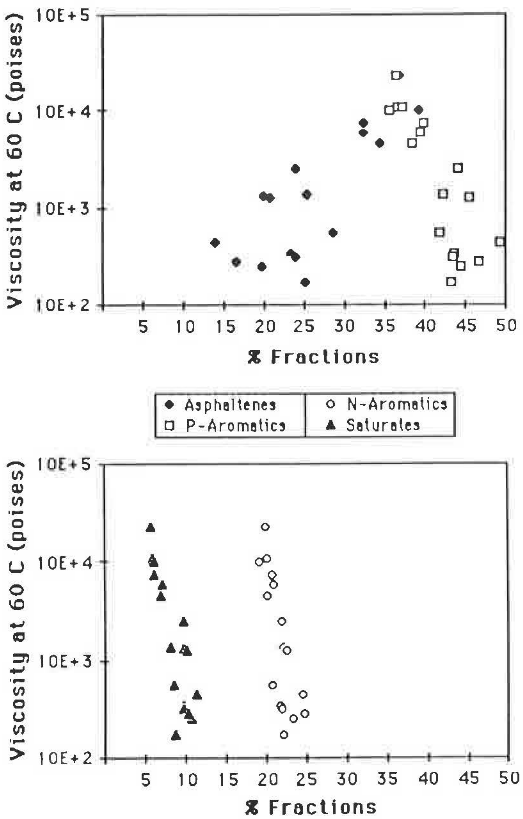

FIGURE 19 Viscosity at 60°C versus chemical fractions: Recovery Method A.

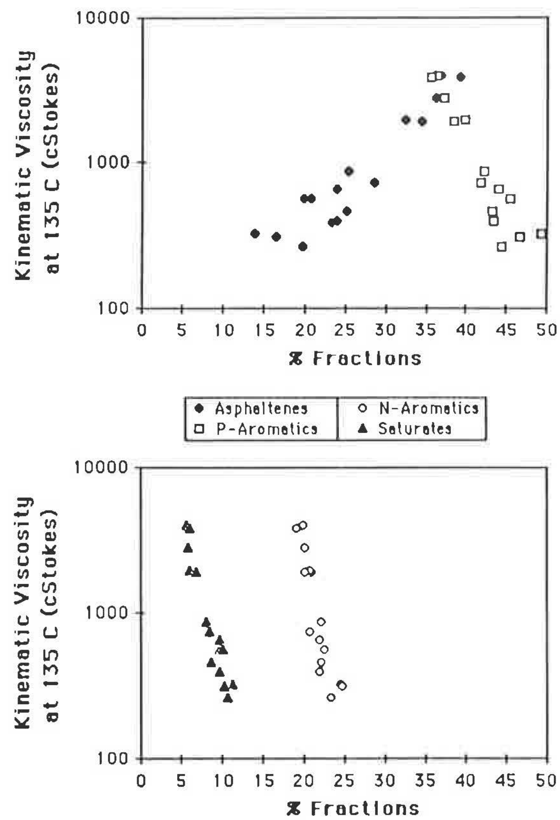

FIGURE 20 Kinematic viscosity versus chemical fractions: Recovery Method A.

Thus, at lower temperatures, the flow property of asphalt may be influenced not only by the percentage concentration of certain types of molecules but also by their polar functionality, spatial arrangement, and geometry.

Relationship Between Chemical Composition and Temperature Susceptibility

Temperature susceptibility can be defined as the rate of change of viscosity (or another measure of asphalt consistency) with temperature and is dependent on the temperature range considered. The influence of temperature susceptibility on pavement construction and performance was addressed in a significant study conducted by Button et al. (*12*).

As was the evaluation of composition versus physical properties, that for composition versus temperature susceptibility was done with two separate groups, original samples combined with RTFO samples (Figures 21–24) and recovered asphalt using Method A (Figures 25–28).

Penetration Index Versus Chemical Composition

Penetration index (PI) values were calculated using penetration at 25°C (Pen 25) and viscosity at 60°C (Vis 60) (*15*). Figure 22 shows the relation for original and RTFO samples and Figure 26 shows the relation for recovered asphalt. It appears that combining Pen 25 and Vis 60 into one index improves the relationships for each of the four fractions. Pen 25 did not show

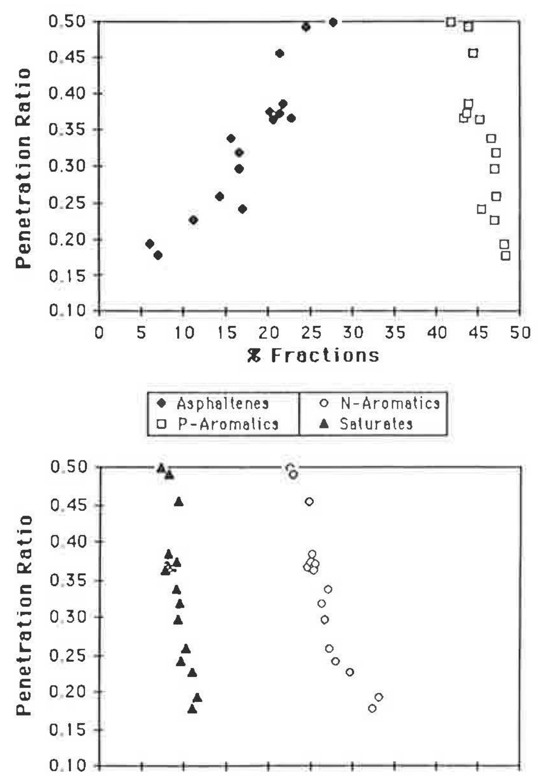

FIGURE 21 Penetration ratio versus chemical fractions: original and RTFO samples.

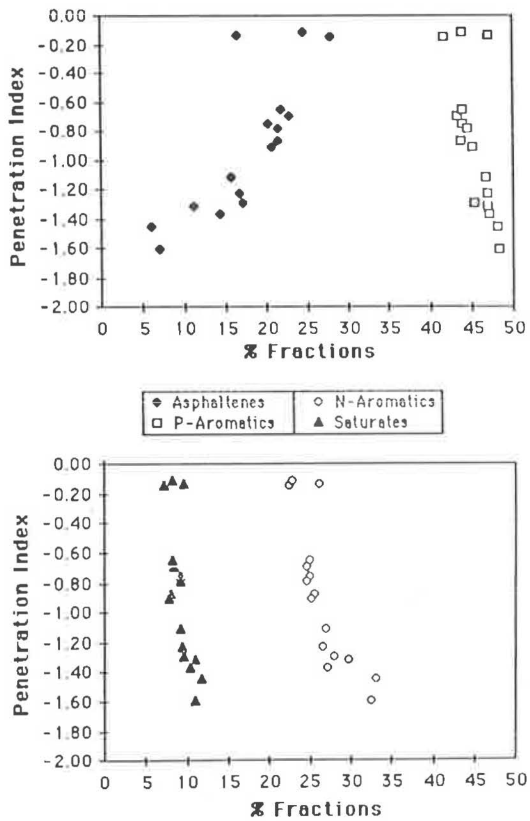

FIGURE 22 PI versus chemical fractions: original and RTFO samples.

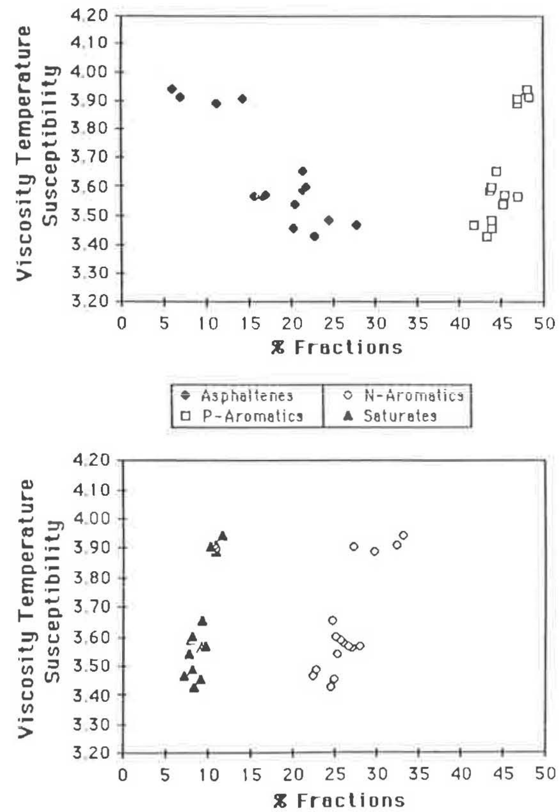

FIGURE 23 Viscosity temperature susceptibility versus chemical fractions: original and RTFO samples.

a good relationship with asphaltene content, but the PI shows a clear dependency on the percentage content of asphaltenes.

Viscosity Temperature Susceptibility Versus Chemical Fractions

The viscosity temperature susceptibility (VTS) parameter was obtained by using the viscosity values measured at 60°C and 135°C. Figure 23 shows the relation for original and RTFO samples, and Figure 27 shows the relation for recovered asphalt.

For both original and RTFO samples there is a correlation between VTS and asphaltene content, although the data are scattered. The percentage content of the other three fractions shows little deviation with changes in VTS-values for both original and recovered asphalt. Although there was some correlation between viscosity at 60°C and 135°C and each of the four generic fractions, it appears that the VTS within this temperature range is not clearly dependent on fractional composition.

Penetration Ratio Versus Chemical Fractions

Penetration ratio (PR) relationships are shown in Figure 21 for original asphalt and Figure 25 for recovered asphalt. The PR parameter used here measures temperature susceptibility of asphalt at 4°C and 25°C.

There is a clear correlation between all four chemical fractions and PR for original and RTFO samples, but poor

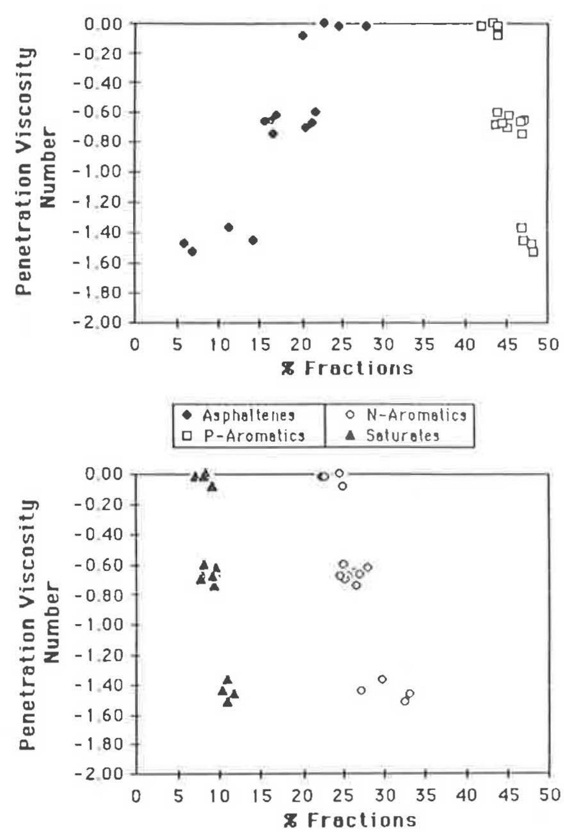

FIGURE 24 Penetration viscosity number versus chemical fractions: original and RTFO samples.

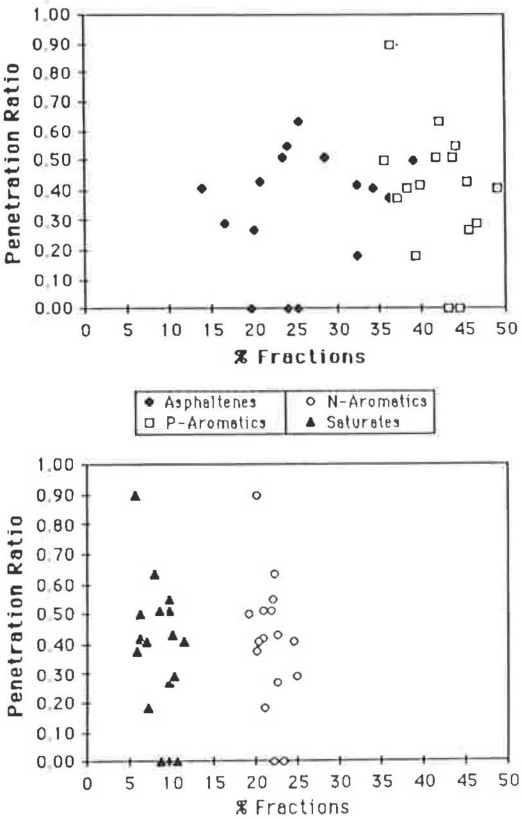

FIGURE 25 Penetration ratio versus chemical fractions: Recovery Method A.

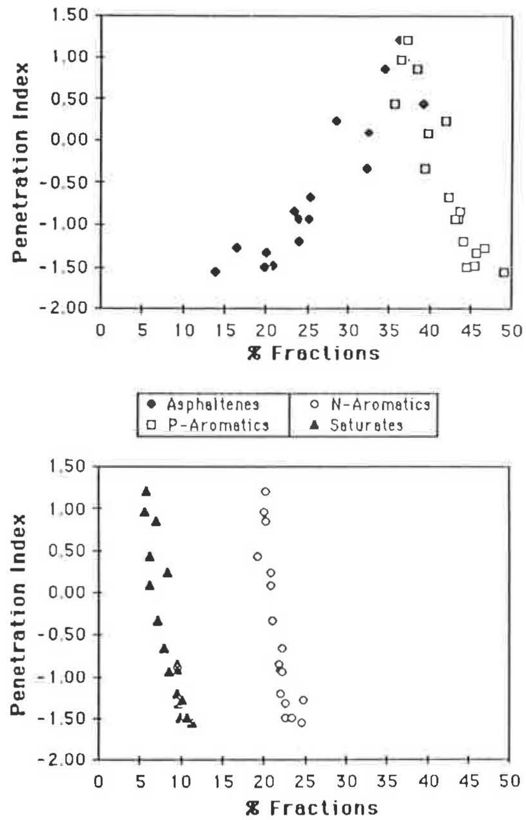

FIGURE 26 PI versus chemical fractions: Recovery Method A.

correlation was observed for recovered asphalt. This big difference among both groups of samples may be explained partly by the arguments given earlier for relationships between fractions. However, the differences may also be due to the recovery procedure during which some kinds of preferential molecular arrangements are destroyed, which would reduce any chance of common behavior within the range of temperature within which penetration values were measured.

Penetration Viscosity Number Versus Chemical Composition

The penetration viscosity number (PVN) parameter is calculated from the penetration value measured at 25°C and the kinematic viscosity measured at 135°C. Figure 24 shows the relation for original plus RTFO samples, and Figure 28 shows the relation for recovered asphalt. PVN covers a larger range of temperatures in comparison with the other three parameters analyzed, and within this temperature range asphalt materials exhibit a wide range of consistency. Thus poor relations were expected between PVN-values and fractional components.

Although, as seen in Figures 24 and 28, there was a poor relation for the PVN-values, the relations with the PI-values determined using a narrower temperature range were similarly bad. The authors have no reasonable explanation for this phenomenon other than to remind the reader that both of these temperature susceptibility parameters (PI and PVN) are based on empirical relationships among the different physical properties and are therefore not fundamental descriptions of material behavior.

Comparison with Other Research—Relationship Between Chemical Composition and Temperature Susceptibility

Values of asphaltene content and temperature susceptibility for 70 asphalts were tabulated by Anderson and Dukatz (*16*) and later plotted by Button et al. (*12*). The temperature susceptibility parameters used in the study were penetration index, viscosity temperature susceptibility, and penetration viscosity number. Button et al. did not observe any relation between any temperature susceptibility parameter and asphaltene content. The differences between the present study and that of Button et al. may be explained as follows:

1. The asphaltene fraction reported by Button et al. was obtained using the Rostler analysis (ASTM D 2006) in which the asphaltene has been precipitated in *n*-pentane. The asphaltene fraction measured in this study, using the Corbett-Swarbrick procedure (ASTM D 4124), was precipitated in *n*-heptane. The amounts of asphaltenes precipitated with each of these two procedures are different (*17*).

2. Button et al. (*12*) plotted all 70 asphalts in one figure in which laboratory asphalt and field asphalt are combined as one set of data. As discussed earlier, field asphalt shows a quite different chemical profile than does "original" asphalt and thus should be treated separately.

3. This research study used a limited number of asphalts, and the data range was artificially increased by using laboratory-aged asphalt with original materials. Nevertheless,

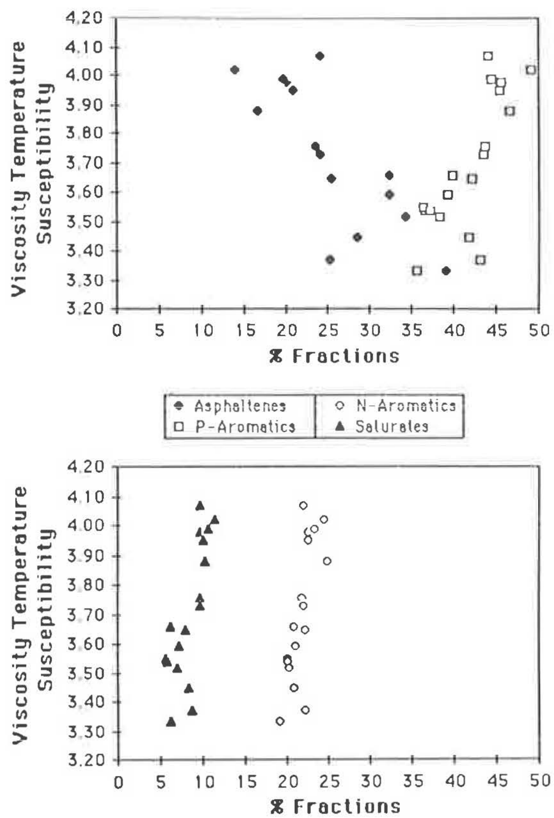

FIGURE 27 Viscosity temperature susceptibility versus chemical fractions: Recovery Method A.

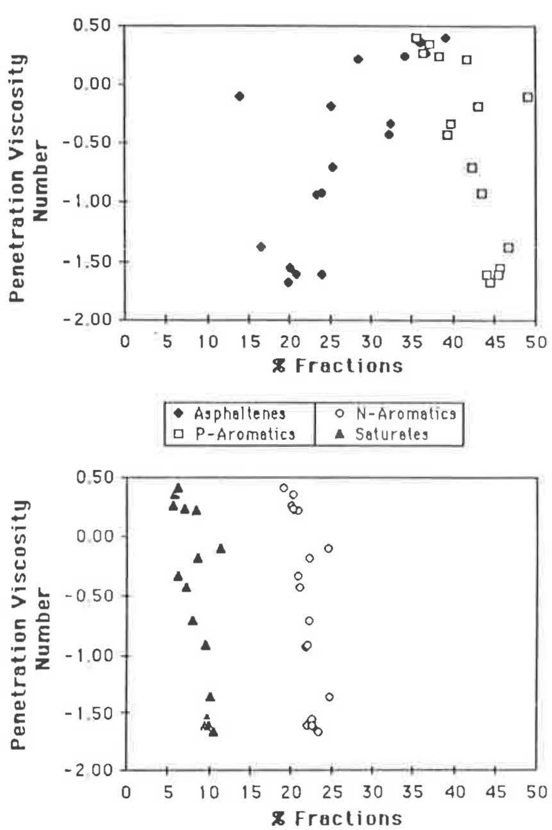

FIGURE 28 Penetration viscosity number versus chemical fractions: Recovery Method A.

this was not done with the field asphalt, and the same kind of correlation was found.

Response of Individual Asphalts

Up to this point, the asphalt analyses of all eight projects have been grouped as one set of data. This permits, to a certain extent, generalization of some of the findings. However, each asphalt used may be studied independently to determine how the aging effect (as measured by increases in hardening) relates to asphalt composition.

For each asphalt sample there are four data points that may be considered: for the original sample, for the sample after RTFO, and for samples recovered from the top and bottom of the core. Unfortunately, the number of points is too small to allow the use of some statistical tools. Thus a descriptive analysis was done to complement the discussion presented previously.

To examine the relationship of each chemical fraction with all of the physical properties and temperature susceptibility parameters used in the present study for all eight projects, 256 plots were created and analyzed. These plots are not included here but will be discussed. The following general observations may be made:

1. All asphalt experienced changes in physical properties and fractional composition with aging. However, asphalts from Projects 2, 4, and 5 experienced relatively small changes in composition, but their physical properties showed significant changes. Asphalt from Project 6 showed the opposite behavior: relatively small changes in physical properties but significant changes in composition. Asphalts from Projects 1, 3, 7, and 8 underwent significant changes in physical properties and composition.
2. For all eight projects, the proportion of asphaltenes increased with consistency as measured by penetration at 4°C and 25°C and viscosity at 60°C and 135°C. The proportions of the other three fractions were reduced.
3. The temperature susceptibility parameters showed quite distinct behavior for all eight asphalts and all four parameters used. Asphalt Samples 1, 3, and 5 showed no variation in VTS and PVN with asphalt composition whereas PR and PI showed significant variations. Asphalt Samples 2, 4, 6, and 8 showed erratic behavior in all four temperature susceptibility parameters. Sample 7 showed some correlation between fractions and all four temperature susceptibility parameters.

From this analysis, the only observation that can be made is that different asphalts behave differently and age differently. The different behavior shown by all of the samples of original and aged material suggests that, to better characterize asphalt properties after aging, more than one aging condition should be studied. For example, asphalt samples could be aged at three or four different RTFO conditions, and, after physical or fractional properties, or both, were measured, the rate of change in measured properties of the different asphalts could be compared. Measuring absolute changes of asphalt properties based on one aging condition may not reflect overall aging behavior.

Comparison of Recovered Asphalt Using Four Different Extraction Procedures

Four methods were used to extract and recover asphalt samples from cores from Projects 3, 5, and 7. The laboratory procedures for these four methods (A, B, C, and D) are summarized in Figure 3. The physical properties and composition analysis of these samples are summarized in Figure 29, which shows that the four methods of extraction and recovery did not give consistent results. The following factors contributed to the differences:

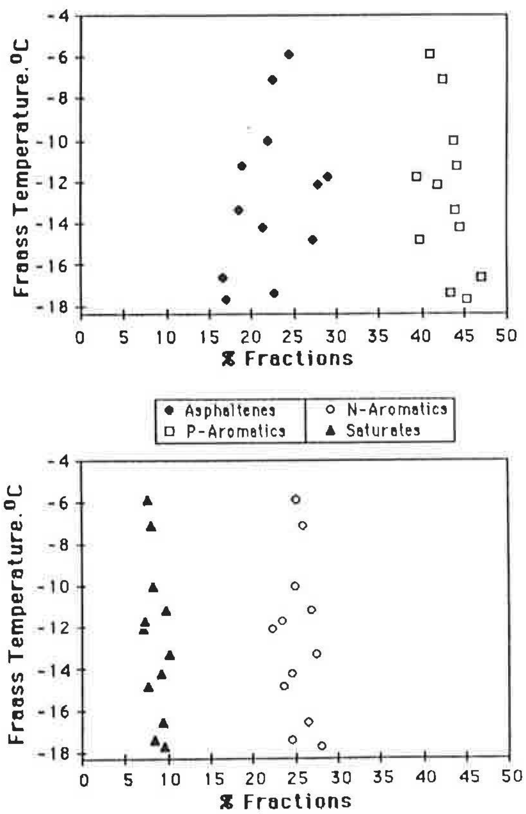

FIGURE 29 Fraass brittle point versus chemical fractions: Projects 3, 5, and 7.

1. The extraction procedure of Methods A and C uses high temperatures (104°C) for a relatively long period of time (as long as 2 hr); for Methods B and D the extraction procedure is done at room temperature. Another difference between the two major extraction procedures is in the filtering devices used. However, the samples are centrifuged in all four methods to decant fine particles that are not filtered properly.

2. The recovery procedure for Methods A and B uses a different gas environment during solvent recovery than is used in Methods C and D. Methods A and B use carbon dioxide at a rate of 2000 mL/min whereas Methods C and D use nitrogen at an unspecified rate. These differences are important because, from OSHD experience, it has been observed that variation of the flow rate of the gas used causes differences in asphalt extracted from the same cores.

3. A third difference to be considered in the present analysis is related to the familiarity of the laboratory technicians with the procedures used. Method A is the only procedure that has been used in routine work for a number of years; the other three methods were used for the first time in this research study.

Analysis of Fraass Test Results and POB Aging Test

This part of the study constitutes an extension of the overall objectives of the research. The POB test was used with the RTFO test to produce accelerated aging of asphalt binders. The POB device was used in conjunction with the Fraass test to evaluate oxidative aging. Samples were prepared on Fraass plates and tested for Fraass breaking point (Institute of Petroleum IP-80/53) before and after aging. Changes in Fraass temperature and in fractional composition were analyzed. It should be noted that the POB test ages asphalt in an oxygen-rich environment in an attempt to simulate long-term aging, whereas the RTFO uses high temperature and simulates short-term construction effects.

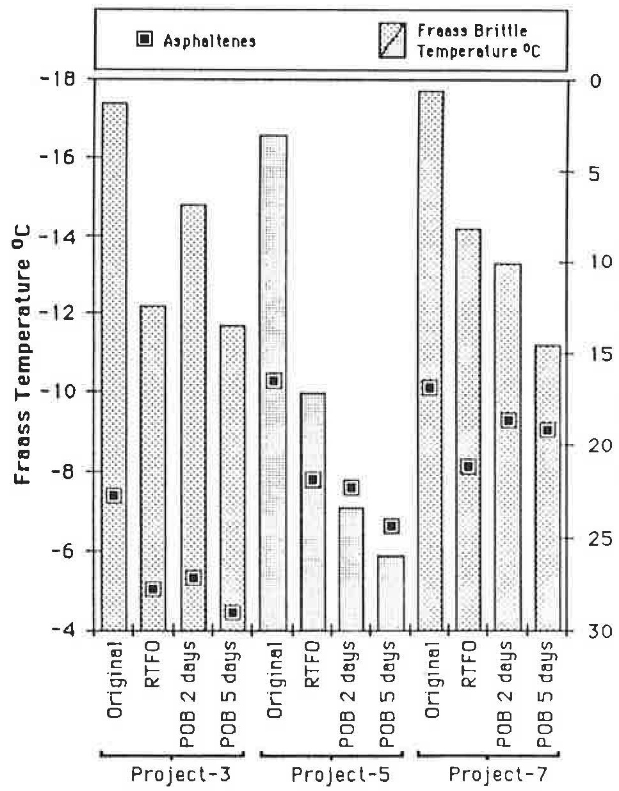

FIGURE 30 Comparison of results of POB test.

Figure 30 shows the relationships between Fraass temperature and all four fractional components for results from three projects studied (Projects 3, 5, and 7) before and after aging. Figure 30 shows that the asphaltenes are the only fraction that has a relationship with Fraass temperature; the other three fractions are more independent. Figure 31 shows similar relationships but is arranged so that each project can be analyzed separately and the effects of each of the aging procedures used can be compared. On the basis of the three samples used (small sample size), the following effects can be observed:

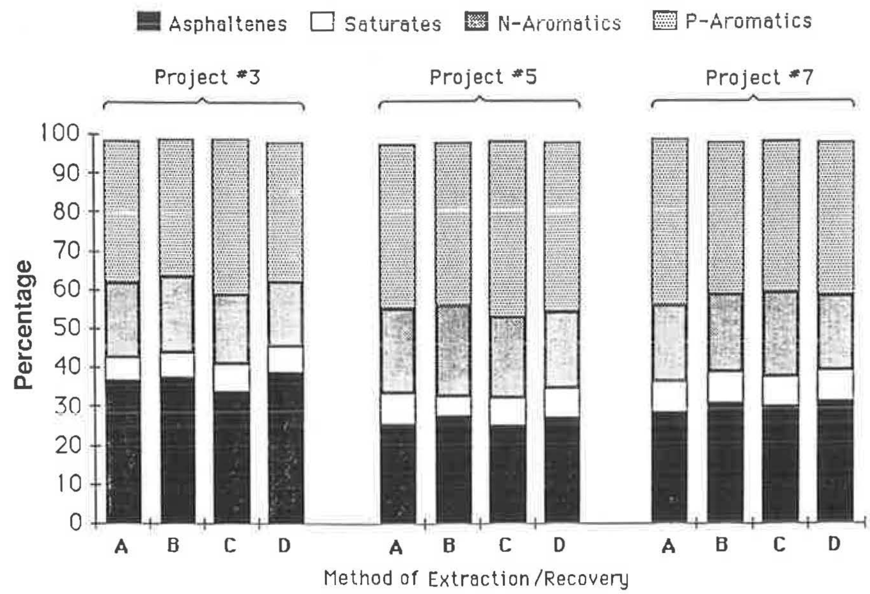

FIGURE 31 Comparison of four extraction and recovery procedures.

- The POB 5-day test was the most severe aging test; it caused much greater changes in composition than did the RTFO test.
- Asphaltene content increases with aging, but the initial asphaltene content of any of the original samples was not related to the total amount of aging after RTFO and POB tests.
- Project 5 was the most susceptible to aging based on laboratory "performance."

CONCLUSIONS

The following conclusions were drawn on the basis of the results of this study:

1. Asphalt samples stored in sealed cans at room temperature for periods of up to 12 years did not show significant variations in their physical properties. Minor variations did not give a clear indication that physical changes were due to aging, and these variations were attributed to the reproducibility of the test results.
2. The analysis of relationships between chemical components showed that recovered asphalt did not have the same profile as original and RTFO samples, which indicates that recovered asphalt, after going through the extraction and recovery procedure, may be chemically altered and no longer represent the in-place asphalt or that the RTFO aging test may not duplicate the changes in asphalt under natural weathering and in contact with mineral aggregates, or both.
3. All physical properties did show some correlation with all four fractions. Better correlations were found at higher temperatures (kinematic viscosity) than at lower temperatures (penetration at 4°C). This may be due to the effects of molecular shape and geometry, which are minimized in the higher temperature range, and to the effect of the testing procedures used at different temperatures.
4. Relatively good relations were found between fractional composition and temperature susceptibility. Better correlations were found for the PR and PI indices in the low temperature range than for VTS and PVN in the high temperature range. Regression analyses showed that the four indices used were distinctly different and that fractional composition had entirely different effects on all four.
5. A certain level of generalization about rheological and chemical behavior of original and aged asphalt was made possible by studying a relatively small group of asphalts. However, analysis of individual asphalts showed that different asphalts do behave differently and age differently. The different types of behavior shown by all of the samples of original and aged materials suggest that more than one aging condition should be studied. For example, asphalt samples should be aged at three or four different RTFO conditions, and the rate of changes in measured properties should be compared for the different asphalts. Measuring absolute changes of asphalt properties under one aging condition may not reflect overall aging behavior.
6. Asphalt extracted and recovered from cores showed a different compositional profile than did the original asphalt. Thus care is advised when using data from recovered and original asphalts together. The four methods used to extract and recover asphalt samples did not give consistent results, and both physical properties and composition measured after recovery were significantly different.
7. Insufficient data were gathered for meaningful conclusions about asphalt low-temperature behavior and its relation to asphalt composition. With the few data available (from Projects 3, 5, and 7), it was observed that, in general, asphalt composition did not show great dependency on the Fraass brittle temperature, which suggests that other molecular properties (e.g., molecular size, molecular structuring, and molecular geometry) may be more important than fractional composition in relation to low-temperature behavior. It was observed that asphaltene content increases with aging in a proportion that is relatively similar to the increase in Fraass temperature. However, the initial proportion of asphaltenes on all three projects was not related to the total amount of laboratory aging.

8. The POB test, in general, did not simulate field aging in terms of the percentage change of asphaltenes. However, the POB showed enough flexibility that it may be adjusted to simulate asphalt aging conditions of zones with different types of environments. The POB did cause greater changes in composition than did the RTFO.

RECOMMENDATIONS

1. Fractional composition of original and laboratory-aged asphalt should be analyzed separately from that of recovered asphalt when studying physical properties of asphalt versus composition.

2. The temperature susceptibility parameters were not comparable because they measured property indices in temperature ranges within which the components of asphalt have different influences. Regression models, based on fractional composition, may be built to predict temperature susceptibility, but a larger set of samples is needed to account for laboratory testing variations.

3. For consistency, the Oregon State Highway Division Laboratory should continue to use the same extraction and recovery procedure (Method A) they have used to date. If interest persists in using the cold vacuum extractor or a Roto-evaporator, or both, for recovery, more research is recommended to produce compatible results or to establish correlations.

4. More testing of the POB device with a larger number of samples is recommended. This will permit a statistical analysis rather than a descriptive discussion of results.

ACKNOWLEDGMENTS

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Improved Quick-Set Slurry Seal Emulsifiers with Tall Oil Derivatives

P. SCHILLING AND H. G. SCHREUDERS

Improved rapid-setting cationic and anionic emulsifiers for bituminous emulsions suitable for slurry seal are described. The cationic emulsifiers are based on condensation products of polymeric ethylene amines with polyfunctional fatty acids. These are obtained by reacting tall oil fatty acids with acrylic acid, fumaric acid, or maleic anhydride. The anionic emulsifiers are obtained by reacting these cationic products with chloroacetic acid or acrylic acid. Emulsions show improved mixing stability with mineral aggregates and good adhesion of the asphalt to the aggregate when the emulsion has set. The reaction mechanisms for obtaining these desired properties are discussed.

Slurry seal is a rapid, low-cost pavement treatment that provides new or existing roads with smooth, antiskid surfaces. One of the most appealing features in addition to cost-effectiveness is swift application, which allows roads to be reopened to traffic less than an hour after treatment. The main benefits are extended life of existing roads afforded by protection from oxidative deterioration and improved skid resistance as the result of eliminating the hazard of loose stones.

Slurry seal is a mixture of fine-graded aggregate of varying coarseness, an asphalt emulsion, and inorganic fillers. In recent years, polymeric latices have been added to further improve durability. The slurry is produced and applied, usually in a thickness of $^1/_8$ to $^3/_8$ in., with a slurry paver. Slurry pavers are self-contained, continuous-flow mixing units capable of accurately delivering predetermined amounts of aggregate, emulsion, inorganic filler, and water to the mixing chamber. The thoroughly mixed materials are discharged into a spreader box and uniformly distributed on the road surface.

A delicate balance of the components must be achieved for satisfactory handling of the slurry seal mix in the mixing chamber and spreader box. Slurry seal must also set and cure properly after it has been laid. The most important factors influencing these properties are the chemical composition and surface charge of the aggregate, the type of asphalt, and the emulsifiers employed.

With a wide variety of surface active chemicals and emulsifiers available, it is possible to tailor the asphalt emulsion properties for individual applications by matching the surfactant system to chemical properties of the aggregate, the inorganic fillers, and the polymer latices. The chemical structure of the surfactant determines its ionic charge in solution and influences its relative solubility in the asphalt and water phase and its degree of adsorption at the asphalt-water interface. A combination of these characteristics determines the ultimate chemical properties of the asphalt emulsions and their behavior in contact with aggregate surfaces.

The purpose of this paper is to familiarize the slurry seal contractor and the asphalt emulsion producer with improved types of cationic and anionic emulsifiers based on tall oil fatty acids, which anticipate the needs of modern slurry seal formulations.

In the early days of slurry seal, modified cement mixers were used to prepare asphalt emulsions. The emulsions had to be extremely stable in the presence of aggregate because of the time necessary for proper lay-down. Anionic emulsions containing rosin soaps and sodium lignate combined with highly oxidized stump wood extracts (complex mixtures of high molecular weight phenolic compounds and oxidized rosin acids) gave adequate mixing performance but were very slow setting. This type of slurry seal emulsion is still being applied in hot, dry regions where set times are accelerated by the rapid evaporation of water.

With the advent of sophisticated continuous mixing slurry seal machines, slurry seals were developed with shorter set times at lower temperatures and with enough stability to prevent premature break in the mixing chamber or spreader box. These new quick-setting slurry seal emulsions are usually cationic in character because better adhesion of the asphalt to negatively charged aggregate is achieved.

CATIONIC EMULSIFIERS

Before specific structures are discussed, the role played by emulsifiers will be explained. The main action during the preparation of an oil-in-water emulsion is the stabilization of small oil droplets by adsorption of emulsifier at the interface. Thus, it is necessary for the emulsifier to be soluble in the aqueous phase and partly soluble in the oil phase. This is achieved by incorporating two distinct regions of opposing solubility in these molecules: the nonpolar hydrophobic region and the highly polar, electrically charged hydrophilic region. When dissolved in water, surfactants form micelles thereby lowering the surface tension of water and facilitating emulsification. The nature and concentration of the emulsifier in an asphalt emulsion determines the coalescence rate (stability) of the emulsion droplets; it also influences particle size distribution, storage stability, rate of setting, and adhesion of the asphalt to the aggregate when the water has evaporated. One of the major difficulties in developing a suitable slurry seal emulsion is the requirement that the emulsion be very stable while it

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is being mixed with the aggregate but unstable after the mix is placed on the road surface.

Asphalt emulsions break on the surface of mineral aggregates by neutralization of the electrical charges of the emulsion droplets by the charges of the aggregate surface. This process causes coalescence of the asphalt droplets resulting in a continuous film of asphalt on the aggregate surface when applied to the road.

Cationic emulsions with highly charged anionic aggregates are generally preferred. Cationic emulsifiers are exclusively fatty, nitrogen-containing molecules with either permanent positive charges (quaternary ammonium salts) or fatty amines that are quaternized by reacting with strong acids to form the corresponding protonated species in the emulsion mixture.

Examples of cationic emulsifiers are fatty amidoamines (*1*), combinations of tall oil fatty imidazolines and ethoxylated nonylphenol (*2*), VINSOL-tetraethylene pentamine condensates (*2*), ethoxylated imidazolines (*3*), fatty mono- and diquaternary ammonium salts (*4*), and combinations of cationic compounds (*5*).

The hydrophobic region of the commonly used asphalt emulsifiers consists of straight C_{12}-C_{20} carbon chains, which can be saturated or contain double bonds. The polar segments can contain one or more nitrogens in the form of primary, secondary, or tertiary amino groups. Structures such as amidoamines, imidazolines, or diaminopropanes also may be used. The fatty amines based on oleic acid shown in Figure 1 serve as examples of these types of emulsifiers.

In general, the ionic strength of the nitrogens in fatty amines increases in the order primary, secondary, tertiary. Deprotonation occurs at a lower rate when the more sterically hindered trialkyl ammonium hydrochlorides approach a negatively charged surface or a hydroxyl anion. A similar situation exists for the amidoamines, where, depending on the amine used for the condensation, primary, secondary, or tertiary nitrogens can be introduced into the molecule. For simple cationic emulsifiers, the more hindered the amine, the more stable the emulsion but the slower the set of the slurry seal.

Emulsions prepared with conventional quaternary ammonium salts that have permanent positive charges interact slowly with aggregates. The positively charged center of the emulsifier is sterically hindered. It is neutralized by the negative charge of the aggregate, which deactivates it as an emulsifier and thereby initiates the "break" of the emulsion.

Condensation products of fatty acids with diethylene triamine result in a mixture of amidoamines (A) and (B) (Figure 2) containing two primary groups or one primary and one secondary amino group. Further reaction results in ring closure to yield an imidazoline (C) with a primary and two tertiary nitrogens in the molecule.

Among these structures, the protonated imidazoline (D) (Figure 3) is the most stable cation because of delocalization of the positive charge over three atoms. Thus protonated imidazolines are relatively stable in the presence of negatively charged surfaces, and emulsions prepared with fatty imidazolines break more slowly when mixed with aggregates than do emulsions prepared with protonated amidoamines (A) and (B).

Cationic emulsions prepared from unmodified tall oil fatty acid–derived amines and imidazoline are still not stable enough to be used for slurry seal mixes. They break prematurely when brought into contact with the aggregate. Successful mixes can be achieved when retarders, such as cement or aluminum sulfate, or excess emulsifier solution, or both, are added.

DIELS-ALDER MODIFIED TALL OIL CATIONIC EMULSIFIERS

Recently, amidoamines and imidazolines, which yield emulsions with improved mixing stability without a retarder in the mix, were found at the Westvaco Corporation. The key to this difference in behavior is the introduction of reactive polar groups in the center of the fatty acid molecules.

Oleic acid, a mono-unsaturated acid, and the dienoic linoleic acid isomers are the major constituents of tall oil fatty acid. By applying a proper catalyst, linoleic acid isomers can be converted to the highly reactive conjugated linoleic acid (E) (Figure 4). This readily undergoes Diels-Alder cycloadditions with dienophiles such as acrylic acid, fumaric acid, or maleic anhydride to give C_{21}-dicarboxylic acid (F), C_{22}-tricarboxylic acid (G), and C_{22}-tricarboxylic anhydride (H) (Figure 4). In this way, additional reactive carboxyl groups can be added to the

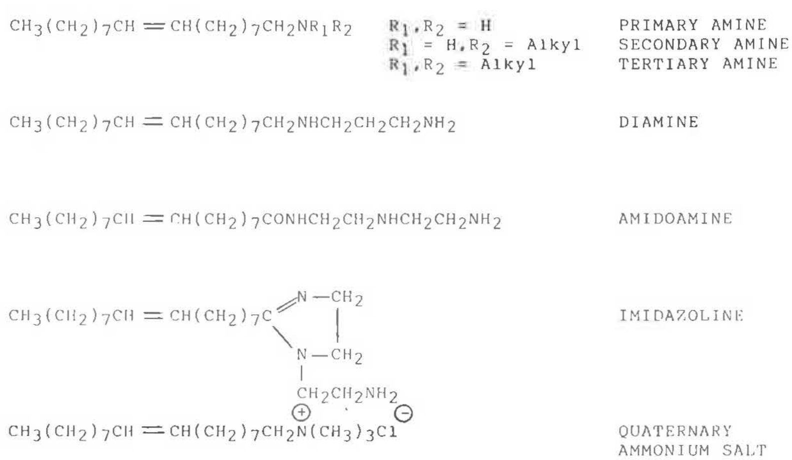


FIGURE 1 Fatty amines based on oleic acid.

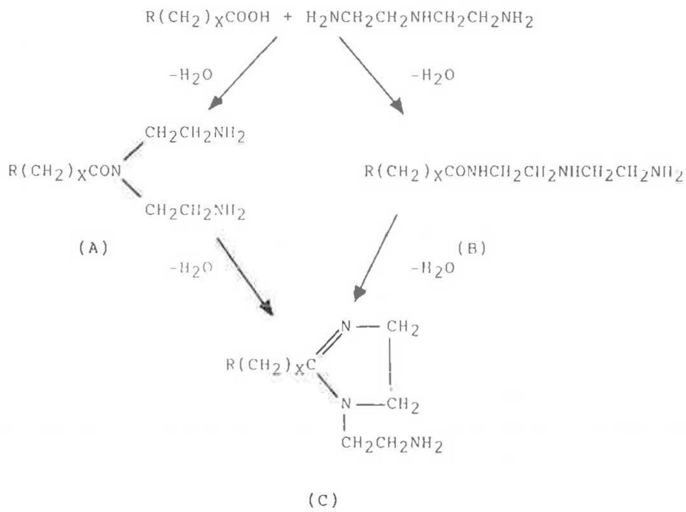


FIGURE 2 Condensation products.

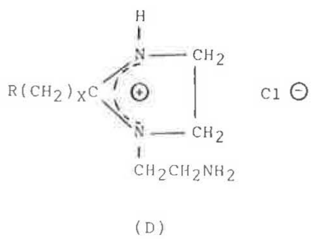


FIGURE 3 Protonated imidazoline.

middle of the fatty acid chain. The other common feature of these molecules is the cyclohexene structure to which the additional carboxyl groups are attached.

Oleic acid also can be modified with maleic anhydride to introduce carboxyl groups on its fatty acid chain. The addition is accomplished by the "ene" reaction to give succinic anhydride derivatives of oleic acid (I) (Figure 5).

Reaction products of these polyfunctional fatty acids with suitable polyamines are effective emulsifiers for cationic quick-set slurry seal. Examples of these cationic emulsifiers are the C_{21}-diamidoamines (J), C_{22}-triamidoamines (K), and C_{22}-amidoimidoamines (L, M) (Figure 6) obtained when the various modified fatty acids are reacted with diethylenetriamine.

Emulsions prepared with these polyamidoamines are stable and can be safely mixed with aggregate without breaking prematurely. The increased nitrogen functionality results in a molecule with higher overall polarity and the ability to form a di-cation in dilute hydrochloric acid. With a second positive charge in the center of the molecule, a different arrangement at the asphalt-water interface can be visualized with the emulsifier attached to the asphalt particle at two sites (Figure 7). Thus the chemical structure of this emulsifier does not allow close packing of the aliphatic oil-soluble chains as is the case with conventional emulsifiers (Figure 8).

When a cationic asphalt emulsion is mixed with an aggregate bearing a negatively charged surface, the emulsion droplets are destabilized by proton transfer from the emulsifier to the aggregate surface, which neutralizes its charge and breaks the emulsion. In the case of the new polycarboxylic acid–derived slurry seal amidoamines, each molecule bears two positive charges. On approaching a negatively charged aggregate surface, both centers must be deprotonated for the emulsifier molecule to have zero charge. Even when the emulsifier molecule loses only one proton from the central or terminal amino group, it still remains an effective emulsifier. In this case, its ratio of molecular weight to protonated amino group is not very different from that of a common protonated fatty amidoamine emulsifier.

These new emulsifiers provide not only superior stability but also improved cohesive strength development of the cured slurry matrix. Comparison of the cohesive strength development, using a modified cohesion tester for a new dicarboxylic acid diamidoamine (*6*), a diquaternary ammonium salt (*4*), and a common tall oil fatty acid monoamidoamine, demonstrates the outstanding performance of these new products in the cured asphalt-aggregate mat (Figure 9).

Emulsions prepared with common tall oil fatty acid monoamidoamine can only be mixed with aggregate in the presence of a retarder such as aluminum sulfate, which neutralizes some of the negative charges of the aggregate. However, the aluminum sulfate also slows down the cohesive strength development of the slurry seal. This situation is avoided with the new dicarboxylic acid diamidoamine because a retarder is not needed for a stable emulsion. Even though the diquaternary ammonium salt required no retarder either, an emulsion produced with this emulsifier had poorer cohesive strength development than that of the dicarboxylic acid diamidoamine (Figure 9).

A retained stone coating of nearly 100 percent after subjecting cured slurry mixes containing di- or polyamidoamine to the hot water boiling test indicates that a strong bond between the asphalt and aggregate is formed with these new emulsifiers (see Appendix). This indicates the superiority of the polyfunctional amidoamines. Mixes prepared with emulsions containing fatty

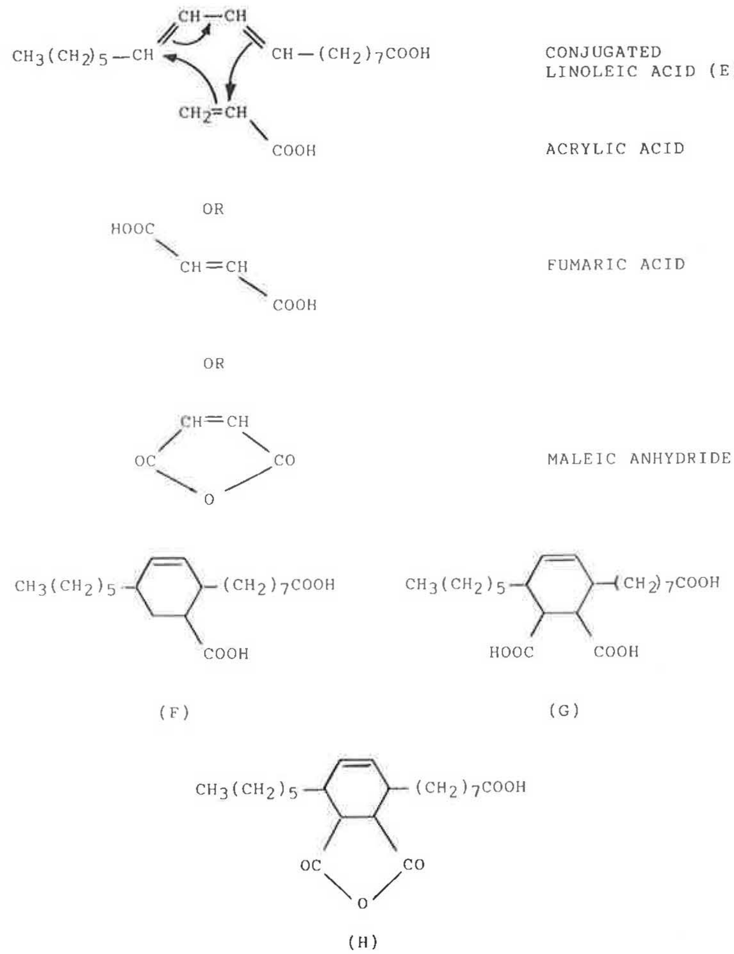


FIGURE 4 Diels-Alder cycloadditions of linoleic acid.

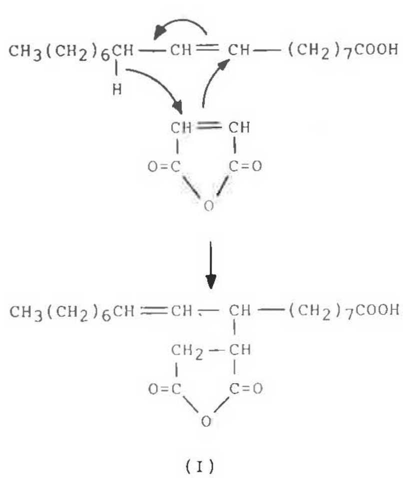


FIGURE 5 Succinic anhydride derivatives of oleic acid.

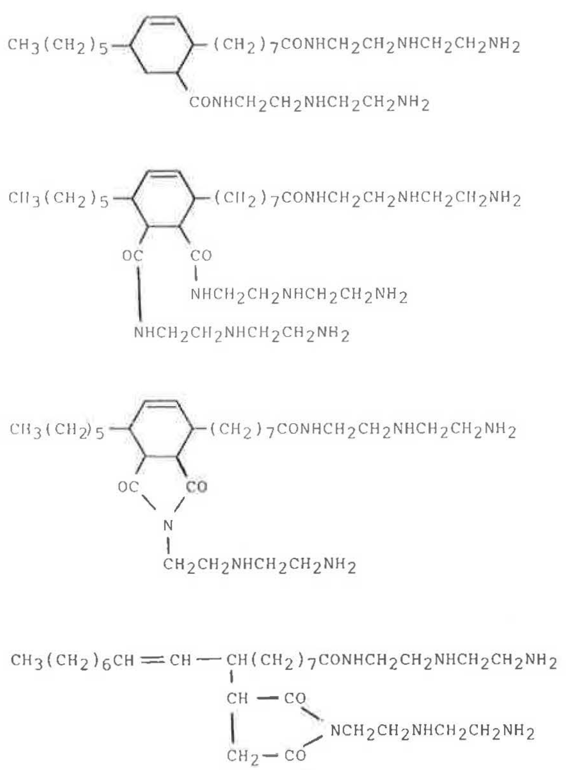
quaternary ammonium salts show a lower degree of retained coating due to weaker bonds between asphalt and aggregate.

ANIONIC EMULSIFIERS

The oldest and most widely used low-cost anionic asphalt emulsifiers are soaps or carboxylates. These compounds are water soluble salts, usually the sodium salt, of long-chain fatty acids derived from animal fats, vegetable oils, and tall oil. Other anionic surfactants used are long-chain aromatic fatty sulfates and sulfonates. As in the case of cationic emulsions, the long nonpolar carbon chain is soluble in asphalt, whereas the hydrophilic head group is soluble in the water phase (Figure 10). Note that the hydrophobic entity R may be obtained from natural fatty or resin acids or from petroleum-based feedstock.

For the manufacture of fast setting anionic emulsions, two types of emulsifiers are used: fatty acid soaps and sulfonates such as -olefin sulfonates, petroleum sulfonates (*7*), and dodecylbenzene sulfonates (*8*). Fatty acid soaps, because of their rapid interaction with Ca^{2+} or other polyvalent cations that render them water insoluble, are not suitable as slurry seal emulsifiers. Their emulsions break prematurely (within seconds) when brought into contact with aggregates. Sulfonic acid salts, on the other hand, do not interact with calcium ions and give emulsions with good mixing stability. However, in slurry seal produced with these emulsions, the asphalt shows inferior adhesion properties due to a lack of interaction with the aggregate surfaces.

Emulsions that were prepared with emulsifiers modified from fatty acids having one or two additional (F, G) carboxylic groups in the center of the molecule broke prematurely. However, with further modification, these di- and tricarboxylic



(J)

(K)

(L)

(M)

FIGURE 6 Cationic emulsifiers.

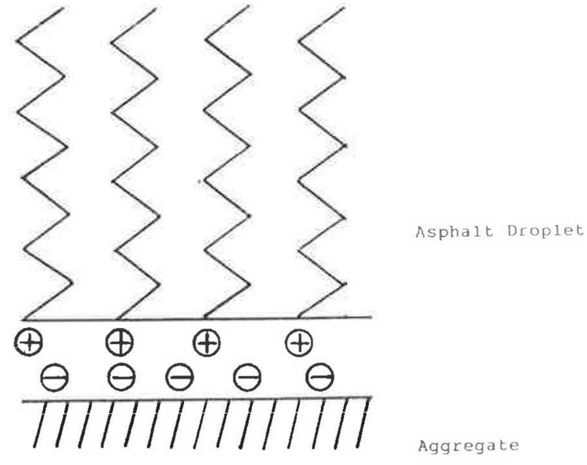


FIGURE 8 Arrangement of conventional cationic emulsifiers in an asphalt droplet.

fatty acids can be converted into workable anionic emulsifiers. These emulsifiers are obtained by reacting the polycarboxylic acid-derived amidoamines or imidoamine (J) through (M) (Figure 6) with the reactive carboxylic acids, acrylic acid, or chloroacetic acid. The resulting amino acids are very effective anionic slurry seal emulsifiers and insensitive to Ca^{2+} ions.

Sulfonic acid groups may also be introduced via sulfomethylation (*9–11*). If a dicarboxylic acid diamidoamine (J) is used as starting material, the reaction products are very complex aminoalkylcarboxylic acids (N, O) or aminomethyl sulfonic acids (P), as shown in Figures 11 and 12.

A benefit of using these complex aminoalkyl carboxylic acids is the flexibility of the pH value at which the emulsions can be prepared. The mixing behavior of the emulsion is readily influenced by the proper choice of the alkali charge. At higher pH values the number of negatively charged carboxyl groups increases, resulting in longer mixing times because of the increase in concentration of repulsive forces between aggregate surface and emulsion droplets (Figure 13).

Conversely, if the emulsions prepared with these surfactants at relatively low pH values (9.5 to 10.0) can be mixed with an aggregate, a short set time will result. Mixes prepared at these lower pH values generally show better adhesion of the asphalt to the aggregate than do mixes prepared at very high pH values. If during production an aggregate requires more mixing time, this can be achieved by the addition of more alkali at the expense of some adhesion. This adhesion loss can be counteracted by the use of small amounts of additives containing calcium ions, which results in good adhesion at high pH. Emulsions prepared with alkylbenzene sulfonates generally result in poor adhesion of the asphalt to the aggregate because at any pH value above 7 the sulfonic acid groups are fully deprotonated.

Figure 14 shows the rate of cohesive strength development of slurries prepared with emulsions containing these aminoalkyl carboxylic acids. It can be seen that these slurries are comparable to a slurry containing cationic emulsions prepared with a diamidoamine. It should be noted that a decrease in pH increases the rate of strength development.

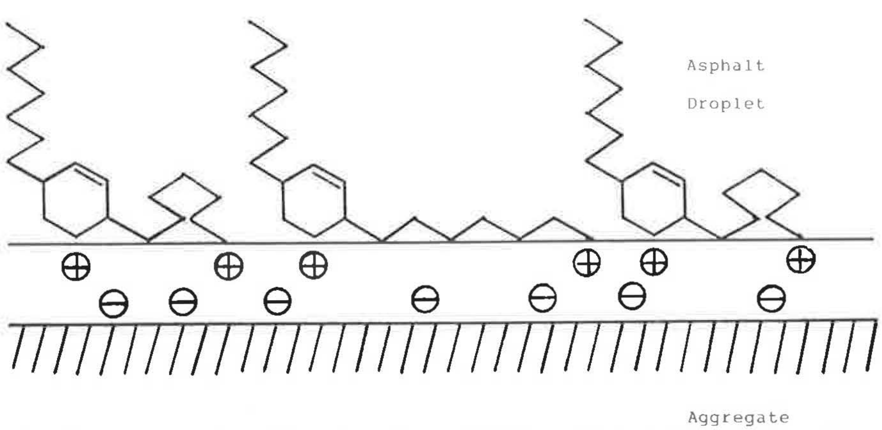


FIGURE 7 Arrangement of fatty acid diamidoamines in an asphalt droplet.

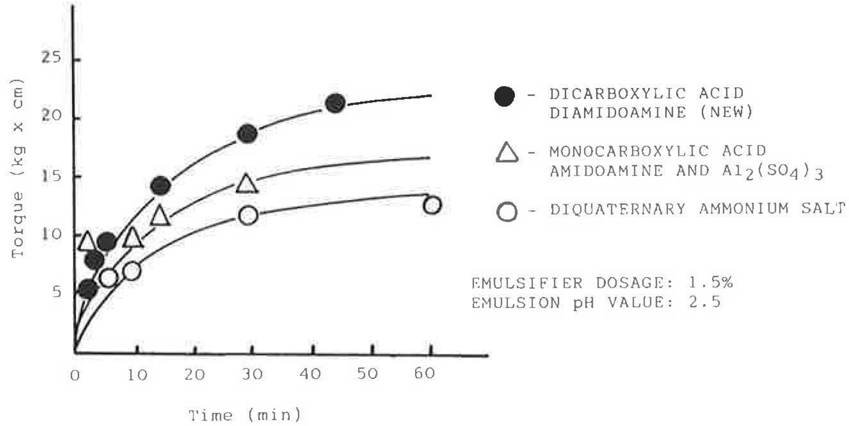


FIGURE 9 Cohesive strength development of cationic slurry seals.

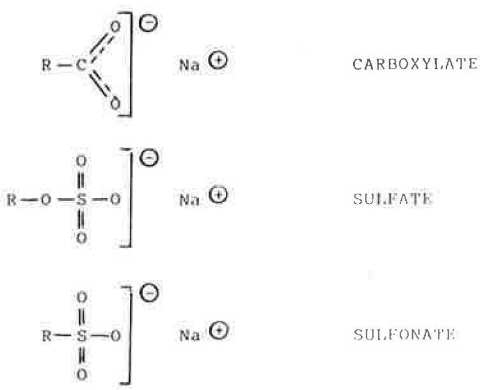


FIGURE 10 Anionic emulsifiers.

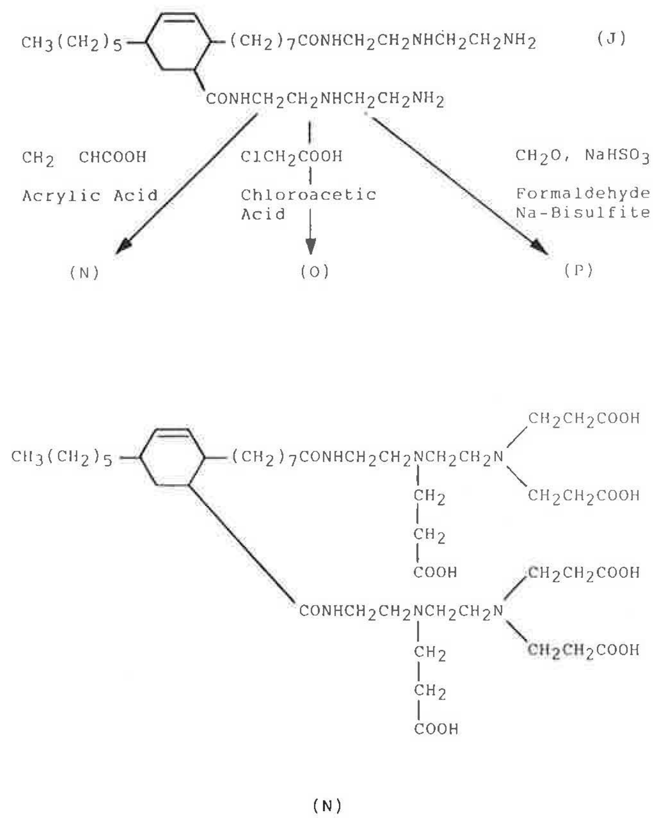


FIGURE 11 Aminoalkylcarboxylic acid.

CHOICE OF PROPER EMULSIFIER

The key to a successful slurry seal application is understanding the total system of emulsified asphalt, mineral aggregate, filler, and water. Therefore the contractor must know the properties of the emulsion and the chemistry of the aggregate. Because aggregate and asphalt sources change constantly, testing is required to identify the proper emulsion-aggregate combination. In this regard, the most flexible component in the system is the emulsifier. By finding the proper emulsifier and by optimizing dosage and pH value, asphalt emulsions can be adjusted to the aggregate so that the slurry will mix, set, and cure to the desired specification.

The factors responsible for the performance of asphalt-aggregate mixes are cohesive forces within the asphalt, adhesive forces at the asphalt-aggregate interface, and coalescence. A very high rate of coalescence and undesirably large cohesive forces result in poor mixing performance. When adhesive forces are greater than cohesive forces, premature breaking of the asphalt emulsion on the aggregate surface occurs. If the opposite is true, stripping of asphalt from the aggregate occurs.

Often slurry properties that actually oppose each other have to be achieved. For example, good mixing stability and rapid-setting characteristics are both desired. The rapid-setting characteristic is obtained by keeping the emulsifier concentration low, which makes mixing difficult. To prepare a conventional medium-setting emulsion, an increase in dosage level of the emulsifier yields the desired mixing stability. However, the break of the emulsion is undesirably slow.

To obtain the necessary mixing stability with aggregates containing an appreciable amount of fines, large amounts of emulsifier have to be used, but the emulsion is still expected to set in less than 30 min. The use of freshly crushed aggregates instead of weathered aggregate requires a very high emulsifier concentration. The use of emulsifiers based on tall oil derived di- and tricarboxylic acid can easily overcome many of these problems and produce stable slurries with the desired set times.

ROLE OF MINERAL AGGREGATES

About 75 percent of the volume of a slurry seal mix is mineral aggregate. Aggregates are generally characterized by physical

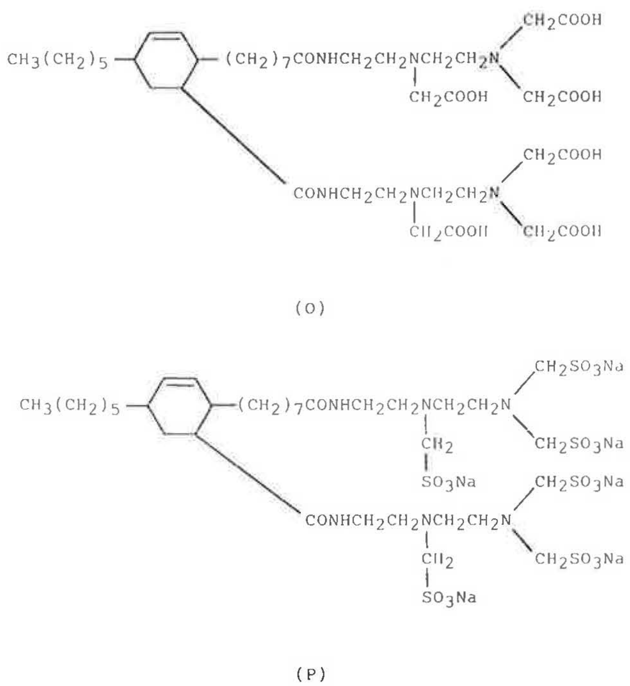


FIGURE 12 Aminoalkylcarboxylic acid and aminomethyl sulfonic acid.

$$R{-}NH(CH_2)_xCOOH + OH^{\ominus} \rightleftharpoons R{-}NH(CH_2)_xCOO^{\ominus} + H_2O$$

FIGURE 13 Reaction between aggregate surface and emulsion droplets.

and chemical testing for hardness, abrasiveness, absorbency, porosity, surface texture, and acidity. The role of the aggregate surface as a chemically reactive site has generally been ignored. The chemical reactions between the aggregate surface and the emulsion droplets determine critical properties of a slurry mix such as adhesion, cohesion, mix stability, compatibility, set, and cure time. Historically, emphasis was placed on the aggregate surface charge, which in the case of calcareous aggregate is electropositive and in case of siliceous aggregate is electronegative.

This may be true if the aggregate is perfectly dry. However, when wetted with water, both types of aggregates become negatively charged according to the reaction scheme shown in Figure 15.

Although the electrical charge is important for adhesion, for slurry seal mixes porosity, absorption of asphalt into aggregate pores, chemical reactivity, ion exchange, and density should be of greater concern to the formulator. Many of these properties can be altered with small amounts of chemically reactive additives to improve mixing and setting characteristics of the slurry seal.

CHEMICAL ADDITIVES—MIX AIDS

The ability of a slurry seal system to be modified by chemically reactive additives plays an integral role in fully using slurry seal technology. Such additives are generally incorporated to improve the strength of the slurry mat; improve the adhesion or cohesion; alter the electric charge of aggregate (partial charge neutralization); improve asphalt properties to achieve longer service life; and increase or decrease the mixing time, rate of setting, and curing time of the mix.

Typical aggregate additives include hydrated lime, portland cement (Types I and III), gypsum, or ammonium sulfate. A wide variety of chemical additives is used in the prewet water at concentrations of 0.01 to 0.05 percent based on the weight of the aggregate. Typical additives are aluminum sulfate, surfactants, or the same emulsifier solution used in the preparation of the emulsion. Fatty amines and latex elastomers are added to the asphalt to improve adhesion and low-temperature flexibility. Toughness and improved emulsifiability of the asphalt are other benefits given by asphalt additives such as fatty amines and tall oil fatty acids.

INERT FILLERS

Inert fillers such as limestone dust, sand, agricultural lime, or fly ash are added to increase the amount of fines in the mix. This is done to provide better compaction of the slurry seal.

SLURRY SEAL PERFORMANCE TESTS

There are many tests available to determine the performance of a slurry system, but most tests do not indicate the compatibility

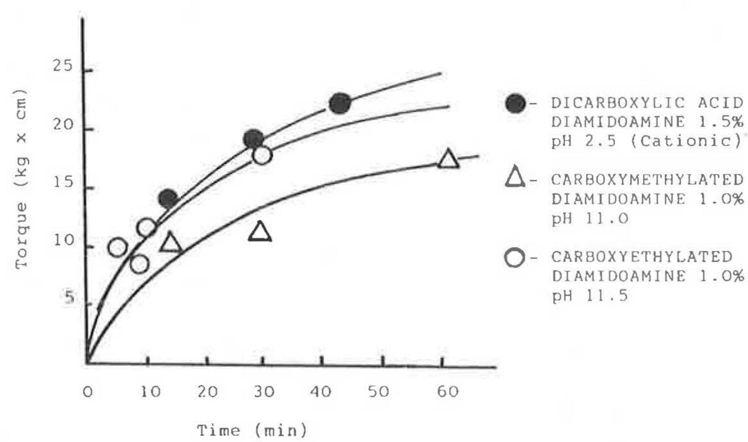
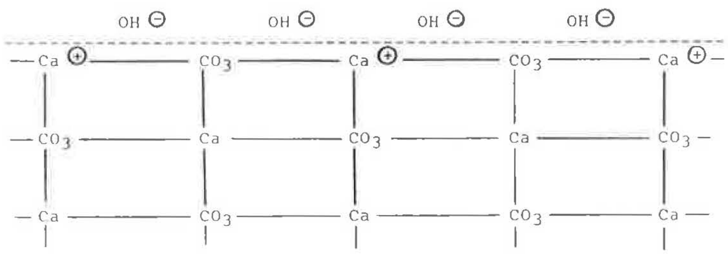


FIGURE 14 Cohesive strength development of anionic slurry seals.

Calcareous Aggregate:

$$CaCO_3 + H_2O \rightleftharpoons Ca^{2+} + 2OH^- + CO_2$$

(OH^- adsorbs onto Ca^{2+})



Siliceous Aggregate:

$$(SiO_2)_n + nH_2O \rightleftharpoons nH^+ + 2SiO_3H^-$$

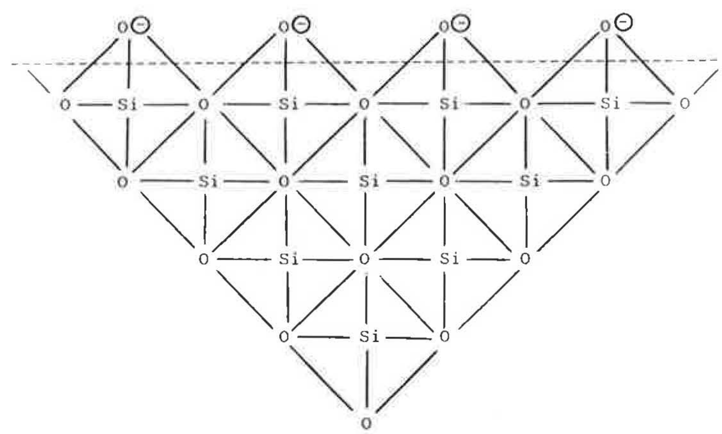


FIGURE 15 Reaction schemes for wetted aggregates.

of emulsion (asphalt) and aggregate in the slurry system. For this reason, the proposed boiling test is applied to cured slurry seal mix specimens. This test allows development of the best emulsifier formulation and simultaneously allows determination of the compatibility of all ingredients in the total system (see Appendix).

The data given in Table 1 indicate how the results of the boiling test correlate with cohesive strength development and the Wet Track Abrasion Test (WTAT) for four different slurry seals. The following conclusions can be drawn from this table:

- Emulsions prepared with a dicarboxylic acid diamidoamine can perform quite well with Camak aggregate, a highly charged aggregate.
- The addition of hydrated lime shortens the setting time (decreases the blotting time by 50 percent) while extending the mixing time.
- When the hydrated lime is replaced with cement, the adhesion is completely destroyed as determined by the boiling test. The increase in aggregate loss shown in the WTAT supports the validity of the boil test.
- The addition of aluminum sulfate extends the handling characteristics of the system while maintaining performance. Although cohesive strength development is slow, actual setting time in the field was about 15 to 20 min. No damage was done when a truck drove over the freshly laid slurry after only 30 min.
- Good overall performance is achieved with the addition of a natural latex.
- Unusual hydroplaning results in cohesion and loss in WTAT with the addition of an accelerator. Although excellent mixing results are obtained with Camak and some other aggregates, mixing time is decreased with other aggregates to less than a minute.

TABLE 1 CORRELATION OF BOILING TEST RESULTS WITH COHESIVE STRENGTH DEVELOPMENT AND WTAT

Additive (%)	Emulsion (%)	Boiling Test Retained Coat (%)	Blot Set (min)	Cohesion (cm · kgs) 15	30	60	90	WTAT (g/m^2)
No Additive								
–	16.0	100	90	14.5	14.5	12.5	16.5	535
–	18.0	100	90	13.0	14.5	16.0	15.5	348
–	19.0	100	105	12.0	14.9	13.5	15.0	243
Lime Addition								
0.5 lime	18.0	90	45	10.0	12.0	16.5	18.2	455
1.0 lime	18.0	100	45	9.5	12.2	13.5	17.0	345
Cement Addition								
0.5 Type III	18.0	20	45	10.5	9.5	11.2	12.0	1251
1.0 Type III	18.0	20	45	8.0	9.0	14.0	17.8	852
Alum Addition								
0.03 alum	18.0	100	90+	10.0	9.2	9.2	11.2	240
0.05 alum	18.0	90	90+	9.8	10.0	11.0	12.1	296
Natural Latex Addition								
1.8	18.0	100	90	9.8	9.8	10.4	10.8	124
1.8	14.0	100	75	9.8	11.5	10.4	10.4	200
Naural Latex + Accelerator								
2.0 accelerator	18.0	100	15	Hyd.	Hyd.	Hyd.	Hyd.	0
2.0 accelerator	14.0	98	15	Hyd.	Hyd.	Hyd.	Hyd.	145

NOTE: Aggregate = Type II granite, Camak, Georgia; emulsifier = dicarboxylic acid diamidoamine (1.5 percent dosage, pH 2.8); asphalt = West Coast AC-20. The hydrated lime and portland cement (Type III) were added to the aggregate. The alum (aluminum sulfate) was added to the prewet water; the concentration was based on the weight of the aggregate. The concentration of 1.8 percent natural latex was based on the solids of the elastomer. Hyd. denotes hydroplaning of the pressure foot on the surface of the test specimen.

These results clearly indicate that the addition of an additive may improve the performance of one slurry system but be deleterious in another. This indicates the complexity of these systems.

APPENDIX: PROPOSED BOILING COMPATIBILITY TEST

1. Sieve job aggregate according to the following specification: No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200.
2. Reconstitute a 100-g sample in the correct proportions as obtained from the sieve analysis.
3. Mix the emulsion, aggregate, and chemically active additive for 1 min (additives: 0.5 percent, 1 percent Type I or III cement; 0.5 percent, 1 percent lime; or 0.03 percent, 0.05 percent aluminum sulfate—all based on the aggregate weight). Higher concentrations of cement and lime may be used when required. (Aluminum sulfate is added to the prewet water.)
4. Place the template (Figure A-1) on aluminum foil.
5. Deposit the mix in the template and level the surface with a squeegee or spatula.
6. Allow the mix to air cure (70°F to 77°F) for 24 hr (minimum 12 to 15 hr).
7. Boil approximately 700 mL of water in a 1000-mL beaker. Use a No. 20 mesh screen in the bottom of the beaker to create a shelf. Bend the edges down so that the sample is raised ½ in. from the bottom of the beaker, avoiding direct contact of the aggregate and heat source. Place the sample in the boiling water. If the sample cannot be easily removed from the foil, cut the foil around the outside of the sample. Place the sample in the boiling water and remove the foil from the water (the boiling water should allow the foil to become detached).
8. Boil for 10 min.
9. Decant the water and spread the mix on a level surface.
10. Observe the coating of the mix.
11. Record the percent retained coating using 2 percent accuracy.
12. Determine mix compatibility:
    - 90 to 100 percent coated surface—good mix compatibility,
    - 75 to 89 percent coated surface—fair to good mix compatibility,
    - 50 to 74 percent coated surface—poor to fair mix compatibility, and
    - Less than 50 percent coated surface—poor mix compatibility.

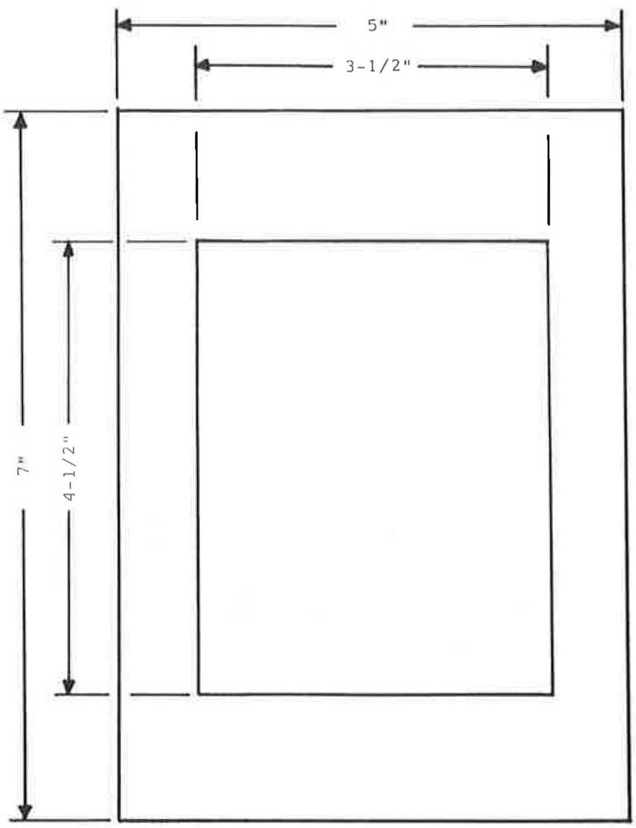


FIGURE A-1 Template (1/4 in. thick).

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Microsurfacing with Natural Latex–Modified Asphalt Emulsion: A Field Evaluation

C. M. "SWEDE" PEDERSON*, WILLIAM J. SCHULLER, AND C. DWIGHT HIXON

This paper presents an evaluation of the use of emulsified asphalt in a relatively new process called microsurfacing. The process was developed in Germany and was first used in the United States in late 1980. Natural latex rubber is incorporated into the asphalt emulsion and mixed with aggregate and other additives in a traveling pug mill similar to but larger than that of a regular slurry seal machine. The test section that was selected for microsurfacing is 3 mi of four-lane divided highway in an urban area. Construction was completed in June 1983. The data indicate that the service life of the test section has been enhanced. It is recommended that microsurfacing be approved for routine use in restoring flexible pavements to fill surface ruts and cracks, seal the surface, and restore skid resistance.

This is a report on the use of emulsified asphalt in a relatively new process called microsurfacing. This process was developed in Germany in 1976 and was first used in the United States in 1980. The original microsurfacing product, Ralumac, incorporates a natural latex into the asphalt emulsion. Microsurfacing consists of

- 2.0 to 4.0 percent latex base modifier (*1*, pp. 4–5) incorporated into the emulsion,
- 1.5 to 3.0 percent portland cement as mineral filler,
- 6.0 to 11.5 percent residual asphalt (the emulsion is 64 percent asphalt) (*1*, pp. 4–5), and
- 82 to 90 percent select aggregate (sand equivalency > 45).

In addition, variable amounts of water and emulsion stabilizer are added during laydown operations. These two additives combined are roughly equivalent to 9 percent of the mix.

The latex-modified asphalt emulsion is mixed with the aggregate and other additives in a traveling pug mill similar to but larger than that of a regular slurry seal machine (Figure 1). The laydown machine uses two different sizes of slurry spreader box. A 5.5-ft-wide box is primarily used for rut filling, and a 13-ft-wide adjustable box is used for surfacing. No roller compaction is required for either rut filling or surfacing.

The laydown machine is serviced by dump trucks that have been modified by the addition of two large tanks in their dump beds. These tanks carry emulsion or water. Each dump truck also carries a load of aggregate between the tanks. The laydown machine has enough on-board storage capacity to allow it to continue operating while servicing trucks are being switched.

FIGURE 1 Microsurfacing laydown machine with truck.

The only items that must be carried by the laydown machine are the portland cement and the set retardant.

The basic crew for operating the microsurfacing machine consists of six people, four on the machine and two following behind. The two following behind carry mops or squeegees and do minor hand work as needed. On the machine, one person adds the portland cement to a small hopper that meters the cement into the mix, a second person hooks up and monitors the service trucks, a third person drives the laydown machine, and a fourth person at the back of the machine controls the actual laydown operation. The operator is able to adjust the amounts of aggregate, portland cement, water, and set retardant going into the pug mill.

Variations are made as dictated by weather and roadway conditions. On dry, warm days, the operator adds more water and set retardant. On cool, overcast, or high-humidity days, the operator adds less water and set retardant. The quality of a finished microsurfacing project depends greatly on the skill of the operator and crew.

A copy of the current Oklahoma specification for microsurfacing may be obtained from the Research & Development Division of the Oklahoma Department of Transportation.

DEMONSTRATION PROJECT

In 1983 the Oklahoma Department of Transportation (ODOT), in cooperation with the FHWA, established a demonstration project to evaluate Ralumac microsurfacing. The evaluation project is located on US-64, a multilane, divided highway in Sand Springs, Oklahoma. Four distinct traffic volumes are carried over the 3.35-mi project. The four sections as distinguished by average daily traffic (ADT) are given in Table 1. The portion of US-64 covered in this study has the typical section shown in Figure 2.

*Deceased.
Research & Development Division, Oklahoma Department of Transportation, 200 N.E. 21st Street, Oklahoma City, Okla. 73105.

TABLE 1 TRAFFIC DATA

	MILEAGE	ADT	COMMERCIAL	COMMERCIAL OVERLOADS
Section I	0.00 - 0.47	23,200	2,552	10%
Section II	0.47 - 1.64	18,400	2,706	10%
Section III	1.64 - 2.00	36,600	4,806	10%
Section IV	2.00 - 3.35	66,100	7,271	10%

Road Condition Before Construction

At the time of microsurfacing, US-64 was 14 years old. The roadway had received only routine maintenance since its original construction. The roadway was badly cracked and had ruts as deep as 0.7 in. The load-supporting ability of the roadway as measured by Benkelman beam deflections was adequate. The very nature of the problems on US-64 pointed to microsurfacing as a workable solution. Restoration of the profile of the roadway and sealing of the roadway surface on an otherwise sound road were needed. These needs could be met by microsurfacing.

Tests on the Project

The entire length of the project was tested for surface friction values, rut depths, load-supporting ability, and ride quality. In addition to these tests, five 300-ft sections were evaluated for cracking. One of the sections in the eastbound lanes evaluated for cracking was treated with a 4-oz/yd^2 nonwoven fabric.

A further test performed on this project was the comparison of the designed microsurfacing application with a thick application of microsurfacing and with an application of hot-mix asphaltic concrete. Each of the three sections for this test consisted of 1,000 ft of roadway in the westbound lanes at the west end of the project. The nominal thickness of the designed microsurfacing treatment was 0.5 in., and that of the thick section of microsurfacing was 1.1 in. The hot-mix asphaltic concrete was laid 1.5 in. thick.

Construction

Work on microsurfacing US-64 began on June 16, 1983, and required 9 workdays to complete. An additional workday was required for laying the test section of 1.5-in.-thick hot-mix asphaltic concrete. During the 9 days of microsurfacing, a total of more than 1,770 tons of material were laid. The job mix formula used on this project is given in Table 2. The yields, as pounds of aggregate per square yard, are given in Table 3.

TABLE 2 JOB MIX FORMULA—AGGREGATE

Sieve Size	Job Formula	Required by Specifications
3/8"	100	99-100
No. 4	87	86-94
No. 8	65	45-65
No. 16	44	25-46
No. 30	30	15-35
No. 50	18	10-25
No. 200	8	5-15
Sand equivalent	77	45 minimum
L.A. Abrasion	19.3	40 maximum

Trial Mixes

Percent Asphalt	Specific Gravity	Hveem Stability
7.0	2.095	38
7.5	2.119	37
8.0	2.133	40

7.5 percent asphalt was recommended.

Note: Aggregate Type: mine chat.

TABLE 3 MICROSURFACING APPLICATION RATES USED ON US-64

Application	WESTBOUND Outside Lane	WESTBOUND Inside Lane	EASTBOUND Outside Lane	EASTBOUND Inside Lane
Rut Filling	29.44	none	35.00	none
Surfacing	15.90	21.20	15.90	21.40
Total	45.34	21.20	50.90	21.40

NOTE: Units are pounds of aggregate per square yard.

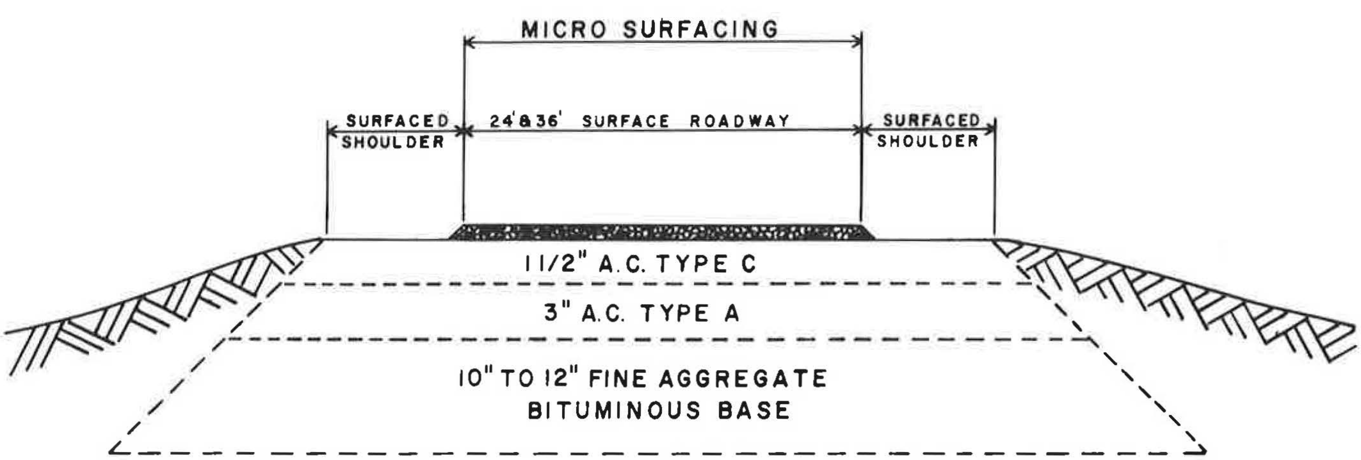

FIGURE 2 Typical cross section of US-64.

TABLE 5 CRACKING

300 Ft. Section	Before L.F.	Year 1 L.F.	Year 1 Pct	Year 2 L.F.	Year 2 Pct	Year 3 L.F.	Year 3 Pct
1	1735	364	21	649	38	526	30
2	1860	335	18	614	33	374	20
3	3170	761	23	983	31	568	18
4	3700	1008	27	1268	34	364	10
Fabric	1610	128	8	421	26	400	25

NOTE: L.F. = Linear Feet of cracking.

Pct = Each years cracking as a percent of the before data.

- Cracking in both microsurfacing sections was appreciably worse than in the 1.5-in. AC section. Both microsurfacing sections exhibited transverse, longitudinal, and random cracking. In addition, the thick microsurfacing section exhibited block cracking. The 1.5-in. AC section exhibited some transverse and longitudinal cracking.

Surface friction data were obtained before, 2 years after, and 3 years after microsurfacing. These data indicate no significant change in the surface friction values before and after microsurfacing. The average friction number for the total project length before microsurfacing was 48. Two years after microsurfacing, the average friction number was 49. Three years after microsurfacing, the average friction number was 45. Surface friction data were not available for 1 and 4 years after microsurfacing.

The ride quality of the microsurfacing 3 years after application, as measured by a Mays ride meter trailer, was at an acceptable level. The average present serviceability index was 3.2. The average inches of roughness per mile was 101.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions about the three 1,000-ft sections can be drawn after 4 years:

1. No benefit was realized from the application of the extra thick microsurfacing as opposed to normal microsurfacing.
2. There was no appreciable difference among the three test treatments in terms of their effect on the load-supporting ability of the roadway.
3. The 1.5-in. AC section resisted cracking better than either microsurfacing treatment.
4. Both microsurfacing treatments performed better than the 1.5-in. AC in resisting rerutting.

Results from the 300-ft fabric test section are inconclusive. ODOT currently has more than 22 mi of microsurfacing on fabric that were laid in 1987. Observation of these sections will be required before any conclusive statement can be made about the use of fabric under microsurfacing.

On the basis of the field data obtained for the entire project, the following statements can be made about the condition of the roadway 4 years after treatment with microsurfacing.

1. The load-supporting ability of the roadway, as measured by Benkelman beam, was generally improved over that measured before microsurfacing.
2. Rut depths were shallower overall than those measured before microsurfacing.
3. Data from the crack map sections indicate that the quantity of cracking after 4 years was below 50 percent of the quantity of cracking measured before microsurfacing.

The results of the field tests conducted on US-64 show that microsurfacing enhanced the life expectancy of the roadway. It is recommended that microsurfacing be used to restore flexible pavements that are rutting or cracking. Microsurfacing is not recommended for use on pavements that lack adequate load-supporting ability.

UPDATED STATUS OF MICROSURFACING

After completion of the evaluation of the demonstration project on US-64, ODOT used microsurfacing routinely on state-aid projects. However, it remained an experimental feature on federal-aid projects; evaluations of several projects are continuing. Several hundred lane miles of the natural latex system were in place by the end of the 1987 construction season, including sections of heavily traveled Interstate in the Oklahoma City area.

A second microsurfacing system, in which synthetic latex was used, was placed under evaluation in the spring of 1987. This system uses different emulsifying and set-retarding agents than those used in the natural latex system.

At the end of the 1987 construction season, 21 lane miles of the synthetic latex system were under evaluation. In one project on rural OK-77, synthetic latex was used on the entire 10 mi. In another project, the synthetic system and the natural system were placed end to end. This was a microsurfacing project on I-40 in Canadian County, Oklahoma, that included a 1/2-mi test section of the synthetic latex material within the 14-mi project in which natural latex microsurfacing was used. Evaluation of the microsurfacing products on these and other projects will continue at least through the early summer of 1988.

This paper has dealt with one paving system that uses a latex-modified asphalt emulsion. Latex and polymer modifiers are also in use with hot-mix asphalt, cold-mix asphalt recycling, and asphalt surface treatments. Modified asphalts are evolving and developing. It is certain that the years to come will bring several new paving materials composed of chemically modified asphalts.

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Development of Improved Mix and Construction Guidelines for Rubber-Modified Asphalt Pavements

H. B. TAKALLOU AND R. G. HICKS

Rubber-modified asphalt pavement mixtures have been used in Sweden and the United States since the 1970s. In these applications ground, recycled tire particles (1/4 in. minus) are added to a gap-graded aggregate and then mixed with hot asphalt cement. The benefits of adding rubber to the mix include increased skid resistance under icy conditions, improved flexibility and crack resistance, elimination of a solid waste, and reduced traffic noise. The major disadvantage of these rubber-modified mixtures is their high initial cost compared with conventional asphaltic concrete pavements. One such rubber-modified asphalt mixture used in the United States is described. The mix ingredients and typical properties are first presented. The requirements for adding and controlling one additional ingredient and for producing an unusual aggregate gradation (gap-graded aggregate) have resulted in construction problems on some projects. These problems can be avoided by proper specifications, controls, and inspection. In the last section of the paper guidelines for use of rubber-modified asphalt mixtures in cold, moderate, and hot environments are presented.

Ground tire rubber has been used as an additive in various types of asphalt pavement construction in recent years (*1*). The use of rubber is of interest to the paving industry because of the additional elasticity imparted to the binder. Resource recycling is an additional benefit of creating a use for waste tires. Each year the United States disposes of about 200 million passenger vehicle tires and 40 million truck tires (*2*). This represents a total of 4 million tons of scrap waste tires. Although a limited number of these 4 million tons of waste tires are used for resource and energy recovery, the vast majority go to landfills or are disposed of in an environmentally unacceptable manner (*2*).

In recent years, rubber-modified asphalt has come to the attention of Congress as a way of solving the ecological problems of disposing of discarded tires. Congress, to stimulate the use of recycled materials, requested the Environmental Protection Agency and the Federal Highway Administration to issue procurement guidelines. In response to the request, the February 20, 1986, issue of the *Federal Register* contains a proposed ruling by the Environmental Protection Agency for Federal Procurement of Asphalt Materials Containing Ground Tire Rubber for Construction and Rehabilitation of Paved Surfaces (*3*). The impact of this proposed guideline remains to be seen. However, many municipalities are currently evaluating the use of discarded tires to modify hot-mix asphalts for road surfacing (*4–8*).

Two different methods of incorporating ground tire rubber into paving mixes have been developed. The method of adding rubber to asphalt mixtures, which will be discussed in this paper, was originally developed in late 1960 in Sweden and patented under the trade names of "PlusRide" in the United States and "Rubit" in Sweden. In this system, rubber-asphalt mixtures are prepared by a process that typically uses 3 to 4 percent by weight relatively large (1/16-in. to 1/4-in.) rubber particles to replace some of the aggregate in the mixture (Figure 1). The benefits of adding rubber to the mix, besides elimination of rubber tire waste, are increased flexibility, resistance to studded tires, increased fatigue life, reduced noise, and crack reflection control. In addition, the increased elastic response of this material also reportedly causes ice formed on the pavement during freezing weather to break under transient vehicle loadings.

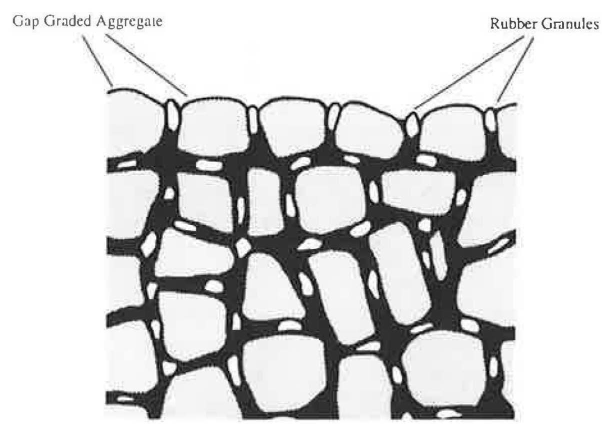

FIGURE 1 Illustration of rubber-modified asphalt (rubber granules are white).

The second type of rubber modification (not discussed here) uses finely ground rubber tire "buffings" that are mixed into the hot asphalt to create a "rubberized asphalt" binder, which is then added to a normal paving aggregate.

The purposes of this paper are to evaluate the use of one type of rubber-modified asphalt paving mixture in road construction and to develop guidelines that indicate how these mixes can best be used in U.S. road systems.

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CURRENT MIX AND CONSTRUCTION GUIDELINES

The rubber-modified asphalt mixture evaluated in this paper is prepared by a process that typically uses 3 percent by weight granulated coarse and fine rubber particles to replace some of the aggregate in the mixture. On the basis of experience in Alaska and Sweden, three different aggregate gradation bands have been recommended for different layer thicknesses to serve different traffic levels (Table 1).

TABLE 1 RECOMMENDED SPECIFICATIONS FOR RUBBER-ASPHALT PAVING MIXTURES FOR DIFFERENT LEVELS OF TRAFFIC (9)

	Mix Designation		
	A	B	C
Average daily traffic	2,500	2,500–10,000	10,000
Minimum thickness (in.)	1.0	1.5	1.75
Sieve size (% aggregate passing)			
3/4 in.			100
5/8 in.		100	
1/2 in.			
3/8 in.	100	60–80	50–62
1/4 in.	60–80	30–44	30–44
No. 10	23–38	19–32	19–32
No. 30	15–27	13–25	12–23
No. 200	8–12	8–12	7–11
1/4-in. to No. 10 size fraction		12 max	12 max
Preliminary mix design criteria			
Rubber, % of total mix by			
Weight	3.0	3.0	3.0
Volume (approx.)	6.7	6.7	6.7
Asphalt (% of total mix by weight)	8–9.5	7.5–9.0	7.5–9.0
Maximum voids (%)	2.0	2.0	4.0

A review of aggregate grading specifications reveals some significant differences between these modified and conventional paving mixtures. The most important difference is indicated by the comparative shapes of the aggregate gradation curves (Figure 2). To provide space for the rubber particles, it is necessary to create a "gap" in the gradation curve of the aggregates, primarily in the 1/8- to 1/4-in. size range. The rubber particles replace a portion of the rock particles that normally occupy this size range.

The rubber particles used in these mixes are specified to be produced in "roughly cubical form" by grinding waste tires, which have first had the steel wires in the tire bead area removed. The rubber may include some tire cord and steel fibers from tire belts and must meet the gradation specifications given in Table 2.

The paving grade asphalt is the same for the rubber-asphalt mixture as for conventional mix. However, rubber-asphalt mix typically requires from 1 1/2 to 2 percent more asphalt than does conventional mix.

Mix Design Considerations

Mix designs for rubber-modified asphalt mixtures are normally arrived at by using the Marshall or Hveem method; however,

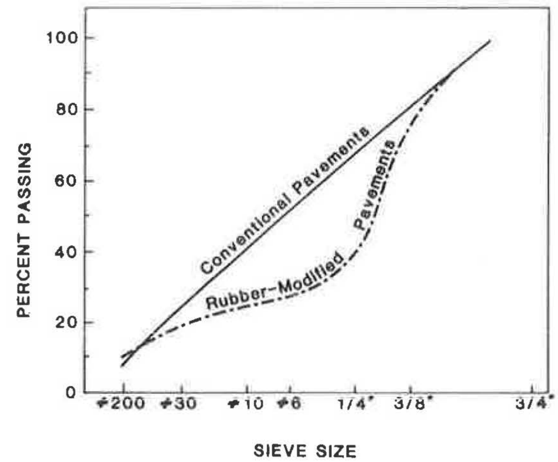

FIGURE 2 Comparative aggregate gradation curves for conventional and rubber-modified asphalt pavements.

TABLE 2 PARTICLE SIZE SPECIFICATION FOR RUBBER (9)

	Percentage Passing		
Sieve Size	Coarse Rubber	Fine Rubber	80/20 Rubber Blend[a]
1/4 in.	100		100
No. 4	70–90		76–92
No. 10	10–20	100	28–36
No. 20	0–5	50–100	10–24

[a]The 80/20 is 80 percent coarse and 20 percent fine rubber in combination.

the criteria for selecting the asphalt content are different for conventional hot-mix asphaltic concrete and rubber-modified asphalt pavements. Most engineers use Marshall stability, flow, cohesion, air voids, and density as criteria for designing conventional hot-mix asphaltic concrete pavements. However, stability values for rubber-asphalt mixes are lower than values obtained for typical asphalt mixes. The flow values for rubber-modified mixes are generally greater than the maximum allowable in asphalt mix design criteria (*10*). Consequently, stability and flow values for rubber-modified mixes may give guidance only in terms of their relative position on design curves, and different criteria should be developed as performance indicators for rubber-modified mixtures.

Experience has shown that the critical factor for successful rubber-modified asphalt installations has been a low percentage of voids in the total mix (*10*). For example, pavements placed in Alaska with low void contents (approximately 4.6 percent) and exhibiting satisfactory performance had stabilities as low as 350 lb and flows of up to 0.19 in. (*10*). In general, the laboratory air voids are recommended to range from 0 to 4 percent maximum depending on the traffic level of the facility being designed (*10*):

- Low traffic—2 to 3 percent,
- Medium traffic—3 percent maximum, and
- High traffic—4 percent maximum.

This required void content is achieved by increasing both the mineral filler and the asphalt cement content until the target value is reached (*10*).

Construction Considerations

Aggregate Production

The most common problems with project batching of acceptable rubber-modified asphalt mixes have been achieving the proper gap in the grading curve and obtaining sufficient fines (No. 200 minus) to serve as a void filler. The lack of mineral filler in the mix causes high air voids and this is of concern to the road agency. Contractors can achieve the (No. 200 minus) requirement by adding baghouse fines or introducing filler such as Cottrell flour, fly ash, limestone dust, or one of several other types of mineral filler (*11*). The percentage by weight of total aggregate for the additional filler material has varied from 2 to 9 percent with an average of 5.3 percent as determined from 15 project summaries (*11*).

Mix Production

Batch, continuous, and drum-dryer plants have been used for mix production (*12, 13*). The experience of the Alaska Department of Transportation and Public Facilities (DOT&PF) indicates that a batch mixing plant is preferable because the required quantities of rubber, asphalt, and aggregates can be measured exactly and added separately to the pug mill or mixing chamber. In this type of plant, preweighed and sacked rubber can be used to advantage, with quantity control by bag count. However, both continuous mix and drum-dryer mix asphalt paving plants have been used without difficulty. In these plants the mixing operation goes on continuously instead of in batches, and the rubber must be added from a separate bin with a belt feed to maintain uniformity. Control in this type of feeding is less accurate. Two additional disadvantages of drum-dryer plants have also been reported. The first problem is the potential for producing smoke, noted on the Lemon Road project in Juneau, Alaska, on a single-entry drum mixer. On this project the flame heat shield had been removed from the drum. Drum mixers set up for the production of recycled mix have generally proven satisfactory. The best drum plants are the double (mid) entry type that allow the rubber to be added in the center of the mixing drum. The second problem occurred when a contractor decided to lower the mixing temperature from 325°F to 305°F. At the lower temperature, asphalt mix began sticking to the flights, which caused the trunnion to slip with the increased load. The slippage was also due to some rubber granules blowing from the feeder belt onto the trunnion. The problem was corrected by cleaning the trunnions and elevating the mix temperature back to 325°F.

Laydown

The laydown of the hot mix must be performed by paving machines equipped with full-width vibratory screens to aid in compaction (*12*). The laydown machinery used includes both hopper and pickup types (*12–14*). Alaska DOT&PF also made one attempt to place the mix by using a motor patrol after end dumping the material (*12*). The mix placed by the grader was too sticky to be easily leveled.

Handwork (such as raking longitudinal joints and placing radii) for the rubber-modified asphalt mixes is affected by the mix gradation and temperatures. According to contractors, the best result of handwork was observed when the mix was at normal laydown temperatures (300°F to 320°F) (*11*).

Compaction

Conventional compaction equipment has been used to roll the rubber-modified asphalt mix. The breakdown rollers are typically 10- to 12-ton vibratory steel drum units (*12–14*). The intermediate and finish rollers are also steel drum units; but they are not always required to be vibratory, nor are they as heavy. Rubber-tired rollers are not recommended according to Swedish engineers. However, experience with rubber-modified asphalt placed in Vancouver, British Columbia, and Anchorage, Alaska, in 1981 indicates that significant surface tightening might be achieved by use of rubber-tired rollers after the mix has cooled below 140°F (*10*).

Current practice is to avoid use of rubber-tired rollers because rutting and pickup problems can occur too easily. Rubber-modified asphalt mix being picked up by the rollers has been reported by several agencies (*12, 13*). The methods used by contractors to prevent or reduce pickup are (*6, 12, 13*)

- Removing rubber-tired rollers from the rolling pattern;
- Making sure all water nozzles are fully operational;
- Using liquid detergent in the drum water; and
- Using a specialty wetting agent, Dewko wetting concentrate, in the drum water.

The most successful method appears to be a combination of making sure that the wetting system is fully operational and including some liquid soap with the drum water (*11*).

REVIEW OF PRIOR PROJECTS

From 1979 to 1987 this process was used in approximately 52 applications throughout the United States. Table 3 gives a summary of the number of tons of rubber-modified asphalt mix placed in the United States.

TABLE 3 SUMMARY OF RUBBER-MODIFIED ASPHALT PROJECTS IN THE UNITED STATES

Year	No. of Projects	Tons of Mix
1979	1	90
1980	1	1,700
1981	4	3,000
1982	8	5,867
1983	6	15,886
1984	7	18,883
1985	14	20,315
1986	11	38,370
Total	52	104,111

As part of the study for Alaska DOT&PF, a survey questionnaire on the performance of these mixes (*15*) was sent to various transportation agencies that had used the rubber-modified asphalt mixes. The questionnaire was designed to obtain the following key items of information from these agencies:

1. Project location and agency in charge;
2. General data, including tons mixed and thickness of paving;
3. Rubber and asphalt content;
4. Construction data and problems encountered;
5. Overall performance and any problems noted;
6. Reasons for using rubberized asphalt; and
7. Project's condition (1987).

A total of 20 experimental projects constructed between 1979 and 1986 were evaluated using the survey questionnaire. Tables 4–6 give summaries of the results of these surveys. As noted, almost all of these projects encountered some difficulties in the construction or performance, or both, of the mix. Many of the performance problems appeared to be related, at least indirectly, to the construction methods used. In a few cases construction was reportedly hampered by "sticky" mixes, which can be attributed to the added rubber. The stickiness appeared to make joint construction difficult. This may have led to reduced rolling and high voids and contributed to early mix raveling. Other possible causes of performance problems included (a) incomplete mixing, (b) excess or insufficient asphalt, (c) high voids, (d) low p-200 content, and (e) erratic rubber content of mix.

TABLE 4 SUMMARY OF MIX DESIGN SURVEY QUESTIONNAIRE

	Average	Range
Asphalt content (%)	7.7	5.0–9.5
Rubber content (%)	3.0	2.5–4.0
Mix temperature (°F)	330	285–360
Total mix time (sec)	30	14–45
Compaction temperature (°F)	320	200–300
Voids in mix (%)	4.8	0.5–12.0

TABLE 5 SUMMARY OF PAVEMENT PERFORMANCE SURVEY QUESTIONNAIRE: PRESENT CONDITION OF RUBBER-MODIFIED ASPHALT MIXES (eight agencies reporting)

	Pavement Condition		
	Severe	Moderate	None
Raveling	1	1	6
Bleeding	0	2	6
Potholing	0	3	5
Wheel track rutting	0	0	8
Cracking	0	0	8

Deicing benefits have been reported by several agencies including the Alaska and Minnesota departments of transportation. Finally, stopping distance tests by the Alaska DOT&PF Research Section showed an average reduction in icy-road stopping distance of 25 percent on rubber-modified pavements for 23 test days over a 3-year period in the Fairbanks area, and a 19 percent reduction in the Anchorage area.

TABLE 6 SUMMARY OF PAVEMENT PERFORMANCE SURVEY QUESTIONNAIRE: OTHER PAVEMENT PERFORMANCE OBSERVATIONS (eight agencies reporting)

Pavement Performance	Noted	Not Noted	Not Evaluated
Ice control	1	6	2
Noise control	4	4	0
Reflective crack control	4	1	3
Skid resistance	3	2	3
Fatigue resistance	3	3	2

EVALUATION OF MIX PROPERTIES

A laboratory study was performed to evaluate the effect of mix variations on properties of rubber-asphalt mixes. The asphalt cement (AC-5 produced by Chevron, USA's Richmond Beach Refinery, primarily from Alaskan North Slope crude) and aggregate (crushed river gravel from Juneau, Alaska) used in this study were obtained from Alaska DOT&PF (Table 7). The recycled rubber was provided by Rubber Granulators in Everett, Washington.

TABLE 7 AGGREGATE GRADATION AND CORRESPONDING SPECIFICATION FOR B MIX

Sieve Size	Percentage Passing: Gap Graded	Percentage Passing: Dense Graded	Specification for B Mix
3/4 in.		10	
5/8 in.	100		100
3/8 in.	70	76	60–80
1/4 in.	37		30–44
No. 4		55	
No. 10	26	36	19–32
No. 30	18		13–25
No. 40		22	
No. 200	10	7	8–12

The two general types of tests used in this study were mix design tests and mix properties tests. The Marshall mix design procedure was used to determine optimum asphalt contents for the different mix combinations. When the optimum asphalt contents had been determined for the different mix combinations, the resilient modulus and fatigue life tests were used to evaluate mix properties.

Mix Design Results

The laboratory mix design results show that the asphalt content required to reach a certain minimum voids level for rubber-modified mixes depends on aggregate gradation, rubber gradation, and rubber content (Table 8). Coarse rubber is defined as rubber particles from ambient-temperature grinding of old tires, of which 80 to 90 percent is in a sieve size range from No. 10 to 1/4 in. The remaining rubber content is tire buffings, primarily in a sieve size range from No. 40 to No. 10. The laboratory results show that the mixture with gap-graded aggregate and 3 percent coarse rubber required the highest design asphalt content (9.3 percent) based on dry aggregate weights. Reducing the rubber content to 2 percent resulted in a reduction in the optimum asphalt content to 8.0 percent. The mixtures with 3 percent coarse rubber and dense aggregate grading required 7.5 percent, and conventional asphalt mix (no rubber) had the lowest design asphalt content (5.5 percent). The design asphalt contents reported were the asphalt contents required to reach the 2 percent air voids level (*16*).

TABLE 8 RECOMMENDED ASPHALT CONTENT AND MIX PROPERTIES AT 2 PERCENT AIR VOIDS (*13*)

Aggregate Gradation	Rubber Content (%)	Rubber Gradation (% coarse/ % fine)	Design Asphalt Content (%)	Marshall Stability (lb)	Flow (0.01 in.)
Gap graded	2	0/000	7.0	920	15
		60/40	7.2	690	21
		80/20	8.0	665	23
	3	0/100	7.5	600	19
		60/40	7.5	650	22
		80/20	9.3	436	33
Dense graded	0	No rubber	5.5	1,500	8
	3	80/20	7.5	550	22

Modulus and Fatigue Results

To evaluate the effect of mix variations on the behavior of rubber-modified asphalt, 20 different mix combinations (Table 9) were tested for diametral modulus (ASTM D 4123) and fatigue at two different temperatures (+10°C and −6°C) (*13*). The mix variables included two void contents, two rubber contents, three rubber gradations, two mix temperatures, two cure times, and use of surcharge. The test results on mix properties show that the modulus and fatigue of rubber-modified asphalt mixes depend on rubber gradation, aggregate gradation, and rubber content.

A summary of the resilient modulus and fatigue life test results at −6°C is given in Table 10. The mix properties test results show that the mixtures with the finer rubber gradations had higher resilient modulus (MR) and lower fatigue life (N_f) values than did mixtures with coarser rubber gradations. In addition, the aggregate gradation affects the mixture properties. The dense-graded aggregate has a higher modulus value. The effect of fatigue on aggregate gradation at two different temperatures (+10°C and −6°C) was reversed. At −6°C the fatigue life was less for mixes with gap-graded aggregate than for mixes with dense-graded aggregate. This unusual performance is mainly due to behavior of rubber particles in the mixture. At +10°C the rubber particles act more as elastic aggregate. However, at −6°C the rubber particles lose some of their elasticity and may work as weak aggregate in the gap-graded mixture. Reducing the rubber content to 2 percent also resulted in higher resilient modulus and lower fatigue life values compared with mixes with 3 percent rubber content.

The findings of this study indicated that rubber gradation, rubber content, and aggregate gradation have a considerable effect on mix design asphalt content, fatigue life, and modulus value. The study also showed that the rubber-modified mixes had a much greater fatigue life than a conventional asphalt concrete mix (*16*).

Creep Behavior

To evaluate the effect of mix variables such as aggregate gradation and rubber gradation on creep behavior, the regression lines for all five mix combinations were compared (Figure 3). In general, the slope of the regression lines for mixes containing rubber are sharper than those for mixes with no

TABLE 9 SPECIMEN IDENTIFICATION (*16*)

Specimen	Rubber Content (%)	Rubber Blend (% fine/ % coarse)	Mixing/ Compaction Temperature (°F)	Asphalt Content (%)	Aggregate Gradation	Cure Time (hr)	Surcharge (lb)
A	3	80/20	375/265	9.3	Gap	0	0
B	3	80/20	375/265	9.3	Gap	2	0
C	3	80/20	375/265	9.3	Gap	0	5
D	3	80/20	425/265	9.3	Gap	0	0
E	3	80/20	425/265	9.3	Gap	2	0
F	3	80/20	425/265	9.3	Gap	0	5
G	3	80/20	375/210	9.3	Gap	0	0
H	3	60/40	375/265	7.5	Gap	0	0
I	3	0/100	375/265	7.5	Gap	0	0
J	3	80/20	425/210	9.3	Gap	0	0
K	2	80/20	375/265	8.0	Gap	0	0
L	2	60/40	375/265	7.2	Gap	0	0
M	2	0/100	375/265	7.0	Gap	0	0
N	3	80/20	375/265	7.5	Dense	0	0
O	3	80/20	375/265	7.5	Dense	2	0
P	3	80/20	375/265	7.5	Dense	0	5
Q	3	80/20	425/265	7.5	Dense	0	0
R	3	80/20	425/265	7.5	Dense	0	0
S	3	80/20	375/210	7.5	Dense	0	0
T	0	No rubber	375/265	5.5	Dense	0	0
U	3	0/100	375/265	7.0	Dense	0	0

TABLE 10 SUMMARY OF RESILIENT MODULUS AND FATIGUE LIFE (16)

Mix	No. of Samples Used in Calculations	Average Value of Air Voids		Average Value of MR		N_f	
		Percentage	SD	ksi	SD	Average Value	SD
A	3	2.17	0.06	1,872	27	29,237	3,629
B	3	2.19	0.12	2,044	128	29,736	2,991
C	3	2.18	0.08	2,084	83	25,070	7,600
D	3	2.14	0.08	2,165	18	22,515	1,504
E	3	2.09	0.03	2,149	52	24,174	1,996
F	4	2.13	0.12	2,047	58	20,768	3,887
G	3	4.08	0.27	1,713	194	46,751	20,326
H	3	2.05	0.08	2,356	175	47,990	256
I	4	2.24	0.09	2,149	74	41,194	5,471
J	3	4.02	0.17	1,787	113	43,271	4,617
K	3	2.12	0.07	2,351	50	89,062	7,012
L	3	2.22	0.05	2,488	127	75,325	4,920
M	2	2.33	0.16	2,588	34	41,788	2,075
N	3	2.22	0.19	2,414	212	118,186	15,670
O	3	2.15	0.24	2,592	161	97,032	18,825
P	3	2.21	0.09	2,225	100	84,153	5,007
Q	3	2.12	0.05	2,116	94	93,651	4,198
R	3	2.02	0.11	1,939	133	81,141	8,354
S	3	4.50	0.23	1,443	177	127,682	24,996
T	3	2.25	0.13	3,163	133	15,536	2,562

NOTE: SD = standard deviation. Specimen U was not tested for MR and N_f. Test temperature was −6°C; strain level was 100 microstrain.

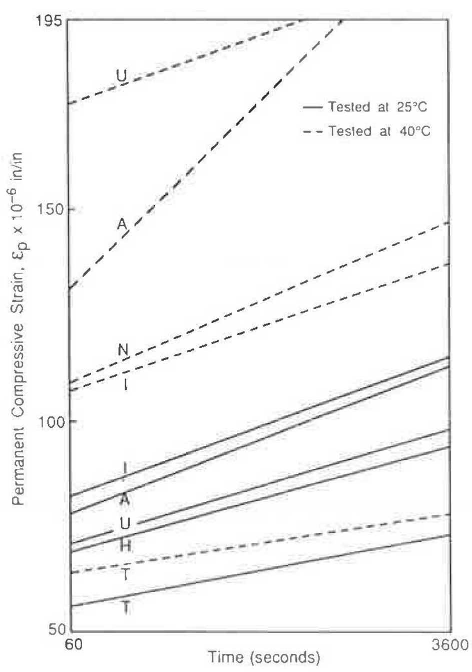

FIGURE 3 Creep behavior of rubber-asphalt mixes.

rubber. Also the intercepts for all rubber-asphalt mixes are at higher values than for mixes with no rubber at both temperatures. These results indicate that the rubber-asphalt mixes have lower creep resistance than the mixes with no rubber.

Among rubber-asphalt mixes, the mix with gap-graded aggregate and coarse rubber (80/20) has the steepest slope, and the dense-graded mix with fine rubber (0/100) has the flattest slope at 40°C. This indicates that the fine rubber improves the creep resistance of rubber-asphalt mixtures. However, there are slight differences among the slopes of all rubber-asphalt mixes, which indicate that the rubber-asphalt mixes have high elasticity.

Permanent Deformation Results

Permanent deformation rates were determined for five different mix combinations. Specimens were tested by cyclic load testing at 100 microstrain (0.01 percent) in a controlled environment at 15°C. Total vertical deformation was measured using a dial gauge accurate to 10^{-3} in.

The test results indicate that the control mix (mix with no rubber) has the steepest slope and the gap-graded mix with 3 percent coarse rubber (80/20) has the lowest slope. In general, all rubber-asphalt mixes have flatter slopes than the control mix (Figure 4). This indicates that the rubber-asphalt mixes have highly elastic behavior.

GUIDELINES FOR USE OF RUBBER-MODIFIED MIXES

On the basis of the results of this study and work by Monismith (17), the following guidelines are suggested for use with rubber-modified mixes.

Mix Design Guideline for Hot Climates

For pavements in hot climates (maximum ambient temperature greater than 100°F) that are subjected to large numbers of heavy vehicles or vehicles operating at high tire pressures, or both, rutting may be a controlling factor in mix design. Suggested steps in the mix design process to mitigate rutting are

1. Use rubber-modified asphalt as a thin overlay layer not as a structural layer.

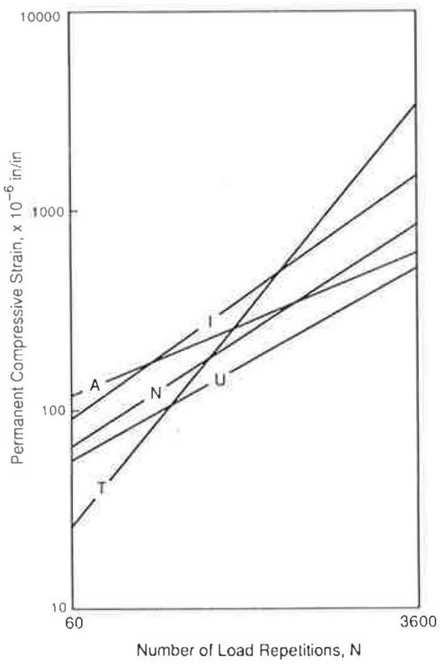

FIGURE 4 Relation between number of load repetitions and vertical strain.

2. The minimum rubber-asphalt layer thickness should not be less than 1 in.
3. Use the same grade of asphalt as is used in conventional cement asphalt pavements.
4. Use rough texture aggregate and Mix A (Table 1) gradation with maximum 9 percent No. 200 filler.
5. Use 3 percent medium rubber (60 percent coarse/40 percent fine, Table 2).
6. Mixing temperatures in the range of 350°F to 375°F and compaction temperatures of from 300°F to 285°F are desirable.
7. Mix the rubber with the aggregate before adding the asphalt.
8. Cure the rubber-asphalt mixture before compaction in an oven (375°F to 350°F) for 1 hr.
9. A preliminary design asphalt content should be selected on the basis of air voids (note that the mix should have an air void content of approximately 3 percent).
10. Determine stiffness of mix at short times of loading (0.1 sec) for expected range in temperatures. Stiffness values should not be less than 300,000 psi at 77°F and 0.1-sec loading time.
11. Perform creep tests on representative specimens to define stiffness of mix as a function of time at 25°C (77°F) and 40°C (100°F). Use 0.5 in. as the criterion for rutting analysis.
12. If the analysis indicates that rutting is at an undesirable level for the expected conditions, the mix must be redesigned and the analysis repeated. The use of fine rubber (0 percent coarse/100 percent fine) can be considered.
13. If the mix is considered suitable, its fatigue performance should be checked.

Mix Design Guidelines for Moderate Climates

For pavements in moderate (maximum ambient temperature of 100°F) climates that are subjected to large numbers of heavy vehicles, fatigue may be a controlling factor in mix design. The following steps represent an approach that can be taken:

1. The minimum rubber-asphalt layer thickness should not be less than 1.5 in.
2. Use the same grade of asphalt as is used in conventional asphalt pavement.
3. Use gap-graded aggregate Mix B (Table 1).
4. Use 3 percent coarse rubber (80 percent coarse/20 percent fine).
5. Mixing temperatures in the range of 320°F to 350°F and compaction temperatures of 300°F to 320°F are desirable.
6. Determine stiffness of mix at short times of loading (0.1 sec) for expected temperatures. Stiffness values should not be less than 250,000 psi at 77°F and 0.1-sec loading time.
7. A preliminary design asphalt content should be selected on the basis of air voids (note that the mix should have an air void content of approximately 3 percent).
8. For all expected traffic and temperature conditions, and for the anticipated range of stiffness (and aging) characteristics of the other rubber or aggregate gradations, perform a fatigue analysis.

Mix Design Guidelines for Cold Climates

In cold climates (minimum ambient temperature of 0°F), low-temperature response will govern the initial selection of mix characteristics. The following steps are suggested for cold climates:

1. The minimum ruber-asphalt layer thickness should not be less than 1.5 in.
2. The rubber-asphalt mixture can be used as an overlay as well as a structural layer.
3. Use the same grade of asphalt cement as is used in conventional asphalt pavement.
4. Use gap-graded aggregate (Mix B or Mix C, Table 1).
5. Use 3 percent coarse rubber (80 percent coarse/20 percent fine) (Table 2).
6. Mixing temperatures in the range of 300°F to 330°F and compaction temperature of 265°F to 300°F are desirable.
7. A preliminary asphalt content should be selected on the basis of air voids (note that the mix should have an air void content of approximately 3 percent).
8. Determine stiffness of mix at short times of loading (0.1 sec) for the expected range of temperatures. Stiffness values should be less than 180,000 psi at 77°F and 0.1-sec loading time.

CONCLUSIONS

On the basis of the results of a laboratory study at Oregon State University and evaluation of the field performance of the pavements made with rubber-modified asphalt mixes, the following conclusions appear warranted:

1. The field survey indicated that most rubber-modified pavements placed to date have not failed in fatigue. Where performance problems have been reported, they have generally been early raveling, or bleeding attributed to excessive voids variation resulting from poor compaction or low or high asphalt contents, or both.

2. Rubber-modified asphalt mixture is more susceptible than conventional mixtures to problems in preparation and compaction because of the need to add and control a third ingredient that has a major effect on overall mix properties and performance. Added plant inspection and increased compaction control efforts are necessary to assure a consistent product. Overall, placement is similar to conventional mixture placement.

3. The laboratory mix design results show that the asphalt content required to reach a certain minimum voids level for rubber-modified mixes depends on rubber and aggregate gradation and rubber content. The asphalt demand of these mixes is much more sensitive to rubber content variations than it is to variation in aggregate size.

4. The increased laboratory fatigue life of these mixes should be further evaluated by comparative field evaluations of underdesigned or overloaded pavement structures.

5. Finally, on the basis of the results of laboratory and field performance of rubber-modified mixes, mix design guidelines for use of these materials in hot, moderate, and cold climates were developed.

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Field Trials of Plastic- and Latex-Modified Asphalt Concrete

K. B. KRATER, D. L. WOLFE, AND J. A. EPPS

Test strips of pavement, which contained seven combinations of a polyolefin (plastic) and a styrene-butadiene rubber, were laid in five states. The states were selected on the basis of climate so that the projects would represent the majority of climatic conditions encountered in the continental United States. Data were collected from preconstruction testing on laboratory-compacted samples and from pavement condition surveys of each site. Data on construction techniques were recorded, and postconstruction data from both field-mixed and laboratory-compacted samples and cores were collected. Testing included resilient modulus tests at 10°F, 34°F, 77°F, and 104°F; Hveem stabilities; tensile strength tests at 10°F and 77°F; and one cycle of the Lottman accelerated conditioning procedure. Preliminary conclusions are that construction using plastic and latex presents no major problems. The addition of plastic and latex in certain combinations will increase resistance to rutting while not increasing thermal cracking at low temperatures. These modified pavements offer better resistance to moisture susceptibility than does pavement with no additives. Finally, these improvements are highly asphalt source dependent.

The poor performance and premature failures of some asphalt paving mixtures in this country have led many field engineers to believe that asphalt cements, and thus asphalt concrete mixtures, have changed over time. Many engineers and field personnel attribute poor pavement performance to the oil companies' taking the "goodies" out of asphalt cement and using them as feedstock for the petrochemical industry. Another belief is that increases in construction and transportation costs have caused a decline in the quality of asphalt cement and asphalt concrete roadways. Field engineers point to several types of pavement distress as evidence of these concerns, including placement difficulties, excessive displacement under traffic (rutting), thermal cracking and raveling, and poor fatigue behavior. Results of these problems are higher maintenance costs, more frequent traffic interruptions during repairs, shorter times between major rehabilitations, and increased numbers of complaints by motorists.

Although the declining quality of asphalt cements is a logical explanation of the poor performance of roadways, several other possible causes must be considered. Greatly increased traffic volumes and loads, along with higher tire pressures, have occurred in the last 10 to 15 years, which has placed an increased demand on roads not originally designed for this magnitude of traffic (*1*). Also, construction equipment has been developed to improve production, air quality, and worker safety, causing an increased demand on materials. Increased transportation and construction costs allow less maintenance and rehabilitation to be done under local budgets that have not gone up proportionally, and quality aggregates are becoming more costly and harder to obtain in many locations where readily available supplies have already been used up (*2, 3*).

The modification of asphalt cements might alleviate several pavement performance problems (*4*). Although construction quality control, improved specifications, asphalt mix designs that correlate with field construction methods, and more accurate pavement design methods need to be examined, improved binder systems offer the potential for substantial improvements in binder systems and roadway performance.

Several asphalt modifiers exist today to improve one or more properties of asphalt cements or asphalt concrete mixes, or both. These modifiers include organic materials, metal compounds, fibers, fillers, sulfur, lime, elastomers, and polymers. Although considerable research has been performed on these modifiers in the laboratory, few field evaluations have been made. This paper gives results from several asphalt overlay projects, constructed in the United States, in which a "functionalized" polyolefin and a styrene-butadiene rubber (SBR) were used. These modifiers were incorporated into the mixtures in seven different combinations.

BACKGROUND

The goal of this research was to construct test strips around the country using two polyolefins (plastics) and an SBR (latex). Two plastics, chosen from an initial screening of twenty, were selected on the basis of mixture properties and high-temperature performance.

ADDITIVES STUDIED

The plastics contain ethylene and acrylic acid. Both plastics are semiclear solid pellets up to a temperature of 180°F and have a specific gravity of from 0.91 to 0.97. No handling problems should occur with expected pavement uses.

Latex had been previously tested and was incorporated into the project because of possible synergistic effects when combined with the polyolefins. Latex is a blend of water and rubber and is similar to water in most of its physical properties. Latex has a specific gravity of from 0.93 to 0.97, a boiling point of 212°F, and solidifies at 32°F. No handling problems should occur with expected pavement uses.

Further testing was done to determine which combinations and concentrations to use, mixing and compaction temperatures, order of additive addition, compaction temperature–air

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void relationship, and construction suitability (*5–8*). The following mixture combinations, listed by amounts of additives by total weight of binder system, were used:

1. A control section with no additives,
2. Five percent Plastic 1,
3. Five percent Plastic 2,
4. Three percent latex,
5. Five percent Plastic 1 + three percent latex,
6. Two percent Plastic 1 + three percent latex, and
7. Two percent Plastic 2 + three percent latex.

By using these combinations of additives, each with different asphalt cements and grades and different aggregates, in a variety of climates it was hoped that the following things could be determined:

1. Construction feasibility,
2. High-temperature performance,
3. Low-temperature performance, and
4. Moisture susceptibility.

RESEARCH PROCEDURE

Preconstruction

The first step was to conduct preconstruction testing on the Texas, Idaho, and Alabama field projects to

1. Determine the compatibility of the modifiers with the asphalt concrete mixtures,
2. Obtain results that could be compared with postconstruction results, and
3. Determine which of two methods to use to combine the modifier with the asphalt cement.

The first method of modifier addition involved retaining the amount of asphalt cement called for in the mix design and adding the modifier by weight of asphalt. The second method involved subtracting a like quantity of asphalt from the mixture so that the modifier and asphalt combined equaled the amount of binder called for in the mix design. Testing showed the latter method of addition to be the best. This method reduced tenderness problems that occurred when the modifier was added to the asphalt, as was done in the first method.

Pavement condition surveys, using the American Public Works Association PAVER method (*9*), were performed at each site. At the times of the surveys, crack maps were drawn to scale so that cracks occurring in the pavement after construction could be traced to determine amounts of reflective versus thermal cracking. The projects consisted of 1,000-ft sections for each of the seven different modifier combinations listed earlier. Each 1,000-ft section was divided into ten 100-ft sample units of which three were selected for sampling of cores and loose mix as well as the PAVER condition survey.

Construction

During construction, a representative from the research team was present to perform a variety of duties. The exact location of all sections was established and referenced for future testing. Samples from each of the selected sample units were obtained for laboratory compaction and testing. Construction rating sheets, listing variables such as mixture temperature, segregation of mix, and ease of compaction and workability of the mixture, were completed. Figure 1 is a copy of the rating sheet used on the projects.

Postconstruction

Nine samples from each of the sections, field mixed and compacted and field mixed and laboratory compacted, were tested from each of the seven sections from all five projects. The testing procedure is shown in Figure 2. The first group of three loose-mix samples was compacted using the standard Hveem compaction effort as described in ASTM D 1561. The *Annual Book of Standards* (*10*) lists all tests used in this project. The other six loose-mix samples were compacted using a reduced Hveem compaction effort of 30 strokes at 250 psi and a leveling load of 10,000 lb to produce sufficient air voids to conduct water sensitivity testing (Figure 3).

The first group of three samples, both cores and laboratory compacted, were tested for resilient modulus at temperatures of 10°F, 34°F, 77°F, and 104°F. Testing was performed according to ASTM D 4123, with a load cycle of 0.1 sec applied load and a 3-sec pause between loads. Splitting tensile strength was

DOW CHEMICAL - FIELD PROJECTS
CONSTRUCTION RATING

Location: Date:

Test Section	1	2	3	4	5	6
Design	CTRL	5P-1	5P-2	3%L	2P-2 3%L	2P-1 3%L
A. TRANSPORT						
1. Segregation in Truck						
2. Flow out of Truck						
3. Adhesion to side of Truck						
4. Temperature of Mix						
B. LAYDOWN MACHINE						
1. Segregation in Hopper						
2. Workability through Screed						
3. Surface Appearance						
4. Temperature						
C. COMPACTION						
1. Shoving						
2. Sticking on Roller						
3. Temperature						
D. JOINT						
1. Overall						
E. WORKABILITY						
1. Overall						
F. AFTER COMPACTION						
1. Surface Appearance						
2. Joint Appearance						

Rating: 8-10 Very Good; 6-8 Good; 4-6 Fair; 2-4 Poor; 0-2 Very Poor

FIGURE 1 Rating sheet.

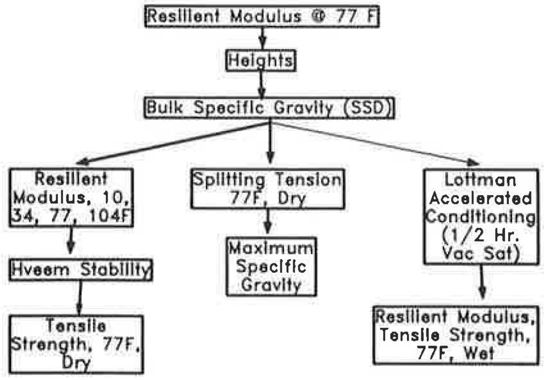

FIGURE 2 Testing outline.

	Construction		Postconstruction
	Standard Hveem	Modified Hveem	Cores
Texas	4.7 S=0.80	7.8 S=0.61	4.5 S=0.73
Idaho	7.4 S=1.03	11.6 S=0.41	9.5 S=0.89
Maine	0.7 S=0.27	3.8 S=0.29	5.9 S=0.78
Alabama	6.4 S=1.04	11.0 S=1.41	12.0 S=1.34
Michigan	0.6 S=0.36	2.8 S=0.74	4.9 S=1.01

FIGURE 3 Air void content.

determined next. Theoretical maximum specific gravities were determined by ASTM D 2041 and air voids were established.

The other six samples were run through one cycle of the Lottman accelerated conditioning procedure (*11*). Resilient modulus values and splitting tensile strengths were determined for dry and wet conditions at 77°F. Retained tensile strengths and modulus values were determined from the ratio of wet to dry test results.

Follow-up testing is expected to continue for 5 years. Cores will be taken at 1, 3, and 5 years, and pavement condition surveys will be conducted annually, using the APWA PAVER method.

PROJECT SITES AND CONSTRUCTION RESULTS

Selection of project sites was based on location, climate, construction equipment and methods available, and cooperation by state highway departments and highway districts. It was desired to find projects that could be constructed using a pug mill asphalt plant. This was necessary because the available plastic is in solid form, and mixing requirements necessitated dumping the plastic pellets directly into the mixer. Significant construction results are discussed separately by state.

Texas

The first highway project is located on US-83 in the lower Rio Grande River Valley near Mission, Texas. US-83 is a four-lane divided facility, with average daily traffic (ADT) of 14,600 in 1985. Extreme summer daytime temperatures are in the 90°F to 99°F range. Winter temperatures are typically in the 50s during the day and drop into the mid-40s at night. Average annual rainfall is 27.0 in. and is evenly distributed throughout the year. Construction consisted of a 2-in. overlay over a double asphalt-rubber seal coat. Existing distress consisted of low to medium severity transverse, longitudinal, and alligator cracking. The asphalt cement used is a Texas Fuel and Oil AC-20 and the aggregate a polished river gravel from the Fordyce pit near Mission, Texas. One percent hydrated lime by dry weight of aggregate was used in the control section to counteract the water susceptibility of the aggregate. Lime was not used in the sections containing modifiers. This was the only project constructed with lime.

The test sections were constructed in July 1986. The hot mix was produced in a plant that uses a continuous pug mill in conjunction with a drum mixer that heats and dries the aggregates. The plant was a DMC Drum Mix Coater manufactured by Astec Industries, Inc. Modifications were made to the plant to facilitate the introduction of the modifiers as necessary. The modifications made were minor and were done the evening before construction. The alteration consisted of cutting a hole on top of the coater unit and bolting on a variable speed auger. The plastic pellets were fed into a chute connected to the auger. The latex was introduced into the asphalt supply line using a metering device fed by a diaphragm pump. This same method was used to add the latex to the asphalt cement on all five projects.

No apparent problems were encountered at the plant when the additives were used. The modified mixes were virtually unnoticed by plant personnel. Three points about this project are worthy of note:

1. The wings of the paving machine were pulled to the center after each load, and the mixture remaining in the truck was emptied on the road surface and shoveled to the center of the lane. This resulted in an area of segregation in the pavement that could be easily identified at the end of each load in the compacted pavement.
2. Some areas of tender pavement occurred; these could be noted by observing the rut depth left by the breakdown roller. The tenderness problem was due to the smoothness of the polished river gravel. This tenderness effect was also observed during laboratory compaction.
3. Slight tearing and pulling behind the screed were noted in sections with 5 percent Plastic 1 plus 3 percent latex, 5 percent Plastic 1, and 5 percent Plastic 2.

Overall, construction personnel were satisfied with both plant operation and field construction.

Idaho

The second project is located in the Boise River Valley in Ada County, Idaho. The site is on Ustick Road between Eagle Road and Cloverdale and is a two-lane facility with a 1986 ADT of 3,700. Summer temperatures are generally mild with highs in the 80°F to 99°F range. Winter temperatures range in the 20s and 30s during the day and drop into the low teens and below at night, December through January. There are nearly 100 air freeze-thaw cycles per year in the area, most of which occur in the spring. Precipitation averages 11.7 in. per year. Construction consisted of a 2-in. overlay over the existing roadway. Existing distress was medium to severe alligator cracking, medium rutting, medium raveling, and low to medium severity longitudinal and transverse cracking. The asphalt used was a Koch AC-10, and the aggregate a smooth polished river gravel obtained from the Nelson pit in Boise.

The test sections were constructed in July 1986. The hot mix was produced in a Barber-Greene 100-ton/hr drum plant modified to produce 150 tons/hr. The plastic pellets were preblended with the asphalt cement in a distributor truck normally used for emulsions. Mixing was not efficiently accomplished because of the small pumps on the truck, and approximately 8 hr were needed to mix the first batch, which contained 5 percent Plastic 1. The plastic pellets normally blend into the asphalt within 20 sec under high agitation during laboratory mixing. The asphalt was introduced as it was in the Texas project.

Because of the mixing problem with the plastic, this project was scaled back to four sections: the control section, 5 percent Plastic 1, 3 percent latex, and 3 percent latex plus 2 percent Plastic 1. No major problems were encountered during construction. Field personnel were only able to identify the section containing 3 percent latex by observing a slightly darker pavement. It was necessary to keep the pneumatic roller well back from the paver in sections containing latex because the mat tended to stick to the rubber tires. However, this was also noticed to some degree in the control section.

Maine

The third project is located on the southbound travel lane of I-95 in Bangor, Maine. I-95 is a four-lane divided facility with a 1985 ADT of 6,695. Summer temperatures range from the mid to the upper 70s, July through September. Winter daytime temperatures range from the low 30s to the low 20s and drop into the low teens and below at night. Precipitation averages 41.6 in. per year. Construction consisted of a 2-in. overlay on a milled and leveled surface. Existing distress consisted of medium to high severity transverse cracking that passed well into the milled surface. The asphalt used was an Irving Oil AC-20, and the aggregate a mixture of coarse sand from the Frink pit in Hermon, Maine, and a ledge sand and stone from the Odlin Road quarry in Hermon.

Construction was performed in September 1986. The asphalt mix was produced in a Stansteel 7,000-lb batch plant. The plastic pellets were introduced by hand into a hole in the pug mill in predetermined quantities. The only noticeable problems were ripples in the final two sections of the project, which were 3 percent latex plus 2 percent Plastic 1 and 3 percent latex plus 2 percent Plastic 2. This was most likely due to a problem with the screed on the paver rather than the mix. The breakdown roller operator commented on how well he thought the sections with plastic in them compacted.

Alabama

The fourth project is located on US-231 in Huntsville, Alabama. US-231 is a four-lane divided facility with a 1986 ADT of 12,000. Summer daytime temperatures range from the low to upper 90s June through August. Winter temperatures are in the 50s during the day and drop into the 30s at night. Precipitation averages 54.6 in. per year. Construction consisted of a 2-in. overlay over the existing roadway. Distress present before construction was low to high severity raveling. The asphalt is an AC-30 from the Hunt Refining Company, and the aggregate is a Vulcan Materials crushed gravel and coarse sand with 10 percent aggregate lime.

Construction was performed in October 1986. The asphalt mix was produced in a pug mill that generates 4 tons per batch. The plastic pellets were introduced by hand into a hole in the pug mill in predetermined quantities. No problems were encountered on this project. Field personnel could not identify any of the seven mixes, except for the 3 percent latex section that was slightly blacker and stickier than the other mixes.

Michigan

The final project is located on State Route M-35 near Marquette, Michigan. M-35 is a two-lane facility with a 1986 ADT of 2,900. Summer daytime temperatures are in the low to upper 70s, June through September. Winter temperatures range from the low to mid 20s during the day to near zero at night, December through January. Precipitation averages 30.4 in. per year. Construction consisted of a 1-in. leveling course and a 2-in. overlay on top of a milled surface. Existing distress included extensive low to medium severity longitudinal and transverse cracking with some alligator cracking. The asphalt is a 120-150 pen asphalt imported from Spain, and the aggregate a coarse gravel.

Inclement weather delayed the Michigan project until late October 1986, which is well into cool weather in northern Michigan. No serious problems were encountered on the Michigan project; research personnel felt comfortable with the operation after the previous four projects. The asphalt mix was produced in a pug mill that generates 3 tons per batch in the same manner as the one used in the Alabama project.

TEST RESULTS

Test results are discussed in terms of the variables investigated.

High-Temperature Performance

Results of high-temperature tests vary depending on which set of data is examined. However, review of preconstruction, construction, and postconstruction air void contents reveals differences in air void content great enough to account for the variance in test data (Figure 3).

Overall, 5 percent Plastic 2 and 2 percent Plastic 1 plus 3 percent latex show the greatest amount of 104°F modulus improvement for both the field-mixed, laboratory-compacted samples and the cores. However, as can be seen in Tables 1–10, the best modifier for any one project varies. There is no one best modifier to increase 104°F resilient modulus for all projects. Improvements are substantial enough to predict improved performance in terms of excessive displacement under traffic, and 104°F modulus gains of up to 50 percent can be expected in the field, according to the data.

Low-Temperature Performance

No long-term performance testing was done to predict low-temperature performance of the mixes in this research project. However, the cold-temperature resilient modulus data suggest that the modifiers are not detrimental to cold-temperature performance of the asphalt concrete. A majority of the results from both the 10°F and the 34°F resilient modulus test are within 10 percent of each other and within one standard deviation. Tables 1–10 give the results of 10°F resilient modulus tests.

Tensile strength tests at 10°F were performed on Maine, Alabama, and Michigan field-mixed, laboratory-compacted samples and Alabama cores. In general, tensile strengths are higher for modified mixes except for the Alabama field-mixed, laboratory-compacted samples, which are the same or slightly lower. Tables 1–10 give the results of 10°F splitting tensile test.

Intermediate Temperatures

Resilient modulus values for modified mixes are considerably higher than for control mixes, except for Texas, which has similar values for both modified and control mixtures. Tensile strengths at 77°F are generally the same for both modified and control mixtures.

Moisture Susceptibility

It was hoped that the polyolefin would coat the aggregate, causing an increased bonding effect between the aggregate and the asphalt cement and generating an increased resistance to stripping in the presence of water. Results of Lottman testing indicate that the modified mixtures have approximately the same retained modulus values (Tables 11–14). The absolute values of the modified mixes after one Lottman cycle are roughly 50 percent higher than those of the control mixes after one Lottman cycle.

The modified mixes have the same or slightly lower tensile strength ratios after one Lottman cycle (Tables 11–14), and the absolute values of the modified mixes after Lottman testing are slightly higher than those of the control mixes after one Lottman cycle.

The Texas data indicate that, although the additives studied offer improved antistripping properties compared with no modifier, lime is a better choice for problem mixtures. However, the modifiers are compatible with lime, as the Alabama study shows. Although the Alabama project contained no added lime, the aggregate did contain 10 percent aggregate lime.

TABLE 1 CONSTRUCTION RESULTS, STANDARD HVEEM COMPACTION, TEXAS

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I	CONTROL	7,175	4,594	494,820	69,510	28.3	147.2	5.0
V	5%P-1	4,531	3,213	451,930	96,800	34.1	137.6	4.3
VI	5%P-2	5,503	3,698	478,400	80,520	29.7	135.5	4.4
II	3%LATEX	6,328	3,561	415,060	72,420	30.2	131.4	5.1
IV	5%P1+3%L	5,165	3,424	611,110	124,740	35.3	152.0	4.6
III	2%P1+3%L	4,999	3,110	462,110	91,230	32.2	140.6	6.0
VII	2%P2+3%L	4,569	3,606	467,810	92,380	28.1	156.3	3.2

TABLE 2 POSTCONSTRUCTION RESULTS, TEXAS

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I	CONTROL	3,331	1,932	271,820	54,480	---	114.1	4.7
V	5%P-1	5,145	1,694	276,040	66,470	---	90.7	4.6
VI	5%P-2	3,123	3,193	334,400	65,890	---	127.2	4.2
II	3%LATEX	3,464	2,043	298,450	58,690	---	110.4	4.9
IV	5%P1+3%L	3,796	1,505	310,360	77,390	---	111.2	4.6
III	2%P1+3%L	3,717	2,013	343,590	66,890	---	106.2	5.7
VII	2%P2+3%L	3,518	2,904	361,980	60,880	---	126.0	3.1

Note: (---) indicates data not available.

TABLE 3 CONSTRUCTION RESULTS, STANDARD HVEEM COMPACTION, IDAHO

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I & IV	CONTROL	2,902	2,586	239,015	31,767	36.8	94.4	6.4
II	5%P-1	3,504	3,239	426,436	75,883	40.1	115.0	7.1
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	2,304	2,384	201,278	35,901	39.6	73.8	9.1
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	2,090	2,838	233,340	44,889	43.4	84.2	6.9
	2%P2+3%L	---	---	---	---	---	---	---

TABLE 4 POSTCONSTRUCTION RESULTS, IDAHO

Test Section Number	Mixture	Resilient Modulus 10 F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I & IV	CONTROL	1,839	1,586	91,616	13,086	---	63.5	9.7
II	5%P-1	1,617	1,081	125,228	22,627	---	58.0	9.4
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	1,685	2,058	132,027	24,482	---	66.1	10.7
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	3,454	---	165,334	---	---	---	8.2
	2%P2+3%L	---	---	---	---	---	---	---

Note: (---) indicates data not available.

TABLE 5 CONSTRUCTION RESULTS, STANDARD HVEEM COMPACTION, MAINE

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 10°F, psi	Air Voids percent
I	CONTROL	4,220	2,927	342,899	55,555	7.8	528.7	0.8
II	5%P-1	5,467	3,489	688,763	145,855	10.0	697.2	0.5
III	5%P-2	5,479	3,176	693,118	173,149	10.4	718.1	0.5
IV	3%LATEX	4,362	3,090	511,072	80,626	5.9	703.9	0.5
VII	5%P1+3%L	3,906	2,511	611,539	128,202	5.9	668.6	0.8
VI	2%P1+3%L	6,326	3,602	726,940	120,522	7.5	689.2	1.3
V	2%P2+3%L	4,550	3,818	749,681	133,291	7.3	713.1	0.6

TABLE 6 POSTCONSTRUCTION RESULTS, MAINE

Test Section Number	Mixture	Resilient Modulus 10°F ksi	Resilient Modulus 34°F ksi	Resilient Modulus 77°F psi	Resilient Modulus 104°F psi	Hveem Stability	Tensile Strength 77°F,psi	Air Voids percent
I	CONTROL	3,302	2,304	102,748	19,187	---	60.8	5.2
II	5%P-1	3,195	2,412	197,378	31,015	---	67.6	5.7
III	5%P-2	2,993	3,104	209,760	41,136	---	67.6	5.7
IV	3%LATEX	3,447	2,600	167,430	29,712	---	67.2	5.0
VII	5%P1+3%L	1,996	1,980	211,008	26,916	---	62.3	6.6
VI	2%P1+3%L	2,869	3,658	210,232	29,841	---	64.4	7.4
V	2%P2+3%L	2,951	3,173	206,651	32,908	---	70.2	5.5

Note: (---) indicates data not available

TABLE 7 CONSTRUCTION RESULTS, STANDARD HVEEM COMPACTION, ALABAMA

Test Section Number	Mixture	Resilient Modulus 10°F ksi	Resilient Modulus 34°F ksi	Resilient Modulus 77°F psi	Resilient Modulus 104°F psi	Hveem Stability	Tensile Strength 10°F,psi	Air Voids percent
I	CONTROL	5,062	2,573	327,959	70,288	35.4	483.6	6.5
II	5%P-1	5,915	4,427	481,658	125,438	37.4	531.2	5.2
III	5%P-2	5,451	3,200	479,141	117,058	33.6	464.2	4.9
IV	3%LATEX	3,546	3,338	369,435	97,829	32.3	395.6	6.6
VII	5%P1+3%L	2,576	2,021	295,591	77,000	32.6	427.9	7.9
VI	2%P1+3%L	5,522	3,966	422,886	110,092	33.1	382.6	6.1
V	2%P2+3%L	3,125	2,320	380,128	95,259	35.0	383.8	7.6

TABLE 8 POSTCONSTRUCTION RESULTS, ALABAMA

Test Section Number	Mixture	Resilient Modulus 10°F ksi	Resilient Modulus 34°F ksi	Resilient Modulus 77°F psi	Resilient Modulus 104°F psi	Hveem Stability	Tensile Strength 10°F,psi	Air Voids percent
I	CONTROL	2,352	2,411	143,071	32,768	11.1	189.6	13.5
II	5%P-1	2,766	1,643	171,360	39,073	20.3	266.7	10.7
III	5%P-2	2,184	1,376	160,766	45,871	---	---	14.2
IV	3%LATEX	2,059	1,303	185,839	44,507	---	---	12.8
VII	5%P1+3%L	2,264	2,284	215,290	37,502	19.0	277.8	11.4
VI	2%P1+3%L	2,362	2,560	230,341	53,015	14.8	274.5	11.1
V	2%P2+3%L	2,797	1,473	200,889	53,715	21.5	288.0	10.6

Note: (---) Indicates data not available.

TABLE 9 CONSTRUCTION RESULTS, STANDARD HVEEM COMPACTION, MICHIGAN

Test Section Number	Mixture	Resilient Modulus 77°F psi	10°F ksi	34°F ksi	104°F psi	Hveem Stability	Tensile Strength 10°F,psi	Air Voids percent
I	CONTROL	145,729	3,864	7,387	22,189	16.0	576.9	0.5
V	5%P-1	225,460	3,428	2,467	34,733	11.4	670.1	0.7
VI	5%P-2	242,775	3,964	2,605	43,321	19.0	642.1	0.3
II	3%LATEX	263,694	4,739	7,331	45,879	14.8	625.8	1.1
IV	5%P1+3%L	183,343	3,700	2,356	36,041	19.9	602.1	0.8
III	2%P1+3%L	338,767	4,149	2,679	70,219	14.7	659.8	1.0
VII	2%P2+3%L	232,462	3,490	2,778	43,777	11.2	671.3	0.0

TABLE 10 POSTCONSTRUCTION RESULTS, MICHIGAN

Test Section Number	Mixture	Resilient Modulus 77°F psi	10°F ksi	34°F ksi	104°F psi	Hveem Stability	Tensile Strength 77°F,psi	Air Voids percent
I	CONTROL	74,237	2,831	1,186	12,803	3.75" COR	51.5	3.2
V	5%P-1	94,803	2,956	1,244	18,201	N/A	50.1	5.9
VI	5%P-2	93,327	2,629	1,562	13,124	N/A	40.9	6.3
II	3%LATEX	129,125	3,123	1,443	17,635	N/A	53.9	4.4
IV	5%P1+3%L	74,332	2,772	1,206	13,494	N/A	46.4	5.7
III	2%P1+3%L	155,395	3,108	1,345	24,097	N/A	65.0	4.3
VII	2%P2+3%L	148,670	3,466	1,704	22,407	N/A	60.7	4.7

TABLE 11 CONSTRUCTION RESULTS, MODIFIED HVEEM COMPACTION, TEXAS

Test Section Number	Mixture	Resilient Modulus,psi 77°F Before	77°F After	Retained Strength percent	Tensile Strength,psi 77°F Before	77°F After	Retained Strength percent	Air Voids percent
I	CONTROL	255,200	86,980	34.1	77.3	73.1	94.6	8.1
V	5%P-1	301,490	49,420	16.4	84.2	22.7	27.0	7.7
VI	5%P-2	305,420	59,850	19.6	95.8	32.0	33.4	7.6
II	3%LATEX	287,370	41,380	14.4	80.2	19.7	24.6	8.3
IV	5%P1+3%L	468,370	77,690	16.6	106.6	36.6	34.3	7.8
III	2%P1+3%L	360,360	55,620	15.4	87.9	25.5	29.0	8.5
VII	2%P2+3%L	318,680	69,500	21.8	104.7	33.1	31.6	6.5

TABLE 12 POSTCONSTRUCTION, LOTTMAN RESULTS, TEXAS

Test Section Number	Mixture	Resilient Modulus, psi 77°F Before	77°F After	Retained Strength percent	Tensile Strength, psi 77°F Before	77°F After	Retained Strength percent	Air Voids percent
I	CONTROL	271,820	120,990	44.5	114.1	88.8	77.8	4.7
V	5%P-1	276,040	74,960	27.2	90.7	33.5	36.9	4.6
VI	5%P-2	334,400	149,420	44.7	127.2	70.8	55.7	4.2
II	3%LATEX	298,450	59,010	19.8	110.4	33.3	30.2	4.9
IV	5%P1+3%L	310,360	104,500	33.7	111.2	48.7	43.8	4.6
III	2%P1+3%L	343,590	83,340	24.3	106.2	36.2	34.1	5.7
VII	2%P2+3%L	361,980	135,040	37.3	126.0	51.0	40.5	3.1

TABLE 13 CONSTRUCTION RESULTS, MODIFIED HVEEM COMPACTION, IDAHO

Test Section Number	Mixture	Resilient Modulus, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Tensile Strength, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Air Voids percent
I & IV	CONTROL	166,945	23,384	14.0	59.2	17.2	29.0	11.8
II	5%P-1	204,454	37,964	18.6	54.1	32.8	60.5	11.5
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	131,565	19,714	15.0	50.8	15.6	30.8	12.1
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	180,764	37,906	21.0	67.1	39.6	59.0	11.0
	2%P2+3%L	---	---	---	---	---	---	---

Note: (---) indicates data not available.

TABLE 14 POSTCONSTRUCTION, LOTTMAN RESULTS, IDAHO

Test Section Number	Mixture	Resilient Modulus, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Tensile Strength, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Air Voids percent
I & IV	CONTROL	91,616	46,733	51.0	63.5	36.2	56.9	9.7
II	5%P-1	125,228	56,620	45.2	58.0	39.7	68.5	9.4
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	132,027	45,758	34.7	66.1	29.2	44.2	10.7
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	165,334	---	---	---	---	---	8.2
	2%P2+3%L	---	---	---	---	---	---	---

Note: (---) indicates data not available.

CONCLUSIONS

Seven mixes, made from various combinations of two "functionalized" polyolefins and a styrene-butadiene rubber, were evaluated in five field test strips in various parts of the country. Effects on high- and low-temperature properties as well as moisture sensitivity and construction feasibility were examined.

The following conclusions can be drawn from this research project:

1. Construction of pavements using polyolefins and latex is certainly feasible. Necessary planning includes testing to determine compatibility of modifiers and asphalt cements, methods of introducing the modifiers into the mixers, and changes in compaction methods to eliminate sticking of the modified mixes to rubber-tired rollers.
2. Resistance to thermal cracking should not decline with addition of a combination of polyolefin and latex.
3. Moisture susceptibility of mixes with polyolefins may or may not be decreased depending on how much higher the absolute strengths of the modified mixes are compared with mixes without modifiers.
4. Properties of modified mixtures are asphalt source dependent, and mix designs should be performed each time a new modifier is used to test for compatibility with the asphalt cement.
5. There is no one best polyolefin or polyolefin-latex combination that will improve all properties for any one mix.
6. To gain the maximum benefit from the inclusion of modifiers, good construction practices, including close attention to air void content, are necessary.
7. Further follow-up testing during the next 5 years of the project will yield much more significant conclusions.

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Probabilistic Approach to Evaluating Critical Tensile Strength of Bituminous Surface Courses

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The strength of the upper layer (surface course) is one of the important indices of pavement serviceability. The serviceability of the pavement will be compromised when the stresses due to axle loads exceed the strength of the upper layer. The probability of failure of a structure is a function of the load effect and the resistance of the structure. In this paper, the load effect on the pavement structure is defined as the radial stress induced by traffic loading, and the resistance is defined as the layer strength. Therefore failure of the surface course is defined as occurring when the radial horizontal stress exceeds the horizontal tensile strength of the surface course. Basic variables for reliability analysis are introduced along with methods for determining the probability distributions of the basic variables. Monte Carlo simulation is used to determine the values for each variable for calculating radial stress using Boussinesq one-layer theory. First-order, second-moment probabilistic methods are used to determine the reliability index and critical tensile strength of the surface course. An example reliability study, using data for I-85 in South Carolina, is presented. Reliability and critical tensile strength of the surface course are obtained for the highway. The critical strength value is evaluated, on the basis of field conditions, to verify its usefulness. Pavement data for rutting and stripping are compared with tensile strength data for the upper pavement layer to evaluate the critical tensile strength value.

The upper layer (surface course) of flexible pavements is in the most severe stress condition because of direct contact of traffic loading. The surface course should be stronger than the other pavement layers. Analyses of field cores taken from I-85 in South Carolina, however, indicated that the tensile strength of the surface course was the lowest of all layers. This study was therefore intended to introduce an approach, using probabilistic concepts, to determine an acceptable value of tensile strength for bituminous surface courses.

Probabilistic design concepts and a general procedure for reliability analysis are first introduced. A first-order, second-moment probabilistic method is used for the reliability analysis (*1*, *2*). An example reliability analysis that uses field data obtained from I-85 is presented. The results of the analysis are evaluated by comparison with current field conditions.

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PROBABILISTIC DESIGN CONCEPTS

The probability of failure of a structure is a function of load effect (S) applied to the structure and resistance (R) of the structure, where S and R are random variables. If $R - S$ is defined as safety margin, then $P(R - S > 0)$ is the probability that the structure will remain safe (*1*, *2*). If the means and standard deviations of R and S are known, then a function, $Y = R - S$, can be defined with mean $Ym = Rm - Sm$ and standard deviation $\sigma_y = (\sigma_r^2 + \sigma_s^2)^{1/2}$ where Rm, Sm, σ_r, and σ_s are the means and standard deviations of R and S, respectively. In Figure 1, probability of failure is then defined as

$$P_f = Pr(R - S < 0) \\ = Pr(Y < 0) \quad (1)$$

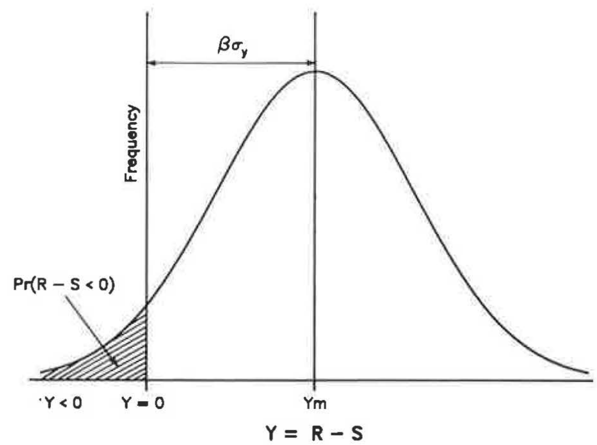


FIGURE 1 Definition of probability of failure.

Subtracting Ym from both sides of the inequality in Equation 1 and also dividing both sides by σ_y results in

$$Pr(Y < 0) = Pr[(Y - Ym)/\sigma_y < -Y_m/\sigma_y] \\ = Pr(u < -Ym/\sigma_y) = F_u(Ym/\sigma_y) \quad (2)$$

where $u = (Y - Ym)/\sigma_y$, and F_u is the cumulative distribution function for the standardized variable (u) with mean $u_m = 0$ and $\sigma_u = 1$. Therefore, probability of failure is

$$P_f = F_u[(Rm - Sm)/(\sigma_r^2 + \sigma_s^2)^{1/2}] \quad (3)$$

In Equation 3, $[(Rm - Sm)/(\sigma_r^2 + \sigma_s^2)^{1/2}]$ is referred to as β, the safety index or reliability index:

$$\beta = (Rm - Sm)/(\sigma_r^2 + \sigma_s^2)^{1/2} \tag{4}$$

Equation 4, introduced in first-order, second-moment probabilistic design concepts (*1*, *2*), can be used for calculating the numerical value of reliability for normal variates. If R and S are log-normal variates, using the mean ratio of the natural logarithm $Ym = [\ln(R/S)]_m$ and the standard deviation $\sigma_y = \sigma_{\ln(R/S)}$, β is defined as $\ln(R/S)]_m/\sigma_{ln(R/S)}$. Using mean and small variance approximations,

$$\beta = \ln(Rm/Sm)/(V_r^2 + V_s^2 + V_r^2 V_s^2)^{1/2} \tag{5}$$

where V_r and V_s are coefficients of variation (COV) of the load effect and the resistance, respectively, and

$$(V_r^2 + V_s^2 + V_r^2 V_s^2)^{1/2}$$

represents the uncertainty associated with load and resistance. If V_r and V_s are smaller than 0.3, the $V_r^2 V_s^2$ term is generally ignored because less than 5 percent error is introduced by doing so (*2*, *3*). Then Equation 5 becomes

$$\beta = \ln(Rm/Sm)/(V_r^2 = + V_s^2)^{1/2} \tag{6}$$

If β is increased with a constant σ_y in Figure 1, then the P_f (shaded area) is reduced. Thus β is a measure of the reliability of a structural member.

Rewriting Equation 5 leads to the following equations:

$$Rm = Sm \text{ Exp } [\beta(V_r^2 + V_s^2 + V_r^2 V_s^2)^{1/2}] = Sm\ \theta \tag{7}$$

where

$$\theta = \text{Exp } [\beta(V_r^2 + V_s^2 + V_r^2 V_s^2)^{1/2}] \tag{8}$$

β is defined as the central safety factor and is used as a safety parameter for member strength (Figure 2).

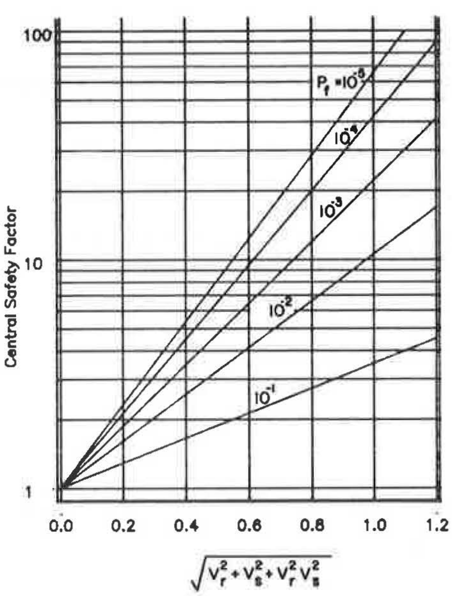


FIGURE 2 Central safety factor.

To apply these concepts of probability of failure to the flexible pavement system, the following assumptions were made in this study: The failure of any layer in a flexible pavement system will cause a functional failure of the pavement structure. Among other factors, failure of the pavement layer is a function of layer strength (resistance) and the stress (load effect) applied to the layer by vehicle loading. Therefore failure of the surface course was defined in this study as occurring when the radial horizontal stress due to traffic loading exceeded the horizontal tensile strength of the surface course.

Reliability Analyses

Data for basic variables for reliability analysis must be collected either from the field or from the literature. Probability distributions for those variables can be determined by a goodness of fit test at a certain level of significance (*4*). On the basis of the established probability distributions, values for each variable can be simulated by computer. The probability distribution for tensile strengths can be used for simulation of layer strength, and the probability distributions for layer thickness and axle load can be used for simulation of applied radial stresses on the layer.

Reliability in this study represents the probability that the surface course will not fail under the current traffic load. The reliability of the surface course can be obtained by comparing that layer's tensile strength with the radial horizontal stress. A critical value of tensile strength for the surface course for a certain level of reliability can then be obtained for the given traffic condition.

Basic Variables

The basic variables for which statistical data must be determined include thicknesses of the pavement layers, tensile strength of the surface course, axle loads, and radial stresses. Data on strength and thickness of the pavement layers and on axle loads of vehicles can be collected from field cores and from the literature to establish appropriate probability distributions.

Layer Thickness

The thickness of the pavement layer is a random variable, differing from one location to another. Data on layer thicknesses can be obtained from field cores. The major portion of the top layer of the flexible pavement in a highway is surface course; the second layer is binder course. Because the intent is to analyze the reliability of the surface course, only the thickness of the first layer is needed to determine probability distributions for layer thickness.

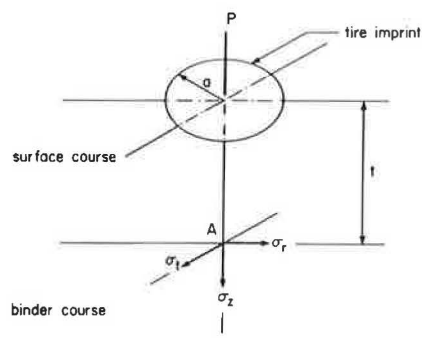
Axle Load

Traffic data on axle loads can be obtained from the state highway department or from truck weight data published by FHWA, or both (*5*, *6*). The probability distribution of weights on a wheel for light vehicles (pickup trucks, passenger cars, and vans) and heavier vehicles must be determined. The probability distribution for weights on a wheel for combined light and heavy vehicles can be determined by combining the two distributions with the appropriate ratio of heavy vehicles to light vehicles.

The weight on a wheel is determined by dividing an axle load (obtained from references) by the number of wheels on the axle. Sixty percent of gross weight is, on average, allocated to the main axle for light vehicles.

Radial Horizontal Stress

Radial stress is a function of axle load and layer thickness, both of which are random variables. Therefore radial stress can be considered a random variable. The stress mechanism under an axle load at the surface course of the pavement is shown in the Figure 3 (7). In Figure 3, the actual value of the radius (a) of tire contact area depends on the magnitude of axle load, and 3 to 6 in. can be used.



t = Thicknesses of surface course
P = Load on a wheel
a = Radius of tire imprint
σ_r = Radial stress in point A
σ_t = Tangential stress in point A
σ_z = Vertical stress in point A

FIGURE 3 Stresses at bottom of surface course under tire loading.

Radial horizontal stress (RHS) in the surface course can be calculated on the basis of Boussinesq one-layer theory. The load at the surface of the flexible pavement is assumed to be distributed over a circular area of tire contact. Because the depth at which the radial stress is measured is less than one-half of the clear distance between tire edges of dual wheels, the equivalent single wheel load (ESWL) concept need not be applied. That is, the maximum thickness of surface course in most highways is less than 5 in., which is approximately one-half the clear distance between tire edges of dual tires for most heavy traffic.

In the original Boussinesq equations, the pavement is considered homogeneous, isotropic, and elastic (7). The following Boussinesq equation can be used to obtain radial horizontal stress:

$$Sr = p[2\,\mu\,A + C + (1 - 2\mu)F] \qquad (9)$$

where

Sr = RHS,
p = pressure at tire-pavement contact,
μ = Poisson's ratio, and
A, C, and F = one-layer elastic function values.

The value for p can be obtained by dividing the weight on a wheel by contact area. An appropriate value for Poisson's ratio can be obtained from experiment or from the literature. The values of A, C, and F are tabulated by Yoder and Witczak (7) as functions of depth and offset distance in radii (z/a and r/a).

Maximum radial stress due to a single wheel occurs at a point along the vertical line beneath the geometric center of the tire imprint (load point). Maximum radial stress due to dual wheels occurs either at a point beneath the load point or at a point beneath the point halfway between the two tires. At a point beneath the midpoint of the two tires of dual wheels, radial stress is duplicated by the loads of the two wheels. Radial stress at this point is sometimes greater than the stress at a point vertically beneath the load point when the depth is greater than the clear distance between the two tires. Because the depth of the surface course is generally less than one-half the clear distance of two tires, maximum stress does not occur at this point. Many trucks are equipped with tandem axles. Because the distance between tandem gears is at least 40 in., however, no stress duplication occurs between tandem wheels at the surface course.

Tensile Strength

Tensile strength of the pavement layer can be used for layer strength (resistance) in reliability evaluation. Tensile strength can be measured for cored specimens by the indirect tensile strength test. The tensile strength of the layer can be considered a random variable that varies from one location to another.

The indirect tensile strength (ITS) test is one type of tensile strength test used for stabilized materials. This test involves loading a cylindrical specimen with a compressive load along the diameter of the specimen. This results in a relatively uniform tensile stress acting perpendicular to and along the diametral plane of the applied load, which results in a splitting failure generally occurring along the diametral plane. For most engineering materials, initial failure occurs by tensile splitting in accordance with the following equation (7).

$$\text{Tensile strength} = 2P/(\pi dt) \qquad (10)$$

where

P = load applied to the specimen,
t = thickness of the specimen, and
d = diameter of the specimen.

For the study discussed in this paper, the indirect tension test was conducted on the cored specimens after the cores were sliced by layer. The testing temperature was 77°F, and load was applied vertically using a Marshall testing machine at a rate of 2 in./min through 0.5-in.-wide curved metal strips on the top and bottom of the specimen.

Determination of Reliability Index and Critical Strength

Probability distributions for resistance (tensile strength) and load effect (radial stress) can be obtained as described previously. The reliability index (β) can be calculated using Equation 4, if both tensile stress and radial stress follow normal distributions, or Equation 5 or 6, if they follow log-normal distributions. Probability of failure (P_f) or reliability ($1 - P_f$) can be obtained from the value of β. If resistance and load effect do not both follow a normal or log-normal distribution, Monte Carlo simulation can be used to obtain an estimated probability of failure.

Given coefficient of variation (COV) values for load and resistance, and target reliability, the central safety factor (β) can be obtained from Equation 8 or Figure 2 and used as a parameter for structural design. The central safety factor represents numerically how many times the resistance should be stronger than the load effect. If a central safety factor is specified on the basis of a target reliability and COV for load and resistance, a minimum or critical value of resistance can be determined.

EXAMPLE STUDY OF RELIABILITY ANALYSIS METHOD

A reliability study was conducted using data from I-85 in South Carolina and is presented in this paper for illustration. The probability distributions for each variable used for the study were determined using a computer program for goodness of fit testing. The program tested the data with *K-S* and χ^2 tests at the 5 percent level of significance (*4*, *8*). The probability distribution for weight on a wheel based on traffic data (*5*, *6*) is shown in Figure 4. The probability distribution for the tensile strength values is shown in Figure 5. Radial stress was obtained using Equation 10 with a Poisson's ratio of 0.35, which is a widely accepted value for asphaltic concrete.

The probability distributions (*9*) selected for each variable are given in Table 1. Because both radial stress and tensile strength data were found to follow log-normal distributions, $\ln(R_t)$ and $\ln(S_r)$ follow normal distributions, where R_t and S_r are tensile strength and radial stress, respectively. Because

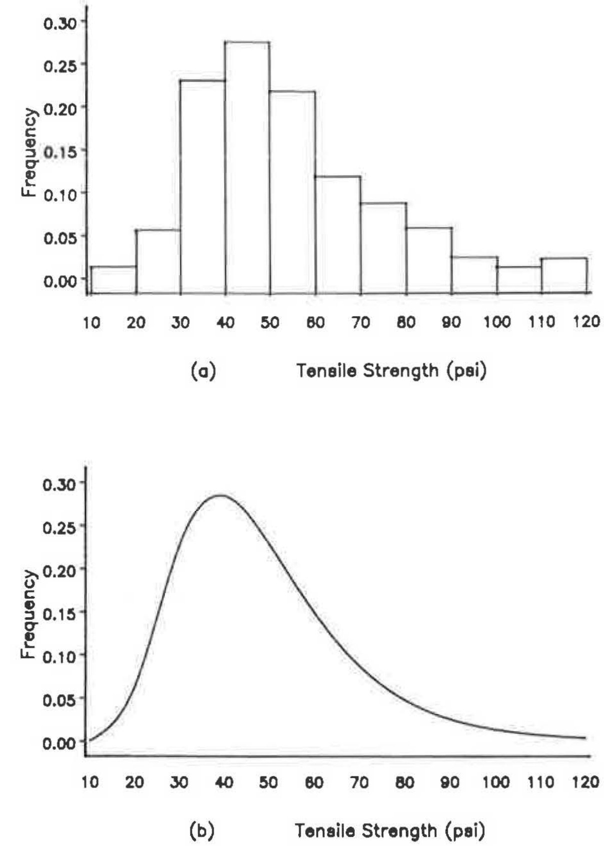


FIGURE 5 (a) Histogram and (b) PDF for tensile strength.

these are random variables, probability of failure can be defined as shown in Figure 6 (*10*, *11*). The reliability index for a given strength value can therefore be obtained from Equation 5 (V_r and V_s are greater than 0.3 as given in Table 2).

The calculated values for β, θ, and reliability, based on the data, are given in Table 2. Therefore a tensile strength of 53.4 psi has a probability of failure (P_f) of 0.015. The safety level, however, for functional failure for most engineering problems requires a probability of failure of less than 0.01 (*10*). The value of tensile strength to satisfy this requirement is $15.25\ \theta = 61$ psi, where $\theta = 3.99$ for $P_f = 0.01$ and $V_r^2 + V_s^2 + V_r^2 V_s^2) = 0.594$. According to the central safety factor in this case, average tensile strength needs to be approximately four times greater than the average radial stress induced by traffic loading. Several example values of tensile strength and their associated P_f-values are given in Table 3. The relationship between reliability and tensile strength is shown in Figure 7.

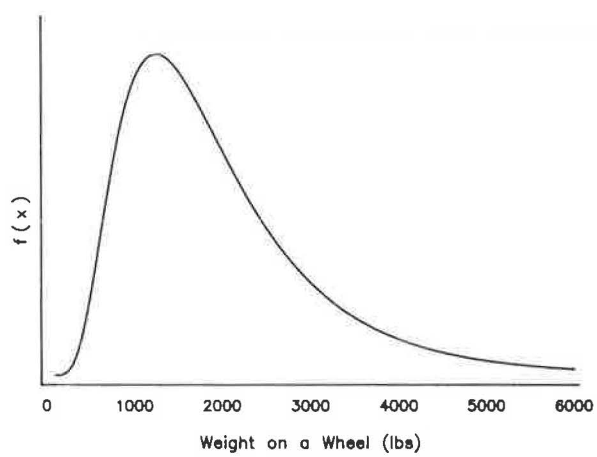


FIGURE 4 Probability distribution for weight on a wheel (total vehicles).

EVALUATION OF TENSILE STRENGTH BASED ON FIELD CONDITIONS

It was found from coring that pavement strength was related to pavement condition in the field. Surface rutting and stripping were the most significant distress mechanisms that were correlated with low-strength pavements. On the other hand, most of the cores from sites where cracks had developed on the surface

TABLE 1 PROBABILITY DISTRIBUTIONS AND THEIR PARAMETERS

Variable Name	Probability Distribution	Parameters
Thickness	Log-Normal	μ = 1.29 inch σ = 0.24 inch
Weight on a Wheel		
Light Vehicles	Weibull*	a = 0 b = 1225 lbs c = 3.69
Heavy Vehicles	Weibull*	a = 0 b = 2784 lbs c = 2.69
Total Vehicles	Log-normal	μ = 1598 lbs σ = 1011 lbs
Radial Stress	Log-normal	μ = 15.25 psi σ = 6.72 psi
Tensile Strength	Log-normal	μ = 53.4 psi σ = 19.49 psi

* Parameters for Wiebull distribution:
a= location factor, b= scale factor, c= shape factor (8,9)

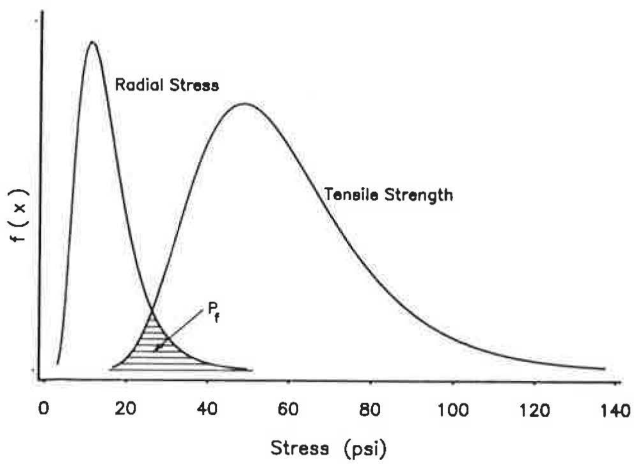


FIGURE 6 Probability of failure for surface course.

showed vertical or horizontal cracks, or both, throughout the cores. Most of them fell apart while being removed from the pavement. Even when the cores were removed without breaking, they were usually so weak that tensile strength values could not be measured. Therefore tensile strength values used in this study did not include values from cracked cores.

Because rutting is caused by consolidation or lateral movement of the materials due to traffic loading, rutting results in permanent deformation of one or more of the pavement layers or in the subgrade. This deformation causes a loss of strength in the mixture, leading to major structural failure of the pavement (*12*). Average tensile strength for the surface layer at sites that were free from rutting was approximately 81 psi. Average tensile strength of the surface layer decreased as surface rutting increased. Following the ITS test on each specimen, visual stripping rate (VSR) was measured on the broken faces of the specimen by the method developed by the Georgia Department of Transportation. Moisture-damaged (stripped) mixtures generally produced lower strengths in ITS tests than did undamaged mixes.

The relationship of rutting and visual stripping ratio with tensile strength values was statistically analyzed using the general linear model (GLM) procedure in the Statistical Analysis System (SAS) (*13, 14*). *F*-tests at the α = 0.05 level of significance were conducted in the analyses of variances of rut depth and VSR. Mean values for tensile strength in various conditions were compared using the least square difference (LSD) method. The average tensile strength value for mixtures that were free from both stripping and rutting was greater than 90 psi. However, the average tensile strength for mixtures that

TABLE 2 CALCULATED RELIABILITY

Strength (psi)		Stress (psi)		β	θ	Reliability
R_t	V_r	S_r	V_s			
53.40	0.365	15.25	0.440	2.107	3.456	0.9743

TABLE 3 TENSILE STRENGTH VALUES AND PROBABILITIES OF FAILURE

Tensile Strength (psi)	β	P_f
40	1.623	0.0530
50	1.999	0.0233
60	2.306	0.0110
70	2.566	0.0051
80	2.790	0.0026
90	2.989	0.0014
100	3.166	0.0008
110	3.326	0.0004
120	3.473	0.0003

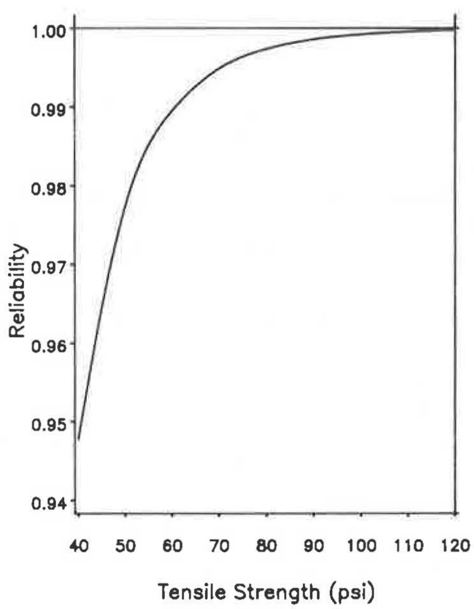


FIGURE 7 Reliability of tensile strength.

were stripped, for example, 10 to 40 percent in either the fine or the coarse aggregates, and also rutted at least 0.5 in. on the pavement surface, was approximately 50 psi.

The results of the analyses, given in Tables 4–6 and shown in Figure 8, reveal that tensile strength values generally decreased as rutting or stripping increased. The tensile strength values for specimens from the distressed pavement area were significantly below (at $\alpha = 0.05$) the tensile strength values from distress-free pavement sections. The combined effect (interaction) of both types of distress on tensile strength, however, was not significant at the same level of α.

Most of the tensile strength values for mixtures from nearly distress-free conditions were above 65 psi, which is 4 psi higher than the critical strength obtained in this study for the 0.01 probability of failure. Because level of probability for serviceability failure should be less than 0.01, the value of 65 psi for which P_f is 0.0073 is acceptable for critical tensile strength. Some of the specimens from areas of minor distress had tensile strength values of approximately 60 psi (for example, rutting = A and VSR = 1.0, rutting = A and VSR = 1.5 in Figure 8). Otherwise, almost all average tensile strength values from distressed conditions were below 60 psi, and many were below 50 psi. Therefore, for the surface course to perform satisfactorily under current traffic loading, a tensile strength of approximately 65 psi for field mixtures appeared to be needed for this section of Interstate highway.

SUMMARY AND CONCLUSIONS

A reliability study, based on first-order, second-moment probabilistic concepts, was conducted to develop a measure of an acceptable value of tensile strength for bituminous surface courses. Probabilistic design concepts and a general procedure for reliability analysis were first introduced, and basic variables to be used in the reliability study were defined. On the basis of the reliability concepts, and using test data from cores drilled from a portion of I-85 in South Carolina, an example reliability study was conducted for bituminous surface courses. Probability distributions for pavement layer strength, layer thickness, axle load, and radial stress were determined from field data and from the literature. The Monte Carlo method was used to determine values for the variables by computer simulation procedures. The reliability of the strength of the surface course

TABLE 4 ANALYSIS OF STRENGTH (tensile strength versus VSR)

		VISUAL STRIPPING RATIO			
		0	1	1.5	2
TENSILE STRENGTH (psi)	MEAN	56.22	48.98	48.27	59.77
	STD.	22.74	15.69	15.96	15.22

Legend for Visual Stripping Ratio

0: Almost no Stripping
1.0: Stripping < 10 %
1.5: 10 % < Stripping < 40 %
2.0: Stripping > 40 %

TABLE 5 ANALYSIS OF STRENGTH (tensile strength versus rutting)

		RUTTING					
		A	B	C	D	E	F
TENSILE STRENGTH (psi)	MEAN	80.98	51.69	51.11	45.46	41.12	45.31
	STD	34.62	14.80	18.73	13.54	17.93	11.98

Legend for Rutting

A: Rutting = 0 B: 0 < Rutting < 0.25 inch
C: 0.25 < Rutting < 0.5 D: 0.5< Rutting < 0.75 inch
E: 0.75 < Rutting < 1.0 F: Rutting > 1.0 inch

TABLE 6 ANALYSIS OF VARIANCE (based on GLM)

SOURCE	DF	SUM OF SQUARES	F VALUE	PR > F
RUTTING	5	12528.3121	8.07	0.0001 *
VSR	3	5022.8908	5.39	0.0012 *
VSR*RUTTING	9	4909.7989	1.76	0.0755
ERROR	334	103712.5993		
CORRECTED TOTAL	351	126173.60112781		

* Significant at α = 0.01

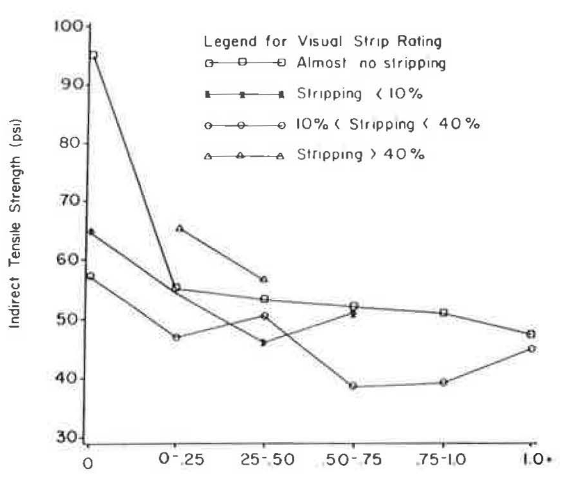


FIGURE 8 Analysis of tensile strength.

was calculated by comparing the tensile strength of the pavement with the load-induced radial stress. A critical value of tensile strength for the surface course was developed from the field data and the results of the simulation procedure.

On the basis of the results of tests on field cores, traffic data obtained from the literature, simulation analyses, and reliability analysis, a critical tensile strength value of 65 psi measured at 77°F for field-cored mixtures was identified as necessary to sustain a given traffic loading for a given section of Interstate highway. The value is based on the assumptions made with respect to traffic volume and make-up. The accuracy of the value is therefore dependent on the correctness of the assumptions made. A more appropriate value could be obtained if traffic studies were conducted to obtain the actual traffic loads carried by the section of pavement.

The procedure and methodology that are presented in this paper can be applied to any surface course and traffic condition. Cores can be drilled and tested to obtain estimated distributions for thickness and tensile strength. Similarly, traffic data can be obtained to determine the appropriate load distribution for the pavement section. Depending on the distributions identified, either simulation or analytical methods can be used to determine the critical tensile strength values for selected levels of reliability.

Although it is recognized that estimation of the potential performance of a pavement is a complex problem and that there are many possible failure mechanisms to consider, the procedures presented in this paper are a first attempt at developing a method for determining a minimum acceptable tensile strength for bituminous surface courses.

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Variables Affecting Marshall Test Results

ZAHUR SIDDIQUI, MARTIN W. TRETHEWEY, AND DAVID A. ANDERSON

The Marshall method of mix design is one of the most widely used methods for designing and controlling hot-mix paving mixtures. However, inconsistencies among results occur when different Marshall equipment is used; this often leads to disputes when verification or acceptance test results vary significantly from the contractor's quality control results. This research identified key equipment-related factors associated with inconsistencies in test results obtained with different compaction and testing equipment. Techniques and procedures for quantifying the effects of these variables and their interactions are currently unavailable. In the absence of such a procedure, several U.S. and Canadian private and public agencies regularly participate in round-robin or mix-exchange programs, which are discussed. To examine the ability to measure the fundamental process parameters of the Marshall compaction operation, several mix specimens were compacted with a mechanical hammer instrumented with accelerometers. From the analysis of data obtained, it was concluded that technology for measuring the amount of energy delivered to the specimen during the compaction process is currently available. Research and development needed for adapting this technology to the field calibration of the Marshall hammer are recommended.

The Marshall method of mix design and control was originally developed in the late 1930s by Bruce G. Marshall of the Mississippi Highway Department. The method evolved between World War II and the late 1950s when the Department of Defense recognized the need for a procedure that could be used for designing asphalt concrete mixes to withstand the increasing wheel loads and tire pressures of military aircraft (*1*). Today the Marshall method of mix design is one of the most widely used methods for the design and control of hot-mix paving mixtures (*2*). However, the current method has evolved through a number of changes and refinements (*1*).

In its current form, the Marshall method of mix design consists of (a) compacting mix specimens, (b) conducting a density-voids analysis on the compacted specimens, and (c) testing the compacted specimens for stability and flow. Details of the procedure and equipment are provided in ASTM (D 1559), AASHTO (T 245), and Military (MIL-STD-620A) standards. The ASTM standard (D 1559) specifies the use of a manual compaction hammer whereas both AASHTO and MIL-STD-620A permit the use of a mechanical hammer, provided it is properly correlated with the standard hand hammer. Currently, however, most highway agencies and contractors use a mechanical hammer for the purpose of design, control, and acceptance of hot-mix asphalt concrete.

Industry and highway agency personnel have long been aware of inconsistencies between test results when mix specimens are prepared and tested in different Marshall equipment (*3–5*). The objectives of the study reported in this paper were (a) to identify the key equipment-related factors associated with inconsistencies in test results obtained by using different compaction equipment and (b) to recommend calibration equipment and techniques that could be used to calibrate Marshall compaction equipment.

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IDENTIFICATION OF VARIABLES

On the basis of their experience and a review of the literature, the research team identified eight compaction equipment–related variables that may have an influence on the level of compaction achieved in the laboratory (Table 1). These eight variables were included in a questionnaire used for conducting telephone interviews with several agency and industry personnel and researchers at universities. A total of 11 persons were interviewed. Two of these were university-based researchers with national reputations; two were from large, private material-testing laboratories; one represented a large paving contractor; one was a consultant currently conducting research on a federally sponsored project related to bituminous concrete; and five were from progressive state highway agencies (including one from Canada). Three of these state highway agencies, and several other people contacted, have also been involved in a series of round-robin (mix-exchange) testing programs with the objective of studying the variability in Marshall test results. Some of these round-robin testing programs are discussed later in this paper.

The frequency with which the persons surveyed rated the eight compaction-equipment variables as important to the level of compaction achieved in the laboratory is summarized in Table 1. All variables were considered by at least three persons to have a significant influence on the level of compaction

TABLE 1 FREQUENCY WITH WHICH VARIABLE WAS CONSIDERED IMPORTANT TO COMPACTION ACHIEVED

Compaction Equipment–Related Variable	No. of Persons Rating Variable as Important to Compaction Achieved
Alignment of hammer	9
Pedestal support	9
Height of free-fall	8
Weight of hammer	7
Pedestal construction	7
Friction between rod and hammer	6
Mold restraint (rotating versus fixed)	3
Dynamic response from energy transfer during impact	3

achieved. Hammer alignment and pedestal support were cited most frequently (9 of 11) followed by the free-fall height of the hammer.

All persons contacted have the base of their Marshall hammers mounted on a standard compaction pedestal fixed to the concrete floor slab of the building. However, several of the people interviewed have encountered situations in which anomalous test results were obtained when the standard compaction pedestal was not used or the equipment was located on an upper floor of the building.

The weight of the hammer and pedestal construction were cited by 7 of the 11 persons interviewed. Hammer alignment, hammer weight, and free-fall height can be standardized by simple length or mass measurements; however, there is no standard or straightforward technique for standardizing pedestal or floor (foundation) stiffness.

Ten of 11 persons interviewed had experienced inconsistencies in test results when hot-mix asphalt concrete specimens were compacted with different Marshall equipment. The age of the Marshall hammers used by the people surveyed ranged between 7 and 20 years. However, their equipment is periodically inspected and parts are replaced or repaired as needed.

The telephone survey revealed that significant differences were perceived in the Marshall compaction equipment made by different manufacturers. One of the differences cited pertains to the mass of the sliding weight, and the experience of the authors confirms this source of difference. Two Marshall hammers from different manufacturers were ordered for their laboratory. When the hammers were received, it was found that the falling weights differed by 266 g. The respondents to the questionnaire also cited differences in the type of reaction (pedestal construction) and the shape of the hammer assembly foot (flat versus beveled) as major differences between manufacturers.

Eight of the 11 persons interviewed attributed differences in compaction test results to both equipment- and operator-related factors. When asphalt concrete mix specimens are compacted in a given compactor, differences in the compaction temperature and the actual preparation of the specimens can significantly influence the test results. Clearly, these are operator-related variables. In addition, a laboratory technician who has been preparing and testing Marshall specimens for several years may, for convenience, develop some shortcuts to the procedure without realizing that he is deviating from the specified procedure.

In addition to compaction density (and the associated voids analysis), asphalt concrete mixes are also tested for stability and flow. The Marshall stability and flow of compacted mix specimens are determined with the help of a breaking head and flowmeter. Nine of the 11 persons interviewed had encountered a breaking head or flowmeter that did not meet ASTM or AASHTO specifications. The major discrepancy was associated with the dimensions of the breaking head, including the dimensions of the bevel. Although the specifications require a $^{1}/_{4}$-in. bevel, breaking heads with $^{3}/_{8}$-in. bevels were encountered. Often the breaking head did not have the standard 2-in. radius. Research has shown that these differences in the breaking head result in significant variability in stability and flow measurements (*3*). Again, operator-related factors, such as conditioning of the specimen and the duration of the actual testing process, were cited as sources of differences between test results.

Marshall Round-Robin and Mix-Exchange Programs

Discrepancies in Marshall test results have long been of concern to both industry and state highway agency personnel. ASTM Subcommittee D04.20, private testing laboratories such as the AASHTO Materials Reference Laboratory (AMRL) and the Chicago Testing Laboratory, and several state highway agencies, both in the United States and in Canada, have conducted extensive interlaboratory testing programs to study the repeatability and reproducibility of Marshall test results. ASTM Subcommittee D04.20 has conducted several round-robin tests, but the test data were not available for analysis by the authors. Georgia and Utah have conducted in-house research to study the variability in Marshall test results. Although these studies have not been published, the researchers have obtained special permission to summarize the studies in this paper.

Georgia Study

In 1980 Georgia conducted an interlaboratory investigation in which five laboratories participated (personal communication with Ronald Collins, Georgia Department of Transportation, 1987). The central laboratory weighed and separately packaged the aggregate for each sample before shipping it to the participating laboratories. Each laboratory prepared and tested the mixes in accordance with the recommended procedure. Each laboratory used both a manual and a mechanical hammer. The graphs shown in Figure 1 represent results for the Marshall properties tested: voids in mineral aggregate (VMA), percentage of air voids, stability, and flow. On each graph, H and M represent the hand hammer and the mechanical hammer, respectively. In each laboratory, the specimens compacted with the mechanical hammer yielded larger VMA and percentage air voids and smaller stability and flow values than those compacted with the hand hammer. The larger specimen densities obtained with the manual hammer may be attributed to the kneading action that takes place when the hammer strikes the sample at a slight angle from the vertical (*2*). These results are in general agreement with the experience of the persons contacted during telephone interviews.

In 1986 four laboratories of the Georgia Department of Transportation and five industry laboratories cooperated in a study to compare test results obtained with the standard 50-blow Marshall procedure (personal communication with Ronald Collins, Georgia Department of Transportation, 1987). Georgia's Type B mix was used in the study. The research results are given in Table 2. Georgia's criteria require a review of the procedure or equipment, or both, if a laboratory average exceeds the following ranges when compared with the overall average:

- Density: ±1.5 lb/ft^3,
- Stability: ±400 lb, and
- Flow: ±0.02 in.

In general, the data meet Georgia's interlaboratory criteria; however, there are instances in which the density, flow, and

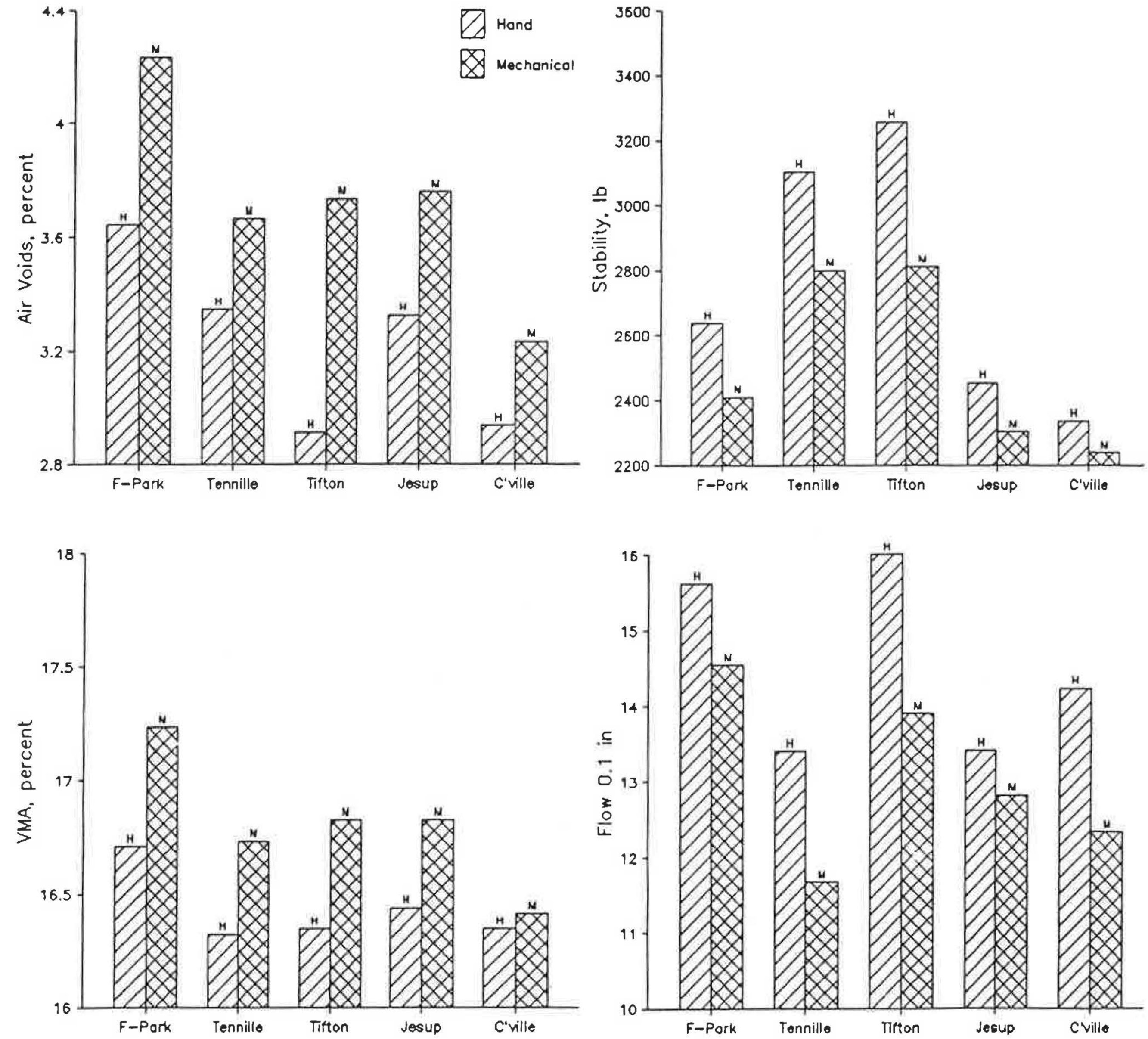

FIGURE 1 Georgia comparative study.

stability do not meet the criteria. The results obtained from the Georgia studies tend to support the experience of the researchers that discrepancies in Marshall test results are due to both equipment- and technician-related factors.

Utah Study

In 1979 a Marshall equipment correlation study was conducted by the Utah Department of Transportation (personal communication with Wade Bentensen, Utah Department of Transportation, 1987). The objective of the investigation was to study the variability that resulted from using different technicians and equipment (Marshall hammer and breaking head). Mix specimens were prepared, compacted, and tested at three levels of asphalt content: 5.5, 6.0, and 6.5 percent.

Table 3 gives a summary of results of tests in which the entire process of preparing, compacting, and testing samples was conducted by one technician from the central laboratory. The technician prepared individual 1200-g aggregate samples at the central laboratory by combining the aggregate in accordance with the design gradation. He performed the balance of the process at each district laboratory using the same Marshall hammer and breaking head.

Table 4 gives data on another set of tests. In this case, the same technician prepared the aggregate samples at the central laboratory. However, these samples were then shipped to the district laboratories, where a district technician prepared, compacted, and tested the mix specimens using the district's Marshall hammer and breaking head. A comparison of the two sets of results indicates that, except for flow, the averages of property values were fairly consistent. However, it is evident from a comparison of the values for range and standard deviation (Tables 3 and 4) that the operator and equipment have a significant effect on the test results. For example, the standard deviation for bulk density in Table 4 was 150 to 260 percent larger than that obtained when the same mix was prepared and tested by one operator using one set of equipment (Table 3).

TABLE 2 1986 GEORGIA LABORATORY COMPARISON STUDY (personal communication with Ronald Collins, Georgia Department of Transportation, 1987)

Laboratory	Location	Height (in.)	Density (lb/ft^3)	Voids (%)	Stability (lb)	Flow (0.01 in.)
District 2	Tennille, Ga.	2.55	154.4	4.3	2,500	12
		2.60	(150.3)	6.8	2,240	12
		2.51	(155.6)	3.6	(3,020)	(15)
District 4	Tifton, Ga.	2.50	154.8	4.1	(2,880)	13.1
		2.50	152.9	5.3	2,350	13.3
District 5	Jesup, Ga.	2.50	154.6	4.2	2,200	(15.0)
		2.50	154.3	4.4	2,175	14.8
		2.50	154.6	4.2	2,275	13.6
District 7	Forest Park, Ga.	2.562	153.0	5.2	2,100	12
		2.555	154.1	4.5	2,460	13
		2.540	154.3	4.4	2,520	11
Producer 1	Macon, Ga.	2.567	153.1	5.1	2,150	10
		2.574	153.6	4.8	2,190	9
		2.562	153.6	4.8	2,340	10
Producer 2	Atlanta, Ga.	2.56	152.6	5.4	2,320	11
		2.56	153.2	5.1	2,470	13
		2.56	152.8	5.3	2,330	13
Producer 3	Doraville, Ga.	2.44	154.4	4.3	2,200	9
		2.50	153.6	4.8	2,050	10
		2.50	152.5	5.5	(1,950)	10
Producer 4	Birmingham, Ala.	2.51	153.0	4.9	2,400	13.5
		2.50	(157.0)	2.9	2,650	13.5
		2.50	155.0	3.8	2,250	12.0
Producer 5	Chattanooga, Tenn.	2.615	152.9	5.3	2,550	11
		2.615	(151.6)	6.0	2,550	10
		2.615	(150.8)	6.5	2,525	12
Average (of all laboratories)			153.6	4.8	2,371	12.0
Acceptable range for a given laboratory		–	152.1 155.1	–	1,971 2,771	10 14

NOTE: Values in parentheses are outside acceptable range.

TABLE 3 UTAH MARSHALL STUDY, SAME OPERATOR AND EQUIPMENT AT VARIOUS LABORATORIES (personal communication with Wade Bentensen, Utah Department of Transportation, 1987)

	Bulk Density (lb/ft^3) at Asphalt Content			Voids (%) at Asphalt Content			VMA Filled (%) at Asphalt Content			Stability (lb) at Asphalt Content			Flow (0.01 in.) at Asphalt Content		
Laboratory	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%
District															
1	2.29	2.29	2.30	3.3	2.4	1.5	78.7	84.7	90.6	2,256	2,064	1,871	10	11	14
2	2.30	2.30	2.30	2.8	2.0	1.5	81.4	87.0	90.6	2,477	2,559	2,216	9	9	12
3	2.29	2.30	2.30	3.3	2.0	1.5	78.7	87.0	90.8	2,538	2,642	2,380	8	9	11
4	2.29	2.30	2.29	3.3	2.0	1.9	78.7	87.0	88.4	2,663	2,678	1,825	10	11	14
5	2.30	2.31	2.30	2.8	1.6	1.5	81.9	89.4	90.6	2,729	2,620	2,045	10	11	14
6	2.29	2.29	2.30	3.3	2.4	1.5	78.7	84.8	90.6	2,367	2,178	2,023	8	11	12
Main laboratory	2.29	2.29	2.29	3.3	2.4	1.9	78.7	84.7	88.4	2,767	1,945	1,826	9	11	12
Average	2.29	2.30	2.30	3.2	2.1	1.6	79.5	86.4	90.0	2,542	2,384	2,027	9	10	13
Standard deviation	±0.005	±0.008	±0.005	±0.024	±0.30	±0.19	±1.4	±1.7	±1.1	±190.1	±310	±211	±0.9	±1.0	±1.3
Range	0.02	0.02	0.01	0.5	0.8	0.4	3.2	4.7	2.4	511	733	554	2	2	2

TABLE 4 UTAH MARSHALL STUDY, DIFFERENT OPERATORS AND EQUIPMENT AT VARIOUS LABORATORIES (personal communication with Wade Bentensen, Utah Department of Transportation, 1987)

	Bulk Density (lb/ft^3) at Asphalt Content			Voids (%) at Asphalt Content			VMA Filled (%) at Asphalt Content			Stability (lb) at Asphalt Content			Flow (0.01 in.) at Asphalt Content		
Laboratory	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%	5.5%	6.0%	6.5%
District															
1	2.28	2.29	2.29	3.3	2.1	1.5	78.3	86.2	90.6	2,776	2,691	2,237	10	10	12
2	2.31	2.31	2.31	2.2	1.5	0.9	84.9	90.0	94.2	3,528	3,194	2,494	16	17	19
3	2.28	2.28	2.28	3.5	2.7	1.9	77.6	83.0	88.2	3,012	3,000	2,664	10	11	13
4	2.29	2.30	2.29	3.3	2.0	1.9	78.8	87.0	88.5	2,450	2,762	2,109	7	10	12
5	2.29	2.30	2.30	3.4	2.1	1.7	78.6	86.6	89.5	2,790	2,455	2,065	10	10	13
6	2.29	2.30	2.29	3.4	2.1	1.7	78.4	86.3	89.4	3,561	3,224	2,572	7	8	11
Main laboratory	2.28	2.30	2.30	3.6	2.0	1.4	77.2	87.0	91.4	2,166	2,158	1,921	14	14	17
Average	2.29	2.30	2.30	3.2	2.1	1.6	79.1	86.6	90.2	2,897	2,783	2,295	11	11	14
Standard deviation	±0.013	±0.012	±0.09	±0.047	±0.36	±0.35	±2.0	±2.0	±2.1	±518	±391	±284	±3.49	±2.91	±2.95
Range	0.03	0.03	0.03	1.4	1.20	0.9	7.7	7.0	6.0	1,395	1,036	734	9	9	8

Characteristics of the equipment used, the procedures employed, and the results obtained during the study were reviewed by personnel at the central laboratory, and the following discrepancies were highlighted:

1. The size (weight) of the individual batches of aggregate and bitumen, and, therefore, the height of the compacted specimens, was not consistent. The standards require that the appropriately compacted specimen have a height of 2.5 ± 0.05 in.
2. Several district laboratories used hydraulic jacks (instead of the testing machine) to extract the compacted specimens from the mold.
3. District laboratories were using nonstandard breaking heads.

Canadian Studies

Canada has an ongoing mix and asphalt exchange program in which private and public laboratories in different parts of the country cooperate in the testing of bituminous mixes. Each year a different agency agrees to be the host and supplies the ingredients (aggregate and bitumen) to the participating laboratories. The laboratories agree to follow a common format or procedure (provided by the host agency) with the objectives of eliminating discrepancies in various laboratory procedures and equipment and ensuring that valid comparisons of data can be made. Nonstandard breaking heads were encountered in the 1984 exchange study in which 31 laboratories participated (*3*). The horizontal dimension of the breaking heads was found to range between 108 and 127 mm, and the vertical dimension ranged between 38 and 63 mm (*3*).

On the basis of a review of the Marshall property test results, the authors of the Canadian study concluded that part of the variation in the test results was due to the variation in the dimensions of the breaking head.

Both manual and mechanical hammers were used in the 1983 Canadian study. Results obtained with the manual hammer were fairly consistent, but large variations were associated with the mechanical hammer. The authors attributed these variations to several equipment-related factors, such as the mass, drop (free-fall), and shape of the hammer (*3*).

The instructions issued by the host for the 1980 study (Manitoba Department of Highways and Transportation) recommended that each face of the specimen be compacted with 75 blows of the manual hammer (*4*). Also, it was required that a description of the compaction pedestal be submitted with the test results. These instructions were issued in light of prior experience that indicated that differences in hammers and compaction pedestals may contribute to the variation in results obtained from different laboratories. Another reason for providing specific instructions to the participants was to eliminate the subtle differences in the manner in which different operators and technicians interpret standard test procedures (*2*).

Once again, significant differences in density and Marshall design properties were found and attributed to equipment variables. Clearly, the results of the studies that have been reviewed indicate that there can be significant variability in Marshall test results and that this variability can be attributed to operator as well as equipment variability. This is in agreement with the opinions expressed by the experts who were interviewed.

TECHNIQUES FOR CALIBRATION

The review of relevant literature, both published and unpublished, and interviews with knowledgeable industry and state highway agency personnel indicate that techniques and procedures for quantifying the effects of these variables and their interactions are currently unavailable. From the literature review and contact with other researchers, the need for a calibration procedure for the Marshall compaction apparatus is readily apparent. It is primarily because of the absence of such a procedure that several private and public agencies, both in the United States and in Canada, regularly participate in round-robin or mix-exchange programs. These mix-exchange programs enable laboratories to evaluate their results with reference to results obtained by the other participating laboratories.

In the Canadian mix-exchange program, results submitted by participating laboratories are evaluated in the following manner: The mean, standard deviation, and ±2 standard deviation limits are calculated for all data received for each test. Any test results from laboratories with data falling outside these limits (i.e., the 95 percent range) are eliminated, and a new mean,

TABLE 5 1983 CANADIAN ASPHALT MIX EXCHANGE (*3*)

	Bulk Specific Gravity			Stability (kN)			Flow (0.01 in.)		
Laboratory No.	Hand	Mechanical	Hand[a]	Hand	Mechanical	Hand[a]	Hand	Mechanical	Hand[a]
1	2.374	2.363	2.375	11.9	11.6	12.9	12	11	12
2	2.372	2.357	2.385	11.4	11.6	15.0	12	13	11
3	2.389	2.384	2.396	11.5	10.8	14.2	11	10	9
4	2.386	2.376	2.381	11.2	11.8	13.0	12	13	10
5	2.370	2.365	2.370	8.6	8.4	9.4	10	10	9
6	2.382	2.299	2.395	10.3	6.0	13.9	13	10	9
7	2.394	2.382	2.398	11.7	11.2	13.8	13	11	11
8	2.359	2.347	2.359	9.6	8.1	9.7	12	11	8
9	2.363	2.347	2.347	9.7	7.7	9.2	8	9	10
10	2.412	2.405	2.395	11.4	12.6	12.5	18	16	9
11	2.390	2.317	2.399	14.0	8.2	13.3	13	13	8
12	2.362	2.358	2.378	14.5	13.3	12.5	14	13	12
14	2.397	2.335	2.384	12.5	7.6	13.4	11	12	10
15	2.382	2.370	2.362	11.0	10.3	10.9	13	13	9
16	2.401		2.393	12.1		14.4	(19)		10
17	2.391	2.396	2.377	11.3	10.2	15.1	7	8	9
18	2.393	2.380	2.378	13.8	12.7	12.0	16	13	12
19	2.393		2.394	11.2		13.6	11		11
20	2.384	2.342	2.354	11.5	9.6	11.6	13	11	9
21	2.377	2.330	2.399	12.9	12.1	14.4	14	12	12
22	2.372	2.356	2.346	10.2	9.6	10.7	13	12	11
23	2.390	2.394	2.372	12.9	14.6	11.8	12	12	9
26	2.383	2.369	2.386	10.5	10.1	13.8	11	10	10
27	2.383	2.337	2.367	11.0	9.8	11.2	14	11	9
28	(2.322)	(2.273)	(2.321)	(5.7)	(4.3)	(8.2)	11	11	10
29	2.394		2.376	11.3		10.7	13		10
30	2.401	2.385	2.397	11.8	13.4	14.1	15	16	11
31	2.377	2.361	2.370	9.4	8.1	11.6	9	9	10
Statistical Summary for All Data									
Mean	2.382	2.357	2.377	11.3	10.2	12.4	12	12	10
Standard deviation	0.017	0.031	0.019	1.8	2.5	1.9	3	2	1
95% confidence interval	2.348–2.416	2.295–2.419	2.339–2.415	7.7–14.9	5.2–15.2	8.6–16.2	6 –18	8–18	8–12
Data range	2.322–2.412	2.273–2.405	2.321–2.322	5.7–14.5	4.3–14.6	8.2–15.1	7 –19	8–16	8–12
Statistical Summary for Select Data (excludes data that were outside 95% confidence interval)									
Mean	2.384	2.361	2.379	11.5	10.4	12.5	12		
Standard deviation	0.014	0.026	0.016	1.4	2.2	1.7		2	1
95% confidence interval	2.358–2.410	2.309–2.413	2.547–2.411	8.7–14.3	6.0–14.8	9.1–13.9	8 –16	8–16	8–12
Data range	2.359–2.412	2.299–2.405	2.346–2.399	8.6–14.5	6.0–14.6	9.2–15.1	7 –18	8–16	8–12

NOTE: Data in parentheses are outside the 95 percent confidence interval and were rejected in calculating the select set of statistical data.
[a]Six specimens were compacted by hand; three were tested in the district laboratory; the specimens in this column were shipped to the central laboratory for testing.

standard deviation, and ±2 standard deviation are determined. The remaining data are checked against these new limits. This procedure is repeated until all data from the remaining laboratories fall within the associated 95 percent range (*4*). Because all participating laboratories are processing and testing the same mix, comparison of results helps each laboratory to assess how well it is performing with reference to other laboratories in the cooperative program. The procedure is given in Table 5 (*3*).

Because of the economy of time and effort, most public and private agencies use mechanical hammers in their laboratories. ASTM D 1559 does not allow mechanical hammers and AASHTO T-245 permits the use of a mechanical hammer only if it is calibrated to give results comparable to those of the manual hammer. A procedure that has been used for calibrating mechanical hammers is described next.

Several samples of a given mix are compacted with a desired compactive effort (e.g., 50 or 75 blows) of a standard, nonsupported manual hammer. The average bulk density achieved is considered the target standard bulk density. Specimens of the same mix are then prepared with the mechanical hammer by using a range of compactive efforts. The relationship between the bulk density and the associated compactive effort is plotted

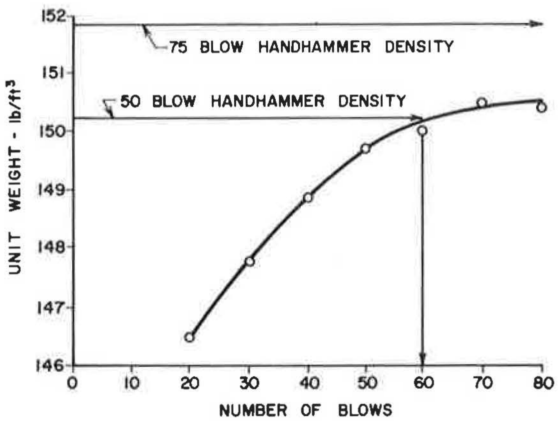

FIGURE 2 Procedure for calibrating a mechanical hammer.

as shown in Figure 2. The number of blows that is required with the mechanical hammer to attain the target bulk density is then determined from the plot.

The calibration (i.e., number of blows) is specific to a given hammer and a given mix, however; and, if more than one mechanical hammer is used in a laboratory, each one should be separately calibrated for each specified compactive effort (i.e., 50 or 75 blows) and for each mix tested. Data on the characteristics of mechanical hammers, listed in Table 6, were collected during a Canadian mix-exchange study (*5*). Table 6 gives the variations in the mass and drop of the hammer and the thickness and type (beveled or flat) of the compaction foot. However, it is possible that the associated pedestal and foundation reactions also varied, creating additional and undocumented sources of variability. Thus, in order to reduce the between-laboratory variation in bulk density results for a given mix, it would be necessary to calibrate each hammer used with the same standard, unsupported manual hammer. Also, the calibration procedure should be periodically repeated to account for wear and repair or replacement of equipment components. Finally, calibration of the hammer can only address the variation in bulk density. It cannot eliminate, or even reduce, the variation in flow and stability associated with a nonstandard or defective breaking head.

As shown in Figure 2, it is possible that a given mechanical hammer may not achieve the target bulk density obtained with a standard manual hammer. This may result from use of a nonstandard compaction pedestal, a nonstandard reaction (foundation), or some other unknown variable. From the data presented and the state-of-the-art review it is clear that a methodology for calibrating the compactive effort of Marshall compaction devices is needed.

A procedure called the "penny test" has been used by some to calibrate pedestal reaction (personal communication with Wade Bentensen, Utah Department of Transportation, 1987). This test consists of placing a copper penny in the mold and subjecting the penny to 35 blows of the hammer; the reaction is gauged by the diameter of the penny. This procedure is empirical and does not merit further consideration as a calibration procedure (*6*).

Further, on the basis of the literature review and results of the telephone interviews, the research team has concluded that

TABLE 6 CHARACTERISTICS OF MECHANICAL HAMMERS FROM 23 LABORATORIES (*4*)

Laboratory No.	Mass of Hammer (kg)	Drop of Hammer (mm)	Thickness of Compaction Foot (mm)	Manufacturer	No. of Blows	Bulk Specific Gravity
1	4.54	457	15.9–17.4	H-D[a]	60	2.411
2	4.54	470	10	M-D	75	2.377
3	4.50	457	13.0–15.0	M-D	60	2.421
5	4.54	457	11.3–14.4	H-T	60	2.443
6	4.57	456	16	H-S	60	2.388
7	4.54	457	11.5–13.5	–	60	2.393
8	4.49	459	6.0–12.0	ML	76	2.407
9	4.54	457	12	M	63	2.454
10	4.53	457	12.0–18.0	M-	60	2.408
11	4.54	453	Flat	P	75	2.421
12	4.54	457	5.0–12.0	ML	75	2.419
13	4.53	457	12.6–15.2	H-D	50	2.380
14	4.50	457	6.5–9.5	R	75	2.390
15	4.51	455	10.2–14.2	R	75	2.437
16	4.68	457	25.4	S-S	80	2.439
17	4.54	457	11.0–14.0	M	70	2.454
18				H	60	2.319
21	4.54	457	12.0	S	75	2.374
22	4.54	442	19.0–79.0	I	75	2.429
24	4.70	456	19.3–19.4	M-D	61	2.403
27	4.55	457	12.3–14.9	H-D	60	2.444
28	4.53	456	12.0			
	4.58	454	12.0	I	100	2.418
	4.56	450	12.0			
29	4.70	453	11.0	I	75	2.376

[a]Letters after dashes have the following meanings: S = single-hammer compactor, D = double hammer, and T = triple-hammer compactor. I = homemade design.

practical and reliable procedures and equipment for calibrating the Marshall apparatus are currently unavailable.

PRELIMINARY EXPERIMENTAL EVALUATION

To examine the feasibility of measuring fundamental process parameters of the Marshall hammer operation, a research study was performed in the Materials Testing Laboratory at the Pennsylvania Transportation Institute. The study was designed to explore the possibility of obtaining meaningful compaction process information with a limited amount of instrumentation and sophistication and was not intended to be a comprehensive experimental evaluation of the compaction process.

The study conducted by the authors consisted of instrumenting a mechanical Marshall compaction hammer with three accelerometers (*6*). The impact-time histories of the accelerometers were recorded with an FM tape recorder. The tape recordings of the accelerations were then analyzed by applying rudimentary digital signal processing techniques. Interpretation of the data allows several conclusions with regard to the compaction process and the associated variables to be drawn. In addition, this preliminary evaluation formed the foundation for recommending further experimental testing and the instrumentation required to properly define the process variabilities between different compaction hammers.

Experimental Testing Procedure and Data Acquisition

The procedures for evaluating the Marshall compaction process paralleled techniques originally developed to examine hot-forging hammer operations (*7*). Shock accelerometers are mounted on the critical components of the hammer assembly that affect energy transfer. These components are the falling mass, the 1-in.-thick steel (base) plate on the top of the pedestal, and the floor in the vicinity of the pedestal. The accelerometers were oriented in the vertical direction to measure the energy transfer of the hammer's structural members during the compaction impact. All of the acceleration data were recorded on a multichannel FM tape recorder to facilitate later analysis (*6*). The instrumentation schematic used for testing is shown in Figure 3.

PCB Piezotronics model 305A shock accelerometers were used to measure the falling-mass and base plate accelerations. A PCB 302A general-purpose accelerometer was mounted on the floor next to the pedestal. The three channels of acceleration data were recorded on a TEAC MR 10 four-channel FM recorder. During recording, the data were simultaneously monitored on an AT&T PC6300 microcomputer with a Computational Systems, Inc., Wavepak data acquisition system. The Wavepak system allows the microcomputer to emulate a digital oscilloscope and dual-channel FFT analyzer. The digital data mode, with its inherent pretrigger data-capture capability, is critical to the analysis of this short-time-duration phenomenon.

The data collection phase commenced with representative impacts of the hammer to ensure that the gain settings on all of the instrumentation were adjusted to the appropriate levels.

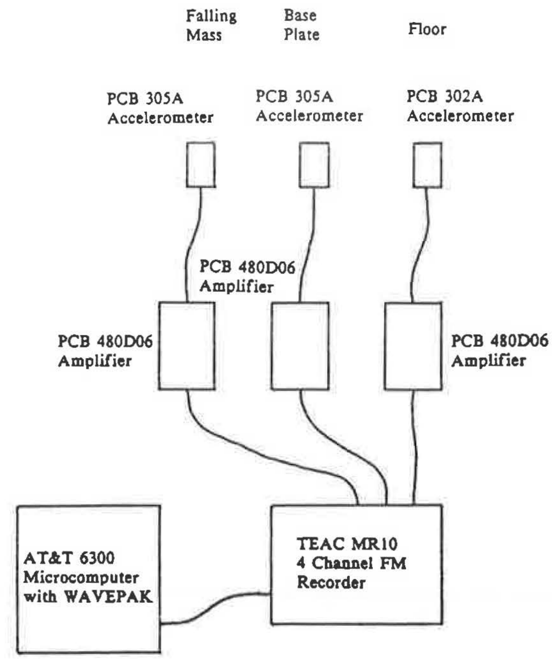

FIGURE 3 Instrumentation schematic for Marshall hammer data collection.

Acceleration data were then recorded for a total of 15 specimens compacted with 35 blows on each side. Pennsylvania's ID-2 dense-graded wearing course mix was used for the study. The testing procedure followed the ASTM standard except that a mechanical hammer was used. The specimen compaction temperature was targeted at 280°F.

Analysis of Marshall Hammer Acceleration Data

The tape-recorded data were further analyzed by using the digital processing capabilities of the AT&T microcomputer and Wavepak system. After the appropriate playback gain and calibration factors had been determined, the representative acceleration time histories for the three channels were captured and analyzed by using several different approaches.

Figure 4 shows typical acceleration signals from the three channels recorded: falling mass, base plate, and floor. The falling-mass acceleration shows that the impact has a very short duration of around 1 msec and a peak acceleration greater than 2000 *g*. The impact excites the longitudinal vibration mode of the falling mass, which appears as longer-duration ringing in the signal. The actual hammer impact is not clearly apparent from the acceleration signal because of the structural ringing. The base plate acceleration is also shown in Figure 4. The acceleration basically shows only the structural ringing of the base plate with peak levels of less than 250 *g*. Figure 4 also shows the acceleration measured on the floor next to the hammer installation; the signal shows a significant acceleration pulse on the order of 25 *g*. The high level of the floor's response to the impact is indicative of the energy flow away from the hammer and into the floor. This indicates that the floor support is, potentially, a major source of variability among different laboratories.

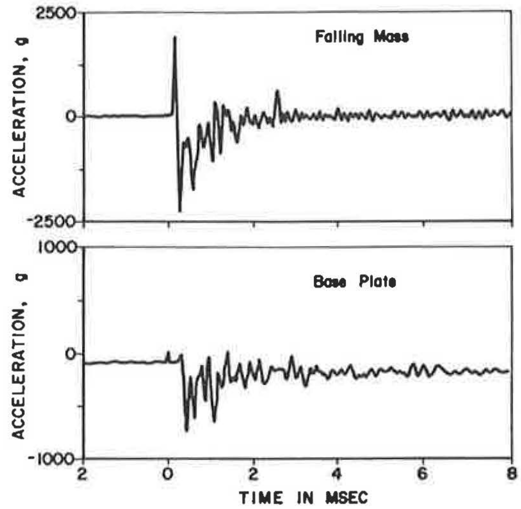

FIGURE 4 Acceleration signals from a typical impact.

Analysis of the acceleration signals recorded for each blow throughout the compaction process confirms that there is little variability between successive blows; the trends are similar between sequences except that the blow strength tends to become greater as the specimen becomes more compacted. In an effort to more quantitatively examine the repeatability of the hammer process, the energy autospectrum of the falling-mass acceleration was estimated for three specimens by considering every fifth blow in the sequence. Two representative spectra for the falling mass and the base plate are shown in Figures 5 and 6. Successive spectra were quite similar, with only slight variations among them. The integral of the area under the spectra curves is proportional to the energy imparted to the sample. Within the tolerance permitted by this experiment, the area

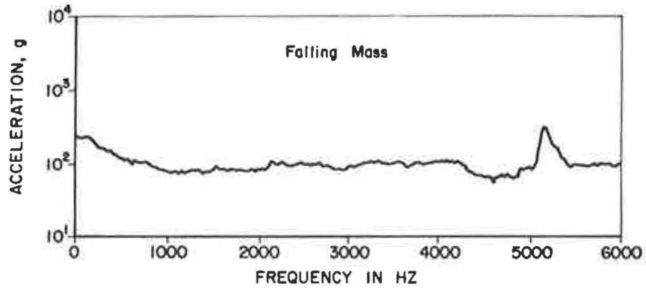

FIGURE 5 Falling-mass acceleration autospectrum, estimated from every fifth blow, Specimen 1.

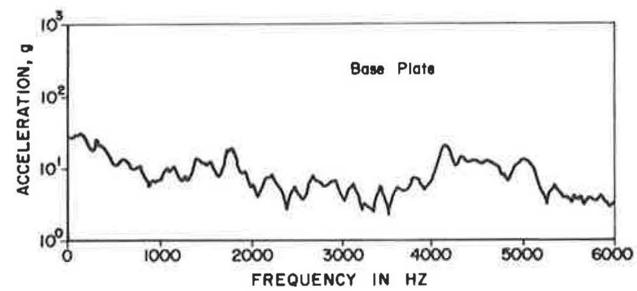

FIGURE 6 Base plate acceleration autospectrum, estimated from every fifth blow, Specimen 1.

under the curves for the different specimens was judged to be equivalent. This situation indicates that a significant degree of process repeatability exists among specimens compacted using the same hammer.

The deformation energy imparted to the mix specimen can be calculated from an integration of the acceleration signals. However, the structural ringing in the signals is sufficiently strong to preclude a direct measurement. In an effort to extract this data, a low-pass, linear-phase electrical filter was introduced to remove the high-frequency ringing. Figures 7 and 8 show typical filtered acceleration time histories for the curves that are shown for the falling mass in Figure 4. After some experimentation, it was found that a filter with a cutoff between 1 and 2 kHz provided the best response. The filter does eliminate the ringing, but it also modifies the signal. Unfortunately, this distortion is sufficient to preclude accurate estimation of the impact energy. With further experimentation, however, the proper filter combination could be determined and calibrated to accurately estimate impact energy from data of this type.

Discussion of Test Results

From the experiment performed, several conclusions can be drawn with regard to the hammer and compaction process. These are outlined next:

1. The compaction process is repeatable for the specimens prepared with the hammer used in the study. For the single compaction apparatus that was studied, random variations that occur during the impacts, such as changes in rod friction, misalignment of the mold, and mold friction, appear not to affect the process.

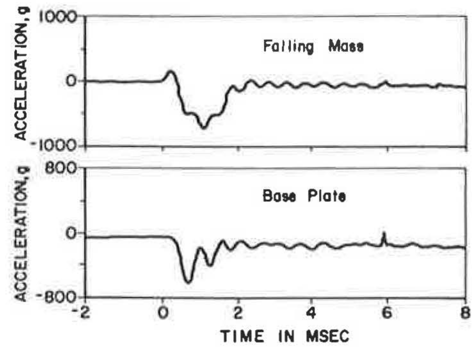

FIGURE 7 Falling-mass acceleration time history with 2-kHz low-pass filter.

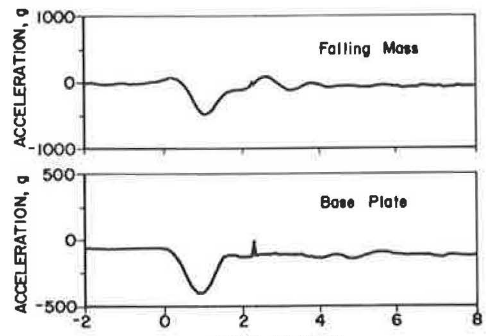

FIGURE 8 Falling-mass acceleration time history with 1-kHz low-pass fileter.

2. The interaction of the hammer, pedestal, and supporting foundation appears to be critical. The acceleration levels recorded indicate a significant interaction with the surrounding support structure. Although the acceleration of the base plate and the floor differed by an order of magnitude, 250 and 25 *g*, respectively, the magnitude of the energy transmitted is unknown because the effective mass of the floor, which is much greater than that of the pedestal, is unknown. This indicates that the relative stiffness of the supporting floor could cause variations in the compaction process and, hence, affect the test results. The authors recommend that the pedestal be mounted on a large block of concrete (e.g., at least 3 ft by 3 ft by 3 ft) rather than on the floor of a building.

3. Reliable process information can be extracted from the hammer with relatively simple instrumentation. The structural ringing makes it difficult to extract the deformation impact from the rest of the signal. Filtering reduces the ringing effect but colors the resulting signal. This distortion makes it difficult to estimate the actual impact energy, but, nevertheless, the signal can be used for comparison purposes.

4. For the hammer evaluated, the impact consisted of a single blow with no repetitive bounces resulting from rebound of the hammer head.

FINDINGS AND CONCLUSIONS

Large variations in compacted density and other Marshall mixture design values may occur when a given mix is compacted with different compaction hammers, and these variations are of concern to both public highway agencies and private industry. Although ASTM and AASHTO procedures for testing Marshall properties were originally written for a hand-held, unsupported hammer, currently, the AASHTO standard (T-245) permits the use of a mechanical hammer. This research team found that several different makes of mechanical hammer are currently in use, and some agencies use homemade hammers. A wide variation in hammer characteristics was found.

Several hammer-related variables that play a key role in influencing Marshall test results were identified. The people surveyed most frequently cited pedestal support and hammer alignment as the equipment characteristics that most significantly affect the level of compaction achieved with a given set of equipment. This finding was verified by the preliminary test results developed in the laboratory study. However, it also was found that inconsistencies in test results could be compounded by subtle differences in the interpretation of the procedures and by the use of nonstandard or defective breaking heads. Operator-related factors, factors associated with the compaction device and the breaking head, and their interactions, together, constitute a fairly complex environment.

A method (procedure and equipment) for quantifying the effect of key equipment-related variables on Marshall test results is currently not available. In the absence of such technology, several agencies, both in the United States and in Canada, regularly cooperate in round-robin or mix-exchange programs, which enable them to evaluate their own performance relative to the performance of other participating agencies. An empirical procedure for measuring the pedestal reaction of a mechanical hammer is currently available. However, this procedure, in which the diameter of a compacted penny is measured, cannot be used to calibrate the Marshall hammer; nor does it address variations in Marshall properties (stability and flow) resulting from equipment variables such as different breaking heads.

The feasibility of developing instrumentation that would measure the amount of energy delivered to the specimen during the compaction process was demonstrated. However, further development is needed to adapt this technology to the field calibration of Marshall hammers. The development and implementation of a field compaction procedure would provide

1. A means for evaluating the characteristics of different compaction devices and the interaction of these devices with the pedestal and base support. (The latter point is important because pedestal type and base support generally vary from site to site.)
2. A means to identify within- and between-operator variability associated with variations in test procedure.
3. A datum that could be used to standardize the compaction process and provide a reference in cases requiring litigation.

ACKNOWLEDGMENTS

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Study on Mix Design Criteria for Controlling the Effect of Increased Tire Pressure on Asphalt Pavement

OK-KEE KIM, CHRIS A. BELL, JAMES E. WILSON, AND GLENN BOYLE

As axle loads have increased, the use of higher tire pressures has become more popular in the trucking industry, and radial tires are predominantly used. However, existing mix design procedures may not produce mixtures capable of withstanding higher tire pressures. They also may not identify potentially highly deformable mixtures. To evaluate the mix design process used by Oregon State Highway Division, aggregate from four different sources was used. One percent lime slurry was added to two aggregates. Six different aggregate gradations, including the Fuller maximum density gradation, were tested. In addition to the routine asphalt mix tests, a simple creep test was run for 3 hr at 40°C, and a compression stress of 0.1 MPa (14.5 psi) was applied. According to the results of creep tests, it is not always true that a mix with a high Hveem stability value resists deformation better than one with low stability. This indicates that current mix design criteria are probably inadequate for producing mixtures capable of withstanding high tire pressures and for identifying potentially highly deformable mixtures. In general, creep stiffness decreases with increases in the percentage of aggregate passing the No. 200 sieve. The effect of the percentage passing the 1/4-in. or No. 10 sieves on creep stiffness is not clear. The results indicate that adding 1 percent lime slurry improves the resistance to deformation of asphalt mixes.

The economics of truck transportation has tended to cause the average gross weight of trucks to increase so that a majority of trucks are operating close to the legal gross loads or axle loads (*1*). In 1982 the federal government permitted 80,000-lb gross vehicle weights, 20,000-lb single axle weights, and 34,000-lb tandem axle weights on Interstate highways. Tandem axle weights of 34,000 lb allowed a potential 12,000-lb load on the steering axle. Many states, including Oregon (*2*), also issue permits for trucks to operate above normal legal load limits.

As axle loads have increased, the use of higher tire pressures has become more popular in the trucking industry. A recent survey in Texas (*3*) indicated that trucks typically operate with tire pressures of about 100 psi in that state. Another study in Oregon (*4*) showed that about 40 percent of radial tires are inflated to more than 110 psi and that the average inflation pressure is 102 psi and 82 psi for radial tires and bias tires, respectively.

Higher tire pressures decrease the contact area between the tire and the pavement, resulting in reduced tire friction or skid resistance and increased potential for pavement damage under the high stress. Higher tire pressures contribute to greater deformation in flexible pavements, manifested as severe wheel track rutting.

In Oregon there have been several occurrences of severe wheel track rutting associated with the high tire pressures that have prevailed in recent years. Rutting is a function of deformation in all layers of a flexible pavement structure, but, with high tire pressures, the deformation in the asphalt concrete mixture is a major contributor. Existing mix design procedures may not produce mixtures capable of resisting high tire pressures. Similarly, they may not identify potentially highly deformable mixtures.

A study of procedures for controlling the effect of increased tire pressure on asphalt concrete pavement damage (*4*) was performed by the Oregon Department of Transportation (ODOT) and Oregon State University (OSU). This paper is about part of this study: the results of mix design evaluation and the results of creep testing to predict rut depth in asphalt pavement. The objectives of this paper are

1. To present and analyze the effectiveness of existing asphalt concrete mix design methods for limiting excessive deformation caused by higher loads and tire pressures and
2. To present and analyze the results of creep testing to predict deformation in asphalt surface layers.

BACKGROUND

Mix Design

The Marshall and Hveem methods of mix design have been widely used with satisfactory results. For each of these methods, criteria have been developed by correlating results of laboratory tests on compacted paving mixes with performance of the paving mixes under service conditions.

However, the limitations of such empirically based methods of pavement mix design have become increasingly apparent in recent years as traffic loads, tire pressures, and numbers of trucks have increased. Increasing demands on asphalt pavements from both higher traffic volumes and higher truck tire pressures have caused highway engineers to examine the foundations of asphalt mix design guidelines and procedures in order to see how best to cope with these challenges.

Existing mix design procedures may not produce mixtures capable of withstanding higher tire pressures. They also may

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not identify potentially highly deformable mixtures. Such a situation was identified by Finn et al. (5) when designing mixtures for heavy-duty airfield pavements on which extremely high tire pressures occur. They used a simple creep test, similar to that developed by Shell researchers (6), to complement Marshall and Hveem mix design procedures and to quantify deformation characteristics of the mix.

Hicks and Bell (7) recently completed a study for the Oregon State Highway Division (OSHD) to evaluate their current specifications and mix design process, which is based on the Hveem procedure. They indicated that gradation of aggregate can be one of the main contributors to producing tender mixes. Many researchers (8) indicate that the potential for constructing tender mix pavements with possible deformation problems increases if gradation values for a 3/4-in. maximum size mix are greater than the following:

Sieve	*Percentage Passing*
No. 4	55
No. 10	37
No. 40	16
No. 200	3–7

Further, they indicate that gradation curves that cross back and forth over the maximum density curve, especially in the region of the No. 30 to No. 80 sieve, tend to produce tender mixes.

Creep Test

In a major effort to develop rational procedures for the design of asphalt mixes, an attempt has been made to develop a test method suitable for judging the stability properties of asphalt mixes. Van de Loo (9) defined stability of an asphalt mix as its resistance to rutting in an actual pavement (i.e., under varying conditions of climate, traffic volume, and traffic load).

Many researchers have used the creep test (static or repeated mode) as a relatively simple test to predict rutting (or permanent deformation) of an asphalt pavement. In 1973 theoretical deformation models of asphalt mixes were formulated by J. F. Hills (10). It was assumed that any deformations in the mix are the result of sliding displacements between adjacent mineral particles, separated by a thin film of asphalt. He interpreted the results in terms of a mix stiffness (S_{mix}) as a function of bitumen stiffness (S_{bit}). Hills stated that, in addition to the effect of the volume concentrations of the mineral aggregate, the gradation, shape, and surface texture of the aggregate play a role, and the state of compaction exerts a strong influence on behavior.

Grob (11) recommends performance of the unconfined, static creep test that was standardized during Colloquium 1977 in Zürich. The recommended sample size is the same as that of normal Marshall specimens (i.e., 4 in. in diameter and 2.5 in high), and a steady temperature of 40°C should be achieved before the test commences. The constant load of 0.1 MPa (14.5 psi) should be applied without any impact and have a duration of 1 hr. A loading time of 1 hr is arbitrary.

The deformation of an asphalt specimen is measured as a function of loading time at a fixed test temperature. The general equation of the creep curves is

$$\log(\varepsilon) = c + n\log(t) \quad (1)$$

where ε is creep strain at time t and c and n are constants. The constants c and n are related to test conditions such as uniaxial stress and temperature, as well as asphalt cement content and the factors indicated by Hills. The constant n represents the inclination of the straight line. Relatively small n indicates less viscous behavior and relatively large n predominantly viscous behavior (11). It has been found that the level of instantaneous response increases with the amount of filler and bitumen (12). Furthermore, the time dependence of the vertical displacement has been associated with the viscosity of the mortar, which is related to the filler-binder ratio.

DESIGN OF EXPERIMENTS—TESTS ON ASPHALT MIXTURES

Variables Considered

Aggregate from four different sources was used for the laboratory mixture study:

1. Morse Brothers Pit (gravel),
2. Cobb Rock Quarry,
3. Hilroy Pit (gravel), and
4. Blue Mountain Asphalt Pit (gravel).

For the mix with the aggregates from Cobb Rock Quarry and Blue Mountain Asphalt Pit, the aggregates were treated with a 1 percent lime slurry and mellowed for a minimum of 24 hr before they were used in the mix.

The variables considered in laboratory mixture preparation for the creep test were

1. Asphalt cement content:
   A: 4, 5, and 6 percent;
   B: 4.5, 5.5, and 6.5 percent; and
   C: 5, 6, and 7 percent.
2. Aggregate gradations A through F (Table 1):
   A: 65 percent passing 1/4 in., 32 percent passing No. 10, and 5 percent passing No. 200;
   B: 60 percent passing 1/4 in., 29 percent passing No. 10, and 5 percent passing No. 200;
   C: Fuller curve—60 percent passing 1/4 in., 36 percent passing No. 10, and 8 percent passing No. 200;
   D: Same as B except 35 percent passing No. 10;
   E: 60 percent passing 1/4 in., 34 percent passing No. 10, and 5 percent passing No. 200; and
   F: Same as E except 8 percent passing No. 200.

Table 2 gives the aggregate gradations considered for each aggregate source. The properties of asphalt cements used are given in Table 3.

Specimen Preparation and Test Program

Following the standard ODOT procedure (13) of using a kneading compactor, specimens 4 in. (100 mm) in diameter by 2.5 in. (63 mm) high were fabricated from four different aggregate sources.

TABLE 1 PERCENTAGES OF AGGREGATE GRADATIONS PASSING SIEVE SIZES

	Morse Brothers Pit Gradation			Cobb Rock Quarry Gradation				Hilroy Pit Gradation						Blue Mountain Asphalt Pit Gradation				
Sieve	A	B	C	A	B	C	D	A	B	C	D	E	F	A	B	C	D	E
1 in.																		
3/4 in.	–	–	–	–	–	–	–	100	100	100	100	100	100	–	–	–	–	–
1/2 in.	100	100	100	100	100	100	100	99	98	99	99	98	98	100	100	100	100	100
3/8 in.	98	97	82	99	99	86	82	86	85	82	85	85	85	87	87	86	87	87
1/4 in.	86	83	72	82	78	73	72	76	72	72	72	72	72	77	74	73	73	73
No. 10	65	60	60	66	60	60	60	65	60	60	60	60	60	65	60	60	60	60
No. 40	32	30	37	32	29	37	37	33	31	37	37	34	34	32	29	36	36	34
No.	13	11	18	13	11	19	19	14	13	19	19	14	14	14	13	16	16	15
200	4.7	4.3	6.8	6.7	6	9	6.9	4.5	4.2	5.9	4.3	5	6.9	5	4.5	7	5	5.2

TABLE 2 AGGREGATE GRADATIONS CONSIDERED FOR EACH AGGREGATE SOURCE

	Aggregate Gradation					
Aggregate Source	A	B	C	D	E	F
Morse Brothers Pit	X	X	X			
Cobb Rock Quarry (with 1% lime slurry)	X	X	X	X		
Hilroy Pit	X	X	X	X	X	X
Blue Mountain Asphalt Pit (with 1% lime slurry)	X	X	X	X	X	

TABLE 3 PHYSICAL PROPERTIES OF ASPHALT CEMENT

Property	I	II	III	IV
Grade	AR 4000	AR 4000	AR 4000	AR 4000
Original				
Penetration at 77°F	68	68	68	61
Absolute viscosity at 140°F (poises)	1339	1349	1349	2111
Kinematic viscosity at 275°F (cSt)	261	248	248	352
Flash point, open cup (°F)	600	605	605	580
After Rolling Thin Film Oven Test				
Penetration	41	40	40	32
Absolute viscosity at 140°F (poises)	3033	3139	3139	5860
Kinematic viscosity at 275°F (cSt)	367	365	365	562
Loss on heating (%)	0.45	0.52	0.52	0.65

NOTE: I = Morse Brothers Pit, II = Cobb Rock Quarry, III = Hilroy Pit, and IV = Blue Mountain Asphalt Pit.

Figure 1 is a flowchart of the test program followed in this study. The ODOT testing program included the conventional mix tests such as the Hveem stability test (AASHTO T-246), Rice maximum specific gravity test (AASHTO T-209), bulk specific gravity test (AASHTO T-166), and repeated load diametral test for resilient modulus (as compacted and after moisture conditioning). OSU performed the creep test with 54 laboratory-fabricated specimens as described in the following subsection.

Test Methods

After laboratory mixes were prepared, repeated load diametral tests and creep tests were performed. The procedures are outlined next.

Resilient Modulus

The resilient modulus test (*4*) was performed using the repeated load diametral test apparatus. The maximum load applied and the horizontal elastic tensile deformation were recorded to determine the resilient modulus using the following equation:

$$M_R = P(0.2692 + 0.9974\nu)/(\Delta H \times t) \tag{2}$$

where

M_R = resilient modulus (psi),
ΔH = horizontal elastic tensile deformation (in.),
P = dynamic load (lb),
t = specimen thickness (in.), and
ν = Poisson's ratio.

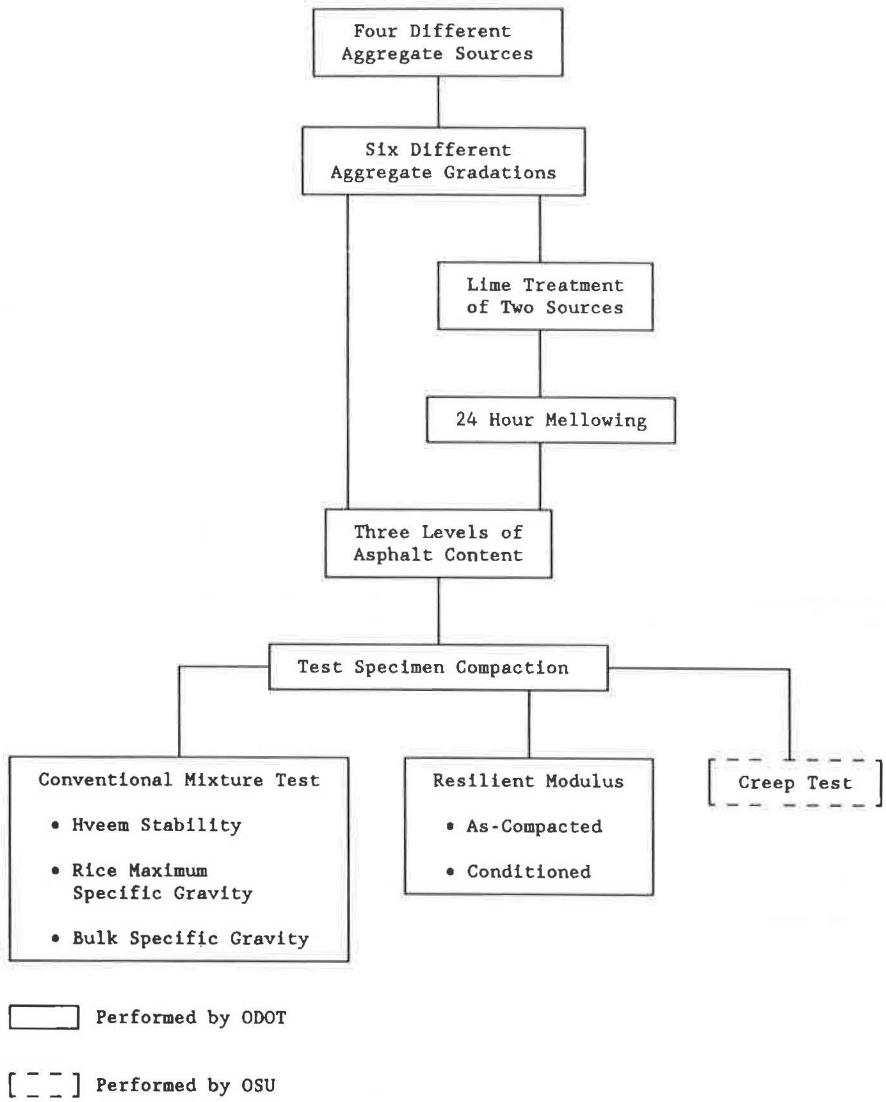

FIGURE 1 Flowchart for test program.

Poisson's ratio was assumed constant and equal to 0.35, which simplified Equation 2 to

$$M_R = 0.6183P/(\Delta H \times t) \tag{3}$$

During the test, the dynamic load duration was fixed at 0.1 sec and the load frequency at 60 cycles per minute. A static load of 10 lb (4.5 kg) was applied to hold the specimen in place. The test was carried out at 77°F (25°C).

Each specimen of each aggregate was tested both before and after conditioning. The specimen-conditioning procedure was based on the moisture damage test defined by Lottman (*14*).

Creep Test

OSU was responsible for developing a simple creep test and running the test. For the creep test, a loading device for soil consolidation and a data acquisition and control unit with a personal computer were used. The creep test was run for 3 hr at 40°C, and a compression stress of 0.1 MPa (14.5 psi) was applied. The creep test procedure is as follows:

1. Put a loading device for soil consolidation in an environmental cabinet and connect to the repeated load test control cabinet. Put the specimens and a dummy specimen with a thermistor in the environmental cabinet. Set the regulator at 0.1 MPa and control the air pressure through the repeated load test control cabinet.
2. Warm the inside of the environmental cabinet to 40°C and check the temperature of the dummy specimen using the data acquisition system and thermistor.
3. After the temperature of the dummy specimen core reaches 40°C, put a specimen on a load plate. Put a linear variable differential transformer on the bottom plate and attach a thermistor to the specimen. Check the level of the bottom plate before running the test.
4. Wait for 5 to 10 min after closing the environmental cabinet door to keep the temperature at 40°C.
5. Apply a pressure of 10 kPa as a preload for 2 min.
6. Apply a pressure of 0.1 MPa and run the computer program.

Kim et al. (*4*) describe the apparatus and the procedure for sample preparation in detail. Also described are the computer programs used to monitor the temperature and measure the deformation of a specimen.

RESULTS

Mix Design

A summary of the mix design for the aggregate from each source with different aggregate gradations is given in Table 4. Table 4 includes the resilient modulus (both as compacted and after conditioning) and the minimum asphalt content for the retained modulus ratio of 0.7. The retained modulus ratio is defined by Equation 4:

$$\text{Retained modulus ratio} = M_R \text{ after conditioning}/M_R \text{ before conditioning} \quad (4)$$

Creep Test

Table 5 gives the creep test results, including the intercept (*I*) and slope (*S*) after regression analysis and creep stiffness at 60 min. The coefficients of determination (R^2) also are given. The regression analysis was performed in the range from 1 to 90 min. Figure 2 shows a typical relationship between creep strain and time.

The intercept and the slope of each sample are obtained by the following equation:

$$\log(\text{strain},\%) = \log(I) + S*\log(\text{time,sec}) \quad (5)$$

Creep strain and creep stiffness can be determined by the following equations:

$$\varepsilon_c = h/H \quad (6)$$

where

ε_c = creep strain,
h = deformation at time t, and
H = thickness of specimen.

and

$$S_{mix}(T,t) = \sigma/\varepsilon(T,t) \quad (7)$$

where

$S_{mix}(T,t)$ = creep stiffness at temperature T and time t,
σ = compressive stress, and
$\varepsilon(T,t)$ = creep strain at temperature T and time t.

The creep stiffness of each sample presented in Table 5 is the predicted value after regression analysis using the measured stiffness. Figure 3 shows mix stiffness (S_{mix}) as a function of bitumen stiffness (S_{bit}). Bitumen stiffness was obtained by using the Van der Poel bitumen stiffness nomograph with the asphalt properties (PI and softening point) and a range of loading time.

Rut Depth

To predict rut depth due to increased tire pressure, the relationships between S_{mix} and S_{bit} resulting from the creep test were used. Physical properties of the asphalt cement and vertical compressive stress (shown in Figures 4 and 5) for a typical asphalt pavement structure in Oregon (SN = 3.0, Figure 6) were used. The Shell method (*6*) was employed to predict the rut depth in the asphalt layer of the given pavement structure. An 18-kip single axle with dual tires and tire pressures of 80 psi (i.e., assumed tire pressure in previous pavement design) and 125 psi (possible tire pressure for future pavement design) were used.

According to Van de Loo (*15*) the permanent deformation in the asphalt layer can be calculated by the following equation:

$$\delta = C_M H_o \sigma_{avg}/S_{mix} \quad (8)$$

where

δ = reduction in layer thickness;
C_M = correction factor for the so-called dynamic effect, which takes account of differences between static (creep) and dynamic (rutting) behavior (this factor depends on the type of mix and must be determined empirically);
H_o = design thickness of the asphalt layer;
σ_{avg} = average stress in the pavement under the moving wheel; and
S_{mix} = value of stiffness of the mix at $S_{bit} = S_{bit.visc}$.

To determine the vertical compressive stress, ELSYM5 (*16*) was used. Values of the input parameters (modulus, thickness, and Poisson's ratio) of each layer were selected to represent Oregon pavements designed for medium traffic levels. Table 6 gives the average vertical compressive stresses calculated from the output of ELSYM5, and Table 7 gives the predicted rut depth for the asphalt surface layer (thickness is 2 in.). The penetration index is −1.4 (for an AR-4000 grade asphalt cement), and the loading time is 0.0125 sec (corresponding to a speed of 50 mph). The number of load repetitions was 1 million and the correction factor (C_M) was 1.2. According to Equation 8, the rut depth for a tire pressure of 80 psi is 0.022 in., and that for 125 psi is 0.034 in. after 1 million load repetitions.

In this paper only one set of calculations for the C gradation mixes of Morse Brothers Pit is presented for the purpose of demonstration. Because the resilient modulus of the asphalt layer is varied with different mixtures, the modulus value for ELSYM5 should correspond to the resilient modulus test results.

More detailed data on the rut depth calculation are presented elsewhere (*4*).

DISCUSSION

Mix Design

Table 4 gives a summary of the mix design results of laboratory-compacted mixes. Their stability is considered to be most significant in this study. ODOT requires a minimum Hveem stability of 30.

TABLE 4 SUMMARY OF MIX DESIGN DATA

Sample ID[a]	Max Sp. Gr.	Bulk Sp. Gr.	Air Voids (%)	AC Content (%)	VMA[b] (%)	Stability[c]	M_R As Comp.[d] (ksi)	M_R Cond.[e] (ksi)	M_R Ratio[f]	Min. AC to 0.7 MRRT[g] (%)	Optimum A/C (%)
Morse Brothers Pit, Gravel, Chevron AR-4000											
A32	2.484	2.26	9.0	5.0		33	258	146	0.56		
A33	2.455	2.30	6.3	6.0		35	227	197	0.87	5.5	6.6
A34	2.408	2.32	3.6	7.0		31	224	189	0.84		
B29	2.463	2.28	7.4	5.0		35	186	102	0.55		
B30	2.446	2.30	6.0	6.0		32	187	139	0.75	5.8	6.6
B31	2.423	2.33	3.8	7.0		33	194	133	0.69		
C26	2.489	2.34	6.0	4.5		36	492	161	0.33		
C27	2.466	2.37	3.9	5.5		37	447	349	0.78	5.3	5.1
C28	2.440	2.40	1.6	6.5		19	303	237	0.78		
Cobb Rock Quarry, 1% Lime, Chevron AR-4000											
A11	2.514	2.25	10.5	4.5	15.1	41	361	172	0.48		
A12	2.476	2.29	7.5	5.5	14.5	37	320	346	1.08	4.9	6.3
A13	2.433	2.33	4.2	6.5	13.9	37	320	312	0.97		
B09	2.506	2.26	9.8	4.5	14.7	33	312	127	0.41		
B10	2.471	2.30	6.9	5.5	14.1	30	240	120	0.50	6.5	6.2
B11	2.433	2.34	4.2	6.5	13.5	37	266	187	0.70		
C09	2.512	2.33	7.2	4.5	12.0	39	465	301	0.65		
C10	2.471	2.37	4.1	5.5	11.5	31	392	501	1.28	4.6	5.3
C11	2.428	2.41	0.1	6.5	10.9	5	282	374	1.33		
D29	2.541	2.31	9.1	4.0	12.3	45	205	76	0.37		
D30	2.497	2.35	5.9	5.0	11.8	38	404	242	0.60	5.2	5.3
D31	2.459	2.39	2.8	6.0	11.2	33	232	302	1.30		
Hilroy Pit, Gravel, Chevron AR-4000											
A30	2.501	2.27	9.2	4.5	15.3	38	362	94	0.26		
A31	2.465	2.31	6.3	5.5	14.7	38	252	115	0.46	6.4	6.4
A32	2.429	2.34	3.7	6.5	14.5	36	239	180	0.75		
B21	2.493	2.27	8.9	4.5	15.3	36	364	93	0.26		
B22	2.459	2.29	6.9	5.5	15.5	35	280	150	0.54		6.2
B23	2.422	2.33	3.8	6.5	14.9	34	265	176	0.66		
C24	2.523	2.33	7.7	4.0	12.6	39	541	66	0.12		
C25	2.477	2.37	4.3	5.0	12.1	44	438	159	0.36	5.8	5.2
C26	2.437	2.41	1.1	6.0	11.5	35	384	302	0.79		
D27	2.474	2.33	5.8	5.0	13.5	40	391	142	0.36		
D28	2.431	2.37	2.5	6.0	13.0	41	403	260	0.65	6.3	5.6
D29	2.414	2.40	0.6	7.0	12.8	18	329	284	0.87		
E29	2.519	2.29	9.1	4.0	14.1	40	752	175	0.23		
E30	2.482	2.34	5.7	5.0	13.2	37	401	199	0.50	7.0	5.9
E31	2.443	2.35	3.8	6.0	13.7	40	396	239	0.60		
F09	2.519	2.30	8.7	4.0	13.8	37	420	89	0.21		
F10	2.482	2.38	4.1	5.0	11.7	39	429	293	0.68	5.3	5.3
F11	2.452	2.40	2.1	6.0	11.9	36	374	272	0.74		
Blue Mountain Asphalt Pit, Gravel, 1% Lime, Chevron AC-20											
A38	2.583	2.33	9.8	4.5	17.9	29	437	214	0.49		
A39	2.545	2.37	6.9	5.5	17.4	30	404	291	0.72	5.4	5.6
A40	2.504	2.41	3.8	6.5	16.9	30	371	289	0.78		
B32	2.590	2.36	8.9	4.5	16.8	37	465	294	0.63		
B33	2.548	2.40	5.8	5.5	16.3	37	425	346	0.81	4.9	5.9
B34	2.510	2.44	2.8	6.5	15.8	38	374	346	0.92		
C29	2.607	2.37	9.1	4.0	16.0	39	679	339	0.5		
C30	2.565	2.41	6.0	5.0	15.5	38	630	353	0.56	5.4	5.3
C31	2.517	2.45	2.7	6.0	15.0	27	601	536	0.89		
D35	2.617	2.36	9.8	4.0	16.4	40	650	317	0.49		
D36	2.568	2.40	6.5	5.0	15.9	38	592	292	0.49	6.0	5.5
D37	2.530	2.44	3.6	6.0	15.4	33	523	372	0.71		
E37	2.607	2.32	11.0	4.0	17.8	37	836	496	0.59		
E36	2.574	2.39	7.1	5.0	16.2	35	728	737	1.01	4.3	5.7
E35	2.528	2.44	3.5	6.0	15.4	33	753	499	0.66		

[a] A–F = aggregate gradation type.
[b] VMA = voids in mineral aggregate.
[c] Stability = stability at first compaction.
[d] M_R As Comp. = resilient modulus at 25°C, as compacted.
[e] M_R Cond. = resilient modulus at 25°C, after conditioning.
[f] M_R Ratio = Resilient modulus after conditioning/Resilient modulus before conditioning.
[g] Min A/C to 0.7 MRRT = minimum asphalt content for the retained modulus ratio (M_R ratio) of 0.7.

TABLE 5 CREEP TEST RESULTS

Sample ID[a]	S_{mix}[b] (ksi)	I[c]	S[d]	R^2[e]
Morse Brothers Pit, Gravel, Chevron AR-4000				
A32	3.47	0.098	0.177	0.961
A33	3.93	0.132	0.126	0.957
A34	3.14	0.116	0.169	0.996
B29	4.14	0.126	0.124	0.929
B30	6.37	0.084	0.122	0.930
B31	2.83	0.146	0.153	0.983
C26	3.57	0.142	0.129	0.951
C27	4.85	0.117	0.114	0.977
C28	5.24	0.069	0.170	0.973
Cobb Rock Quarry, 1% Lime, Chevron AR-4000				
A11	4.76	0.135	0.099	0.940
A12	3.68	0.171	0.102	0.929
A13	5.40	0.105	0.115	0.997
B09	5.15	0.096	0.134	0.940
B10	3.33	0.206	0.091	0.931
B11	7.33	0.069	0.128	0.948
C09	3.95	0.075	0.194	0.998
C10	2.80	0.114	0.185	0.985
C11	1.47	0.307	0.143	0.962
D29	5.03	0.107	0.121	0.942
D30	3.81	0.093	0.172	0.964
D31	3.73	0.113	0.151	0.985
Hilroy Pit, Gravel, Chevron AR-4000				
A30	5.06	0.127	0.099	0.898
A31	3.50	0.128	0.143	0.929
A32	2.05	0.073	0.227	0.983
B21	6.07	0.058	0.173	0.889
B22	4.85	0.064	0.188	0.944
B23	3.75	0.051	0.247	0.938
C24	4.05	0.101	0.155	0.960
C25	4.62	0.056	0.210	0.979
C26	3.59	0.091	0.182	0.984
D27	5.72	0.058	0.180	0.990
D28	8.06	0.046	0.167	0.945
D29	2.70	0.135	0.169	0.973
E29	5.90	0.027	0.271	0.977
E30	7.56	0.018	0.292	0.964
E31	7.77	0.018	0.283	0.976
F09	4.87	0.025	0.303	0.971
F10	4.70	0.020	0.336	0.980
F11	4.58	0.130	0.109	0.803
Blue Mountain Asphalt Pit, Gravel, 1% Lime, Chevron AC-20				
A38	5.34	0.137	0.084	0.939
A39	4.91	0.182	0.059	0.922
A40	2.31	0.148	0.176	0.991
B32	2.24	0.270	0.107	0.942
B33	2.99	0.188	0.116	0.945
B34	2.57	0.175	0.143	0.984
C29	2.61	0.182	0.137	0.965
C30	2.42	0.243	0.110	0.984
C31	1.48	0.358	0.123	0.970
D35	3.90	0.094	0.169	0.956
D36	2.17	0.206	0.143	0.968
D37	2.88	0.190	0.119	0.967
E38	5.01	0.031	0.273	0.943
E39	5.86	0.027	0.269	0.941
E40	4.25	0.012	0.409	0.952

[a]A–F = aggregate gradation type.
[b]S_{mix} = predicted creep stiffness at 60 min after regression.
[c]I = intercept; strain, percentage at 1 sec.
[d]S = slope; strain, percentage = I*(time,sec)**S.
[e]R^2 = coefficient of determination.

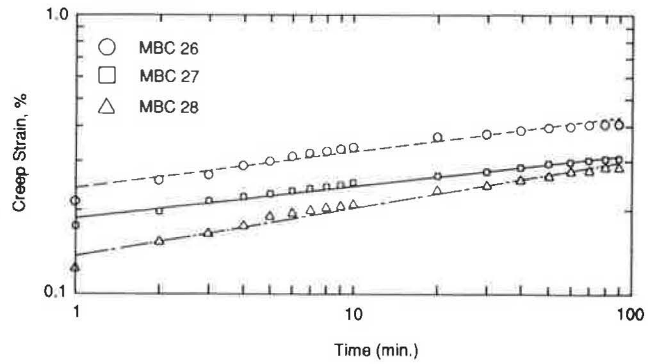

FIGURE 2 Creep strain versus time.

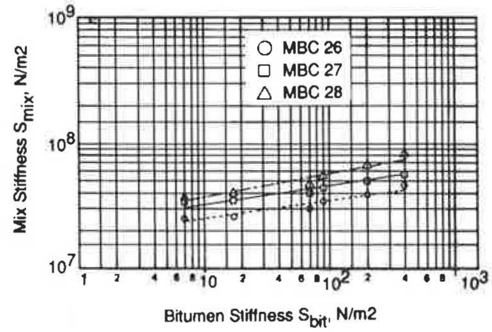

FIGURE 3 S_{mix} versus S_{bit}.

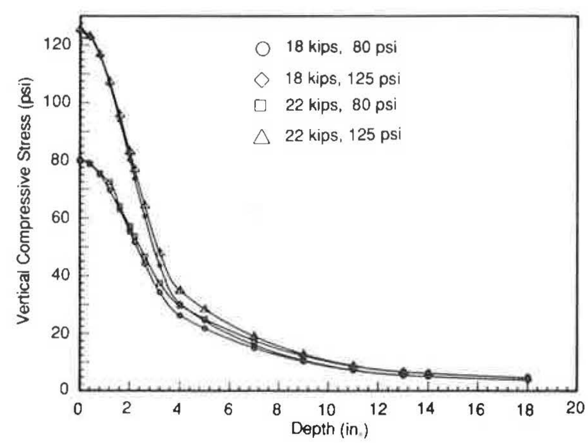

FIGURE 4 Vertical compressive stress: single axle dual tires.

As indicated by the data in Table 8, the correlation between log (Hveem stability) and log (creep stiffness) is not strong except for the Cobb Rock mixes. According to the results of creep tests, it is not always true that a mix with a high stability value resists deformation better than one with low stability. This indicates that the current mix design criteria are probably inadequate for producing mixtures capable of withstanding high tire pressures and for identifying potentially highly deformable mixtures.

It is noted that Gradation C mix (the Fuller maximum density gradation) requires the smallest optimum asphalt content for aggregate from each source according to the existing mix design method. Also, Gradation C has the smallest VMA.

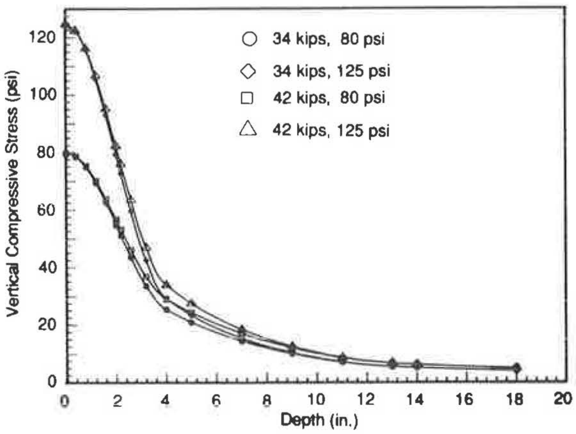

FIGURE 5 Vertical compressive stress: tandem axle dual tires.

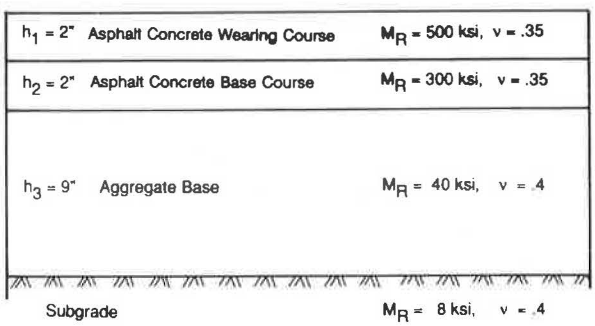

FIGURE 6 Typical asphalt pavement in Oregon (SN = 3.0).

TABLE 6 AVERAGE VERTICAL COMPRESSIVE STRESS

Axle Configuration	Tire Pressure (psi) 80	125
Single axle, dual tires		
18 kips	70.7	108.2
22 kips	71.8	109.4
Tandem axle, dual tires		
34 kips	70.4	107.6
42 kips	71.1	108.8

NOTE: Values are psi.

TABLE 7 PREDICTED RUT DEPTH UNDER GIVEN CONDITIONS

Tire Pressure (psi)	Rut Depth (in.)
80	0.022
125	0.034

NOTE: Conditions are as follows: AR 4000 (PI = −1.4), asphalt pavement (SN = 3.0) shown in Figure 6, H_o = 2.0 in., number of repetitions = 10^6, and MAAT = 20°C.

TABLE 8 CORRELATION ANALYSIS

Variables	Morse Brothers Pit	Cobb Rock Quarry	Hilroy Pit	Blue Mountain Asphalt Pit
Correlations with log (Creep Stiffness, ksi)				
log (stability)	−0.3141	0.8176	0.4878	−0.0482
log (M_R; as comp., ksi)	0.0636	−0.0859	0.5004	−0.2771
log (M_R; cond., ksi)	0.2664	−0.4886	−0.1981	−0.7012
log (M_R ratio)	0.2428	−0.5353	−0.3665	−0.3592
log (AC, %)	−0.0906	−0.2440	−0.4839	−0.3310
log (max sp. gr.)	0.1638	0.2542	0.4825	0.3015
log (air voids, %)	−0.1736	0.7529	0.3890	0.5625
log (VMA)	N/A	0.5805	0.0615	0.7465
log (pass 1/4 in., %)	−0.4197	0.2196	−0.4026	0.5609
log (pass No. 10, %)	0.1970	−0.5034	0.0897	−0.1731
log (pass No. 200, %)	−0.3141	−0.6766	−0.1416	−0.3799
log (intercept)	−0.6955	−0.7780	−0.3532	−0.7038
log (slope)	−0.4395	−0.2410	−0.3908	−0.5329
Correlations with log (Slope)				
log (stability)	−0.5737	−0.1073	−0.0163	0.4056
log (creep stiff., ksi)	−0.4395	−0.2410	−0.3908	−0.5329
log (M_R; as comp., ksi)	−0.1814	0.5060	−0.3602	0.2600
log (M_R; cond., ksi)	−0.0878	0.4838	0.3963	0.2604
log (M_R ratio)	0.0993	0.3077	0.4671	−0.0078
log (AC, %)	0.3817	−0.0476	0.5256	0.0436
log (max sp. gr.)	−0.3476	0.0459	−0.4687	0.0079
log (air voids, %)	−0.3252	−0.2107	−0.2363	−0.2589
log (VMA)	N/A	−0.7506	−0.0819	−0.4468
log (pass 1/4 in., %)	0.4420	−0.5647	−0.2625	−0.4437
log (pass No. 10, %)	−0.0317	0.6751	−0.0743	0.2183
log (pass No. 200, %)	−0.5737	0.6777	−0.0215	0.0439
log (intercept)	−0.3388	−0.4176	−0.5332	−0.2156
Correlations with log (Intercept)				
log (stability)	0.7761	−0.7241	−0.3974	−0.2351
log (creep stiff., ksi)	−0.6955	−0.7780	−0.3532	−0.7038
log (M_R; as comp., ksi)	0.0714	−0.2807	−0.1320	0.1409
log (M_R; cond., ksi)	−0.2211	0.1370	−0.4243	0.5916
log (M_R ratio)	−0.3393	0.3109	−0.2871	0.3855
log (AC, %)	−0.2007	0.2766	−0.1660	0.2983
log (max sp. gr.)	0.0961	−0.2882	0.0705	−0.2930
log (air voids, %)	0.4335	−0.6008	0.1323	−0.4049
log (VMA)	N/A	−0.0696	0.2844	−0.5163
log (pass 1/4 in., %)	0.0795	0.1491	0.5427	−0.3455
log (pass No. 10, %)	−0.1846	0.0341	−0.1375	0.0135
log (pass No. 200, %)	0.7761	0.1812	−0.1531	0.4049
log (slope)	−0.3388	−0.4176	−0.5332	−0.2156
Correlations with log (Stability)				
log (creep stiff., ksi)	−0.3141	0.8176	0.4878	−0.0482
log (M_R; as comp., ksi)	0.0471	0.1153	0.3026	0.0361
log (M_R; cond., ksi)	−0.2101	−0.3435	−0.2735	0.0017
log (M_R ratio)	−0.2987	−0.4685	−0.3810	−0.3332
log (AC, %)	−0.4433	−0.4636	−0.4805	−0.4824
log (max sp. gr.)	0.3579	0.5197	0.4139	0.5657
log (air voids, %)	0.7820	0.9546	0.6501	0.3984
log (VMA)	N/A	0.4529	0.0179	−0.0909
log (pass 1/4 in., %)	0.1302	0.2283	0.0664	−0.6330
log (pass No. 10, %)	−0.2928	−0.2220	0.0104	−0.0017
log (pass No. 200, %)	1.0000	−0.4696	0.2500	−0.1198
log (slope)	−0.5737	−0.1073	−0.0163	0.4056
log (intercept)	0.7761	−0.7241	−0.3974	−0.2351

NOTE: N/A = not available.

In general, the optimum asphalt content from the existing mix design method is higher than that required to achieve the retained modulus ratio (MMRT) of 0.7 except for the mixes with Hilroy Pit aggregate.

It appears to be necessary to study further which mix design criteria, including creep stiffness, should be considered and how to determine the optimum asphalt content of a mix for resistance to rutting and good durability.

Creep Behavior of Mixes

The creep behavior of an asphalt mixture can be determined from the slope obtained after regression analysis and creep strain or creep stiffness. To analyze the effect of some mix variables, including aggregate gradation, on creep behavior, a correlation analysis among the variables (Table 8) was made. In general, creep stiffness decreases with increasing percentage of aggregate passing the No. 200 sieve, as indicated in Table 8.

Because of the limited data, the effect of the percentage of aggregate passing the 1/4-in. or No. 10 sieve on creep stiffness is not clear. With regard to the percentage passing the 1/4-in. or No. 10 sieve, however, the results concerning the creep stiffness of the aggregates from the Morse Brothers Pit and the Hilroy Pit show a similar trend (i.e., negative correlation with the percentage passing the 1/4-in. sieve and positive correlation with the percentage passing the No. 10 sieve). The results on the Cobb Rock Quarry and the Blue Mountain Asphalt Pit aggregates, which were mixed with 1 percent slurry lime, indicate another similar trend (i.e., positive correlation with the percentage passing the 1/4-in. sieve and negative correlation with the percentage passing the No. 10 sieve).

For aggregates from four sources, the creep stiffness has negative correlation with the intercept (which shows the deformation characteristics at the initial stage) or slope (which shows resistance to deformation).

The slope decreases with an increase in the percentage of aggregate passing the 1/4-in. sieve, except for aggregate from the Morse Brothers Pit.

Mixes made with the Morse Brothers Pit aggregate show a trend similar to that of those made with the Hilroy Pit aggregate (i.e., the slope has negative correlations with percentages passing both the No. 10 and the No. 200 sieves), and mixes with the Cobb Rock aggregate have a trend similar to that of the Blue Mountain Asphalt Pit aggregate (i.e., the slope has positive correlations with percentages passing both the No. 10 and the No. 200 sieves).

For the Cobb Rock aggregate and the Blue Mountain Asphalt Pit aggregate mixed with 1 percent lime slurry, the slope increases with increases in the percentage of aggregate passing the No. 10 and the No. 200 sieves.

From the results of the mix design, it can be noted that adding 1 percent lime slurry improves not only the durability of the asphalt mix, as seen by the retained modulus ratio in Table 4, but also its resistance to deformation. This may be due in part to the increased strength imparted to the mix by the addition of the lime. However, the effect of lime slurry on the permanent deformation of asphalt mixes still needs to be investigated.

It should be noted that the creep stiffness of Gradation C mix (Fuller maximum density gradation) is not the highest in the range of asphalt content tested in this study as shown in Figure 7, even though the mix with Gradation C has the smallest VMA (Table 4).

For Hveem stability, the mix with the Cobb Rock aggregate has high correlation between log (stability) and log (creep stiffness).

As can be seen in Figure 7, the relationship between asphalt content and creep stiffness (at 60 min) is not clear. The stiffness of a mix made with aggregate from different sources or of different gradations, or both, is unique.

Rut Depth

The Shell method was employed to predict rut depth in an asphalt surface layer. For the rut depth calculation, the creep test results of C gradation mixes of Morse Brother Pit were used.

The average vertical compressive stress in an asphalt surface layer shown in Figure 6 is about 90 percent of the inflation tire pressure given in Table 6. As the data in Table 7 indicate, the rut depth in the asphalt surface layer increases by 52 percent as the tire inflation pressure increases by 56 percent. Therefore it can be said that the increase in rut depth of an asphalt layer is approximately proportional to the increase in tire inflation pressure.

As indicated by Van de Loo (*17*), it is essential that the creep curve that is used as input in the calculation procedure be representative of the mix that will be present in the pavement. Because the creep behavior (i.e., slope of the curve) of laboratory-prepared specimens may be quite different from that obtained on cores from pavements, because of differences in compaction effort and heating process, core samples should be obtained shortly after construction and used for the creep test. Because of this, the prediction of rut depth with laboratory specimens is meaningless. However, laboratory-prepared specimens can be used to determine the ranking of different mixes.

In this paper emphasis has been mainly on the stability of mixes. For the overall performance of asphalt pavement, however, durability and fatigue characteristics of asphalt mixes as well as their stability should be considered in the mix design process.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The mix design process used by Oregon State Highway Division was investigated to evaluate its ability to minimize damage from higher tire pressure. For this study aggregate from four different sources was used. Six different aggregate gradations, including the Fuller maximum density gradation, were tested.

A simple method of creep testing to predict deformation of an asphalt mixture, which used a loading device for soil consolidation and a data acquisition system with a microcomputer, was used. The major findings and conclusions of this study follow:

1. Gradation C (the Fuller maximum density gradation) requires the least amount of optimum asphalt for aggregate from each source.

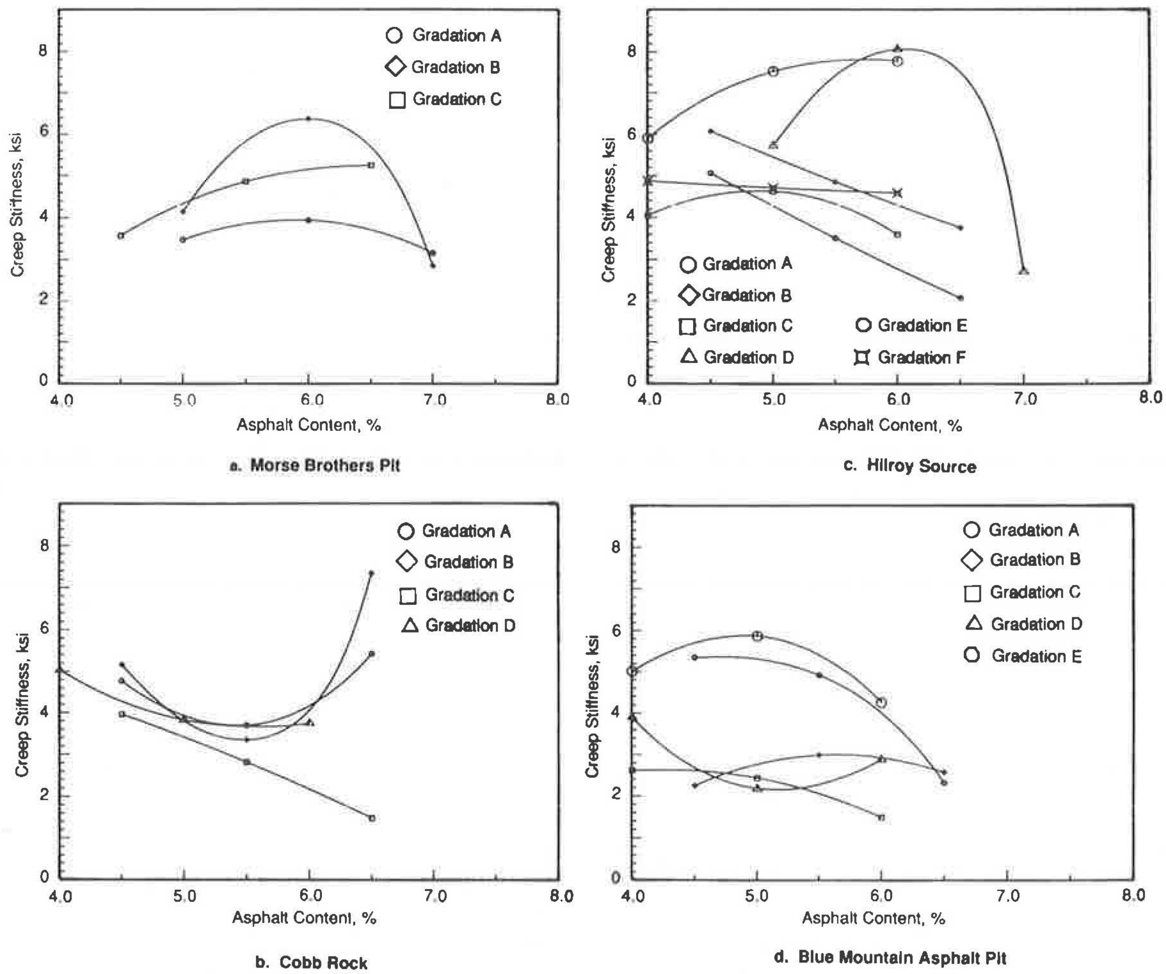

FIGURE 7 Effect of asphalt contents on creep stiffness.

2. Hveem stability has little relationship with creep stiffness. The results of creep tests show that it is not always true that a mix with a high Hveem stability value resists creep deformation better than one with low stability. Therefore, for projects on which deformation is a major concern, the use of creep tests in the mix design process should be of benefit.

3. Creep stiffness decreases with an increasing percentage of aggregate passing the No. 200 sieve. However, the effect of the percentage passing the 1/4-in. or the No. 10 sieve on creep stiffness is not clear. Control of the passing No. 200 material clearly contributes to deformation resistance and should be given more emphasis in mix design and construction.

4. Using 1 percent lime slurry results in some improvement in creep stiffness.

Recommendations

The following recommendations are made for controlling the effect of increased tire pressure on asphalt concrete pavement:

1. Include the creep behavior of a mix in mix design criteria, such as creep stiffness, to predict the rut depth due to increased tire pressure or to rank candidate mixes, or both. As the Shell manual indicates, it is essential that the creep curve that is used as an input in the calculation procedure be representative of the behavior of the mix in the pavement. A study to correlate laboratory mixture stability (i.e., Hveem stability, Marshall stability, and creep stiffness) with field deformation is recommended.

2. More investigation is needed into the effect of lime slurry on the permanent deformation of asphalt mix. The results of this study indicated that there was some improvement in creep stiffness in mixtures that contained lime slurry.

3. The use of other additives to increase creep stiffness of mixtures should be considered.

4. Further study of the process of designing mixes to withstand higher tire pressure is necessary in laboratory and field.

ACKNOWLEDGMENTS

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Asphalt Mix Design: An Innovative Approach

GILBERT Y. BALADI, RICHARD W. LYLES, AND RONALD S. HARICHANDRAN

Using the results of Marshall and constant and cyclic load indirect tensile tests, it is shown that the indirect tensile tests can be used to infer the structural properties of compacted asphalt mixes without the need to use other expensive test apparatus. It is also shown that if the Marshall test is modified, the asphalt mix procedure can be tailored to optimize the structural properties of the mix. Correlations between the asphalt mix design parameters and the structural properties and the respective statistical matrix are presented and discussed. All tests were conducted on Marshall-sized specimens made from various asphalt concrete mixes.

Structural properties of asphalt mixes have a direct bearing on pavement performance under traffic loading and environmental conditions (*1–7*). Determining the relevant structural properties can be tedious and involved because these properties change with environmental conditions. Unlike the properties of mineral aggregate in the mix or in the base and subbase layers, which are relatively constant, physical and chemical properties of asphalt binder are dynamic in nature and are influenced by temperature, moisture, and time. In addition, the response of asphalt mixes to load is the result of three different mechanisms: elastic, viscoelastic, and plastic (*8–13*). Thus some of the relevant structural properties of asphalt mixes needed for the design of asphalt pavement include resilient characteristics, plastic (permanent) deformations, creep, and fatigue behavior.

Asphalt mixes are largely composed of coarse and fine aggregates, mineral filler, asphalt binder, and air voids. The proportions of these components in any given mix (the asphalt mix design) affect its structural properties and dictate its behavior under traffic loading. Existing practices divorce asphalt mix design procedures from those used to obtain structural properties. Thus a major problem facing the pavement engineer today is tailoring the asphalt mix design to optimize its structural properties to result in the best pavement performance under the anticipated traffic loading and environmental conditions.

In recognition of this need, researchers have developed several equations correlating the structural properties of the compacted mix and the mix design parameters. Some of these correlations were based only on the stiffness of the binder. Others were based on proportioning of the different materials in the mix. Still others were based on the Marshall stability and flow values (*14–23*). Each of these correlations was found to be limited to certain mixes or binder stiffness. In addition, fatigue properties of compacted asphalt mixes still need to be evaluated or estimated, or both.

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In recognition of these shortcomings, a research project sponsored by the FHWA was undertaken to identify, evaluate, and document a laboratory test procedure or procedures whereby asphalt mix design can be examined from the structural viewpoint. The results of the study should help the highway engineer to determine the structural properties of asphalt concrete mix that are needed in the design of flexible pavements.

EXPERIMENTAL PROGRAM

To accomplish the objectives of the study, an experimental program was undertaken to evaluate the structural and other parameters of compacted asphalt mixes. The test results of this study are tabulated elsewhere (*24*). This paper addresses the problems associated with Marshall and indirect tensile tests. Also, a new approach to obtaining the structural parameters of asphalt mixes needed in the pavement design methods is presented. A companion paper by Baladi et al. in this Record addresses variations of the structural properties with respect to variations in the specimen and test variables. To avoid unnecessary duplication, the reader is referred to the work of Baladi (*24*), Baladi et al. (*25*), and the other paper in this Record by Baladi et al. for sample preparation and test procedures and for a description of a new indirect tensile test apparatus that was developed during the course of the investigation. For completeness and convenience however, the test types and test materials are briefly described here.

Laboratory Tests

The following tests were conducted using the new indirect tensile test apparatus (*24*):

1. Indirect tensile tests (INTT) using a standard Marshall loading frame and deformation rate. Some of the test specimens were conditioned as standard Marshall specimens. Others were tested dry at 60°C, 25°C, and 5°C (140°F, 77°F, and 40°F).
2. Indirect constant peak cyclic load (INCCL) tests using an MTS hydraulic system. The specimens were subjected to a constant sustained load followed by a constant peak cyclic load of 500 lb. Some of the test specimens were subjected to a maximum of 500,000 cycles at a frequency of two cycles per second with a loading time of 0.1 sec and a relaxation period of 0.4 sec. Measurements of elastic, total, and plastic (permanent) deformations were collected along the vertical and horizontal diameters and the thickness of the specimen. The data were then analyzed to obtain the resilient and total characteristics of the specimens and their fatigue lives.

3. Indirect variable peak cyclic load (INVCL) tests using an MTS hydraulic system. Basically, the test procedure is the same as that of the INCCL test except that after the application of the sustained load, the specimen was subjected to 100-, 200-, and 500-lb peak cyclic loads; each load was applied for only 1,000 cycles.

4. Marshall tests at the design asphalt content. Some of the test specimens were conditioned as specified by the standard Marshall test procedures. Others were conditioned dry at 40° F and then tested at the same temperature. For all of the Marshall tests, an equivalent Marshall stiffness (ES) was defined using the load-deformation record and Equation 1.

$$ES = S/2\ [F_{0.5(s)}] \tag{1}$$

where

ES = equivalent stiffness (lb/in.),
S = Marshall stability (lb), and
$F_{0.5(s)}$ = flow at half the value of Marshall stability (in.).

A total of 125 samples (375 specimens) were made and tested using the new indirect test apparatus (75 for each of the INCCL, INVCL, and Marshall tests, and 150 for the INTT). In the remaining parts of this paper, the term "sample" is used to describe one test sample that was later cut to three (triplicate) test specimens. It should be noted that

1. All samples were made using several materials as described in the next section.
2. All indirect tests were conducted using a new indirect tensile test apparatus.
3. For all of the tests and for each combination of the test materials, a constant (design) asphalt content was used. This design asphalt content corresponds to that at 3 percent air voids as determined by using separate standard Marshall mix design procedures.

Test Materials

The test materials used in this study were

1. Three different types of aggregate were used: crushed (angular) limestone, rounded natural (river deposit) aggregate, and a mix of 50 percent by weight per sieve of crushed limestone and natural aggregate;
2. Fly ash mineral filler;
3. Two aggregate gradations (*24* and the other paper in this Record by Baladi et al.); and
4. Three viscosity-graded asphalt cements (AC-10, AC-5, and AC-2.5).

For each material combination, a constant percentage of asphalt content was used (the percentage of asphalt content at 3 percent air voids as determined from the standard Marshall mix design procedures). The samples were compacted near three values of percentage of air voids (3, 5, and 7 percent) by varying the foot pressure and number of tampings of a kneading compactor. For each material combination and percentage of air voids, a cylindrical sample 10.16 cm in diameter and 22 cm high (4 in. by 8.5 in.) was made. Later, the sample was cut into three 6.3-cm-high (2.5-in.) specimens. The three specimens (a triplicate) were then tested under the same conditions (test temperature and test type) using the new indirect tensile test apparatus for the INTT, INCCL, and INVCL tests and a standard Marshall apparatus for the Marshall tests.

ANALYTICAL MODELS

The analytical models used to calculate the resilient and total moduli and Poisson's ratios and the tensile and compressive strengths were developed on the basis of linear, homogeneous, and isotropic elastic models. Details of these models may be found elsewhere (*24*). Equations 2 and 3, to calculate the compressive and tensile strengths of compacted asphalt mixes, are relevant to the following discussion.

$$INCS = 0.475386\ (P/L) \tag{2}$$

$$INTS = 0.156241\ (P/L) \tag{3}$$

where

$INCS$ = indirect compressive strength (psi),
$INTS$ = indirect tensile strength (psi),
P = maximum load (lb), and
L = specimen thickness (in.).

STATISTICAL MATRICES

The main objective of the statistical analysis was to determine if results from one type of test (e.g., the Marshall test) can or cannot be used to infer the results from other tests (e.g., cyclic load indirect tensile tests). Traditionally, this was accomplished by a simple statistical correlation between one set of results and another. Such correlations ignore the physical interpretation of the data and simply relate one set of numbers to another. Consequently, the resulting correlation equations are naturally limited to a specific set of data. A better method is to independently examine each set of data and analyze its variations with the given set of variables. For example, if the Marshall results show that stability and flow are related to one variable (e.g., the percentage of air voids) and the INCCL results show dependency on another variable or other variables, then perhaps the results from the latter tests cannot be related to stability and flow. If the results from both tests are not satisfactory, any correlation between them is problematic.

To this end, results from each test were examined for specific patterns with regard to the independent variables. The factors that influence the test results are summarized in Table 1. Numbers under the variables indicate the order of significance of that variable for the test results. For example, the percentage of air voids is the second most significant factor affecting Marshall stability for all tests that were conducted at the design asphalt content but different percentages of air voids. Flow value, on the other hand, is influenced by (in order of decreasing significance) load (stability), gradation, percentage of air voids, and aggregate angularity. Equations 4–10 relate Marshall stability, flow, equivalent stiffness, indirect compressive strength, indirect tensile strength, resilient modulus, and total modulus to the specimen and test variables, respectively. It resulting equations. Nevertheless, results from INTT (Equations 7 and 8) were statistically related to those from INCCL tests (Equation 9), which resulted in the following equations:

$$\ln(MR) = 7.1949 + 1.01341 \times \ln(INCS) - 0.0003409 \times CL \quad (12)$$

$R^2 = 0.974$; $SE = 0.220$

$$\ln(MR) = 8.3145 + 1.01511 \times \ln(INTS) - 0.0003409 \times CL \quad (13)$$

$R^2 = 0.974$; $SE = 0.220$

where all variables are as before.

The estimated values of the resilient modulus, using Equations 12 and 13, were found to vary by 13 percent from the measured ones. The equation overestimated the resilient modulus for the 3 percent air voids specimens and underestimated it for the 7 percent air voids specimens. These observations suggested that a correction to Equation 12 should be derived to account for the percentage of air voids. Examination of the regression coefficients of Equations 7–9 indicated that *AV* affects the values of ln(*INCS*) and ln(*INTS*) by a factor of about –0.26 and the values of ln(*MR*) by a factor of –0.14. This implies that the effect of *AV* is much greater on *INCS* than on *MR*. This was expected because, as noted previously, *INCS*-values are based on the load at failure (Equation 2); the values of *MR* are based on smaller loads and the elastic component of the deformation. Thus the relationship between *MR* and *INCS* should also account for the effects of *AV*. A second regression was done in which the percentage of air voids was included as one of the independent variables. This yielded the following equations:

$$\ln(MR) = 6.1776 + 1.08108 \times \ln(INCS) + 0.14145 \times AV - 0.0003409 \times CL \quad (14)$$

$R^2 = 0.996$; $SE = 0.085$

$$\ln(MR) = 7.3667 + 1.08335 \times \ln(INTS) + 0.14218 \times AV - 0.0003409 \times CL \quad (15)$$

$R^2 = 0.996$; $SE = 0.083$

where all variables are as before.

It should be noted that *INCS* in Equations 14 and 15 is also dependent, in part, on the percentage of air voids. That is, a collinearity is introduced into the equation. Also, the positive values of the regression coefficient of the percentage of air voids (0.14145 and 0.14218) does not mean that increasing *AV* increases *MR*. On the contrary, increasing *AV* yields a decrease in *MR*. This is mainly related, as stated earlier, to the interpretation of the equation. The *AV* term in Equations 14 and 15 accounts for the difference between the effects of the air voids on *INCS* and the effects on *MR*. Stated differently, the percentage decrease in the value of *INCS* due to an increase in *AV* from 3 to 7 percent is greater than the percentage decrease of *MR* for the same range of *AV*. To relate the *INCS*- and *MR*-values, this difference has to be accounted for; the *AV* term in the equations accounts for this difference. From an engineering viewpoint, Equations 14 and 15 should not be used because interpretation of the equation may mislead the user (higher values of the percentage of air voids cause higher resilient modulus). The equations are introduced here for one reason, to be able to estimate the resilient modulus for the same specimen subjected to the indirect tensile test. Hence laboratory costs are minimized. Again, the equations should not be used for physical or mathematical interpretations of the sensitivity of the test results to the independent variables or other interpretations. This is simply a tool for estimating the resilient modulus of specimens subjected to INTT.

Similarly, Equations 16 and 17 express the total modulus (Equation 10) in terms of *INCS* and *INTS*, respectively.

$$\ln(E) = 7.0327 + 1.0205 \times \ln(INCS) - 0.1108 \times AV - 0.0003339 \times CL \quad (16)$$

$R^2 = 0.996$; $SE= 0.080$

$$\ln(E) = 8.1552 + 1.0227 \times \ln(INTS) - 0.1153 \times AV - 0.0003339 \times CL \quad (17)$$

$R^2 = 0.996$; $SE= 0.078$

where all variables are as before.

As was the case for Equations 14 and 15, introducing the *AV* term in Equations 16 and 17 introduces collinearity because *INCS* is also a function of *AV*. Again, the *AV* term in the equation simply accounts for the difference in its effects on *INCS*, *INTS*, and *E*. It is strongly recommended that Equations 14–17 be used only for estimation of the values of *MR* and *E* and not for mathematical or physical interpretations and manipulation.

Figure 1 shows the measured and calculated values (using Equation 14) of the resilient modulus. The figure is divided into two quarters for results at 77°F and 40°F. The straight line in the figure represents equality between the calculated and measured values. It was found that the maximum percentage difference between the measure and the calculated values (of Equation 14) is 3.2 percent for the 77°F tests and 9.2 percent for those at 40°F. These percentage differences are, respectively, 6.0 and 4.1 percent for Equation 16. It should be noted that the measured values of the resilient and total moduli were found to be dependent on the number of load applications (a higher number of load applications yields lower values of the moduli). The values of *MR* and *E* at cycle 500 were used to derive Equations 14–17. The reason is that, in practice, the resilient modulus tests are conducted for only 500 cycles at which the value of the modulus is calculated and the test is terminated. Nevertheless, the effects of the number of load repetitions can be incorporated into the equations without any complications. The calculated and measured values of the total modulus showed a trend similar to that of Figure 1.

Analysis of the deformations along the vertical (*DV*) and horizontal (*DH*) diameters (Table 1) indicated that

1. The *DH* measured from the indirect tensile tests can be used to estimate the magnitude of the permissible maximum

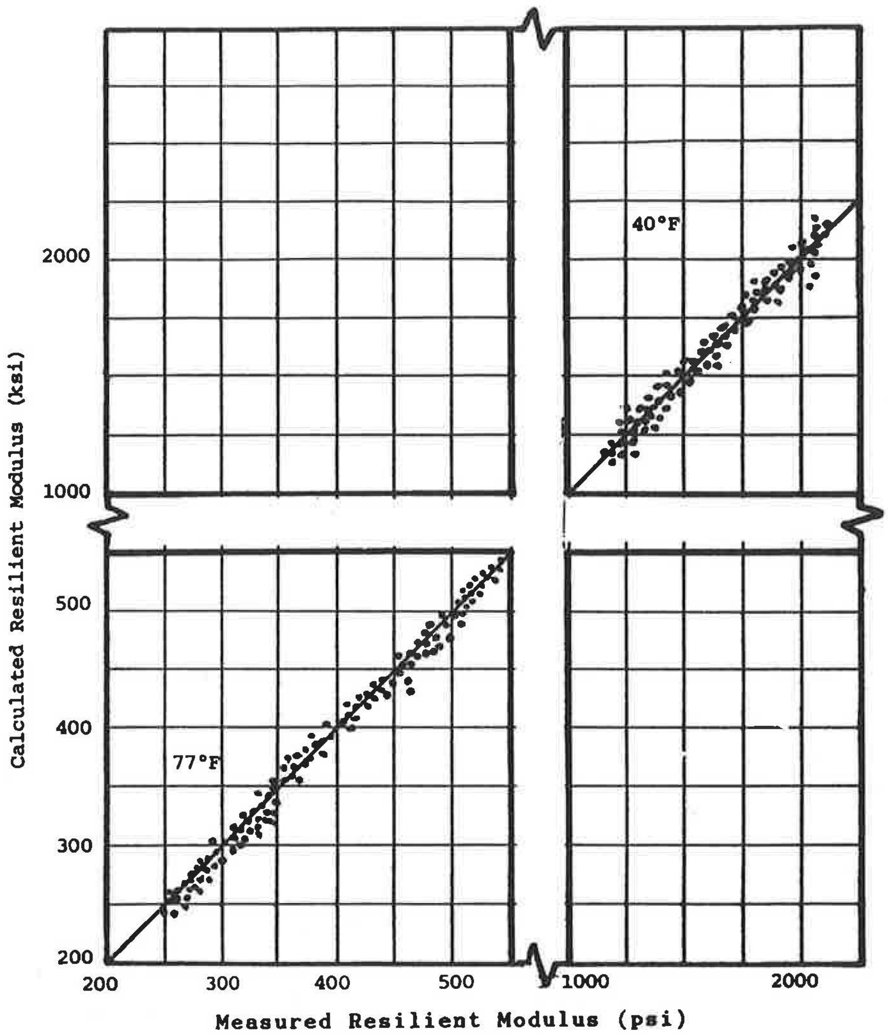

FIGURE 1 Calculated (using Equation 14) versus measured resilient modulus for three magnitudes of cyclic loads and two test temperatures.

cumulative tensile plastic strain to control fatigue cracking. In most specimens subjected to the INCCL test, a hair-sized tensile crack was initiated when the value of the measured cumulative plastic tensile *DH* was about 95 percent of the value of *DH* of a compatible specimen subjected to INTT. This observation implies that the fatigue life of the mix can be defined by the number of load applications at which the cumulative plastic tensile strain reaches a value of 95 percent of the *DH*.

2. The vertical deformation (*DV*) measured from the indirect tensile test is a measure of the compressibility of the asphalt mix. Again, the completed analysis has indicated that the *DV* can be related to the permanent deformation measured in the INCCL tests.

Because the final analysis was completed after this paper was submitted, the findings and equations (fatigue life and permanent deformation) will be published elsewhere and may be found in Baladi (*24*).

ASPHALT MIX DESIGN

As noted previously, statistical analyses indicated a poor correlation between Marshall stability and *MR*. A better and more accurate correlation was obtained between *MR* and *ES*, and *MR* and *INCS* or *INTS*. This implies that Marshall stability cannot be used to accurately estimate the structural properties of a mix. Because the objective of the study is to tailor the asphalt mix design procedure to optimize the structural properties of the mix, the rejection or acceptance of an asphalt mix design should not be partly based on Marshall stability. Given this scenario and these findings, what criteria should be used to accept or reject a design of an asphalt mix? Two alternatives are offered here. The first is based on a slightly modified version of Marshall mix design. The second is based on the indirect tensile test.

In the first alternative, two modifications to the standard Marshall mix design procedures are suggested:

1. Replace Marshall stability by the equivalent Marshall stiffness (*ES*) to select the optimum asphalt content. That is, replace the plot of Marshall stability versus asphalt content by equivalent stiffness versus asphalt content. The design asphalt content can then be determined using, for example, the Asphalt Institute criteria except that the asphalt content corresponding to the optimum value of *ES* should be used rather than that at optimum stability.

2. Marshall tests can be conducted at room temperature thus eliminating the need for a water bath.

After the design asphalt content has been determined, the values of *ES* and *AV* that correspond to design asphalt content should be used in Equation 11 to estimate the resilient modulus of the mix.

In the second alternative, the INTT is highly recommended. In these tests, the specimen deformations in two directions should be measured. The design asphalt content should then be selected on the basis of the values of *INCS*, *INTS*, *DV*, *DH*, and the median limits given in Article 3.14 of the Asphalt Institute Manual Series 2 for the percentage of air voids. A detailed sample preparation and test procedure may be found in Baladi (*24*).

The INTT requires neither expensive and complex equipment nor a new training or personnel. The INTT can be conducted at room or any other temperatures. Indeed, the test and test procedures are similar to the Marshall test. The AASHTO T 245-82 procedure can be followed step by step except as noted in the following list.

1. After the test specimens (triplicate for each combination of material and asphalt content) have been prepared, bring their temperature to the test temperature (room temperature is recommended). If other temperatures are desired, a temperature-controlled chamber is required.
2. Place the indirect tensile test apparatus under the loading head of a standard Marshall loading frame.
3. Place the test specimen on the lower curved platen of the apparatus and lower the loading strip to make contact with the specimen (5 lb of load will ensure a good contact).
4. Adjust and balance the electronic measuring system as necessary. This includes the load cell and the vertical and horizontal linear variable differential transformer or transformers.
5. Apply the load to the specimen by means of the constant rate of movement of the Marshall machine head (2 in./min) until the maximum load is reached and the load decreases as indicated by the measuring system.
6. Calculate the indirect compressive and tensile strengths (*INCS* and *INTS*) using Equations 2 and 3, respectively.
7. Calculate the resilient and total moduli using Equations 14 and 16 or 15 and 17, respectively.
8. The tests can be conducted at different temperatures to infer the effects of test temperature on test results.
9. As in the standard Marshall mix design, analyze the *INCS*, *INTS* (or *MR* and *E*), *DV*, *DH*, the percentage of air voids, the percentage of voids in mineral aggregate, and the density of the specimens as a function of the percentage of asphalt content.
10. The design asphalt content should be selected on the basis of the optimum structural properties and the specified percentage of air voids.

The values of the optimum structural properties vary and depend on the particular project under consideration. Nevertheless, the results of these tests can be directly used to establish the mix design and to obtain the structural properties for the pavement design. Further, the procedure eliminates the need for a water bath. Tests at 140°F or any other temperature can be conducted on specimens conditioned dry in a temperature-controlled chamber.

It should be noted that because the recommended equations are based on a limited data base, verification of the estimated values of the resilient and total moduli is strongly recommended. Such verification should also help the engineer to calibrate the equations.

CONCLUSIONS

The following conclusions, based on laboratory test results and analytical and statistical analyses, can be drawn:

1. The resilient and total moduli of asphalt mixes can be expressed in terms of the indirect compressive strength of the mix, the test temperature, and the magnitude of the applied cyclic load.
2. The need for complex tests and testing equipment to estimate the resilient modulus of asphalt mixes can be eliminated.
3. Marshall stability cannot be used to properly characterize the structural parameters required in the mechanistic pavement design models. The equivalent Marshall stiffness is a better descriptor of these parameters.
4. The asphalt mix design procedure can be tailored to optimize the structural properties of asphalt mixes.
5. Indirect tensile tests can be used to obtain an asphalt mix design based on the structural properties of the mix.

SUMMARY

Structural properties of asphalt mixes have direct bearing on pavement performance. Knowledge of these properties is essential for the structural design of pavements. Existing asphalt mix design procedures are divorced from those used to obtain structural properties. It was shown that the Marshall mix design method can be tailored to optimize the structural properties of a mix. Modifications of the method are suggested. Further, relationships to estimate the total and resilient moduli from the indirect tensile test results are presented. Hence, the need for complex and expensive tests can be eliminated.

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New Relationships Between Structural Properties and Asphalt Mix Parameters

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Improved relationships between some of the fundamental mechanical properties and the mix variables of asphalt are presented and discussed. It is shown that the properties of asphalt mixes can be predicted from a knowledge of several parameters of compacted asphalt mix, magnitude of applied load, and test temperature. During the course of the investigation, a new indirect tensile test apparatus was designed, fabricated, and tested. The apparatus was then used to conduct cyclic load indirect tensile tests using Marshall-type specimens and various asphalt concrete mixes. The indirect tensile test can be used to characterize the elastic, total, creep, permanent deformation, and fatigue behavior of asphalt concrete mixes.

The design of flexible pavement has rapidly evolved from empirical and semiempirical procedures to design methods based on elastic or viscoelastic theories, or both (*1–7*). Today, many highway agencies use such methods in one form or another for the design of new pavements and overlays. This use requires a thorough knowledge of the basic mechanical properties of asphalt paving materials, which are functions of the asphalt mix variables (*8–13*). A variety of tests and test equipment has been developed and employed in laboratories to evaluate these properties (*14–23*). Regardless of the complexity of the tests, test procedures, and test equipment, it was found that different tests yield different results and that test results are difficult to reproduce (*12*). Further, existing asphalt concrete mix design procedures are based on parameters that do not necessarily have any relationship to the structural design of asphalt pavements (*12, 15*).

EXPERIMENTAL PROGRAM

In recognition of the need to tailor asphalt mix design procedures to optimum structural properties and to be able to obtain these parameters from simple tests, a research project sponsored by the FHWA was undertaken at the Department of Civil and Environmental Engineering at Michigan State University (MSU). The objectives of the program included

1. The selection of a simple test and test procedure that will allow the highway engineer to determine the fundamental engineering properties required for the structural design of asphalt pavements.
2. A study of the repeatability of the test results and the number of tests required to reliably obtain the mechanical properties (resilient modulus and Poisson's ratio, fatigue life and permanent deformation characteristics, creep, and viscoelastic properties) of asphalt materials.

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Several tests and test procedures were employed. These included triaxial tests (constant and repeated cyclic loads), cyclic flexural tests, Marshall tests, indirect tensile tests (constant and variable cyclic loads), and creep tests. The test data indicated that

1. Repeatability of test results is poor,
2. Material properties obtained from the different tests are substantially different, and
3. Results from the indirect tensile test were the most promising although they were not consistent.

The last observation was made after examining the results of 24 tests (12 tests at a cyclic load of 500 lb and 12 at 100 lb) that were conducted using existing (Schmidt) apparatus. The tests were conducted in triplicate using 100- and 500-lb cyclic loads. The test results are shown in Figure 1. It can be noted that the values obtained for the resilient modulus from a triplicate vary by a factor of 1.9, and those obtained from the 100-lb cyclic load tests are higher than the values from the 500-lb cyclic load tests. Further, during the tests, it was noticed that the measured horizontal deformations were dependent on the placement of the specimen on the lower curved platen (two sequential placements yielded different measurements) and that the upper head of the instrument experienced a slight rocking motion. Hence the inconsistency in the indirect tensile test results appeared to be related to the equipment rather than the test mode. Existing indirect test apparatus have one or more of the following problems:

1. A rocking motion of the loading head, which distorts the accuracy of the vertical deformation of the test specimen.
2. Because of equipment configuration, the test specimen may roll over the lower curved platen resulting in erroneous horizontal deformations.
3. The position of the test specimen on the lower curved platen is arbitrary and differs from one specimen to another.
4. During a repeated load test, the horizontal axis of the test specimen may rotate relative to the vertical axis of the loading head resulting in a smaller measurement of the radial deformation on one side of the diameter than of the opposite side.
5. Lack of capability to measure specimen deformations in three directions indicates that some information available from the test cannot be recorded.

In recognition of these problems, a new indirect tensile test apparatus was designed at MSU and fabricated by the

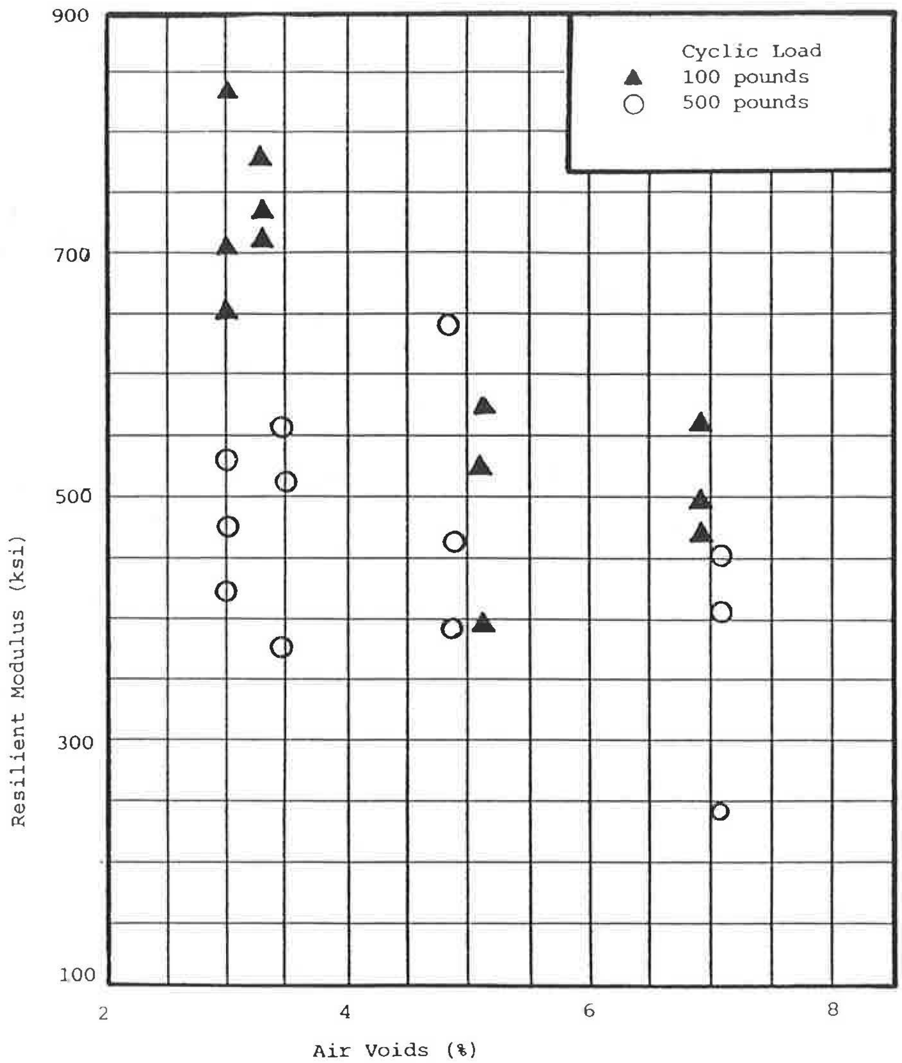

FIGURE 1 Resilient modulus versus percentage of air voids for specimens tested using existing indirect tensile test apparatus.

Michigan Department of Transportation (MDOT). A brief summary of some of the features of that apparatus is presented next.

NEW INDIRECT TENSILE TEST APPARATUS

To overcome the problems associated with existing apparatus, a new simple and inexpensive indirect tensile test apparatus was designed at MSU and later modified and fabricated by personnel of the Division of Testing and Technology at MDOT. The new apparatus possesses the following features:

1. Deformation of the indirect test specimen can be measured in one, two, or three directions using either one or two linear variable differential transducers (LVDTs) in each direction.
2. The apparatus can be used under any existing loading frame (e.g., Marshall, hydraulic system, unconfined, triaxial), and it has a guiding system that consists of four frictionless pistons.
3. The function of the frictionless guiding system is to prevent rotation or rocking, or both, of the upper curved platen of the loading head.
4. The apparatus has four reference positions for easy placement of the apparatus under the center of the loading mechanism of a standard loading frame.
5. The apparatus has a sample stopper for easy positioning of the test specimen on the curved lower platen and for perfect alignment of the horizontal diameter (axis) of the specimen with the axis of the horizontal LVDT or LVDTs.

Complete details of the apparatus along with engineering drawings may be found elsewhere (*24*).

LABORATORY TESTS

The following tests were conducted using the new indirect tensile test apparatus:

1. Indirect tensile tests (INTT) using a standard Marshall loading frame and deformation rate. Some of the test specimens were conditioned as standard Marshall specimens. Others were tested dry at 60°C, 25°C, and 5°C (140°F, 77°F, and 40°F).
2. Indirect constant peak cyclic load (INCCL) tests using an MTS hydraulic system. The specimens were subjected to a

constant sustained load followed by a constant peak cyclic load of 500 lb. Some of the test specimens were subjected to a maximum of 500,000 cycles at a frequency of two cycles per second with a loading time of 0.1 sec and a relaxation period of 0.4 sec. Measurements of the elastic, total, and plastic (permanent) deformations were collected along the vertical and horizontal diameters and the thickness of the specimen. The data were then analyzed to obtain the resilient and total characteristics of the specimens and their fatigue lives.

3. Indirect variable peak cyclic load (INVCL) tests using an MTS hydraulic system. Basically, the test procedure is the same as that of the INCCL test except that, after the application of the sustained load, the specimen was subjected to 100-, 200-, and 500-lb peak cyclic loads each of which was applied for only 1,000 cycles.

A total of 150 specimens were tested in the INTT, 75 in the INCCL test, and 75 in the INVCL test. The results from the last two tests are discussed in this paper.

TEST SPECIMENS

A total of 125 samples (375 specimens) were made and tested using the new indirect test apparatus (75 for each of the INCCL, INVCL, and Marshall tests, and 150 for the INTT). In the remaining parts of this paper, the term "sample" is used to describe one test sample that was later cut to three (triplicate) test specimens. The samples were made using the following materials:

1. Three different types of aggregate (crushed and angular limestone, relatively rounded natural aggregate, and a mix of 50 percent by weight per sieve of the crushed limestone and natural aggregates);
2. Fly ash mineral filler;
3. Two aggregate gradations (Figure 2); and
4. Three viscosity-graded asphalt cements (AC-10, AC-5, and AC-2.5).

For each material combination, a constant percentage of asphalt content was used (the percentage of asphalt content at 3 percent air voids as determined from the standard Marshall mix design). The samples were compacted near three values of percentage air voids (3, 5, and 7 percent) by varying the foot pressure and number of tampings of a kneading compactor. For each material combination and percentage of air void, a cylindrical sample 10.16 cm in diameter and 22 cm high (4 in. by 8.5 in.) was made. Later, the sample was cut into three 6.3-cm-high (2.5-in.) specimens. The three specimens (a triplicate) were then tested under the same conditions (test temperature and test type) using the new indirect tensile test apparatus.

ANALYTICAL MODELS

Several analytical models were developed to calculate the resilient (*25, 26*), total (*27, 28*), plastic (*27, 28*), and fatigue characteristics of the compacted mixes. The resilient and total models are presented in this paper. Other models and analyses are presented elsewhere (*24*). It should be noted that the term "resilient modulus" relates the applied cyclic load to the resilient deformation (immediately recoverable after removing the load). Total modulus, on the other hand, relates applied cyclic load to total deformation.

In the development of the analytical models, it was assumed that the load is applied normal to the contact area between the specimen and the loading strip. This implies that there is no friction between the loading strip and the test specimen (*26*). Assuming plane-stress conditions for homogeneous, isotropic, and linear elastic materials, it can be shown that the resilient

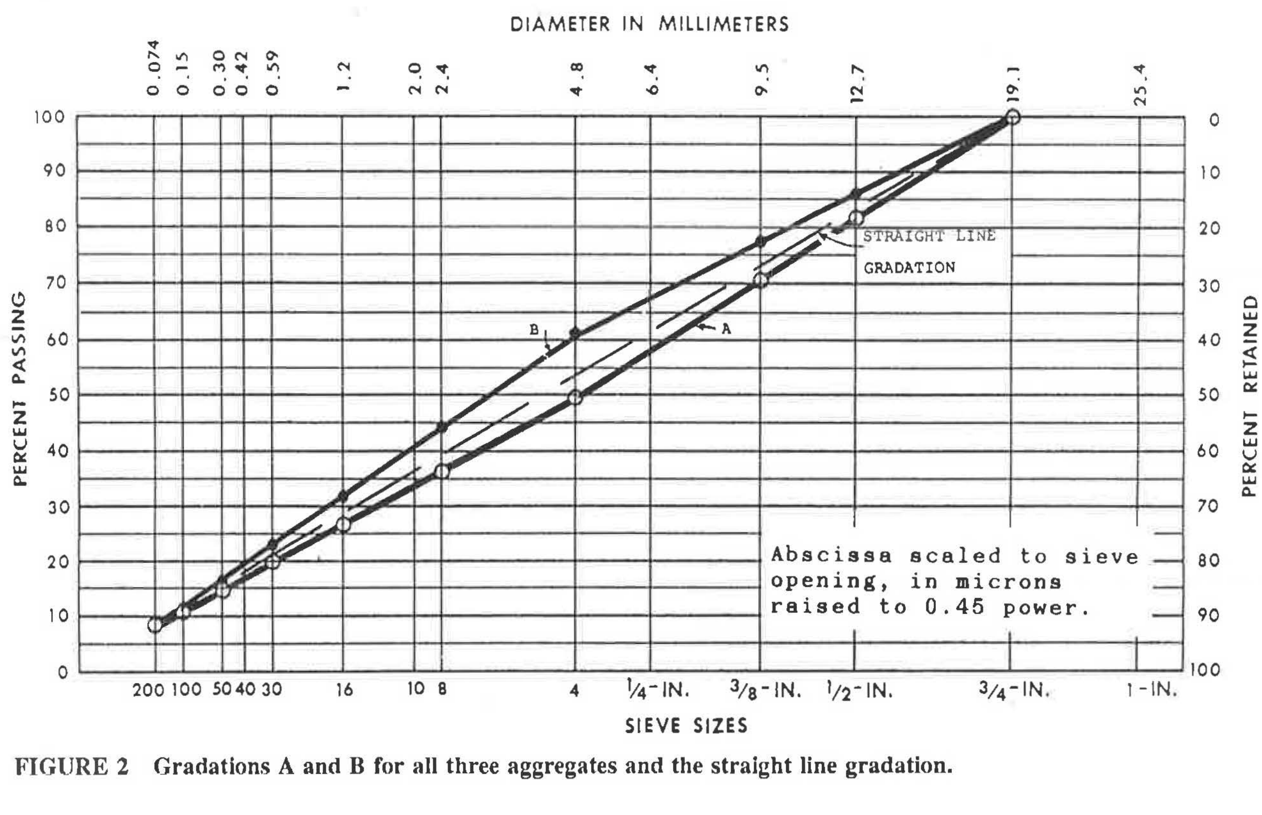

FIGURE 2 Gradations A and B for all three aggregates and the straight line gradation.

and total moduli and Poisson's ratios are given by the following equations:

$$U = \frac{3.58791 - 0.269895\ (DR)}{0.062745 + DR} \tag{1}$$

$$M_R \text{ or } E = \frac{P\ [3.58791 - 0.062745\ (U)]}{L(DV)} \tag{2}$$

$$M_R \text{ or } E = \frac{0.319145\ (P)\ (U)}{DL} \tag{3}$$

$$INCS = \frac{0.475386\ (P)}{L} \tag{4}$$

$$INTS = \frac{0.156241\ (P)}{L} \tag{5}$$

where the constants in the equations result from integration, and

U = Poisson's ratio,
DR = deformation ratio = DV/DH,
DV = vertical resilient or total deformation of the specimen along the vertical diameter (in.),
DH = horizontal resilient or total deformation of the specimen along the horizontal diameter (in.),
DL = radial deformation along the longitudinal axis (thickness) of the specimen (in.),
M_R = resilient modulus (psi),
E = total modulus (psi),
L = sample thickness (in.),
P = magnitude of the applied load (lb),
$INCS$ = indirect compressive strength at the center of the test specimen (psi), and
$INTS$ = indirect tensile strength at the center of the test specimen (psi).

Theoretically, the values of the resilient modulus from Equations 2 and 3 should be exactly the same for homogeneous, isotropic, and linear elastic material. Asphalt mixes are heterogeneous and anisotropic. As a result of this and measurement errors, differences between the two calculated values should be expected. Equation 2 can be used if the radial deformation (DL) is not measured. If the sample deformations in all three directions are measured, the two values of the calculated resilient modulus from Equations 2 and 3 should be compatible (a maximum difference of 5 percent was observed in this study). Substantial difference between these two values may mean that the test results are not accurate. A better procedure is to estimate the modulus and Poisson's ratio using all measurements. The following equations were developed using least square techniques and Equations 1–3:

$$\begin{aligned} U = {} & [0.225127(H^2) - 0.269895(V^2) - 0.0447676(A^2) \\ & + 3.570975(H)(V) + 0.086136(A)(H) \\ & + 1.145064(A)(V)]/\mathrm{D} \end{aligned} \tag{6}$$

$$\begin{aligned} M_R \text{ or } E = {} & [0.253680(H) + 3.9702876(V) \\ & - 0.0142874(A)]/D \end{aligned} \tag{7}$$

where

$$\begin{aligned} D &= 1.105791\ (H^2 + V^2 + A^2) \\ &\quad - [H - 0.0627461(V) + 0.319145(A)]^2; \\ H &= DH(L/P); \\ V &= DV(L/P); \\ A &= DL/P; \text{ and} \end{aligned}$$

M_R, E, DH, DV, DL, L, and P are as before.

If the sample deformations are measured in all three directions, Equations 6 and 7 yield the best estimates of the resilient modulus and Poisson's ratio. However, if one of these three deformations is not available, Equations 1 and 2 or 3 should be used.

STATISTICAL MODEL

Recall that 125 samples were compacted and 375 specimens were made. Seventy-five specimens were tested using the INCCL test and another 75 using the INVCL test. For each test specimen, the resilient and total moduli and Poisson's ratios were then calculated by using Equations 1–7. It was found that

1. The maximum difference between values of the moduli obtained from Equations 2 and 3 was less than 5 percent;
2. Equations 6 and 7 yielded more consistent results than did Equations 1–3; and
3. The maximum difference in the values of the moduli (elastic and total) obtained from a triplicate was less than 7 percent.

Nevertheless, the values calculated from Equations 6 and 7 were statistically correlated with the test and specimen variables using the stepwise procedure of a multivariate linear regression program SPSS/PC+ (*29*). In this procedure, the most highly correlated variable (to the test results) was analyzed first (all other variables were not included); the second most significant was then added, and so forth; the least significant was the last variable included in the analysis. At each step, an equation relating the test results and the variable or variables was produced along with the coefficients of correlation and standard error and a partial correlation matrix (PCM). Variables that did not have a significance level higher than 0.05 percent relative to the previous variable were automatically excluded from the final equation. The advantages of this method follow:

1. At each step, the variables in the equation are listed in order of significance;
2. The interaction between variables can be qualitatively assessed by comparing the regression coefficients from two consecutive steps and by examining the values of the partial correlation coefficient listed in the PCM; and
3. The method produces the simplest and most efficient possible equation.

The specimen and test variables for all tests are

1. Air voids (AV) of the test specimens,
2. Kinematic viscosity (KV) of the asphalt binder,
3. Gradation ($GRAD$) of the aggregates,
4. Aggregate angularity (ANG),

5. Magnitude of cyclic load (*CL*),
6. Number of load repetitions (*N*), and
7. Test temperature (*TT*).

During the analysis, for each independent variable, several transformations (arithmetic, exponential, and logarithmic) relating the dependent and independent variable were also explored. The final form was selected on the basis of its simplicity, physical interpretation, and the values of the coefficients of correlation and standard error.

Table 1 gives a summary of the regression matrix (regression coefficients, coefficients of correlation, and the standard error of the resulting equation from each step of the analysis) of the resilient modulus. It can be noted that

1. The most significant variable affecting the resilient modulus is *TT*.
2. The value of the regression coefficient of *TT* is only slightly affected as more variables are included in the analysis. This observation implies that *TT* is independent and there is little or no interaction between this variable and the others.
3. Apparently there is some interaction between plastic deformation (*CD*1) and percentage of air voids (*AV*), and between *CD*1 and the magnitude of the cyclic load (*ACL*). This was expected because as plastic deformation increases, the air voids either decrease (densification) or increase (dilatation). Similarly, as plastic deformation increases, response of the specimen to the cyclic load will change.
4. The values of the coefficient of correlation and standard error have indicated consistency in the test results. This could be related to the extra care exercised during the tests and to the features of the new apparatus.

Equation 8 is the final regression equation relating the resilient modulus to the specimen and mix variables.

$$\begin{aligned}\ln(M_R) = {} & 16.097 - 0.03634(TT) - 0.1349(AV) \\ & - 0.0002653(CL) + 0.04586(ANG) \\ & + 0.0009142(KV) - 0.00004688(CD1) \\ & - 0.02456(GRAD) - 0.005924[\ln(N)] \\ R^2 = {} & 0.999;\ SE = 0.017 \end{aligned} \tag{8}$$

TABLE 1 REGRESSION MATRIX FOR RESILIENT MODULUS OF COMPACTED ASPHALT MIXES USING CONSTANT AND VARIABLE PEAK CYCLIC LOADS

DEPENDENT VARIABLE RESILIENT MODULUS (M_R)	INTERCEPT	REGRESSION COEFFICIENT OF THE INDEPENDENT VARIABLE								R^2	S.E.
		TT (10^{-2})	AV (10^{-1})	ACL (10^{-4})	AG (10^{-2})	KV (10^{-4})	CD1 (10^{-5})	GR (10^{-2})	lnN (10^{-3})		
$\ln(M_R)$	15.724	-3.778	-	-	-	-	-	-	-	0.885	0.2190
	16.266	-3.578	-1.403	-	-	-	-	-	-	0.983	0.085
	16.401	-3.582	-1.428	-3.398	-	-	-	-	-	0.991	0.061
	16.291	-3.628	-1.393	-3.397	3.969	-	-	-	-	0.994	0.049
	16.092	-3.658	-1.401	-3.409	4.353	8.793	-	-	-	0.997	0.033
	16.029	-3.601	-1.363	-2.684	4.441	8.625	-5.829	-	-	0.999	0.022
	16.043	-3.617	-1.334	-2.663	4.644	9.125	-5.909	-2.547	-	0.999	0.019
	16.097	-3.634	-1.349	-2.653	4.586	9.142	-4.688	-2.456	-5.924	0.999	0.017

ln = natural log;
TT = test temperature;
AV = air voids;
ACL = actual cyclic load;
AG = angularity;
KV = kinematic viscosity;
CD1 = plastic deformation along the vertical diameter (inches);
GR = gradation;
N = number of applications;
R^2 = coefficient of correlation;
S.E. = standard error; and
\- = not applicable.

where

$\ln$ = natural logarithm;
M_R = resilient modulus (psi);
TT = test temperature (°F);
AV = percentage of air voids (AV = 1, 2, 3, etc.);
CL = applied cyclic load (lb);
L = actual sample thickness (in.);
ANG = aggregate angularity (angularity was assigned a scale of from 1 to 4; a value of 1 represents perfectly spherical and smooth particles, and 4 represents highly angular particles); for this study the angularity of the crushed limestone is 4, that of the rounded natural aggregate is 2, and the angularity of the 50 percent by weight mix was given a value of 3;
KV = kinematic viscosity of the asphalt at 135°C (275°F) (cSt) (AASHTO T201);
$CD1$ = cumulative vertical plastic deformation (in.);
$GRAD$ = gradation factor ($GRAD$ = 1 for Gradation A and 0.98 for Gradation B); because only two gradations were used in this project, a meaningful relationship between $GRAD$ and the resilient modulus cannot be obtained;
N = the number of load repetitions;
R^2 = coefficient of correlation; and
SE = standard error.

From a practical viewpoint, Equation 8 can be simplified by considering only five variables: TT, AV, CL, ANG, and KV. This is because the additional variables add little to the explanatory power, in a statistical sense, and their net impact on the actual predicted values of $\ln(M_R)$ is minimal. Therefore the equation reduces to (the fifth step of the stepwise procedure)

$$\begin{aligned}\ln(M_R) = {} & 16.092 - 0.03658(TT) - 0.1401(AV) \\ & - 0.0003409(CL) + 0.04353(ANG) \\ & + 0.0008793(KV) \\ R^2 = {} & 0.997;\ SE = 0.033 \end{aligned} \tag{9}$$

where all variables are as before.

The sensitivity of the values of the resilient modulus predicted by Equation 9 to variations in the values of the independent variables (TT from 40°F to 77°F; AV from 3 to 7 percent; CL from 100 to 500 lb, ANG from 2 to 4, and KV from 159 to 270 cSt) was studied. It was noted that

1. The values of the resilient modulus increase by a factor of 3.76 as the temperature decreases from 77°F to 40°F;
2. A decrease in the percentage of air voids from 7 to 3 percent results in an increase in the resilient modulus by a factor of 1.76;
3. An increase in the applied load from 100 to 500 lb results in a reduction in the modulus by a factor of 1.15;
4. The values of the resilient modulus increase by a factor of 1.09 as ANG increases from 2 (rounded aggregate) to 4 (crushed aggregate); and
5. Increasing KV from 159 to 270 cSt leads to increasing M_R by a factor of 1.1.

These observations imply that the asphalt mix has a nonlinear elastic behavior (greater load yields lower modulus values). The significance of this is that existing standard test procedures (e.g., ASTM D4123) specify that the resilient modulus tests be conducted using low magnitudes of the cyclic load. This will result in higher estimates of the resilient modulus than would be the case at greater loads. A correct evaluation of the values of the total or resilient modulus should include the sensitivity of these values to the range of load anticipated in the field (i.e., obtain the relationship between M_R or E and CL). Further, it is stated in ASTM D4123 that "if Poisson's ratio is assumed, the vertical deformations are not required. A value of 0.35 for Poisson's ratio has been found to be reasonable for asphalt mixtures at 77°F." The values of the resilient modulus in this study were also calculated (as specified by ASTM D4123) using the measured horizontal deformation and an assumed value of Poisson's ratio of 0.35. It can be seen that the assumption of Poisson's ratio of 0.35 consistently yields higher modulus values. An assumed Poisson's ratio of 0.27 yields better estimates of the values of M_R for most data points. It should be noted that a change in the assumed value of Poisson's ratio of 0.01 results in a change of approximately 2 percent in the value of M_R. Because Poisson's ratio of asphalt mixes is a function of the mix and test variables (see Equation 12), it is strongly recommended that Poisson's ratio be calculated using measured vertical and horizontal deformations.

Nevertheless, Figure 4 shows the measured and calculated resilient modulus using Equation 9. The straight line in the figure represents equality between measured and calculated values. It should be noted that the maximum difference between the arithmetic (not logarithmic) values of the measured and calculated data (using Equation 9) is 8 percent. This difference is only 3 percent for Equation 8.

A regression matrix for the total modulus (E) was obtained. It was noted that the order of the variables in this matrix was the same as that for the resilient modulus (Table 1). Comparison of the values of the regression coefficients from the two matrices indicates that the test temperature and the percentage of air voids have slightly greater effects on the total modulus than on the resilient modulus. This was expected because the total modulus reflects elastic and viscoelastic behavior whereas the resilient modulus reflects only elastic behavior. This implies that higher temperatures increase the viscoelastic component of the deflection (strain) more than they increase the resilient component. Nevertheless, Equation 10 expresses E in terms of the sample and test variables.

$$\begin{aligned}\ln(E) = {} & 16.385 - 0.04529(TT) - 0.1549(AV) \\ & - 0.0003339(CL) + 0.04258(ANG) \\ & + 0.0008364(KV) \\ R^2 = {} & 0.998;\ SE = 0.034 \end{aligned} \tag{10}$$

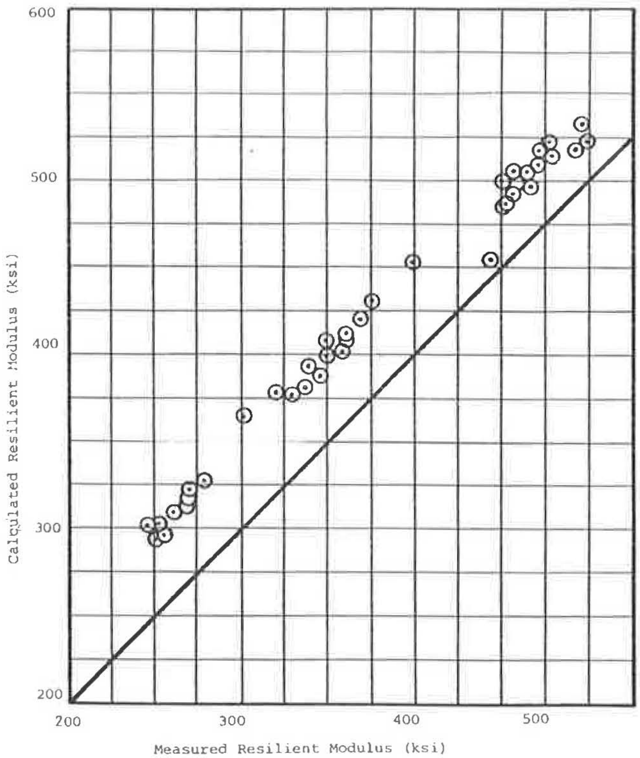

FIGURE 3 Calculated versus measured resilient modulus using the ASTM D4123 standard test procedure.

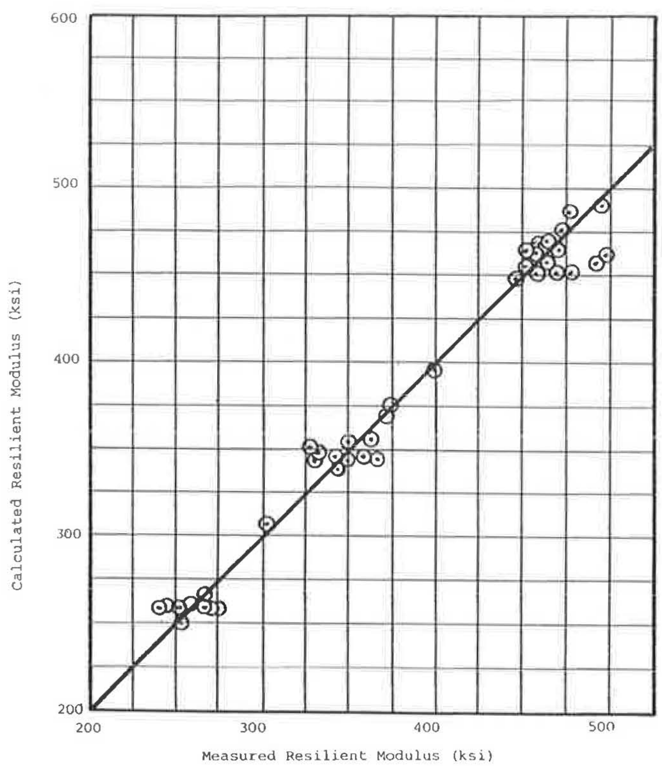

FIGURE 4 Calculated versus measured resilient modulus.

A plot of the measured versus calculated values of E (using Equation 10) showed a trend similar to that of the resilient one.

The regression matrices for the resilient and total Poisson's ratio are given in Tables 2 and 3, respectively. It should be noted that

1. The order of significance of the variables in the matrices is different; TT is the most significant factor affecting the total Poisson's ratio, but it is fourth for the resilient Poisson's ratio, and
2. The value of the coefficient of correlation for the resilient Poisson's ratio is quite low (0.275).

The first observation was expected because the values of the total Poisson's ratio reflect both elastic and viscoelastic behavior; viscoelastic behavior is more temperature sensitive than is elastic. The second observation does not mean that the values of the resilient Poisson's ratio are inconsistent. It should be recalled that the variable peak cyclic load tests were conducted using 100-, 200-, and 500-lb cyclic loads. The horizontal resilient deformations for the 100-lb cyclic load (at 400°F) were on the same order of magnitude as the accuracy of the measurement system (0.00001 in.). Hence these measurements are not reliable for analytical purposes. Total deformations, on the other hand, were higher and consequently the values of the total Poisson's ratio are more consistent. When the data for the 100-lb cyclic load were excluded, the regression matrix of Table 4 was obtained. Note that the order of the variables in this matrix is different from that in Table 2. Also, values of the regression coefficients and the coefficients of correlation and standard error are considerably different.

These comments support the statement made concerning the ASTM D4123 standard: conducting the INCCL tests using a low value of cyclic load will yield inconsistent and misleading results unless the accuracy of measurement is improved.

Using the results given in Tables 3 and 4, and neglecting the last two variables in both tables because they have negligible impact on the values of $\ln(PT)$ and $\ln(PR)$, Equations 11 and 12 can be obtained for the total and resilient Poisson's ratios, respectively.

$$\begin{aligned}\ln(PT) &= -0.43228 - 0.01940(TT) - 0.06329(AV) \\ &\quad + 0.001332(KV) + 0.0001236(CL) \\ \mathrm{R}^2 &= 0.913;\ SE = 0.108\end{aligned} \tag{11}$$

$$\begin{aligned}\ln(PR) &= -1.370 - 0.04243(AV) + 0.000885(KV) \\ &\quad + 0.004662[\ln(N)] + 0.0004489(TT) \\ R^2 &= 0.720;\ SE = 0.045\end{aligned} \tag{12}$$

It can be noted that although the resilient Poisson's ratio is dependent on the number of load repetitions (N), the total Poisson's ratio is independent. The reason for this is that increasing N yields higher resilient deformation and lower viscoelastic deformation. Hence total deformation is more or less independent of N. Figure 5 shows the measured and calculated resilient Poisson's ratio using Equation 12. The line representing the value of Poisson's ratio of 0.35 is also shown. It is clear that assuming a Poisson's ratio value of 0.35 for all mixes is not appropriate. Therefore it is strongly recommended that values of Poisson's ratio be calculated using measured horizontal and vertical deformations. A similar plot can be obtained for total Poisson's ratio.

TABLE 2 REGRESSION MATRIX FOR RESILIENT POISSON'S RATIO OF COMPACTED ASPHALT MIXES USING CONSTANT AND VARIABLE PEAK CYCLIC LOADS

DEPENDENT VARIABLE RESILIENT POISSON'S RATIO (PR)	INTERCEPT	REGRESSION COEFFICIENT OF THE INDEPENDENT VARIABLE						R^2	S.E.
		AV (10^{-2})	KV (10^{-3})	ACL (10^{-4})	TT (10^{-3})	CD1 (10^{-5})	lnN (10^{-2})		
ln(PR)	-1.179	-3.088	-	-	-	-	-	0.076	0.156
	-1.428	-3.280	1.075	-	-	-	-	0.153	0.150
	-1.511	-3.117	1.071	2.150	-	-	-	0.208	0.145
	-1.623	-3.485	0.979	2.170	2.231	-	-	0.254	0.141
	-1.654	-3.297	0.969	2.539	2.529	-2.972	-	0.260	0.140
	-1.793	-2.929	0.957	2.510	3.028	-6.182	1.566	0.275	0.139

ln = natural log;
TT = test temperature;
AV = air voids;
ACL = actual cyclic load;
KV = kinematic viscosity;
CDI = plastic deformation along the vertical diameter (inches);
N = number of applications;
R^2 = coefficient of correlation;
S.E. = standard error; and
\- = not applicable.

TABLE 3 REGRESSION MATRIX FOR TOTAL POISSON'S RATIO OF COMPACTED ASPHALT MIXES USING CONSTANT AND VARIABLE PEAK CYCLIC LOADS

DEPENDENT VARIABLE TOTAL POISSON'S RATIO (PT)	INTERCEPT (10^{-2})	REGRESSION COEFFICIENT OF THE INDEPENDENT VARIABLE						R^2	S.E.
		TT (10^{-2})	AV (10^{-2})	KV (10^{-3})	ACL (10^{-4})	ANG (10^{-2})	GRAD (10^{-2})		
ln(PT)	-34.333	-1.986	-	-	-	-	-	0.829	0.151
	-38.948	-1.915	-6.218	-	-	-	-	0.887	0.123
	-38.554	-1.937	-6.433	1.328	-	-	-	0.910	0.110
	-43.228	-1.940	-6.329	1.332	1.236	-	-	0.913	0.108
	-37.314	-1.915	-6.501	1.295	1.241	-1.905	-	0.915	0.106
	-39.370	-1.887	-6.916	1.211	1.236	-2.197	3.618	0.917	0.105

ln = natural log;
TT = test temperature;
AV = air voids;
ACL = actual cyclic load;
KV = kinematic viscosity;
ANG = aggregate angularity;
grad = aggregate gradation;
R^2 = coefficient of correlation;
S.E. = standard error; and
\- = not applicable

TABLE 4 REGRESSION MATRIX FOR RESILIENT POISSON'S RATIO OF COMPACTED ASPHALT MIXES USING CONSTANT AND VARIABLE PEAK CYCLIC LOADS (100-lb load excluded)

DEPENDENT VARIABLE RESILIENT POISSON'S RATIO (PR)	INTERCEPT	REGRESSION COEFFICIENT OF THE INDEPENDENT VARIABLE						R^2	S.E.
		AV (10^{-2})	KV (10^{-4})	lnN (10^{-3})	TT (10^{-4})	ACL (10^{-4})	CDI (10^{-6})		
ln(PR)	-1.101	-4.109	-	-	-	-	-	0.503	0.059
	-1.310	-4.269	9.003	-	-	-	-	0.705	0.046
	-1.343	-4.186	9.035	4.030	-	-	-	0.713	0.045
	-1.370	-4.243	8.850	4.662	4.489	-	-	0.720	0.045
	-1.765	-4.281	8.901	4.471	4.181	7.954	-	0.722	0.045
	-1.797	-4.168	8.856	6.219	5.948	8.117	-9.146	0.724	0.044

ln = natural log; TT = test temperature; AV = air voids; ACL = actual cyclic load; KV = kinematic viscosity; CDI = plastic deformation along the vertical diameter (inches); N = number of applications; R^2 = coefficient of correlation; S.E. = standard error; and - = not applicable.

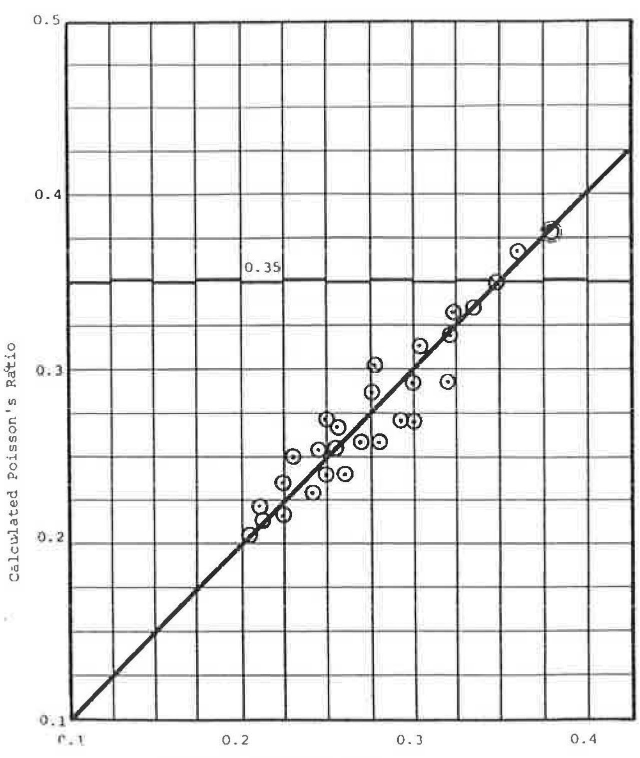

FIGURE 5 Calculated versus measured Poisson's ratios.

CONCLUSIONS

On the basis of test results and analytical and statistical analyses, the following conclusions were drawn:

1. The resilient characteristics of asphalt mixes can be expressed in terms of the mix and test variables.
2. The test temperature and the percentage of air voids in the mix have the greatest influence on the resilient characteristics of the mix.
3. The resilient modulus decreases as the number of load repetitions increases (softening effects).
4. Poisson's ratio of the mixes is dependent on the air voids in the mix and the kinematic viscosity of the binder.
5. Increasing aggregate angularity causes an increase in the resilient modulus.
6. Higher asphalt binder viscosity produces stiffer mix and higher resilient characteristics.
7. For any test specimen, the test results are consistent and quite reasonable.
8. The maximum difference in the values of the resilient or total moduli obtained from a triplicate is only 7 percent.
9. The values of Poisson's ratio for all 414 specimens vary from 0.23 to 0.32. Further, for any triplicate, Poisson's ratio is almost constant.
10. For any combination of variables, the test results can be accurately reproduced.

11. The ASTM standard test procedure D-4123 is inadequate and may lead to misleading results.

SUMMARY

A knowledge of the fundamental mechanical properties of flexible pavement materials is essential for the structural design of pavements. This basic knowledge becomes increasingly important as more highway engineers use pavement design systems based on elastic or viscoelastic theories, or both, that require estimates of these properties. Further, a proper asphalt concrete mix design procedure should be based on these fundamental properties. A new indirect tensile test apparatus was designed and used in this study to calculate asphalt mix properties. The test is simple and yields consistent results. Values of calculated resilient and total characteristics were compatible with those measured during actual tests.

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Reproduction of Thin Bituminous Surface Course Fabric by Laboratory Compaction Procedures

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The purpose of this study was to determine which laboratory compaction procedure best reproduces the pore size distribution of rolled bituminous sand mix surface courses in the field. Mercury intrusion porosimetry tests were performed on laboratory-compacted samples and field cores to determine their pore size distributions. Comparison of fabric was accomplished by using curve descriptors constructed on differential and cumulative pore size distribution curves. Two gradations, two asphalt types, and four aggregate combinations were evaluated. Laboratory compaction was accomplished by varying the parameters of four laboratory compactors and by compacting at three mix temperatures. Analysis of the results of all compaction methods indicated that the gyratory compactor with a vertical ram pressure of 20 psi and 35 revolutions with a 1-degree gyratory angle plus 5 revolutions of leveling pressure (no applied gyratory angle) best reproduces the fabric produced by the rolling procedures presently used in the field.

Surface sand mixes were used by the Indiana Department of Highways (IDOH) in the late 1950s to economically provide smooth, long-lasting riding surfaces on high-volume roads. These $^5/_8$-in.-thick overlays have 95 percent of the aggregate finer than the No. 4 sieve and, in general, have an asphalt content of 7.5 percent. Pavements with an 8 to 12 percent air voids content display good friction numbers without compromising the stability of the pavement (*1, 2*). After having performed satisfactorily for many years, sand mix overlays have begun to display problems in service, especially on a section of Interstate highway. These problems consist of blistering and delamination distress under high-speed truck traffic. Investigations indicated a difference of densities between the top and the bottom portions of the surface course and a horizontal plane of failure at middepth of the overlay. These problems prompted a study of the effects of compaction procedures and mix designs on the performance of these sand mix overlays.

It was believed that the distribution of pore sizes within the compacted surface course could be used to characterize the "fabric" of that material and that the fabric could be related to performance. This is a carry-over from geotechnical engineering, which has further found that compaction equipment and procedures uniquely determine the distribution of pore sizes. Mercury intrusion porosimetry is commonly used to determine the pore size distribution of porous materials, and it can also be used for compacted bituminous paving mixtures. The pore sizes of a bituminous mixture are generally larger than several tens of microns, sizes commonly termed macropores, and are rapidly and economically intruded by low-pressure porosimetry techniques. The asphalt pavement industry has long debated which laboratory compaction procedure best replicates the fabric developed by field compaction procedures. The answer to this question is the major objective of this study. Other investigators have been able to show good correlation between porosimetry results and behavior characteristics (*3–5*). Thus reproduction of the fabric in the laboratory and correlation of fabric to in-service performance of the material could provide good feedback for the mix design process.

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THEORETICAL BASIS OF MERCURY INTRUSION POROSIMETRY

The theory of Mercury intrusion porosimetry has been attributed to Washburn (*6*, pp. 52–60), who pointed out that mercury would not voluntarily intrude pores of particulate materials. The high surface tension of nonwetting liquids, such as mercury, requires such liquids to be forced into the pores. Using a cylinder as a geometric model of a pore, a simple equation was developed relating the size of pore being intruded to the applied pressure. The resulting Washburn equation yields the diameter of an equivalent cylindrical pore that would be intruded at that pressure (in real materials pores are not cylindrical):

$$P = \frac{-2\gamma \cos(\theta)}{r} \tag{1}$$

where

P = pressure causing the intrusion (psi),
r = radius of equivalent cylindrical pore (in.),
γ = surface tension of mercury (lb/in.), and
θ = contact angle between mercury and pore wall (degrees).

MATERIALS

Asphalt Binders

A 7.5 percent asphalt content is the midpoint of the range allowed by the Indiana State Highway Commission (IDOH) Standard Specifications (*7*) and was used throughout this study. Two asphaltic binders are used in sand mix overlays in Indiana:

AC-20 and AE-60. The AC-20 was obtained from the Amoco Oil Company. The AE-60 (IDOH designation) is a high-float, medium-setting asphalt emulsion formulated and provided by the laboratory of McConnaughay Emulsions in Lafayette, Indiana. The basic properties of these binders follow: Kinematic viscosity of the AC-20 is 236.0 cSt at 135°C (275°F); its penetration is 60 at 25°C (77°F), 100 g, and 5 sec-min. The AE-60 has 70.7 percent residue by distillation; penetration of residue is 60 after distillation at 25°C (77°F), 100 g, and 5 sec-min.

Aggregates

Two materials were used as aggregates for the bituminous mixture in this study: local material and crushed agricultural limestone (agg lime). The local pit-run gravel-sand was obtained from the Western Material Company in West Lafayette, Indiana. This material is a terrace gravel-sand deposited by the early Wabash River and consists of approximately equal amounts of calcareous and siliceous minerals. The crushed agg lime was quarried in Indiana and is used in proportions of up to 20 percent of the total aggregate in the mix for the purpose of obtaining required Florida bearing values (*8*). This study combined the two aggregates in proportions of 100 percent local material and 90 percent local, 10 percent agg lime to obtain the required comparisons with field cores. The properties of the aggregate blends are given in Table 1.

TABLE 1 PROPERTIES OF MINERAL AGGREGATE

Property	Local Gravel-Sand	90% Local and 10% Agg Lime
Apparent specific gravity	2.661	2.758
Bulk specific gravity (SSD)[a]	2.615	2.632
Absorption (%)	1.08	0.96
Florida bearing values		
G2	64.0	66.0
FM	67.0	69.0

[a]Saturated surface dry.

Job Mix Formula

Two gradations, representing those commonly used by the IDOH on sand mix projects, were examined. The first gradation, G2, was based on pavement sections showing good performance characteristics. The second gradation, FM, was based on a fineness modulus concept that has only recently been introduced. The FM and G2 gradation curves, as well as the Indiana State Highway Commission (IDOH) standard specifications, are shown in Figure 1.

COMPACTION PROCEDURES

Field Compaction

The procedures commonly specified by IDOH for compaction of HAE Type IV and HAC Type D surface sand mixes require two passes of a three-wheeled steel-wheeled roller followed by two passes of a tandem roller (*9*). The three-wheeled steel-wheeled rollers must have dry roller weights (the two rear rollers) of 300 lb per inch width of roller, and the tandem roller is required to have a weight of 10 tons (*7, 10*).

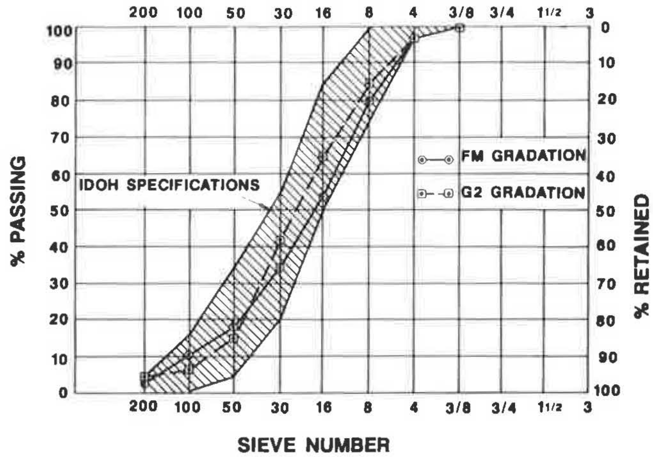

FIGURE 1 Gradation curves and specifications.

Laboratory Compaction

Laboratory specimens were compacted in gyratory, California kneading, vibratory, and Marshall (impact) compactors. Each of these uses significantly different procedures for imparting energy to densify the mix, and this offers the possibility of producing different fabrics. The compaction parameters used in this study to create samples with a density of 135 pcf, the target density for controlling compactive effort, follow:

- Gyratory compactor (at 1-degree gyratory angle)
  20-psi vertical pressure, 35 revolutions
  50-psi vertical pressure, 20 revolutions
  70-psi vertical pressure, 15 revolutions
- Kneading compactor (with 5,000-lb leveling load)
  60-psi contact pressure, 150 tamps
  80-psi contact pressure, 120 tamps
  90-psi contact pressure, 90 tamps
- Marshall compactor
  30 blows per side, 10-lb hammer, 18-in. free-fall
  25 blows per side, 10-lb hammer, 18-in. free-fall
- Vibratory compactor (50-psi air pressure)
  ¼ in./min advance rate, 15,000-lb max load
  ½ in./min advance rate, 15,000-lb max load

The parameters for all compactors are well below typical mix design compaction parameters. In addition to variations in the independent compactor variables, the lift thickness can influence the efficiency with which a compactor arranges and compacts particles. Thus the amount of material compacted to form a sample was varied between 500 and 1050 g. Three compaction temperatures, 230°F, 180°F, and 150°F, were investigated as well.

The major operational aspects of the gyratory testing machine (GTM) have been thoroughly discussed in the literature (*11, 12*) and in ASTM D 3387-74T. The GTM has three variable parameters: vertical ram pressure, number of revolutions, and gyratory inclination angle. As a guide, a minimum of 20 revolutions is required to account for initial roller compaction (*13*). To achieve the target density only, low compaction energies were required. The gyratory angle was held constant at 1 degree as suggested (*10*) because the gyratory action was ineffective with less applied angle. An additional five revolutions of leveling pressure (no applied gyratory angle) were applied at the end of each compaction procedure.

Operation of the kneading compactor was modified for this study, but it basically followed the procedure of ASTM D 1561. The variable parameters for this procedure are maximum contact pressure, number of tamps, and leveling load. The tolerable maximum contact pressure was 90 psi, well below the 500-psi standard pressure required by the Hveem method. Higher pressures resulted in the tamping foot punching through the mixture and pulling it out of the mold on the upstroke. A maximum leveling load of 5,000 lb, applied after the mix was kneaded, was necessary to produce samples that met the density criterion. The number of pressure applications required to sufficiently work the mixture was previously determined to be 150 (*14*). Because of physical limitations of the kneading compactor, 60 psi was the minimum contact pressure capable of compacting the material.

Samples were compacted in the Marshall compactor as prescribed in ASTM D 1559. This compaction procedure has become a major component of procedures for bituminous mix design. The variable parameters are weight of hammer, free-fall distance, and number of impacts per sample side. The limited number of hammers and the fact that the automatic Marshall compactor used in this study could only accommodate a free-fall distance of 18 in. limited the number of combinations of parameters investigated.

The vibratory compactor was designed to simulate the action of vibratory rollers. The variable parameters for this procedure are maximum applied load, rate of vibrating unit advancement, and air pressure causing vibration. The vibratory action was obtained by compressed air causing a piston to vibrate vertically resulting in the acceleration of the compactor head. Vibration during compaction was possible because of the insertion of a neoprene cushion on top of the vibrating piston unit. The vibratory action was most effective at low vertical pressures. As the vertical load was increased to its limiting value of 15,000 lb (due to the compressibility of the neoprene cushion) the vibrating action decreased. Compaction of the samples is achieved by the advance of the vibrating unit through the use of the Riehle Machine static compactor.

MERCURY INTRUSION POROSIMETRY

The porosimeter system consists of high-vacuum pump, vacuum-to-atmospheric manometer, McCleod gauge, immersing device, and vacuum stopcock that opens to the atmosphere, all of which are connected by vacuum tubing (Figure 2). The manometer shows the absolute pressure in the system to the nearest millimeter of mercury. The McCleod gauge is used for readings of high vacuum pressure, to the nearest 0.01 mm Hg, in the system. The immersing device houses the penetrometer and a quantity of mercury sufficient to surround and fill the penetrometer when the immersing device is tilted into its filling position. One vacuum stopcock allows the entry of pressure into the system by opening to the atmosphere. The atmospheric pressure enters slowly because of the small orifice through which it must pass, allowing the desired pressure increments to be easily attained. The penetrometer is an accurately calibrated glass stem with a sample-holding chamber. The entire system must be tightly sealed.

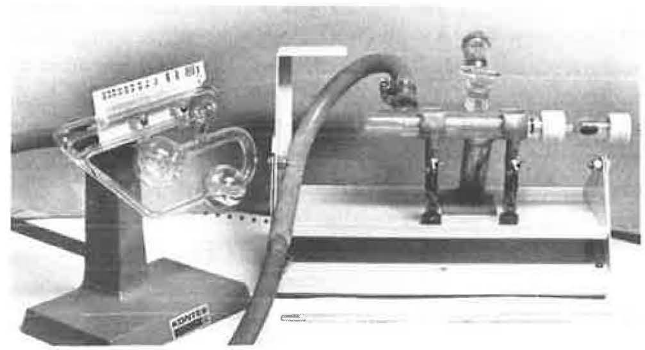

FIGURE 2 Porosimeter apparatus.

Sample Preparation and Testing

Porosimetry specimens about 1 cm × 1 cm × 1 cm in size were obtained from compacted material, either field cores or laboratory samples. This was accomplished by freezing the compacted material to −10°F to produce a brittle condition that made it possible to crack the material without disturbing the particle arrangement. Specimens from laboratory-compacted samples were then desiccated for not less than 24 hr to remove most of the volatile gases. The field samples were thought to be sufficiently cured already because 1 month or more had elapsed from the time of construction to the time the cores were tested.

After the weight measurements were obtained, a specimen was placed in the apparatus, the system was sealed from the atmosphere, and the vacuum pump was turned on to evacuate the air within the system. A vacuum pressure of at least 0.02 mm Hg was obtained within the system before 20 mm of atmospheric pressure was applied to force mercury into the penetrometer to surround the test specimen. Light tapping of the penetrometer end was required until the reading on the penetrometer stem stabilized to assure the mercury intruded all voids at this and each additional increment of pressure applied to the system.

The pressure increments applied to a test specimen must reflect the predominant pore diameters within the sample. The spacing must be small enough to obtain sufficient curve descriptor accuracy, yet large enough to allow the differential volumes intruded to be significant and the variability to be kept reasonable. The quantity controlling this even spacing on the logarithmic x-axis is c in Equation 2. Equally spaced data points along this axis allow simple geometric constructions for fabric descriptors to be comparable anywhere on these plots. A value for c of 0.099 was selected because previous investigations indicated that this would produce meaningful, yet sensitive, descriptors.

$$\frac{P_i}{P_{(i-1)}} = 10^c \qquad (2)$$

where

P_i = ith increment of pressure,

$P_{(i-1)}$ = previous pressure increment, and
c = constant of increment.

Comparison of Pore Size Distribution Data

The differential and cumulative pore size distribution (PSD) curves (Figures 3 and 4) were generated by a computer program in which each of the data points from a porosimetry test constituted one data point. Descriptors, characterizing the distribution curves, were created by simple geometric constructions; these were used to compare curves from various test conditions. Work performed previously (*15*) determined that the greatest effort should be directed toward the differential curve because this plot appears to be the most sensitive to differences in fabric. The descriptors used for this study were selected to measure the quantities believed to be influential in the performance of sand mixes and permit the differentiation of material fabrics. These descriptors are defined next.

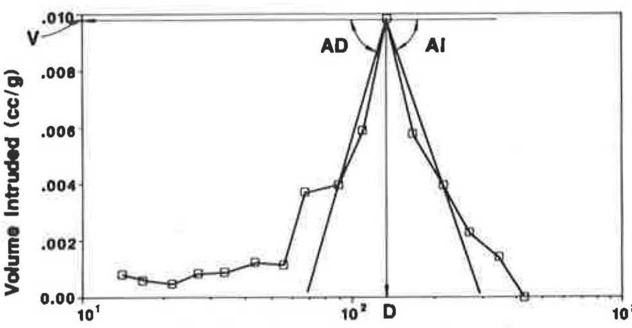

FIGURE 3 Differential PSD curve and descriptors.

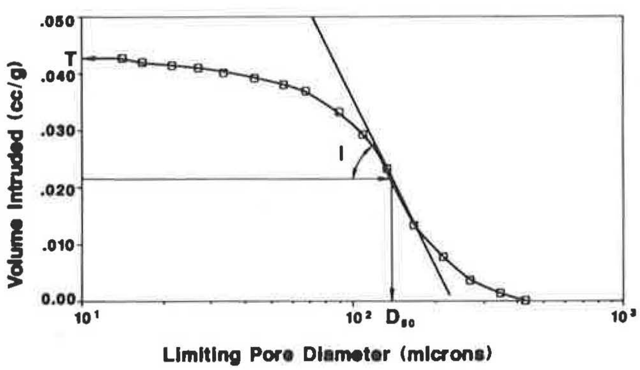

FIGURE 4 Cumulative PSD curve and descriptors.

LD is the logarithm of the pore diameter, shown as D in Figure 3, at which the peak frequency occurs. The conversion of D to its logarithm occurs after the value of D has been interpolated from the x-axis. This descriptor is probably related to the permeability of the mixture.

V/T is the ratio of the volume intruded at the peak frequency to the total volume intruded in the sample. The two descriptors, V and T, in Figures 3 and 4, respectively, are combined to form this descriptor. This measurement and the descriptors AI and AD determine the concentration of pores about the maximum frequency diameter.

AI is the onset angle and AD is the offset angle to the peak; both are measured on the differential plot. These descriptors are obtained by constructing a line through the peak frequency and two data points on either side of this peak frequency.

LD_{50} is the logarithm of the pore diameter, shown as D_{50} in Figure 4, where 50 percent of the intruded volume lies in pores of larger sizes. Again, D_{50} is interpolated from the x-axis and is converted to its logarithmic value for comparison purposes. This quantity is found by multiplying T by 0.5 to obtain H. The diameter corresponding to the intersection of H and the cumulative porosity curve was defined as D_{50}.

T/P, the percentage of total voids intruded, is not shown on these plots. This percentage of interconnected voids is believed to be a critical parameter in sand mixes for reasons of permeability and possible performance. This descriptor uses P, the total volume of voids in the sample, a value computed separately, and T, shown in Figure 4.

ANALYSIS OF RESULTS

The results presented represent work performed with the following combinations of materials: G2, AE-60 with both local and 90-10 aggregate blends as well as FM, AC-20, and local material. The means and standard deviations for each set of porosimetry samples, laboratory and field, having similar gradation and aggregate combinations are given in Tables 2–4. The number of porosimetry tests performed on the various compacted samples was 12 tests on field samples and from 5 to 7 tests on laboratory samples. Within each table of results, the means of the various laboratory descriptors were checked against that of the field descriptors using a 95 percent confidence interval to determine the equivalence, or otherwise, of the laboratory and field mean values. The underscored numbers indicate the descriptor means that are statistically equivalent to that of the field cores. The comparisons that were made between laboratory and field samples for FM gradation (Table 4) have only limited applicability because of the higher densities of the specimens compacted in the laboratory. This higher density was obtained even though the compaction parameters were reduced to extremely low values.

The results for samples with a 90-10 combination of aggregate and AE-60 as a binder indicate that as the applied pressure is reduced and the time of compaction increased, the fabric obtained approaches that of the field cores. This is true for both the gyratory and kneading compactors with the first being more sensitive. The trend is not followed, however, for the compacted samples containing only local material and is, indeed, reversed.

Gradation changes produced noticeably different pore size distribution curves in both the field and the laboratory samples. The FM gradation specimens had less pore volume concentrated about the peak frequency diameter. The differences in curves produced by the two gradations are much more pronounced in the laboratory samples.

Compaction temperatures had no influence of the fabric created in the laboratory. Lower compaction temperatures have a slight tendency to increase the percentage of voids intruded in a test specimen. This could be the result of nonuniform distribution of binder within a compacted sample. Also, the amount of material compacted to form a sample had little influence on the resulting pore size distribution.

TABLE 2 90-10 G2 BLEND RESULTS

	LD	V/T	AI	AD	LD_{50}	T/P
Field cores	2.122 (0.106)	0.215 (0.022)	65.5 (6.72)	65.2 (9.16)	2.164 (0.069)	0.752 (0.084)
Gyratory at 230°F						
20 psi for 35 revs	1.984 (0.126)	0.221 (0.026)	37.8 (14.72)	44.5 (21.16)	2.082 (0.046)	0.427 (0.066)
30 psi for 20 revs	2.130 (0.065)	0.244 (0.022)	53.2 (11.65)	65.2 (6.89)	2.129 (0.118)	0.457 (0.122)
500-g sample, 30 psi	1.919 (0.106)	0.165 (0.019)	53.7 (14.60)	56.9 (10.50)	2.016 (0.084)	0.674 (0.088)
Kneading at 230°F, 90 psi for 3 min	1.913 (0.136)	0.224 (0.047)	46.3 (7.61)	39.6 (16.90)	1.979 (0.076)	0.395 (0.086)
Kneading at 180°F, 90 psi for 3 min	2.047 (0.107)	0.216 (0.019)	42.4 (19.19)	43.3 (18.70)	2.112 (0.057)	0.459 (0.115)
Vibratory at 230°F, 15,000 lb 0.25 in./min	2.102 (0.096)	0.143 (0.021)	38.6 (24.30)	23.8 (13.40)	2.055 (0.040)	0.671 (0.153)
Marshall at 230°F, 25 blows/ side	2.040 (0.056)	0.228 (0.043)	49.2 (9.40)	52.1 (16.20)	2.108 (0.038)	0.553 (0.051)

NOTE: Standard deviations are in parentheses. Underscored values are equivalent to the field mean within a 95 percent confidence interval.

TABLE 3 LOCAL G2 BLEND RESULTS

	LD	V/T	AI	AD	LD_{50}	T/P
Field cores	2.069 (0.069)	0.222 (0.030)	57.3 (11.86)	55.9 (15.80)	2.110 (0.051)	0.674 (0.149)
Gyratory at 230°F						
20 psi for 35 revs	2.034 (0.110)	0.200 (0.024)	44.0 (10.59)	51.4 (9.23)	2.070 (0.072)	0.486 (0.040)
30 psi for 20 revs	2.019 (0.115)	0.196 (0.022)	39.8 (21.19)	41.9 (18.51)	2.050 (0.052)	0.484 (0.075)
70 psi for 15 revs	1.969 (0.110)	0.197 (0.026)	32.8 (12.10)	26.1 (11.35)	2.030 (0.061)	0.430 (0.080)
Gyratory at 180°F, 20 psi for 35 revs	1.927 (0.155)	0.174 (0.028)	46.2 (15.02)	41.9 (19.47)	1.907 (0.096)	0.530 (0.029)
Kneading at 230°F						
90 psi for 3 min	1.870 (0.095)	0.150 (0.014)	44.6 (11.14)	44.7 (11.70)	1.908 (0.087)	0.563 (0.046)
60 psi for 5 min	1.883 (0.092)	0.152 (0.014)	45.7 (11.90)	32.6 (13.15)	1.889 (0.064)	0.572 (0.031)

NOTE: Standard deviations are in parentheses. Underscored values are equivalent to the field mean within a 95 percent confidence interval.

TABLE 4 LOCAL FM BLEND RESULTS

	LD	V/T	AI	AD	LD_{50}	T/P
Field cores	1.934 (0.090)	0.227 (0.036)	52.3 (14.59)	62.3 (9.35)	2.010 (0.044)	0.568 (0.058)
Gyratory at 230°F, 20 psi for 35 revs	1.706 (0.138)	0.124 (0.028)	19.8 (14.24)	17.9 (13.36)	1.785 (0.042)	0.306 (0.045)

NOTE: Standard deviations are in parentheses.

A consistent and important observation made from the results is that when the pores were concentrated about the peak, a higher percentage of the voids was intruded (interconnected). This trend is consistent for compaction temperatures, compactors, and pressure-time relationships.

Tests previously conducted on cores from pavement surfaces that performed well and poorly indicate that the pavements that performed poorly had larger volumes of pores at the peak. The set of samples obtained from areas that performed well had pores more evenly distributed among the various pore diameters. On the basis of these limited results, it is believed that the FM gradation could prove to be stable when subjected to service loads.

SUMMARY AND CONCLUSIONS

Two conclusions can be drawn from the results obtained in this study. First, the gyratory compactor with a 20-psi vertical ram pressure for 35 revolutions at a 1-degree angle of inclination plus 5 leveling revolutions (no gyratory angle) best duplicates the fabric obtained by the field compaction procedures currently in use with Gradation G2 mixes. The kneading compactor also produces some fabric characteristics that are not too different from field fabric. Although the fineness modulus gradation fabric was not matched as well by the GTM procedure, these investigators believe that the GTM, used at low vertical ram pressures, offers the best opportunity to reproduce sand mix fabric for a range of gradations.

Second, a high concentration of pores near the peak frequency diameter, a characteristic common in the fabric created by field rolling of G2 surface courses, allows the possibility of redistribution of these pores on compaction by traffic. These pores are not always redistributed, which results in mixed performance of these pavements. When compaction due to the action of traffic changed the fabric to a less pronounced peak on the pore size distribution curve, acceptable performance was observed. When no redistribution of the pore sizes occurred in the fabric, poor pavement performance was observed. Tests performed on newly paved overlay sections that consisted of aggregate meeting the FM gradation specifications revealed less pore volume located near the peak frequency diameter. This lower initial volume near the peak should be associated with less variability and better in-service performance characteristics. The authors believed that the compaction of thin overlays and the gradation used should allow a low concentration about the peak frequency diameter to obtain pavements that perform well. One note of caution: only a very few test results were available on the characteristics of pavements that performed well and poorly in the field.

ACKNOWLEDGMENTS

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Characterization of Hot-Mixed Open-Graded Asphalt Mixtures

CHAOUKI A. GEMAYEL AND MICHAEL S. MAMLOUK

A detailed laboratory investigation was undertaken to study the effects of different mix components on several engineering properties of open-graded asphalt mixtures. Among these properties are density, air voids, Marshall stability and flow, resilient modulus, tensile strength, and permeability. Properties of the open-graded mixture were compared with those of dense-graded mixtures and open-graded cores obtained from a porous pavement experimental test section. The asphalt content and aggregate gradation influenced the density, air voids, Marshall stability, instantaneous and total resilient moduli, and coefficient of permeability of laboratory-prepared open-graded specimens. The tensile strength was not affected by either the asphalt content or the aggregate gradation at a significance level of 0.05. The resilient characteristics were found to be highly affected by temperature. Lower densities and consequently higher air voids were measured on the open-graded cores than on laboratory-prepared specimens containing similar material components. Lower Marshall stability and resilient moduli were measured on the cores. Theoretical analysis revealed significant differences between the predicted performance of open-graded and dense-graded asphalt pavements. Thicker layers were required for the open-graded asphalt mixture to produce the same vertical deformation on top of the subgrade. The study resulted in understanding basic engineering properties of open-graded asphalt mixtures so their use in pavement structural layers would be based on a more rational procedure.

Among the many effects of urbanization on the environment is the increased runoff caused by the lower infiltration of paved surfaces such as streets, parking lots, and building roofs (*1*). As a result, existing sewer systems are overloaded, and new systems are expensive to construct. These problems are especially obvious in arid areas where typical storms are of high intensity and short duration in the summer, which results in high peak discharge, and of long duration and low intensity in the winter, which creates a large volume of runoff (*2*). Both cases require large highway drainage structures that typically account for more than 35 percent of the total cost of construction of highway projects in urban areas.

Porous pavements consisting of open-graded surface, base, and subbase layers have been suggested as a less costly alternative to conventional pavements (*1*). The concept behind this alternative is to eliminate highway runoff or to design the runoff to be the same as that of the original site. In addition to adequately carrying traffic loads, the main purpose of the porous pavement is to absorb, store, and dissipate storm waters into the ground. Water supplied to the pavement surface drains vertically through interconnected voids and is stored in the base and subbase until it can percolate into the subgrade (*3*). The permeability of the porous pavement, which is typically controlled by that of the surface layer, is designed so that water passes through at a rate faster than the rate of water supply to the pavement surface in order to prevent water ponding. Other advantages of porous pavements include higher skid resistance, better visibility of pavement marking during rain, and lower hydroplaning potentials in comparison with conventional asphalt concrete pavements (*4*).

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BACKGROUND, PROBLEM STATEMENT, AND OBJECTIVES

Previous porous pavement applications have been generally limited to parking lots and driveways (*1, 3*). Some problems were reported, such as scuffing of the pavement surface due to steering and braking actions of traffic and reduction in permeability with time due to clogging of the pavement pores.

In an effort to study the feasibility of the porous pavement for controlling runoff on major roads, the Arizona Department of Transportation (ADOT) established a 3,500-ft-long three-lane-wide experimental test section on the northbound lanes of Arizona Avenue in Chandler, Arizona. The average daily traffic (ADT) is 25,000 to 30,000 vehicles with 7 to 8 percent trucks. The pavement consisted of a thick open-graded surface course supported by an open-graded asphalt-stabilized base on top of an open-graded aggregate subbase (Figure 1). The surface layer

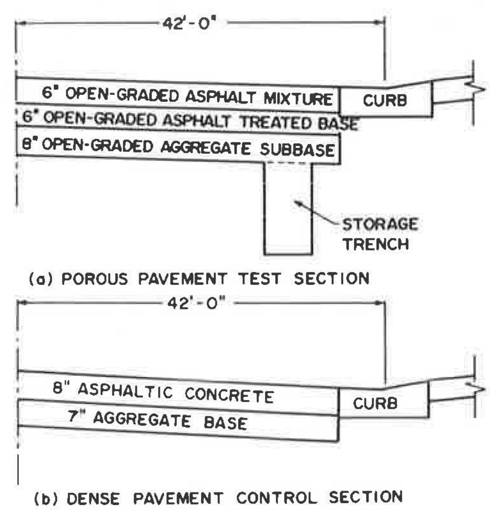

FIGURE 1 Typical cross section of ADOT's experimental porous pavement.

consisted of 6 in. of hot-mixed, open-graded asphalt mixture with a maximum aggregate size of 3/8 in. The open-graded mix was compacted using static rollers to avoid breaking of aggregate particles. The subgrade consisted of clayey material with low permeability, and it was not saturated during construction. The pavement was designed to control the runoff expected from a 100-year storm or two 10-year storms on consecutive days. The conventional asphalt concrete pavement located on both ends of the porous pavement test section was designated as the control section. It should be noted that the ADOT experimental pavement is quite different from most previous porous pavements, which consist of a porous aggregate base and a thin porous asphalt surface. Also, the open-graded asphalt mixture has been previously used in thin friction courses to increase skid resistance and to improve riding quality (*5, 6*).

Shortly after the construction of the two outside lanes of pavement in May 1986 and after a few weeks of stage construction detour traffic, rutting problems occurred that led to the temporary closure of the roadway as a stage construction detour. Rut depth ranged between 3/8 and 1 in. as shown in Figure 2. To eliminate these depressions and to satisfy design thickness requirements, the open-graded surface layer was further compacted using vibratory rollers and then overlaid with a 1-in.-thick layer of the same open-graded mixture and compacted with a vibrating roller.

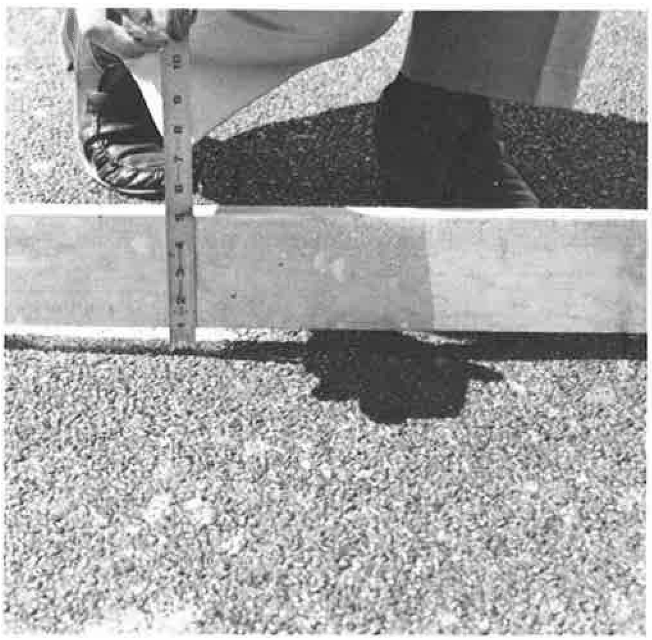

FIGURE 2 Example of rutting resulting from stage construction detour traffic before final 1-in. lift of surface.

Rutting was not the only problem encountered. Difficulties were observed during the placement and compaction of the surface layer. The mixture appeared to be rich in asphalt and failed to harden.

MIX DESIGN OF HOT-MIXED, OPEN-GRADED ASPHALT MIXTURES

The mix design method employed by many highway agencies has produced satisfactory results for thin friction courses. However, the response of these thin layers to load is different than that of thick layers. Most of the load applied to thin layers, with approximately the same thickness as the maximum aggregate size, is carried by large aggregate particles that transmit the load to the next layer of the pavement structure. On the other hand, the stability of thick layers is governed by other factors, especially the interlocking effect of aggregate particles. Therefore open-graded asphalt mixtures that are intended for use in thick layers require different mix design procedures than do open-graded mixtures placed in thin layers.

For friction courses, the optimum asphalt content of open-graded asphalt mixtures is usually computed from empirical formulas developed from field experience. These formulas are mainly a function of the surface capacity (K_c) of the coarse aggregate fraction of the mixture. In addition to a design formula developed by FHWA (*7*), several states have developed their own formulas based on their past experience. The formula used by ADOT to determine the asphalt content for friction courses was also used for the open-graded mixture of the porous pavement experimental project. This formula is (*8*)

$$AC = (1.5\, K_c + 3.5) \times 2.65/G_{oc}$$

where

AC = asphalt content by weight of total mix,
K_c = surface capacity of the coarse aggregate fraction, and
G_{oc} = combined aggregate oven-dried specific gravity.

Little information is available in the literature on the structural ability of the hot-mixed, open-graded asphalt mixture. Most of the available literature on the strength of open-graded mixtures is concerned with emulsion mixtures (*9*) and asphalt-stabilized aggregate bases (*10*). A better understanding of the mechanical properties of open-graded asphalt mixtures and the factors affecting these properties is still needed.

The objectives of this study are to

1. Evaluate the mechanical characteristics of cores obtained from the surface layer of the experimental porous pavement constructed by ADOT;
2. Compare the properties of field and laboratory specimens prepared with the same materials and proportions in order to detect construction deficiencies;
3. Examine the effect of asphalt content and aggregate gradation on the permeability, strength, and stability characteristics of the hot-mixed, open-graded asphalt mixture; and
4. Compare the strength parameters and the thickness requirements of the hot-mixed, open-graded mixture with those of the dense-graded mixture in order to evaluate the feasibility of placing the open-graded mixture in thick layers.

LABORATORY CHARACTERIZATION

The main part of this investigation was a laboratory evaluation of certain engineering properties of the hot-mixed, open-graded asphalt mixture. The study also involved the use of the mixture properties to examine the structural ability of the open-graded asphalt mixture compared with that of the dense-graded asphalt concrete.

Response Variables

The response variables or the measured mixture properties were selected in order to achieve a better understanding of the stability, strength, and permeability characteristics of the hot-mixed, open-graded asphalt mixture. These properties include

1. Density,
2. Air voids,
3. Marshall stability and flow,
4. Resilient modulus at 41°F and 77°F,
5. Tensile strength at 77°F, and
6. Coefficient of permeability.

Independent Variables

Two independent variables or factors were selected for the evaluation of the open-graded asphalt mixture, asphalt content and aggregate gradation. A third factor, test temperature, was also considered during the resilient modulus evaluation.

Because field observations indicated that the mixture was too rich in asphalt, four levels of asphalt content, based on the weight of oven-dried aggregates, were considered in this study: 4.0, 4.5, 5.0, and 5.7 percent. The latter asphalt content is the one used in the field. Two aggregate gradations were also used.

Initially, the resilient modulus evaluation was intended to be performed at three temperatures: 41°F, 77°F, and 104°F according to the ASTM D 4123 method. However, the open-graded mixture was unstable at temperatures higher than 90°F. For this reason, the evaluation was performed only at the lower two temperatures.

The dense mixture examined in the course of this investigation was similar in both aggregate grading and asphalt content requirements to the dense mixture used in the control section of ADOT's porous pavement project. The laboratory evaluation was performed only at the Marshall optimum asphalt content, which is equal to 5.7 percent by weight of aggregate. Note that the optimum asphalt content was the same for both open-graded and dense-graded mixes, although they were designed using two different methods. Because the surface area of the open-graded aggregate is less than that of the dense-graded aggregate, the amount of asphalt needed to coat the aggregate particles should be smaller for the open-graded mix. However, the open-graded design formula incorporates additional asphalt to counter the fast deterioration of open-graded mixes.

Aggregates

Representative aggregate samples were obtained from approved stockpiles for ADOT's porous pavement project. Strict specifications applied to the quality of the aggregates in the field as well as in the laboratory. At least 90 percent of the aggregates retained on the No. 8 sieve was required to have one or more fractured face, which was produced by crushing, in an effort to reduce mixture stability problems. Two open and one dense aggregate gradations were used in the laboratory (Figure 3). The first open gradation, with a maximum size of 3/8 in. (Gradation A), was identical to the gradation used in the field. This gradation was determined from quality control field records such as those on binder extraction tests performed on production samples. The second open gradation with a 3/4-in. maximum size (Gradation B) was selected from the middle of the open-gradation specification band of ADOT. The third gradation was dense (Gradation C) with a 3/4-in. nominal maximum aggregate size, and was similar to the gradation used on the control section of the porous pavement project. Table 1 gives a summary of the apparent (G_{APP}), saturated surface dry (G_{SSD}), and oven dry (G_{OD}) bulk specific gravities, as well as water absorption (P) for Gradations A, B, and C.

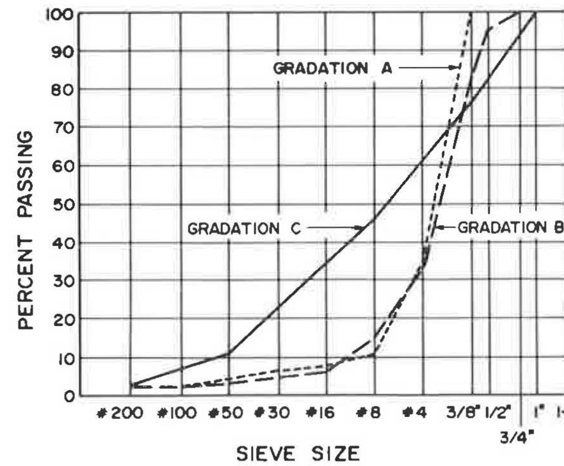

FIGURE 3 Aggregate gradations used in this study.

TABLE 1 SPECIFIC GRAVITIES AND ABSORPTION OF AGGREGATE GRADATIONS A, B, AND C

Gradation	Size[a]	G_{APP}	G_{SSD}	G_{OD}	P (%)
A	Coarse	2.729	2.688	2.665	0.86
	Fine	2.711	2.660	2.629	1.18
B	Coarse	2.735	2.676	2.642	1.30
	Fine	2.684	2.622	2.585	1.42
C	Coarse	2.712	2.661	2.631	1.13
	Fine	2.683	2.616	2.577	1.54

[a]Coarse is +No. 8; fine is −No. 8.

Asphalt Binder

AC-40 asphalt cement samples were obtained from the same source as the binder used in the field. Table 2 gives the asphalt binder characteristics.

Cores

Six 4-in.-diameter cores were obtained from the center of the outside lane of the existing porous pavement in order to compare their properties with those of laboratory-prepared samples. Dry ice was placed on top of the selected core location for approximately 15 min before drilling to stiffen the material during the coring operation. The cores were kept in a low-temperature-controlled room to prevent disintegration during the short storage time before they were trimmed to 2.5-in. thicknesses and tested.

Specimen Preparation

Marshall-size open-graded specimens were prepared according to Arizona Method 814 (*8*) using 950 g of aggregates blended to the specified gradations (A and B). The mixture was compacted at a temperature of 230 ± 5°F using the Marshall hand compactor and applying 75 blows on each side. The same compactive effort was used on the dense-graded specimens that were prepared according to the ASTM D 1559 procedure. Three replicate specimens were prepared for each combination of asphalt content and aggregate gradation for each test.

TABLE 2 ASPHALT BINDER PROPERTIES

Test	AASHTO Method	Test Results	ADOT Specification
Absolute viscosity at 140°F (poises)	T202	4004	3200–4800
Kinematic viscosity at 275°F (centistokes)	T201	327	300 min
Penetration at 77°F, 100 g, 5 sec	T49	44	40 min
Flash point, Penesky-Martin (°F)	T73	537	450 min
Absolute viscosity of aged residue (poises)	T202	10 197	20 000 max
Ductility at 77°F (centimeters), aged residue	T51	+145	75 min
Specific gravity	T228	1.023	

Laboratory Testing

The tests performed included

1. Density
   - Dense specimens: saturated surface dry method (ASTM D 2726).
   - Open-graded specimens: weight of the specimen in air divided by its volume as determined from physical dimensions (i.e., height and diameter).
2. Marshall test (ASTM D 1559).
3. Resilient modulus test at 41°F and 77°F (ASTM D 4123) with a pulse duration of 0.1 sec and a rest period of 2.9 sec. Poisson's ratios of 0.3 and 0.35 were assumed at the two temperatures, respectively.
4. Indirect tension test at 77°F (ASTM D 4123) with a rate of loading of 2 in./min. The Instron electrohydraulic closed-loop loading machine was used. The same specimens used in the resilient modulus test were used in the indirect tension test.
5. Constant head permeability test (*11*) (Figure 4).

RESULTS AND DISCUSSION

The results were statistically analyzed using the ANOVA technique at a significance level of 5 percent. A two-factor factorial design was employed for all response variables except the resilient modulus, which was analyzed as a three-factor factorial. The statistical analysis yielded the following results.

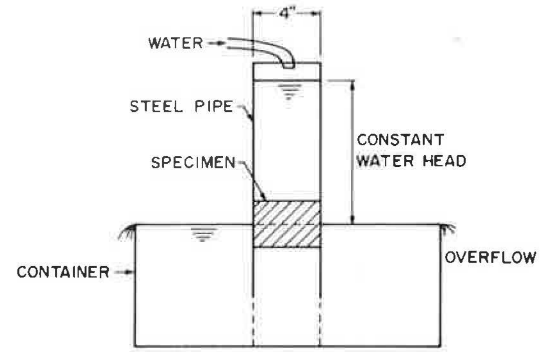

FIGURE 4 Constant head permeability apparatus.

Density and Air Voids

The density of laboratory-prepared open-graded specimens ranged from 118.7 to 124.5 pcf with an average of 121.7 pcf. The average densities obtained for the different asphalt contents and aggregate gradations are given in Table 3. Both asphalt content and aggregate gradation significantly affected the compactibility of the mixture. In general, because of the high air voids content that characterizes the open-graded asphalt mixture, the added asphalt cement fills the voids and leads to higher densities.

TABLE 3 AVERAGE DENSITIES OF SPECIMENS INCLUDED IN THE STUDY

Asphalt Content (%)	Gradation A	Gradation B	Gradation C	Cores
4.0	119.2	122.8		
4.5	119.6	122.7		
5.0	120.6	123.6		
5.7	121.3	123.7	143.4	112.3

NOTE: Units are pounds per cubic foot.

The change in aggregate gradation from A to B yielded higher densities. Also, the cores showed a significant decrease in density compared with specimens prepared in the laboratory and containing similar material ingredients. It should be noted that a number of compaction techniques were used in the field and that the low densities observed in the cores may have been caused by the difficulty of compacting the mix in the field, especially at the high temperature encountered during construction, or by the lack of support of the subbase layer of cohesionless unbound aggregate.

The densities of the dense-graded specimens were, as expected, much higher than those of the open-graded specimens. Obviously, the larger fine aggregate fraction of the dense mixture, which results in a low air voids content, is the major cause of the higher densities.

The percentage of air voids of all laboratory-prepared open-graded specimens ranged between 17.8 and 23.7 percent with an average of 20.8 percent. Table 4 gives a summary of the average results for specimens prepared at various asphalt contents using different aggregate gradations. The percentage of air voids at the time of testing was significantly affected by the

TABLE 4 AVERAGE PERCENTAGE OF AIR VOIDS OF SPECIMENS INCLUDED IN THE STUDY

Asphalt Content (%)	Gradation A	Gradation B	Gradation C	Cores
4.0	23.4	20.9		
4.5	22.6	20.4		
5.0	21.3	19.2		
5.7	20.0	18.4	6.0	25.9

asphalt content and the aggregate gradation. In addition, the average percentage of air voids of specimens prepared with Gradation A was slightly higher than that of specimens prepared with Gradation B.

The low densities of the cores led to a higher air voids content than was found in laboratory-prepared specimens. Laboratory specimens containing similar material ingredients had air voids contents of 20.0 percent, which is significantly lower than the average air voids content of 25.9 percent of the cores. On the other hand, an average air voids content of 6.0 percent was obtained for dense-graded specimens; this is lower than that for open-graded specimens.

Marshall Stability and Flow

The stability and flow results for asphalt mixtures evaluated in this study are shown in Figures 5 and 6. The typical stability of laboratory-prepared open-graded specimens ranged between 575 and 975 lb with an average of 705 lb. Asphalt content and

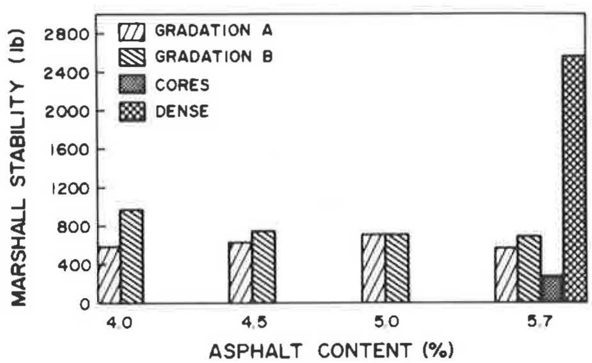

FIGURE 5 Effect of aggregate gradation and asphalt content on Marshall stability.

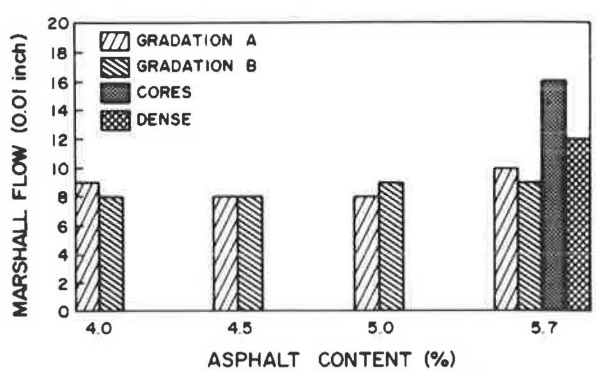

FIGURE 6 Effect of aggregate gradation and asphalt content on Marshall flow.

aggregate gradation as well as their interaction significantly affected the stability of the mixture.

The maximum stability value measured for the open-graded mixture was lower than that specified by most highway departments for dense-graded mixtures. However, it is not known if the same criterion should be applied to the open-graded mixture because no field performance data are available. Moreover, the average stability values of the open-graded cores were lower than those obtained for laboratory specimens prepared with the same material ingredients. The average stability of the cores was 270 lb, which is equal to 47 percent of the average stability of the laboratory specimens.

As expected, the average stability obtained for the dense-graded mixture (Gradation C) was significantly higher than for the open-graded mixture. The higher stability is caused by the larger fine aggregate fraction that provides a "choking" action between the larger aggregate particles. The average stability of the dense mixture was equal to 2,560 lb, which represents an increase of 265 percent over the overall average stability of the laboratory-prepared open-graded mixtures.

The typical flow values of the laboratory-prepared open-graded specimens ranged between 8 and 10. Figure 6 shows the average flow results of all asphaltic materials tested during this investigation. The narrow range of the flow indicates that neither gradation nor asphalt content had any practical effects on the flow of laboratory-prepared open-graded specimens. Further, the average flow obtained for the dense mixture was slightly higher than that obtained for the open-graded laboratory-prepared mixture. The flow values obtained for the cores had an average of 16, which is 60 percent greater than the average of laboratory specimens prepared with similar material components.

Resilient Modulus

Diametral resilient modulus at different temperatures is a measure of the stiffness and of the temperature susceptibility of an asphalt paving mixture. The trend toward rationally designing flexible pavements through numerical analysis of layered systems for strains, stresses, and deflections necessitates the use of the resilient modulus to characterize asphalt mixtures. In general, the resilient modulus is defined as the ratio of the applied stress to the recoverable or resilient strain after many repetitions when the load is applied in a pulsating form.

Figures 7 and 8 show the average instantaneous resilient modulus (E_{RI}) results determined for the laboratory-prepared specimens as well as the field cores at the two test temperatures.

The E_{RI} results of the laboratory-prepared open-graded specimens were significantly affected by test temperature, aggregate gradation, and asphalt content. As expected, an increase in temperature caused a decrease in E_{RI}-values because the higher temperature reduces the viscosity of the asphalt binder and therefore decreases the stiffness of the mixture. A reduction of 65 percent in the average E_{RI}-values was observed for both Gradations A and B when the testing temperature was increased from 41°F to 77°F.

At 77°F, a change in the aggregate gradation from A to B caused a 14 percent decrease in the average E_{RI}-value. Also, the change in asphalt content had no practical effect on the E_{RI}-

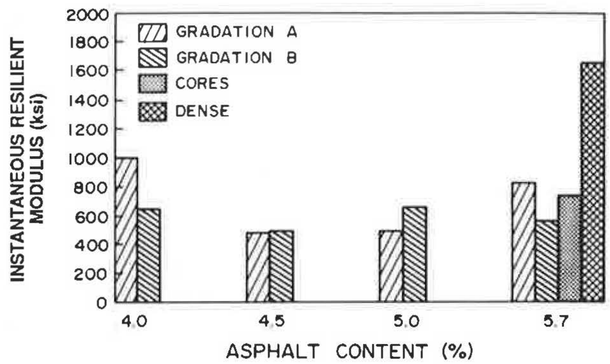

FIGURE 7 Effect of aggregate gradation and asphalt content on the instantaneous resilient modulus obtained at 77°F.

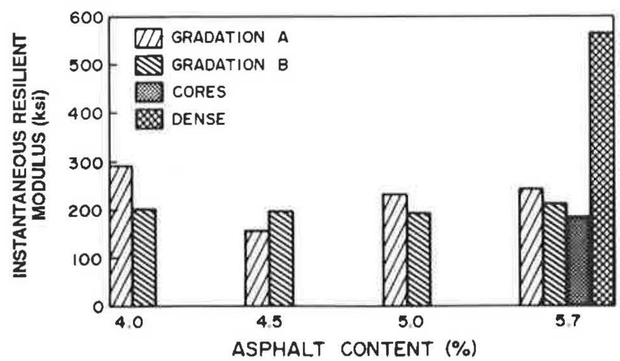

FIGURE 8 Effect of aggregate gradation and asphalt content on the instantaneous resilient modulus obtained at 41°F.

values measured at 77°F for specimens prepared with Gradation B. However, a change in asphalt content significantly affected the E_{RI} results of Gradation A. On the other hand, a change in aggregate gradation from A to B decreased the average E_{RI}-value by 19 percent from 705 to 595 ksi at a test temperature of 41°F.

The E_{RI} results of the open-graded cores were lower than those of the laboratory specimens prepared with similar material components. The decrease in the E_{RI} results is mostly attributed to the low core densities. Furthermore, the dense mixture exhibited E_{RI}-values that were much higher than those obtained for the open-graded specimens as shown in Figures 7 and 8.

The same factors and interaction of factors that affect the instantaneous resilient modulus results also influence the total resilient modulus (E_{RT}) results. Although the individual values might be different, the same trends observed for the E_{RI}-values are also observed for the E_{RT}-values.

Tensile Strength

Tensile strength is a measure of the maximum tensile stress an asphalt paving mixture can withstand. This parameter is related to thermal and shrinkage cracking resistance. In this study typical values of the tensile strength (S_T) of the laboratory-prepared open-graded specimens ranged between 100 and 120 psi with an average of 105 psi at a test temperature of 77°F. The average S_T-values obtained during this investigation are summarized in Figure 9. Neither aggregate gradation, asphalt content, nor the interaction of the two factors had any significant effect at a significance level of 0.05.

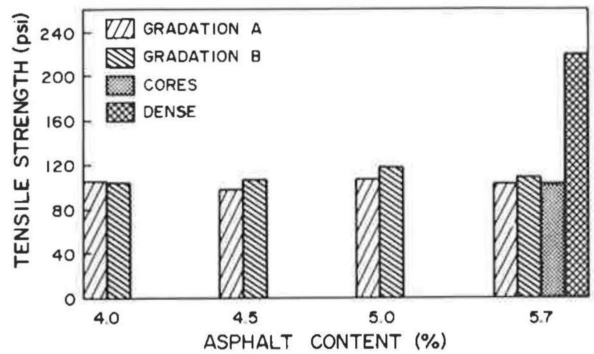

FIGURE 9 Average indirect tensile strength, obtained at 77°F, of specimens included in the study.

Unlike the other engineering properties investigated during this study, the average indirect tensile strength of the cores was similar to that of laboratory open-graded specimens prepared with similar material components. However, the variability of the S_T results for the cores was greater than that for the laboratory specimens because of the lower quality control that is usually experienced during field construction. The S_T results of the cores ranged between 85 and 130 psi with an average of 105 psi. In addition, the tensile strength of the dense-graded specimens was equal to 220 psi, which is high compared with an average of 105 psi for the open-graded specimens.

Coefficient of Permeability

The coefficient of permeability is an important parameter of open-graded asphalt mixtures. The coefficient of permeability determined in the laboratory may not match those expected in the field because of construction practices and poor quality control, operational problems, and maintenance activities. Tack coats, larger fine aggregate fraction than specified, and dust and patching are some of the problems that may be encountered in the field.

In this study, the coefficient of permeability (K) of laboratory-prepared specimens ranged between 321 and 621 ft/day with an average of 467 ft/day. Note that these values are much larger than the permeability value indicated in the design, which was 20 ft/day. The statistical results indicate that asphalt content, aggregate gradation, and their interaction largely affected the results. The average K-value of Gradation A was equal to 482 ft/day compared with 452 ft/day for Gradation B.

Asphalt content had a significant effect on the K results. Increasing the asphalt content reduced the permeability of the open-graded mixture (Figure 10). This effect can be explained by the fact that the excess asphalt is replacing the air voids and clogging some of the interconnected void channels.

No cores were tested for permeability by the authors. Data are available in the ADOT and the design company records about permeability of cores; however, these data were not available to the authors.

FIVE-MONTH FIELD EXPERIENCE

As of October 1986 (5 months after construction) the average rutting of the open-graded ADOT experimental section was about 0.2 in. in the wheel tracks and as high as 0.5 in. at some spots. The average rutting of the control (dense) section was 0.04 in. No cracking was observed in either the open-graded or the control sections. Also, minor roughness was noticed in the open-graded section. Noticeable improvement in drainage, which reduced glare and increased stripe visibility during rainfall, was observed in the open-graded section.

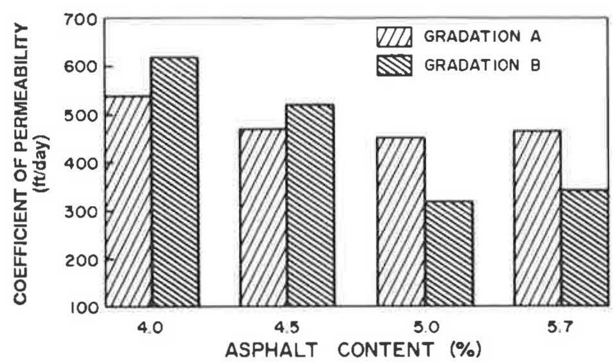

FIGURE 10 Effect of aggregate gradation and asphalt content on the coefficient of permeability of laboratory-prepared hot-mixed, open-graded asphalt mixture (note: 1 ft/day = 0.5 in./hr).

DETERMINATION OF LAYER THICKNESS EQUIVALENCY

The results of laboratory evaluation of the resilient modulus and tensile strength indicated a large difference between the stiffness of the open-graded asphalt mixture and that of the dense-graded mixture. The lower stiffness of the open-graded material means that higher stresses are transmitted to underlying layers. As a result, a greater structural thickness is required than for dense-graded pavements.

The experimental porous pavement section was designed using the AASHTO method for flexible pavements that requires a knowledge of the structural layer coefficient (a_1) of the hot-mixed, open-graded asphalt mixture. The a_1-value was selected between 0.4 and 0.44. This selection was based on recommendations made by Hicks et al. (*12*) for the cold-mixed, open-graded emulsion mixtures that were used under traffic and environmental conditions different from the ADOT project conditions.

In this study, the equal mechanistic response approach was used to evaluate the layer thickness equivalency (LTE) factor of the open-graded asphalt mixture (*13*), which can be further used to determine the structural layer coefficient, a_1. The maximum vertical deformation on top of the subgrade under a dual 4,500-lb wheel load was selected as the response parameter. Little and Epps (*13*) indicated that the vertical deformation on top of the subgrade is directly correlated with pavement performance, particularly in terms of riding quality and possibly rut depth. The multilayered elastic computer program ELSYM5 (*14*) was used in this study to model the pavement structure.

The LTE factor of the open-graded asphalt mixture was evaluated for the pavement structure shown in Figure 11. The structural cross section represents the porous pavement experimental test section. The maximum compressive vertical deformation on top of the subgrade was determined at different surface layer thicknesses of the open-graded and the dense-graded asphalt mixtures. The deformations were plotted versus the corresponding surface layer thicknesses of each mixture. Consequently, the thickness of the open-graded layer (H_1) and that of the dense-graded layer (H_2) required to obtain equal deformation on top of the subgrade were determined from these plots. The ratio of H_1 to H_2 was then computed. This ratio is equal to the LTE factor (*13*).

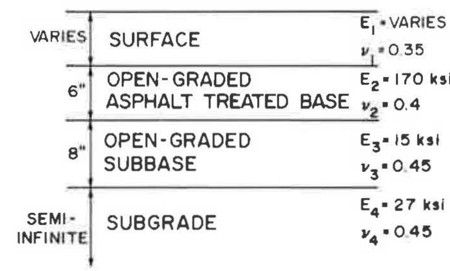

FIGURE 11 Pavement structure used in LTE computations.

The instantaneous resilient moduli of both hot-mixed, open-graded and dense-graded asphalt mixtures evaluated in the laboratory at temperatures of 41°F and 77°F were used to characterize the surface layers of the two pavement structures using the ELSYM5 computer program. The moduli of the base and subbase were estimated from the design charts produced by Van Til et al. (*15*). The modulus of the subgrade was backcalculated from the falling weight deflectometer data measured on the experimental and the control pavement structures. Typical Poisson's ratios of various materials were assumed. The moduli and Poisson's ratios of various layers are shown in Figure 11. It should be noted that the subgrade modulus is relatively large, which might be due to the inappropriate assumption of semi-infinite subgrade.

The computed LTE factors are given in Table 5. The overall LTE average of the cores was 1.7, which is much greater than 1.1 that was used during the design of the porous pavement. This indicates that the open-graded surface layer should have been 9.3 in. instead of the 6 in. used in the field. The overall LTE average of the laboratory mixture was 1.45, which requires a surface layer thickness of 7.9 in.

TABLE 5 AVERAGE LTE-VALUES OF OPEN-GRADED ASPHALT MIXTURE

Temperature (°F)	Based on Core Testing	Based on Laboratory-Prepared Specimens	Average
41	1.41	1.33	1.37
77	1.98	1.57	1.78
Average	1.70	1.45	1.57

Finally, the increase in temperature resulted in higher LTE factors. This was expected because the increase in temperature causes a large decrease in the modulus of the open-graded mix compared with the relatively small decrease in the modulus of the dense mix. This causes a large increase in stresses, strains, and deformations in the lower pavement layers under the open-graded surface layer.

It should be noted that the thickness requirements computed in this study were based on subgrade strain. However, rutting in the field may have resulted from several factors, and subgrade strain is only one of those factors.

CONCLUSIONS AND RECOMMENDATIONS

1. Core densities were much lower than the densities of laboratory specimens, which affected most of the properties evaluated in this study.
2. The average Marshall stability of the cores was equal to 270 lb compared with 575 lb for laboratory specimens. The Marshall flow decreased from 16 for the cores to 10 for the laboratory specimens.
3. The average instantaneous resilient modulus of the cores was equal to 745 and 185 ksi at test temperatures of 41°F and 77°F, respectively, compared with 830 and 245 ksi for the laboratory specimens prepared with similar ingredients. Similar trends were observed for the total resilient modulus.
4. Asphalt content and aggregate gradation had large effects on density, air voids, Marshall stability, resilient moduli, and coefficient of permeability. However, tensile strength was not greatly affected by these two factors.
5. The open-graded mixture was extremely sensitive to test temperature. The instantaneous and total resilient moduli significantly decreased with an increase in temperature. The open-graded mixture was unstable at high temperatures, which prevented the determination of the resilient modulus characteristics of the mixture at 104°F as recommended by ASTM D 4123.
6. The coefficient of permeability of all specimens evaluated ranged between 321 and 621 ft/day. It was significantly affected by asphalt content and aggregate gradation.
7. A large difference was observed between the properties of the dense-graded mixture and those of field and laboratory open-graded mixtures. Higher density, stability, tensile strength, and resilient moduli and lower air voids were obtained for the dense mixture.
8. The layer thickness equivalency factors indicate that 1.7 in. of the open-graded mixture is required to replace 1 in. of the dense mixture in the field. The factor could be reduced to 1.45 if construction practices were able to reproduce the material properties found in the laboratory.

On the basis of these laboratory measurements and theoretical analyses as well as observations made in the field at the site of the experimental porous pavement, the following conclusions are drawn:

1. The open-graded asphalt mixture appears to be capable of draining storm waters as indicated by its high coefficients of permeability.
2. The open-graded asphalt mixture is more susceptible to temperature than is dense-graded asphalt concrete. At high temperatures, the stability of asphalt mixtures is dependent mostly on aggregate interlock rather than on the binding ability of the asphalt binder because the increase in temperature reduces the viscosity of the binder. Aggregate interlock is far less available in open-graded asphalt mixtures than in dense mixtures because of the fewer points of contact between aggregate particles. Therefore the use of open-graded asphalt mixtures in pavement structural layers in hot climates, such as in Arizona desert areas, should be discouraged for roads that carry a high traffic volume and a high percentage of trucks.
3. The open-graded surface layer of the experimental porous pavement quite likely contributed to the rutting observed during construction especially given the high temperature of the pavement. This contribution was partly caused by the inability to obtain the same level of compaction in the field as that obtained in the laboratory. Furthermore, shear deformation may have occurred in the open-graded surface layer because of its low stability. On the other hand, the open-graded base and subbase layers of the porous pavement may have significantly contributed to the problem.
4. Because of the low resilient modulus of the open-graded asphalt mixture especially at high temperatures, its load-spreading capability is lower than that of the dense asphalt mixture. To decrease the stresses and deformations in the subgrade, thicker open-graded surface layers are required. However, the thicker open-graded surface layer may not guarantee lower susceptibility to rutting because of the potential of compaction and shear-induced strain within the surface layer itself.
5. If a permeable mix is a necessity, a mixture consisting of aggregates conforming to Gradation B and a 4.0 percent asphalt content by weight of aggregate is recommended for use under light or medium traffic in favorable climates.

ACKNOWLEDGMENTS

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Modulus of Asphalt Mixtures—An Unresolved Dilemma

MICHAEL S. MAMLOUK AND RAMSIS T. SAROFIM

Several moduli have been used to characterize asphalt mixtures. However, little information is available about which modulus is most appropriate to use in a given situation. This paper provides rational understanding, based on principles of strength of materials, of the moduli commonly used for structural evaluation of asphalt mixtures. Young's, shear, bulk, complex, dynamic, double-punch, resilient, and Shell nomograph moduli are discussed. The assumptions and limitations of these moduli are evaluated. It is concluded that the resilient modulus is more appropriate for use in multilayer elastic programs than are other moduli because it represents the elastic stiffness of the material after many load repetitions. Also, moduli predicted from the dynamic nondestructive testing of pavements are most representative of the in situ resilient moduli of the materials. If other moduli are used in the theoretical analysis of pavements, caution should be exercised to avoid inconsistency of assumptions.

The technology for characterizing asphaltic materials has been developed largely on an empirical basis, which has resulted in index-type values. Empirical methods of characterization are useful for comparison of materials under specific conditions. However, empirical correlations are valid only for conditions similar to those under which the values were originally developed. Further, empirical methods of characterization do not provide the information that is needed for fundamental or theory-based structural analysis of pavements. Given the continuing increase in truck weights, tire pressures, and traffic volumes and the fast deterioration of the nation's highway system, a more rational philosophy of asphalt concrete characterization is needed for optimal pavement design.

Several test procedures and theories for determining the moduli of asphalt mixtures are available in the literature. Among these moduli are Young's modulus, resilient modulus, complex modulus, dynamic modulus, and modulus obtained from the Shell nomograph. According to Brown and Barksdale (*1*), the term "modulus" has been "loosely" used in the literature on pavement, and the term "elastic stiffness" has been recommended. In addition, different test conditions, such as load frequency, magnitude, and duration and equipment type, may result in different values for the same modulus and for the same material. Little information, however, is available about which test method is most appropriate to use under specific conditions. Even though reasonably accurate values of test results may be obtained in the laboratory under carefully controlled conditions, extreme care must be exercised in understanding the basic nature of the test and in incorporating the test data into mechanistic design procedures. Unless such understanding is developed, arbitrary and inconsistent moduli may be selected by different researchers, and the dilemma will remain: "which modulus is suitable to use under such conditions?"

The objective of this paper is not to list various moduli used to characterize asphalt mixtures but to provide a rational understanding of the moduli that are commonly used for structural evaluation of asphalt mixtures. A number of moduli are defined and evaluated, and the relations between these moduli and material linearity, elasticity, and viscoelasticity as well as recent mechanistic methods of pavement evaluation are discussed. The paper is intended to clarify the definition and use of terms and procedures.

STRESS-STRAIN RELATIONSHIP

A material is said to be perfectly elastic if strains completely appear and disappear immediately on application and removal of stresses. This definition does not imply a linear stress-strain relation. Effects of temperature are usually neglected in the theory of linear elasticity. In the classic theory of linear elasticity, a material can be fully characterized by two constants. These are usually Young's modulus of elasticity and Poisson's ratio.

On the other hand, viscoelastic materials exhibit a combination of elastic and viscous (time-dependent) responses. As a minimum, a modulus, Poisson's ratio, and a time-dependent term are required to describe their response. Viscoelastic materials are usually highly temperature dependent. Time-dependent responses such as creep and relaxation are typical of viscoelastic materials.

The responses of asphaltic mixes are time and temperature dependent and consequently they should be analyzed as such. Under some special conditions, however, this behavior can be approximately considered as elastic, and reasonable solutions of stresses and deformations can be obtained. It is, therefore, justifiable to employ the theory of elasticity and indicate the usefulness as well as the limitations of its application. In other cases, the theory of viscoelasticity can also be used to explain the time-dependent response of material. The choice between the two theories depends on the intended use of the results and the level of accuracy required in the analysis.

A typical stress-strain diagram for an asphalt concrete specimen monotonically (statically) loaded and unloaded during an unconfined compression test is shown in Figure 1. A small concave upturn of the curve at the beginning of loading is usually encountered because of voids (*2, 3*). Figure 1 shows

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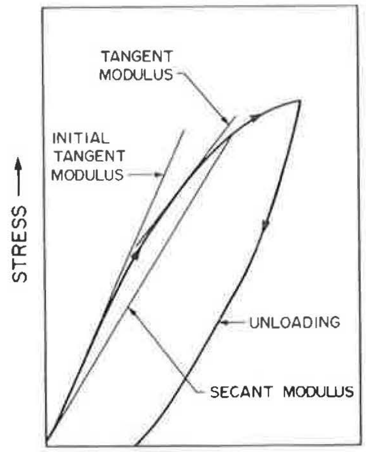

FIGURE 1 Typical unconfined compression stress-strain curve for asphalt concrete.

that asphalt concrete is a nonlinear inelastic material. The stress-strain relation depends on the rate of load application and is largely temperature dependent.

Although the monotonic (static) uniaxial compression test is commonly used to characterize soils and portland cement concretes, it has been used for asphalt concretes on a very limited basis. Likewise, the monotonic triaxial load test has not gained wide acceptance among researchers in the asphalt concrete area (*4*). Some of the reasons for not using the monotonic unconfined compression or triaxial load tests follow:

1. The test results are largely dependent on the rate of load application; therefore rational data interpretation is not simple;
2. Preparing the required large-sized specimens and running the test are difficult;
3. Controlling the test temperature is difficult;
4. The nonrepetitive nature of the monotonic load test does not simulate traffic load; and
5. Empirical tests such as Marshall and Hveem tests predominate.

Repetitive load–type laboratory tests have recently been developed in an attempt to simulate traffic loads. Among these tests are the sinusoidal unconfined compression tests, the triaxial resilient modulus test, and the diametral resilient modulus test. Several moduli that are not rationally understood by many highway engineers are obtained from these tests. Some of these moduli are used interchangeably without distinction and without an understanding of the relation between them and field response of material under actual traffic loading. Some of these moduli are described in the following sections.

YOUNG'S, SHEAR, AND BULK MODULI

If a material is perfectly linear elastic, Young's modulus (E) is defined as the slope of the straight line representing the stress-strain relation, and Poisson's ratio (ν) is defined as the absolute value of the lateral strain divided by the axial strain when only an axial stress is applied to a cylindrical specimen. Other elastic moduli can also be defined: shear modulus (G), bulk modulus (K), or the Lame constants (λ and μ). The interrelationships between these constants are (*5*)

$$G = \frac{E}{2(1 + \nu)} \tag{1}$$

$$K = \frac{E}{3(1 - 2\nu)} \tag{2}$$

$$\lambda = \frac{E\nu}{(1 + \nu)(1 - 2\nu)} \tag{3}$$

$$\mu = G \tag{4}$$

It should be recognized that these elastic parameters have precise meanings in the linear theory of elasticity. However, in the literature on pavement, other moduli that are descriptive of laboratory and field data have been defined.

Because the stress-strain relation of asphalt concrete is not linear (Figure 1), the term Young's modulus of elasticity can strictly be applied only to the straight part of the stress-strain curve or, when no straight portion is present, to the tangent to the curve at the origin. This is the initial tangent modulus, but it is of little practical importance. It is also possible to find a tangent modulus at any point on the stress-strain curve, but this modulus applies only to quite small changes in load above or below the load at which the tangent modulus is considered. Another modulus that can be defined is the secant modulus, which is the slope of the line from the origin to any point on the stress-strain curve. The secant modulus represents an average modulus between zero load and the load at which the modulus is determined. It should be noted that these moduli are applicable to static (or quasi-static) loading conditions as opposed to dynamic (or repetitive) traffic loading conditions.

COMPLEX AND DYNAMIC MODULI (ASTM D 3497)

When a viscoelastic cylindrical specimen is subjected to a sinusoidal axial load, the strain lags the stress as shown in Figure 2. The complex modulus is the complex quantity (i.e., real and imaginary number) that relates axial stress to axial strain in a cylindrical specimen subjected to sinusoidal loading. The imaginary part of this quantity represents the internal (material) damping property of the material, and the real part characterizes its stiffness (Young's modulus). Considerable

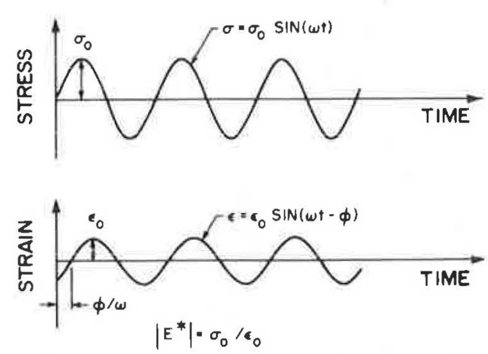

FIGURE 2 Typical plot of stress and strain versus time during the complex (dynamic) modulus test.

care is needed to carry out the requisite dynamic tests because of the hazards of resonance and inertial resistance in the sample and test equipment that may swamp the true response characteristics of the material, particularly at high frequencies. In classic elastodynamics (*5*), the stiffness of materials is assumed to be frequency invariant, but frequency-dependent material properties can be accommodated within such analyses by solving for each component frequency separately. Frequency-dependent material damping can be treated similarly. The complex modulus is usually denoted by E^*:

$$E^* = \frac{\sigma_0 \text{ Sin } \omega t}{\varepsilon_0 \text{ Sin } (\omega t - \phi)} \tag{5}$$

$$= E(1 + 2i\beta) \tag{6}$$

where

ω = angular frequency of vibration (radian/sec),
ϕ = phase difference between the stress and the strain,
i = unit imaginary number, and
β = damping ratio.

The transformation from circular functions to complex numbers is accomplished by means of the Euler transformation, whence

$$\beta = \frac{1}{2} \text{Tan } \phi \tag{7}$$

and

$$E = \frac{\sigma_0}{\varepsilon_0} \text{Cos } \phi \tag{8}$$

The absolute value (magnitude) of the complex modulus, $|E^*|$, is commonly referred to as the dynamic modulus. With respect to the parameters described,

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \tag{9}$$

Papazian (*6*) has measured a substantial frequency effect on the complex modulus of asphaltic materials. If this is true, and not an artifact of the test procedure, these data should be incorporated into elastodynamic analyses of pavements. Figure 3 shows the apparatus used to determine the complex and dynamic moduli of asphalt concrete; it is essentially an electrohydraulic machine (capable of applying sinusoidal loading) and strain-measuring devices.

A different procedure for obtaining the dynamic modulus, which is known as the double-punch modulus, was used by Al-Juraiban and Jimenez (*7*). In this test, a 4-in.-diameter specimen is subjected to a sinusoidal load through two 1-in. cylindrical punches along the specimen axis. This load creates tensile stresses in the radial direction. The amplitude of the radial expansion is obtained at midheight of the specimen. The double-punch modulus is obtained by dividing the stress amplitude by the strain amplitude assuming linear elastic response.

FIGURE 3 Complex (dynamic) modulus apparatus.

In general, the dynamic modulus is insuffcent to explain true material response because it ignores the loading frequency and the phase lag between deformation and load. The complex modulus, however, can yield valuable information on fundamental material properties such as stiffness and damping provided that it is properly interpreted. From the theoretical point of view, neither the dynamic modulus nor the complex modulus is suitable for use in elastic multilayer computer programs used in pavement analysis because they do not represent elastic parameters. The complex modulus can be used in viscoelastic pavement models in which the time-dependent response is considered.

RESILIENT MODULUS

Pavement materials are subjected to repetitive loading that tends to "shake down" their response through strain hardening. Thus, after several repetitions of loading, subsequent deformations become predominantly recoverable and, often, substantially linear. The resilient modulus is the ratio of repeated stress to corresponding recoverable or resilient strain during such loading; that is, it is the elastic stiffness of the material after many load repetitions have been applied. Therefore the resilient modulus of asphalt mixtures is more appropriate for use in multilayer elastic programs than are other moduli. Also, the moduli predicted from the dynamic nondestructive testing of pavements are more representative of the in situ resilient moduli of the materials. The resilient modulus can be determined in the laboratory using the axial, triaxial, or diametral method.

Axial or Triaxial Method

The axial or triaxial resilient modulus test on asphaltic materials is performed by applying a pulsating axial load to a cylindrical specimen and recording the recoverable axial deformation while keeping the confining pressure zero or constant (*8*). The resilient modulus is the ratio of the repeated axial stress to the recoverable axial strain after many load repetitions. Figure 4 shows the typical relation between stress and strain in an asphalt concrete specimen when repeated load is applied. The figure shows that the stress-strain relation is essentially linear after several load applicatons. The resilient

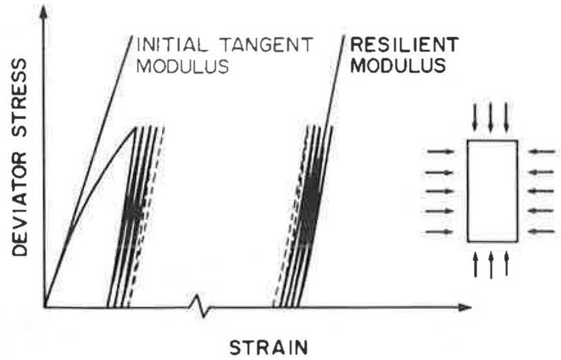

FIGURE 4 Typical stress-strain response during triaxial resilient modulus test.

modulus is, therefore, the slope of the stress-strain curve after many load repetitions are applied. This indicates that the resilient modulus is the "current" modulus of the material, given the repetitive nature of the traffic load. The resilient modulus of asphalt concrete can be used in the elastic layer analysis of pavements with a large degree of accuracy. A typical relation among load, deformation, and time is shown in Figure 5. Note that two resilient moduli can be obtained, instantaneous and total, depending on which response is considered. The instantaneous resilient modulus, however, is more consistent with elastic theory than is the total resilient modulus. Currently, there is no standard test procedure for the axial or triaxial test method. A standard test procedure is, however, available for determining the resilient modulus of subgrade soils (AASHTO T-274).

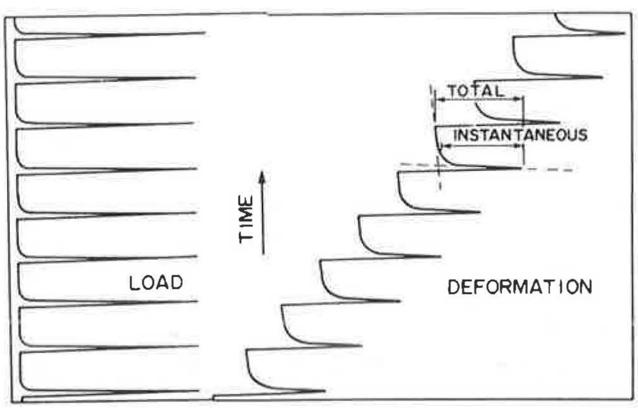

FIGURE 5 Typical plot of load and deformation versus time during resilient modulus test.

Diametral Method

In the diametral resilient modulus test (ASTM D 4123), a pulsating load is applied along the vertical diameter of a Marshall-sized specimen of asphalt concrete, and the horizontal or the vertical deformation, or both, are recorded (Figure 6). The stress distribution in the specimen when the load is applied is shown in Figure 7 assuming plane stress condition (*9*). It should be noted that the plane stress condition is valid only when the thickness of the specimen is quite small in comparison with its diameter (e.g., disk). Therefore the validity of this assumption is questionable for a Marshall-sized specimen.

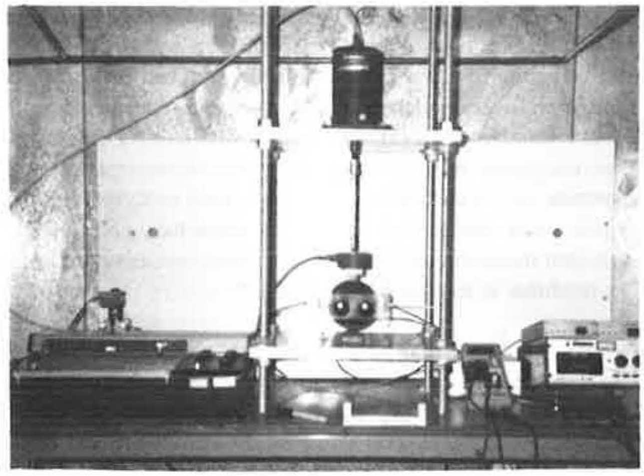

FIGURE 6 Diametral resilient modulus apparatus.

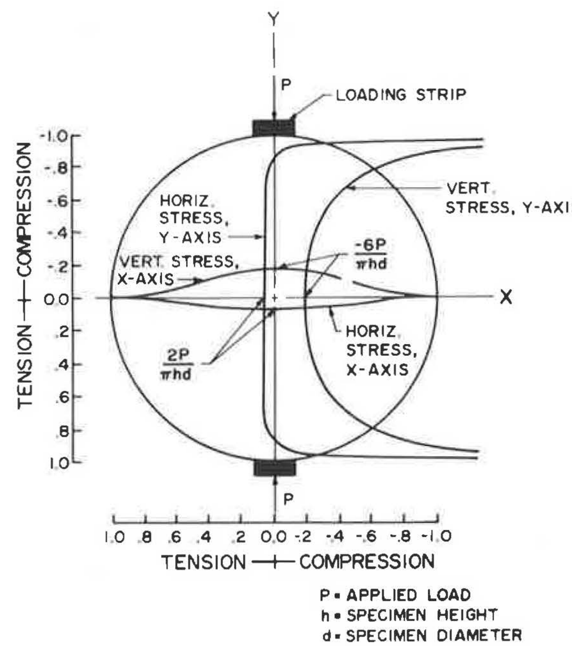

FIGURE 7 Stress distribution along the principal axes of specimen during diametral resilient modulus test.

The relation among load, vertical deformation, horizontal deformation, and time is similar to that of the axial resilient modulus test (Figure 5). Assuming linear elastic material response, the resilient modulus can be computed as reported in the ASTM D 4123 procedure. As is the case with the triaxial method, both instantaneous and total moduli can be determined.

Comparison of the triaxial resilient modulus method with the diametral method indicates that the triaxial method is more representative of field conditions because of the triaxial nature of the load. This is particularly important at high temperatures for two reasons. The first reason is that the asphalt gets soft at high temperatures and the modulus would be largely dependent on the amount of confining pressure. Thus, if the correct confining pressure is used, a better approximation of the in situ modulus of the material is obtained. The second reason is that a large amount of tensile stress is developed in the specimen as shown in Figure 7. Consequently, when the asphalt binder gets soft at high temperatures, artificially low moduli are obtained that do not represent the true stress-strain relation of the asphalt-aggregate mixture in the field; instead, they represent responses that are largely affected by the tensile strength of the binder.

SHELL NOMOGRAPH MODULUS

The modulus of an asphaltic mix obtained by the Shell Oil Company (Van der Poel) nomograph (*10*) is based on more than 20 years of laboratory work and correlation analysis. The method uses a nomographic solution to obtain the modulus of the asphalt and equations to convert this value to the modulus or stiffness of the asphalt mix. It is assumed that the asphalt mix is a viscoelastic material, properties of which are dependent on the duration (or frequency), of loading and on the temperature at which the load is applied. Thus, if the load time (or frequency), temperature, properties of asphalt cement, and mix ingredient are known, the stiffness modulus can be determined from the nomograph.

During the original development of the Shell nomograph the "two-point" bending apparatus was used to determine the modulus of asphalt concrete (Figure 8). In this test a trapezoidal specimen fixed in a cantilever form is subjected to sinusoidal loads at its free end. A continuous plot of load and deformation is obtained, and the stiffness modulus is defined as the ratio of the amplitudes of stress to strain. Therefore the stiffness modulus obtained from the Shell (Van der Poel) nomograph is a prediction of the dynamic modulus discussed previously, with the exception that a trapezoidal specimen was used instead of a cylindrical specimen.

The Van der Poel nomograph concept was further extended by Bonnaure et al. (*11*) to predict the phase lag in addition to the stiffness modulus of the asphalt mix. By using the stiffness modulus and the phase lag, the complex modulus can be obtained as discussed previously.

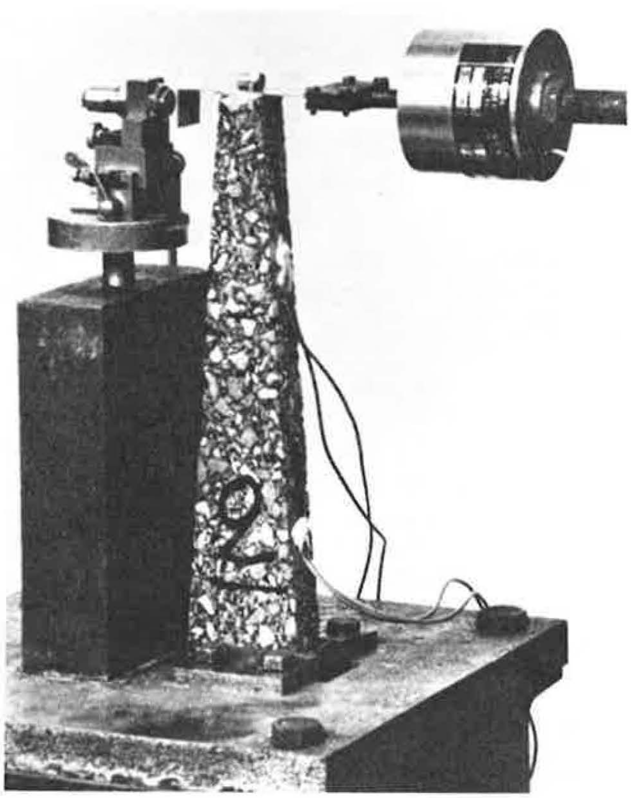

FIGURE 8 Two-point bending apparatus used in Shell method (*10*).

SUMMARY AND CONCLUSIONS

The moduli commonly used to characterize asphaltic mixtures have been discussed in terms of the basic theory of strength of materials. A rational understanding of stiffness parameters—Young's, shear, bulk, complex, dynamic, double-punch, resilient, and Shell nomograph moduli—has been presented. The assumptions used to define these moduli, such as material linearity, elasticity, and viscoelasticity, have been specified.

The appropriate modulus to be used is dependent on assumptions of material response, method of analysis, and required accuracy. In some cases, asphaltic materials can be assumed to be linear elastic especially in multilayer elastic analyses in which repeated loads are applied to pavements. In such cases the triaxial or diametral resilient modulus can be used to characterize the material. If more accuracy is needed, the nonlinearity of the material (or the change in the modulus with change in the state of stress) can be considered. In such cases, the triaxial resilient modulus is more appropriate to use than is the diametral resilient modulus. Also, the triaxial modulus is useful at high temperatures at which the viscosity of the binder gets small and the effect of confining pressure becomes significant. In addition, the moduli predicted from the dynamic nondestructive testing of pavements are more representative of the in situ resilient moduli of the materials.

The dynamic modulus is a frequency-dependent parameter and ignores the phase lag between load and deformation. However, the complex modulus considers both stiffness and phase lag. From the theoretical point of view, neither the dynamic modulus nor the complex modulus is suitable for use in elastic multilayer computer programs because of the inconsistency of the assumptions. The double-punch modulus or the modulus predicted from the Shell nomograph is conceptually similar to the dynamic modulus except that a different load configuration or specimen shape is used.

In conclusion, for the purpose of comparing different asphalt mixtures, any modulus (dynamic, resilient, etc.) can be used as long as the same modulus is used for all mixtures. If the modulus is to be used in a specific empirical formula, the test procedure used during the original development of the formula should be followed. On the other hand, if the modulus is to be used in a specific empirical formula, the test procedure used during the original development of the formula should be followed. On the other hand, if the modulus is intended to be used in a theoretical or mechanistically based analysis, any test procedure can be followed as long as an interpretation of the test results that is consistent with the assumptions of the analysis is used. When proper material characterization is used, reliable performance models, which are important for both pavement design and pavement management, can be developed.

Further research is needed to provide typical laboratory results of different moduli of asphalt mixtures. It would also be useful if these different moduli were input into the same multilayer computer programs and the relative outcomes (stresses, strains, deflections, etc.) were compared.

ACKNOWLEDGMENTS

The authors wish to thank the Center for Advanced Research in Transportation and the Department of Civil Engineering at Arizona State University for making their facilities available for this study. Appreciation is extended to Trevor G. Davies of the University of Glasgow for his advice and cooperation.

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Effect of Inclusion of Oil Shale Ash on Behavior of Asphalt Concrete

TAISIR S. KHEDAYWI

Because of a shortage of asphaltic material and the increasing price of asphalt, researchers are investigating different methods of using available materials, such as waste materials, and substitute binders or extenders in roadway construction. The Jordanian government is concerned with the exploration of every possible source of energy, especially oil shale. Oil shale ash is one of the few materials that is expected to be in vast oversupply in the future in Jordan. The purpose of this study is to investigate the effect of oil shale ash on properties of bituminous concrete. To achieve this objective, oil shale ash was added to bituminous mixtures in different ratios. Marshall stability, Marshall immersion, and resilient modulus tests were performed on the mixtures. Results indicate that oil shale ash produces paving mixtures that have higher Marshall stability and resilient modulus than do paving mixtures without ash. The experiments also indicated that oil shale ash reduces the potential of stripping in bituminous mixtures.

In view of the growing energy problem in Jordan, which is becoming a major threat to its national economy, the Jordanian government is concerned with the exploration and development of every possible domestic source of energy, especially oil shale. The oil shale reserves of the El-Lajjun area are estimated to be more than one billion metric tons. The El-Lajjun area is located in west central Jordan midway between Qatrana and Karak (see Figure 1 for location). Oil shale ash is one of the few materials that is expected to be in vast oversupply in the future.

To date, considerable use has been made of ash as a substitute for or supplement to asphalt cement. One of the successful uses of ash has been as a mineral filler in bituminous concrete mixes. Studies done on mineral fillers indicate that it is possible for a mineral filler such as fly ash to act as an inexpensive asphalt substitute in bituminous mixtures.

Because changes in mix composition usually influence mix properties, work is needed to determine how the addition of oil shale ash to bituminous paving mixes would alter Marshall stability, flow, unit weight, voids in mineral aggregate (VMA), air voids, and resilient modulus of a mix. Therefore the primary objective of this study was to investigate the feasibility of using oil shale ash as a substitute for part of the asphalt in bituminous paving mixtures in order to achieve economic advantage and good performance.

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The effects of mineral fillers on the behavior of asphalt concrete have been studied by many researchers for many years. Richardson (*1*) was one of the earliest to report on mineral fillers. He hypothesized that mineral filler not only filled voids but also caused a physicochemical interaction between the asphalt and the filler. From extensive studies on mineral fillers, Puzinauskas (*2*) also concluded that mineral fillers play dual roles in asphalt mixtures: (a) they are part of the mineral aggregate that fills the interstices and provides contact points between larger aggregate particles and (b) when mixed with asphalt they form a high-consistency binder or matrix that cements larger aggregate particles together.

Anderson and Goetz (*3*) found that the size and mineralogy of the filler affected rheological behavior. The temperature susceptibility of the asphalt was also increased with the addition of the mineral fillers. Baghouse fines have also been used as a mineral filler. The California Department of Transportation (*4*) was one of the first to report on baghouse fines in 1976. Their study indicated that using a maximum of 2 percent baghouse dust had very little effect on an asphaltic mixture, provided that the maximum amount passing the No. 200 sieve, including the added baghouse dust, did not exceed the limits established in their 1975 standard specification. Ishai and Craus (*5*) studied the effects of physicochemical properties of fillers on bituminous mixes. They found that, for most types of fillers tested, all physicochemical effects result in greater geometric irregularity correlated with greater intensity of absorption. They concluded that this combined effect causes a strengthening of the filler-bitumen bonds and a relative increase in the amount of fixed bitumen; the result is mastics with higher consistency or mixtures with higher strength. The West Virginia Department of Highways (*6*) conducted a study on baghouse fines, from different sources, that had different particle size distribution and physical properties. Their study showed that (a) baghouse fines when used in proper quantity can improve the stability and cohesion of the paving mix, (b) the finer sizes of dust act as an asphalt extender, and (c) changes in either the size or the amount of the dust added to the paving mix have about the same effect as changing the asphalt content.

EXPERIMENTAL INVESTIGATIONS

The laboratory investigation undertaken in this study was intended to quantitatively evaluate the effect of ash on the behavior of bituminous concrete mixes. The tests used were

1. Marshall design method,

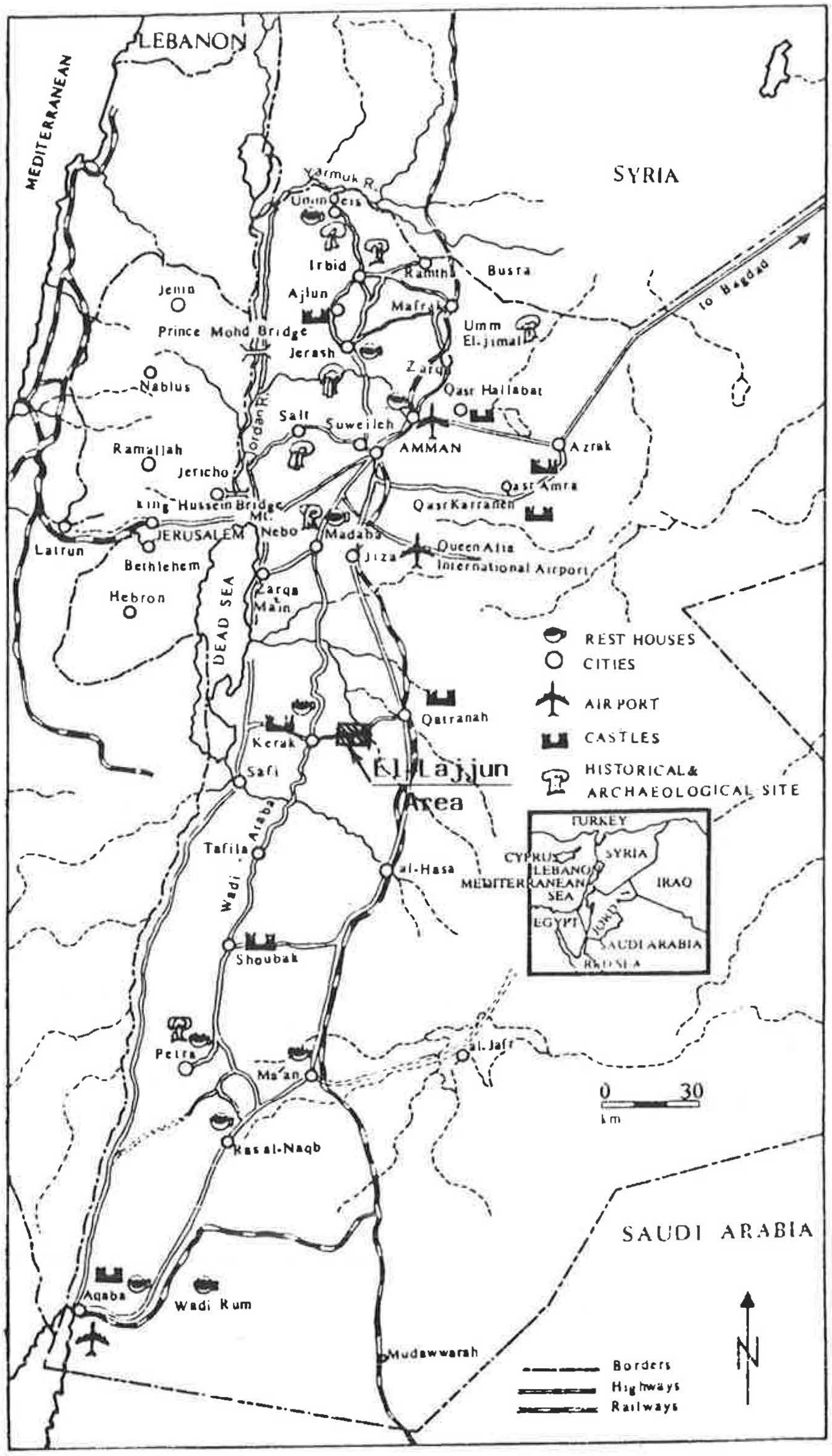

FIGURE 1 General map of Jordan.

2. Water sensitivity of bituminous mixtures, and
3. Resilient modulus test.

Material Used

Asphalt

One penetration grade asphalt cement (80-100) was used in this study. This asphalt was obtained from Jordan Refinery, Zerka, Jordan. This grade was chosen because it is widely used in pavement construction. Table 1 gives a summary of the results of some tests run on the asphalt.

Aggregate

One type of limestone aggregate was used in this study. This aggregate is the most commonly used in pavement construction

TABLE 1 PROPERTIES OF ASPHALT CEMENT

Property	ASTM Test Designation	Test Result
Specific gravity at 77°F	D 70	1.010
Penetration (0.1 mm) at 77°F, 100 g, 5 sec	D 5	81
Softening point (°F), ring and ball	D 36	122
Ductility (cm) at 77°F	D 113	130
Kinematic viscosity (cSt) at 275°F	D 2170	310

NOTE: 1 mm = 0.0394 in.

in Jordan. It was obtained from local quarries in the north of Jordan. The gradation used in this study conformed to the Jordanian Ministry of Public Works specification. This gradation is given in Table 2. Table 3 gives a summary of the properties of the aggregate.

Oil Shale Ash

The ash used in this study was obtained by burning oil shale at 600°C. The oil shale was supplied by the Department of Natural Resources of Jordan. The specific gravity of this ash is 2.47 (Table 3). Table 4 gives the results of a hydrometer analysis of a selected ash sample.

TABLE 2 AGGREGATE GRADATION

Sieve Size	Percentage Passing	Specification[a]
3/4 in.	100	100
1/2 in.	96	90–100
3/8 in.	85	75–90
No. 4	60	45–70
No. 8	45	33–53
No. 30	25	15–33
No. 50	14	10–20
No. 100	6.5	4–16
No. 200	5	2–9

NOTE: 1 in. = 2.54 cm.
[a]Jordanian specification limits.

TABLE 3 PROPERTIES OF AGGREGATE

Type of Aggregate	ASTM Test Designation	Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
Limestone coarse aggregate	C 127	2.40	2.67	3.20
Limestone fine aggregate	C 128	2.43	2.70	3.70
Limestone mineral filler	C 128	2.53	2.78	4.20
Oil shale ash	C 188	–[a]	2.47	–[a]

[a]Data not applicable.

Marshall Test

A series of tests was performed on the bituminous mixtures using the Marshall method to find the optimum binder content for the mix. The variables involved were

TABLE 4 HYDROMETER ANALYSIS OF SELECTED ASH SAMPLE

Size (microns)	Percentage Finer
75	100
56	97
21	90
20	80
8	76
3	27
1	10

NOTE: 1 μm = 0.0394×10^{-3} in.

1. Percentage of ash by volume of binder (0, 5, 10, 15, and 20 percent) and
2. Binder content equivalent by weight of aggregate (5, 6, 7, and 8 percent).

The term "binder content equivalent" means that for a given experiment the volume of asphalt plus the volume of ash was kept constant.

Preparation of Ash-Asphalt Binders

The following steps were followed in preparing ash-asphalt binders:

1. Ash was heated in a stainless steel beaker and maintained at a temperature between 145°C and 150°C (293°F and 302°F).
2. Asphalt cement was heated in an oven at a temperature of about 145°C (293°F).
3. The stainless steel beaker used for mixing was cleaned and kept in the oven at a temperature of about 145°C.
4. The required amount of asphalt was weighed into the beaker; then, the amount of ash required to yield the desired ash-to-asphalt ratio was weighed.
5. The beaker was placed on a hot plate to maintain a constant mixing temperature. The mix temperature varied between 140°C and 145°C (284°F and 293°F).
6. The laboratory mixer used was then placed so that the propeller was about 1.5 cm above the bottom of the beaker.
7. The mixer was started, and mixing was continued for 5 min at 1,600 RPM. During mixing it was necessary to watch for any undesirable splashing, which might introduce small air bubbles into the mix. By changing the position of the mixer's propeller within the mixing beaker, it was possible to stop the splashing.
8. At the end of the mixing operation, the ash-asphalt binder was mixed with the heated aggregate to prepare ash-asphalt concrete specimens.

Preparation and Testing of Marshall Specimens

The procedure used in preparing and testing the bituminous concrete Marshall specimens was that outlined in the Asphalt Institute Manual Series (*7*, pp. 17–32). Table 5 gives the Marshall properties of mixtures without ash. An example of the calculation of asphalt and ash weights for one specimen follows: Assume that a given mix will be used. This mix had a binder content equivalent of 5.0 percent by total weight of

TABLE 5 MARSHALL PROPERTIES OF MIXTURES WITHOUT ASH

Asphalt Cement (%)	Marshall Stability (lb)	Flow (0.01 in.)	Unit Weight (lb/ft^3)	VMA (%)	Air Voids (%)
5	2216.6	13.4	133.2	16.9	8.7
6	1908.6	14.6	135.4	16.4	5.6
7	1541.4	17.4	136.4	16.7	3.7
8	1410.4	18.2	134.9	18.5	3.5

NOTE: 1 lb = 0.454 kg; 1 in. = 2.54 cm; 1 lb/ft^3 = 16.019 kg/m^3.

aggregate. Assume that an ash-asphalt binder with 10 percent ash (by volume of binder) is to be used instead of the original asphalt. This asphalt cement has a specific gravity of 1.01, and the ash has a specific gravity of 2.47. All mixes were prepared from a batch of which the weight of the aggregate was 1200.0 g. The weight of the asphalt cement (AC) for this mix without ash was calculated as follows:

$1200 \times 5/100 = 60.0$ g

Therefore

Volume of AC = $60.0/1.01 = 59.4$ cm^3

This means that the volume of asphalt plus the volume of ash should be 59.4 cm^3 for this mix.

The volume of ash for this mix was calculated as follows:

$59.4 \times 10/100 = 5.94$ cm^3

Volume of asphalt for this mix = $59.4 \times 90/100 = 53.46$ cm^3. Weight of ash for this mix = $5.94 \times 2.47 = 14.67$ g. Weight of asphalt for this mix = $53.46 \times 1.01 = 54.0$ g. Therefore weight of binder = $14.67 + 54.0 = 68.67$ g.

Table 6 gives a summary of the weights of asphalt and ash calculated using this procedure for each Marshall specimen. The freshly prepared ash-asphalt binder was added, in the desired proportion, to 1200 g of the hot aggregate, and the combination was mixed until all aggregate was covered completely with binder.

TABLE 6 WEIGHTS OF ASPHALT AND OIL SHALE ASH FOR MARSHALL SPECIMENS

BCE[a] (% by weight of aggregate)	Ash by Volume of Binder (%)				
	0	5	10	15	20
5	60.0[b]	57.0	54.0	51.0	48.0
	0.0[c]	7.3	14.7	22.0	29.3
6	72.0	68.4	64.8	61.2	57.6
	0.0	8.8	17.6	26.4	35.2
7	84.0	79.8	75.6	71.4	67.2
	0.0	10.3	20.5	30.8	41.1
8	96.0	91.2	86.5	81.6	76.8
	0.0	11.7	23.5	35.2	46.9

NOTE: Units are grams; 1 g = 2.2×10^{-3} lb; weight of aggregate for each specimen = 1200 g.
[a]BCE = binder content equivalent.
[b]Weight of asphalt.
[c]Weight of ash.

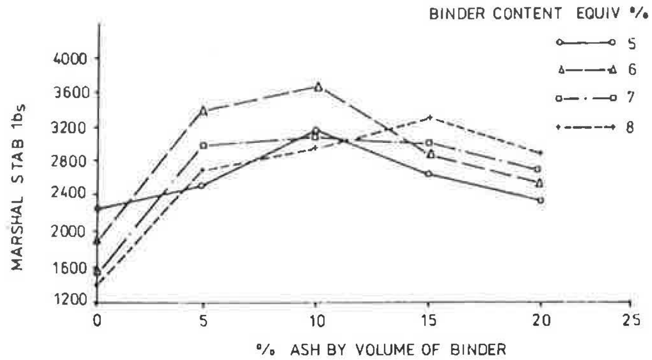

FIGURE 2 Effect of replacing asphalt with oil shale ash on Marshall stability of bituminous mix (1 lb = 0.454 kg); each point is an average of three values.

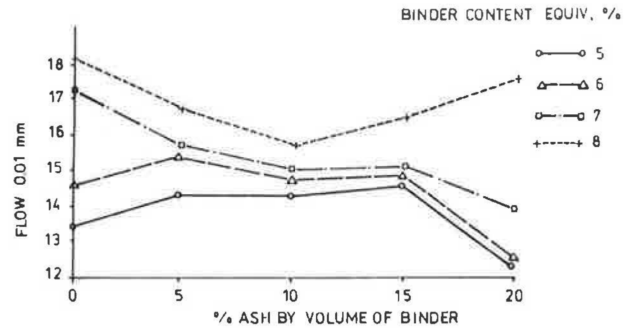

FIGURE 3 Effect of replacing asphalt with oil shale ash on Marshall flow of bituminous mix (1 in. = 2.54 cm); each point is an average of three values.

TABLE 7 OPTIMUM BINDER CONTENT EQUIVALENT FOR ASH-ASPHALT CONCRETE MIXES

Ash by Volume of Binder (%)	Specific Gravity of Binder	Optimum Binder Content (% by weight of mix)	Asphalt Content (% by weight of mix)	Ash Content (% by weight of mix)
0	1.01	5.89	5.89	0.00
5	1.083	6.29	5.57	0.72
10	1.156	6.69	5.26	1.43
15	1.229	7.08	4.95	2.13
20	1.302	7.47	4.64	2.83

Three specimens were prepared for each mix. One day after the specimens were prepared, their bulk specific gravities were measured. Then the specimens were subjected to Marshall tests. Figures 2–6 show the results of these tests. Table 7 gives a summary of the optimum binder content equivalent for ash-asphalt concrete mixes.

Water Sensitivity of Ash-Asphalt Concrete Mixes

The susceptibility of asphalt concrete mixes to attack by water is considered a primary design criterion. This is especially true in areas in which the pavement is in contact with water. Water can get between the binder and the surface of the aggregate. If this happens, the pavement starts to break up as the result of loss of cohesion in the mix. Before ash-asphalt concrete mixes can be used for highway construction, their susceptibility to loss of cohesion in the presence of water must be evaluated.

The Marshall immersion test was used to evaluate the influence of ash on the moisture resistance of ash-asphalt concrete mixes. This test was developed by Taylor and Khosla (*8*). In this test, Marshall stability is measured for wet and dry specimens. The optimum binder contents given in Table 8 were used to design the Marshall specimens. Prepared specimens were divided into two groups. One group of specimens was cured in air at room temperature for 24 hr (unconditioned) and then put in a water bath at 60°C for 30 min. The other group of specimens was immersed in a water bath at 60°C for 24 hr (conditioned).

Conditioning of specimens was performed to investigate the effect of hot water on the cohesion of compacted mixtures. Both groups of specimens were tested using the Marshall apparatus to determine their stability and binder flow values after conditioning. Moisture damage is evaluated on the basis of the ratio of Marshall stability for conditioned and unconditioned specimens. Figures 7 and 8 show the results of these tests.

Resilient Modulus

The resilient modulus test was used to determine the effect of ash on the resilient modulus of ash-asphalt paving mixes. Marshall size specimens were prepared as outlined previously.

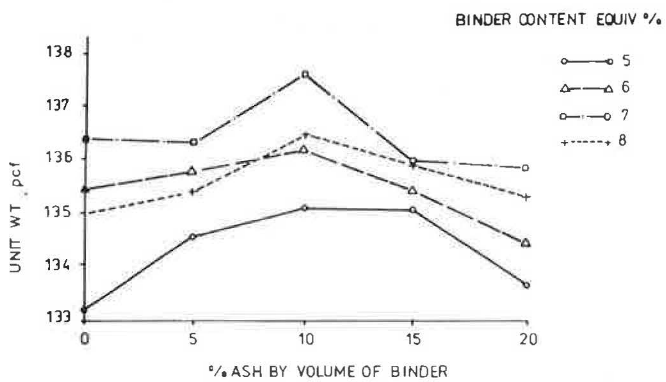

FIGURE 4 Effect of replacing asphalt with oil shale ash on unit weight of bituminous mix (1 lb/ft^3 = 16.019 kg/m^3).

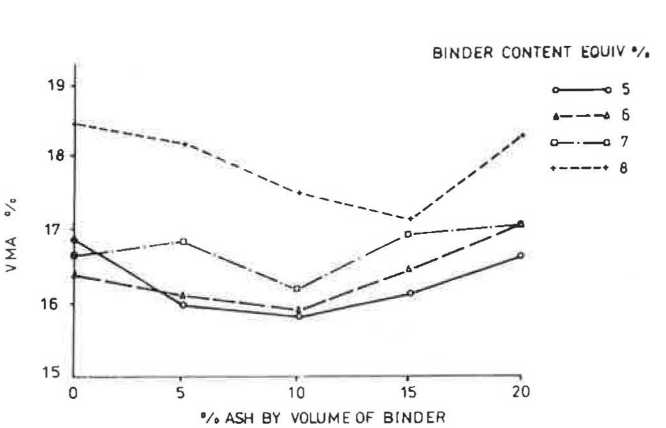

FIGURE 5 Effect of replacing asphalt with oil shale ash on voids in mineral aggregate of bituminous mix.

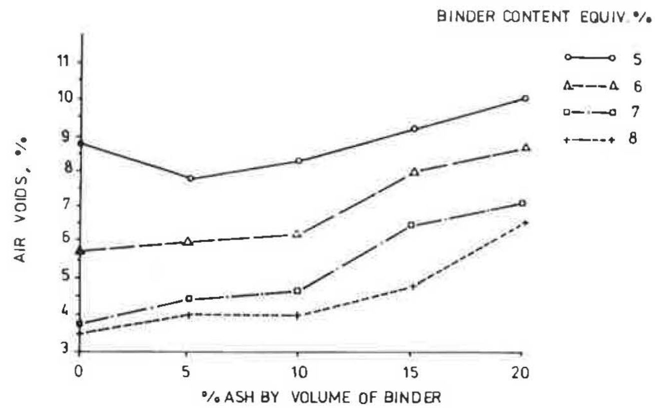

FIGURE 6 Effect of replacing asphalt with oil shale ash on air voids of bituminous mix.

TABLE 8 BINDER, ASPHALT, AND ASH CONTENT FOR DURABILITY AND RESILIENT MODULUS TESTS

Ash by Volume of Binder (%)	Optimum Binder Content (% by weight of mix)	Asphalt Content (% by weight of mix)	Ash Content (% by weight of mix)	Weight of Binder (g)	Weight of Asphalt (g)	Weight of Ash (g)
0	5.89	5.89	0.00	75.10	75.10	00.00
5	6.29	5.57	0.72	80.55	71.33	09.22
10	6.69	5.26	1.43	86.04	67.65	18.39
15	7.08	4.95	2.13	91.43	63.93	27.51
20	7.47	4.64	2.83	96.88	60.18	36.70

NOTE: 1 g = 2.2 × 10^{-3} lb.

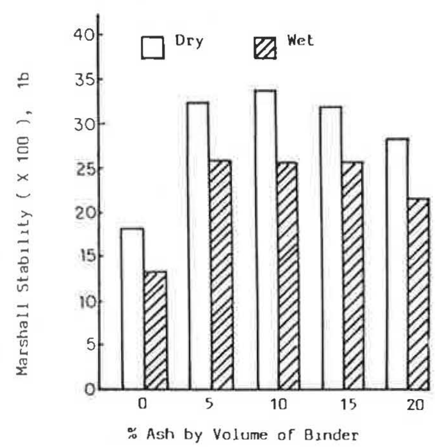

FIGURE 7 Effect of oil shale ash on dry and wet stability of bituminous mixtures (1 lb = 4.448 N).

Two specimens were used to determine the resilient modulus for each mix considered. The binder, asphalt, and ash content (percentage by weight of total mix) given in Table 8 were used. One day after the specimens were prepared, the resilient modulus (M_r) of the specimens was measured in dry condition at 77°F (*9*).

A nondestructive resilient modulus test reported by Schmidt (*9*) was used to determine the resilient modulus of the specimens. In this test a light 0.1-sec-duration pulsating load is applied across one diameter of a cylindrical specimen while the

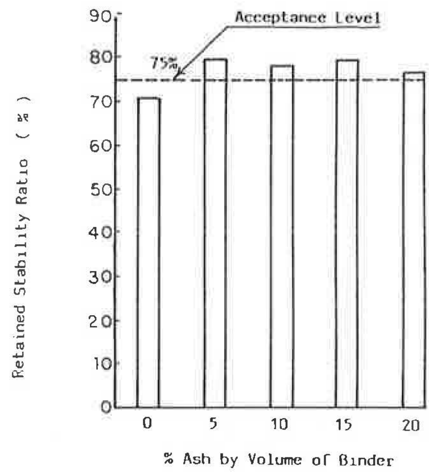

FIGURE 8 Relationship between percentage of ash and retained stability ratio.

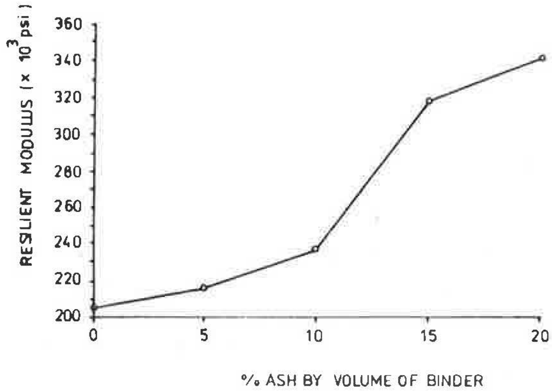

FIGURE 9 Effect of replacing asphalt with oil shale ash on resilient modulus of bituminous mix (1 psi = 6.89 kPa); each point is an average of two values.

resultant elastic deformation across the opposite diameter is measured. A pulsating load of 50 lb was used, and two measurements were taken on each specimen. The resilient modulus was then calculated using the following equation:

$$M_r = [P(\nu + 0.2734)]/t\Delta$$

where

P = applied load (50 lb),
ν = Poisson's ratio (assumed to be 0.35),
t = thickness of specimen (in.), and
Δ = elastic deformation measured (in.).

Figure 9 shows the results of this test.

TEST RESULTS AND DISCUSSION

Marshall Stability of Bituminous Mixtures

Figure 2 shows the effect of oil shale ash on Marshall stability of bituminous mixes for each binder content equivalent. From the results obtained, it was found that stability increases and then decreases with increasing percentages of ash in the binder. It was also found that for a particular binder content equivalent there is an optimum ash content that gives maximum stability. For the ash used in this study, the optimum ash content was around 10 percent for 5, 6, and 7 percent binder content equivalent and 15 percent for 8 percent binder content equivalent.

Marshall Flow of Bituminous Mixes

Figure 3 shows the effect of oil shale ash on Marshall flow of bituminous mixes for each binder content equivalent. At 5 and 6 percent binder content equivalent, the flow increases then decreases with increasing percentages of ash by volume of binder. At 7 percent binder content equivalent, flow decreases with increasing ash concentration in the binder. At 8 percent binder content equivalent, flow decreases and then increases with increasing percentages of ash by volume of binder.

Unit Weight of Bituminous Mixtures

Figure 4 shows the effect of oil shale ash on the unit weight of bituminous mixes for each binder content equivalent. The unit weight increases then decreases with increasing ash concentration in the binder for all curves of binder content equivalent.

Voids in Mineral Aggregate

Figure 5 shows the effect of oil shale ash on VMA in bituminous mixtures. VMA decreases then increases with increasing ash concentration in the binder. This increase in VMA can be due to the rough surface texture of aggregate particles that was developed after addition of ash to the bituminous mixtures.

Air Voids in Bituminous Mixes

Figure 6 shows the effect of oil shale ash on air voids in bituminous mixtures. Air voids increase with increasing ash content in the binder.

Optimum Binder Content

The optimum binder content was determined on the basis of the Marshall method of mix design (ASTM D 1559). The compactive effort was 50 blows on each face of the specimen. These values of binder contents were used in preparing Marshall immersion test and resilient modulus test specimens. Table 6 gives the effect of oil shale ash on optimum asphalt content of the bituminous mixtures.

Stability Test on Dry and Wet Specimens

Dry stability is the value after the specimen has been immersed in hot water at 60°C for 30 min (unconditioned) when tested by the Marshall test apparatus (*7*). Wet stability is the reading measured with the Marshall test apparatus after immersing the specimen in a water bath at 60°C for 24 hr (conditioned).

Figure 7 shows the effect of oil shale ash on dry and wet stability of bituminous mixes. Figure 7 shows that the stability of conditioned specimens was less than that of unconditioned specimens because hot water had softened the mixture, especially when it had been exposed to hot water for 24 hr. Using oil shale ash in bituminous mixes reduced the percentage difference in stability between dry and wet specimens. This is an indication that durability was improved.

Retained Stability Ratio

Evaluation of stripping is mainly based on the retained stability ratio (RSR), which is the ratio between wet and dry stabilities. Most researchers indicate that this ratio should not be less than 75 percent. Therefore 75 percent RSR was used as the acceptance/rejection criterion. Results are summarized in Figure 8. Figure 8 indicates that the RSR-value has been increased by adding oil shale ash to the normal mix (untreated). This increase indicates a reduced potential for stripping in bituminous mixtures. The retained stability of mixtures without ash was not acceptable.

Resilient Modulus of Bituminous Mixes

Figure 9 shows the effect of oil shale ash on the resilient modulus of bituminous mixes. Resilient modulus increases with increasing ash concentration in the binder. This is because the adhesive forces between asphalt and aggregate and the mechanical interlock between aggregate particles have been improved as the result of treatment.

Stiffness Ratio

Table 9 gives the stiffness ratio (SR) for ash-asphalt concrete. SR is defined here as the resilient modulus of the ash-asphalt concrete mix at 1 day divided by the resilient modulus of a similar mix with no ash (control mix) at 1 day. It was found that the SR increases as the percentage of ash in the binder is increased. In other words, the more ash used, the higher the resilient modulus of the mix. This is because the addition of ash enhances the adhesive forces between asphalt and aggregate.

TABLE 9 EFFECT OF REPLACING ASPHALT WITH OIL SHALE ASH ON STIFFNESS RATIOS

Ash in Binder (%)	Stiffness Ratio[a]
5	1.06
10	1.16
15	1.56
20	1.66
25	1.73

[a]Stiffness ratio = (Resilient modulus of ash-asphalt concrete mix)/(Resilient modulus of control mix with no ash).

CONCLUSIONS

From the results obtained in this study, the following conclusions were drawn:

1. Oil shale ash significantly increases the stability and cohesion of bituminous mixtures. The use of 10 percent ash by volume of asphalt could be considered optimum.

2. The unit weight of bituminous mix increases then decreases with increasing ash concentration in the binder.

3. Voids in mineral aggregate decrease then increase with increasing ash concentration in the binder.

4. Air voids increase with increasing percentages of ash in the binder.

5. Use of oil shale ash reduces the potential for stripping in bituminous mixtures because this additive converts the aggregate surface to one that is more easily wetted with asphalt than with water.

6. Oil shale ash improves the stiffness of bituminous mixtures because the adhesive forces between asphalt and aggregate and the mechanical interlock between aggregate particles are improved by treatment.

7. Ash as an additive to asphalt cement plays a dual role in paving mixtures. First, it and asphalt form a high-consistency binder that cements the aggregate. Second, it acts as a part of the mineral aggregate, fills the interstices, and provides contact points between particles, thereby strengthening bituminous mixtures.

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