

TRANSPORTATION RESEARCH RECORD 843

**Asphalts, Asphalt
Mixtures, and
Additives**

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1982

Transportation Research Record 843
Price \$13.40
Edited for TRB by Naomi Kassabian

modes

- 1 highway transportation
- 4 air transportation

subject areas

- 24 pavement design and performance
- 31 bituminous materials and mixes
- 62 soil foundations

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Asphalts, asphalt mixtures, and additives.

(Transportation research record; 843)

1. Pavements, Asphalt—Addresses, essays, lectures, 2. Asphalt—
Addresses, essays, lectures. I. National Research Council (U.S.)
Transportation Research Board. II. Series.

TE7.H5 no. 843 [TE270] 380.5s [625.8'5] 82-12579
ISBN 0-309-03320-9 ISSN 0361-1981

Sponsorship of the Papers in This Transportation Research Record

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES

R. V. LeClerc, consultant, Olympia, Washington, chairman

Bituminous Section

J. York Welborn, consultant, Rockville, Maryland, chairman

Committee on Characteristics of Bituminous Materials

Rowan J. Peters, Arizona Department of Transportation, chairman
Peggy L. Simpson, Sahuaro Petroleum and Asphalt Co., secretary
David A. Anderson, Joe W. Button, Richard L. Davis, Robert L. Dunning, Jack N. Dybalski, E. Keith Ensley, Woodrow J. Halstead, C. W. Heckathorn, M. V. Hunter, Prithvi S. Kandhal, Narendra P. Khosla, L. C. Krchma, Robert P. Lottman, C. A. Pagen, J. Claine Petersen, Charles F. Potts, Vytautas P. Puzinauskas, B. A. Vallerga, J. York Welborn, Leonard E. Wood

Committee on Characteristics of Nonbituminous Components of Bituminous Paving Mixtures

Gene R. Morris, Arizona Department of Transportation, chairman
John E. Huffman, Sahuaro Petroleum and Asphalt Co., secretary
Oliver E. Briscoe, John J. Emery, Nyla M. Ford, Douglas J. Hanson, Bobby J. Huff, Prithvi S. Kandhal, L. C. Krchma, Bobby D. Lagrone, Dah-Yinn Lee, Donald W. Lewis, Robert P. Lottman, Charles R. Marek, Charles F. Potts, Vytautas P. Puzinauskas, Donald Saylak, Russell H. Schnormeier, Scott Shuler, Garland W. Steele, Egons Tons, Richard D. Walker, Leonard E. Wood

Committee on Characteristics of Bituminous-Aggregate Combinations to Meet Surface Requirements

Leonard E. Wood, Purdue University, chairman
Fred C. Benson, Sabir H. Dahir, John J. Emery, Moreland Herrin, Rudolf A. Jimenez, Bernard F. Kallas, Nabil Kamel, Prithvi S. Kandhal, Michael S. Mamlouk, G. W. Maupin, Jr., James A. Scherocman, Peggy L. Simpson, Stewart R. Spelman, Loren M. Womack

Committee on Characteristics of Bituminous Paving Mixtures to Meet Structural Requirements

Bernard F. Kallas, The Asphalt Institute, chairman
Grant J. Allen, Oliver E. Briscoe, R. N. Doty, Jack N. Dybalski, Jon A. Epps, K. P. George, William O. Hadley, R. G. Hicks, J. M. Hoover, Rudolf A. Jimenez, Edwin H. Jones, Ignat V. Kalcheff, Thomas W. Kennedy, Narendra P. Khosla, Larry L. Kole, Raymond K. Moore, C. A. Pagen, Richard D. Pavlovich, David W. Rand, James A. Scherocman, Charles G. Schmitz, Donald R. Schwartz, Jack E. Stephens, Ronald L. Terrel, David G. Tunnicliff, B. A. Vallerga, Leonard E. Wood, Walter H. Zimpfer, James D. Zubiena

William G. Gunderman, Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each report. The organizational units, officers, and members are as of December 31, 1981.

Contents

SPECIFICATION REQUIREMENTS FOR ASPHALT-RUBBER Scott Shuler	1
LABORATORY MEASUREMENTS OF ASPHALT-RUBBER CONCRETE MIXTURES R.A. Jimenez	4
FIELD EVALUATION OF RUBBER-MODIFIED BITUMINOUS CONCRETE Jack E. Stephens	11
METHODS OF INCREASING FRACTURE TOUGHNESS OF ASPHALT CONCRETE R.T. Woodhams	21
EVALUATION OF FILLER EFFECT OF SULFUR IN ASPHALT BINDER Amir F. Bissada	26
PREDICTING SURFACE FRICTION FROM LABORATORY TESTS W.H. Parcels, Jr., T.M. Metheny, and R.G. Maag	33
DESIGN AND PERFORMANCE OF BITUMINOUS FRICTION-COURSE MIXES Nabil Kamel, G.R. Musgrove, and A. Rutka	40
PERFORMANCE OBSERVATIONS ON OPEN-GRADED BITUMINOUS CONCRETE OVERLAYS IN CONNECTICUT Charles E. Dougan	51
EFFECTS OF BAGHOUSE FINES AND MINERAL FILLERS ON PROPERTIES OF ASPHALT MIXES Bassam A. Anani and Hamad I. Al-Abdul-Wahhab	57
EFFECT OF MIX VARIATIONS ON ASPHALT PAVEMENT LIFE: NORTH OAKLAND-SUTHERLIN PROJECT Jean Walter, R.G. Hicks, Joe P. Mahoney, and J.E. Wilson, Jr.	64
DUNE SAND-AGGREGATE MIXES AND DUNE SAND-SULFUR MIXES FOR ASPHALT CONCRETE PAVEMENTS M.N. Fatani and H.A. Sultan	72
ASPHALT CONCRETE MIXTURES MADE WITH CEMENT-COATED AGGREGATES H.R. Guirguis, O.E.K. Daoud, and S.K. Hamdani	80

FACTORS AFFECTING UNCONFINED COMPRESSIVE STRENGTH OF LIME-BITUMINOUS-EMULSION-TREATED CLAY (Abridgment) Raymond K. Moore	85
LABORATORY TEST METHOD FOR PREDICTING MOISTURE-INDUCED DAMAGE TO ASPHALT CONCRETE Robert P. Lottman	88
CHEMISTRY OF ASPHALT-AGGREGATE INTERACTION: RELATIONSHIP WITH PAVEMENT MOISTURE-DAMAGE PREDICTION TEST J.C. Petersen, H. Plancher, E.K. Ensley, R.L. Venable, and G. Miyake	95
SILANE PRETREATMENT OF MINERAL AGGREGATE TO PREVENT STRIPPING IN FLEXIBLE PAVEMENTS Joseph A. DiVito and Gene R. Morris	104
EVALUATING ASPHALTENE SETTLING TEST AND RELATING RESULTS TO PHYSICAL PROPERTIES OF PAVING ASPHALTS Freddy L. Roberts and Thomas W. Kennedy	111
ASPHALT TEMPERATURES SUSCEPTIBILITY AND ITS EFFECT ON PAVEMENTS Joe W. Button, Jon A. Epps, Dallas N. Little, and Bob M. Gallaway	118

Authors of the Papers in This Record

- Al-Abdul-Wahhab, Hamad I., Department of Civil Engineering, University of Petroleum and Minerals, Box 220, Dhahran, Saudi Arabia
- Anani, Bassam A., Dhahran International Airport, P.O. Box 144, University of Petroleum and Minerals, Box 220, Dhahran, Saudi Arabia
- Bissada, Amir F., Department of Civil Engineering, College of Engineering and Petroleum, University of Kuwait, P.O. Box 5969, Kuwait
- Button, Joe W., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
- Daoud, O.E.K., c/o Mr. Sadi Farwana, Ministry of Public Works, Kuwait
- Di Vito, Joseph A., Arizona Transportation Research Center, Arizona State University, Phoenix, AZ 85007
- Dougan, Charles E., Bureau of Planning and Research, Connecticut Department of Transportation, Wethersfield, CT 06109
- Ensley, E.K., Laramie Energy Technology Center, U.S. Department of Energy, P.O. Box 3395, University Station, Laramie, WY 82071
- Epps, Jon A., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
- Fatani, M.N., Department of Civil Engineering, King Abdulaziz University, Jeddah, Saudi Arabia
- Galloway, Bob M., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
- Guirguis, H.R., Resource International, Ltd., P.O. Box 8021, Salmya, Kuwait
- Hamdani, S.K., Resource International, Ltd., P.O. Box 8021, Salmya, Kuwait
- Hicks, R.G., Oregon State University, Corvallis, OR 97331
- Jimenez, R.A., Arizona Transportation and Traffic Institute, College of Engineering, University of Arizona, Tucson, AZ 85721
- Kamel, Nabil, Gulf Canada Limited, 2489 N. Sheridan Way, Sheridan Park, Mississauga, Ontario L5K 1A8, Canada
- Kennedy, Thomas W., Center for Transportation Research, University of Texas at Austin, Austin, TX 78712
- Little, Dallas N., Texas Transportation Institute, Texas A&M University, College Station, TX 77843
- Lottman, Robert P., Department of Civil Engineering, University of Idaho, Moscow, ID 83843
- Maag, R.G., Research and Material Laboratory, Kansas Department of Transportation, 2300 Van Buren Street, Topeka, KS 66611
- Mahoney, Joe P., Department of Civil Engineering, University of Washington, Seattle, WA 98105
- Metheny, T.M., Colorado Department of Transportation, 4201 East Arkansas Avenue, Denver, CO 80222
- Miyake, G., Laramie Energy Technology Center, U.S. Department of Energy, P.O. Box 3395, University Station, Laramie, WY 82071
- Moore, Raymond K., School of Engineering and Engineering Experiment Station, Auburn University, Auburn, AL 36849
- Morris, Gene R., Arizona Transportation Research Center, Arizona State University, Phoenix, AZ 85007
- Musgrove, G.R., Ontario Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario, Canada
- Parcells, W.H., Jr., Research and Materials Laboratory, Kansas Department of Transportation, 2300 Van Buren Street, Topeka, KS 66611
- Petersen, J.C., Laramie Energy Technology Center, U.S. Department of Energy, P.O. Box 3395, University Station, Laramie, WY 82071
- Plancher, H., Laramie Energy Technology Center, U.S. Department of Energy, P.O. Box 3395, University Station, Laramie, WY 82071
- Roberts, Freddy L., Center for Transportation Research, University of Texas at Austin, Austin, TX 78712
- Rutka, A., Ontario Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario, Canada
- Shuler, Scott, Texas Transportation Institute, Texas A&M University, College Station, TX 77843
- Stephens, Jack E., Department of Civil Engineering, University of Connecticut, Storrs, CT 06268
- Sultan, H.A., Department of Civil Engineering, University of Arizona, Tucson, AZ 85721
- Venable, R.L., University of Missouri, Rolla, MO 65401
- Walter, Jean, Oregon State University, Corvallis, OR 97331
- Wilson, J.E., Jr., Highway Division, Oregon Department of Transportation, Salem, OR 97310
- Woodhams, R.T., Department of Chemical Engineering and Applied Chemistry, University of Toronto, Toronto, Ontario M5S 1A4, Canada

Specification Requirements for Asphalt-Rubber

SCOTT SHULER

Asphalt-rubber has been used in the United States to fabricate seal coats and interlayers since 1967. Current estimates put the amount of asphalt-rubber in service at approximately 10 000 lane miles of both highway and aircraft pavements. Specifications used since 1967 are a result of experience gained with materials applied. Empirical results from paving projects traditionally have provided the engineer with information necessary to devise a descriptive account of proper construction procedures. Information is provided regarding knowledge gained since 1967 on proper practices in asphalt-rubber construction. Some of the tempting variances to recommended procedures are cited as well as predictable results if specified descriptions are not adhered to.

Current specifications for asphalt-rubber mixtures are derived from descriptions of materials produced by companies involved in asphalt-rubber construction. Through many years of empirical development, asphalt-rubber membranes have evolved from the combination of a limited source of ground scrap tires and asphalt cement to a sophisticated art and science that uses many sources of raw materials. This empirical development led to increased understanding regarding potentially beneficial asphalt and rubber combinations, effective concentrations, and methods of fabrication that include enumerable preparation techniques. This technology allows the leaders in asphalt-rubber production to gain expertise in order to improve their products and consequently enhance pavement quality. However, current specifications remain a description of the products from a few suppliers of asphalt-rubber.

Research activities have been under way in asphalt-rubber development since its inception in 1964. Experimentation has been helpful in determining physical and chemical characteristics that might be used to describe the differences in mixtures, beneficial attributes, and properties desirable for field use. Developing specifications has been the goal of many researchers, but difficulties encountered with a new hybrid engineering material such as asphalt-rubber provided much challenge in their development. Initially, laboratory tests had to be developed for a material that behaves neither like asphalt cement nor like tire rubber. Research and development activities aimed at identifying adequate laboratory and field tests continue. Later, after a few promising laboratory procedures were discovered that could differentiate between rubber mixes, interpretation of results was necessary to provide meaning to these new tests.

The mechanism that allows asphalt-rubber or any other low-modulus interlayer to provide for crack attenuation is understood in principle; however, the solution to the problem that describes how the mechanism operates is extremely complex. An understanding of this mechanism is required before any phenomenological or mechanistic specification can be prepared. This specification would not be a description of an empirical product but a rational procedure for the preparation and construction of a new engineering material.

SUMMARY OF ASPHALT-RUBBER SPECIFICATIONS

Asphalt-rubber as defined in this paper is the combination of hot asphalt cement and recycled passenger-car or truck tires. It is understood that uses for this product are numerous and include applications other than pavement interlayers and surface treatments. However, this paper is concerned with the latter applications only, and references to specifications included here deal specifically with

membrane construction. Asphalt-rubber as defined includes between 18 and 24 percent by total weight of dry rubber in an asphalt cement matrix. The methods of combining these two principal ingredients vary, and distillate additives are allowed in current specifications, but the component composition of the various types of mixtures is essentially equivalent. The product obtained after the two principal components have been combined, however, varies so dramatically between various mixtures that methods for controlling fabrication are essential.

The first mixtures of asphalt and rubber as defined here included only the two principal ingredients. Rubber was added to hot asphalt cement and mixed for various time periods at temperatures considerably higher than those customarily used on paving projects. Results of these early projects varied, but the concept was sound and from these early experiments came the first specifications for a new paving material.

The proceedings of the 45th Annual Meeting of the Highway Research Board document a specification by C.H. McDonald (1) that called for use of 33 percent by weight of rubber and the remaining proportion occupied by 85-100 penetration-grade asphalt cement. The asphalt was heated to 420°F and the rubber added and mixed until a jell consistency was acquired. This composition was applied to the pavement in amounts of 1 gal/yd² by using squeegees. Then approximately 45 lb/yd² of aggregate chips were applied to complete the membrane. Some experimentation was done with varying proportions of rubber to asphalt by using mixes with 25 percent rubber by total weight.

Later, slurry seal equipment was used to apply the mixtures. Asphalt temperatures were raised to 445°F prior to mixing with rubber, but proportions remained at two parts 85-100 asphalt to one part rubber. Difficulties getting uniform application of the final asphalt-rubber mixture led investigators away from this form of construction.

After early work that used slurry seal equipment to construct membranes, use of conventional seal-coat distributor equipment began. The success of this process was mixed, since the conventional equipment had application difficulties with the very high viscosity material (2). These difficulties led to the addition of kerosenes to the hot asphalt-rubber during the mixing process. This process aided uniformity during distribution, and experimentation began by using various types of diluents. One major difference between modern specifications and previous ones is the provision for diluents.

As the technology of asphalt-rubber applications progressed, so did the descriptions of the materials, fabrication techniques, and application processes. The most recent specification (3) to date provides a description of major differences among the various types of asphalt-rubber membranes. The differences among these specifications are due to several types of rubber currently available for use in membrane production. These types include possible combinations of vulcanized and devulcanized rubber materials. Included in these specifications in addition to rubber differences are details that surround the use of various principal ingredients. As field experience indicates, use of one of the rubber types means using a specific method for fabrication and application of the membrane. Current

specifications are considerably more complex than previous ones and allow more flexibility and consequently more margin for error in inexperienced hands. Several rubber gradations, types of rubber, rubber manufacturing differences, asphalt sources, diluent types, and fabrication processes make the construction of asphalt-rubber membranes a highly refined technique that requires considerable experience by the applicator. Clearly, the need for comprehensive specifications is implicit in successful production by contractors who have had little or no previous experience with asphalt-rubber.

REQUIREMENTS OF SPECIFICATIONS

Specifications for asphalt-rubber have through necessity been designed as specific component and method outlines aimed at describing in detail every aspect of fabrication. Performance specifications to date do not exist, due to an insufficient knowledge that relates field performance with laboratory test results. The aim of many researchers in asphalt-rubber technology is to develop such specifications. This type of specification appears worthy of development for a material as complex as asphalt-rubber. It seems unlikely that the relatively few asphalt-rubber formulations in use are the only potentially successful combinations available. Indeed, performance attributes that the successful mixtures have in common should also be available in some yet-undiscovered method of combining raw materials.

These ideal performance attributes that combinations of asphalt and rubber should display will in the future be the basis for specifications. Research in progress is dedicated to determining what engineering properties these mixtures should display and what techniques may be used to measure them. Only by developing this type of construction specification will the seemingly insurmountable task of combining the myriad of asphalts and rubber be simplified. However, the knowledge necessary to establish such a specification remains in the future, and until adequate information becomes available regarding materials properties, the recipe specification is essential to competent construction.

The most concise specification to date has been fabricated based on proven materials, manufacturers, and construction methods. It seems of interest to note some of the more important aspects of this specification. It is understood that as soon as a performance-oriented specification becomes available, many items necessary in the asphalt-rubber recipe specification may not be required. However, until more-mechanistic methods are discovered, the current technology will dictate the procedures and components for use.

Generally, a specification, for whatever use, is designed to guide an agency through the construction process by indicating the types of materials and methods of fabrication required to produce the desired result. Asphalt-rubber specifications are no different in this respect and consequently begin with definitions essential to understanding the specialized terminology of asphalt-rubber technology. Examples of terms that may be included in this type of glossary are listed below:

1. Ambient ground rubber: Tire rubber ground or processed at ordinary room temperatures.

2. Asphalt-rubber: A mixture of paving-grade asphalt and recycled vehicular tire rubber and certain additives. The rubber component is at least 15 percent by weight of the total mixture and is reacted in the hot asphalt sufficiently to cause swelling of the rubber particles.

3. Automobile tires: Tires that have an outside diameter less than 26 in and are used by automobiles or light trucks.

4. Cryogenically ground rubber: Tire rubber that has been subjected to temperatures below the embrittlement temperature of the rubber during the grinding process.

5. Devulcanized rubber: Tire rubber that has been subjected to treatment by heat, pressure, or the addition of softening agents to alter properties of the recycled material.

6. Reclaimed tire rubber: Rubber obtained by processing and recycling used automobile, truck, and bus tires. Solid tires, fork-lift, aircraft, earth-moving equipment, and other nonautomobile (truck) tires and non-tire-rubber sources are excluded.

7. Tread rubber: Tire rubber that consists primarily of tread rubber or peel with less than 5 percent sidewall rubber.

Descriptions of the materials to be used are an essential part of the job specification. Asphalts available around the country vary considerably, and experience with different asphalts, although considerable, pales in comparison with the number available. Therefore, the type of bitumen to be used should be described in some detail to avoid confusion. Rubber types and manufacturing methods differ much less abundantly than do asphalts. However, several choices of rubber types are available. Depending on which rubber is used, the method of combining rubber with asphalt will vary. For example, the rubber types available from the recycling industry are numerous. Even after all sources other than tires have been discounted, choices remain. Finally, after the choice has been limited to pneumatic tires from passenger cars and heavy trucks and earth-mover tires, aircraft tires, or pneumatic industrial-machinery applications have been eliminated can the final selection be made.

The difference between mixes made from automobile versus truck tires is due in part to the chemical balance in the rubber. One constituent of tire rubber known to affect asphalt-rubber behavior is the natural rubber component. Whole truck tires contain approximately 18 percent natural rubber compared with 9 percent for whole automobile tires and 2 percent for automobile tire tread (4).

Rubber-recycling methods are different. Some processes produce a totally vulcanized product, whereas others produce a chemically treated rubber tire product known as devulcanized or depolymerized rubber. Combinations of the two products are also readily available. Some recyclers grind up tires in shredders and put the whole tire through the mill in successive passes until the desired gradation of ground rubber is produced. Some recyclers put only the tread rubber through the grinder; the resultant rubber product has virtually no sidewall material and consequently different properties than the particulate whole tire. Some producers use cryogenic methods to fabricate the particulate tires, whereas others grind at relatively ambient conditions. All these processes result in different particulate rubber materials.

Sizes of rubber particles may differ in asphalt-rubber mixtures. The most recent specification includes four gradations. The type of rubber to be used in the asphalt-rubber mix dictates which gradation or gradations to choose. The four gradations currently in use appear in Table 1. Types I, II, and III are used with vulcanized rubber products. Type IV is used when devulcanized materials are present in the mixture.

The third constituent used in asphalt-rubber production is a viscosity-reducing agent. This mater-

Table 1. Gradations of particulate tire rubber for use in asphalt-rubber.

Sieve No.	Percent Passing			
	Type I	Type II	Type III	Type IV
8	100	100		100
10	95-100	95-100		
16		70-80	100	
20			95-100	
30	0-10	5-15	60-80	60-80
40	0-5	0-5		
50			0-10	15-40
100				0-15

ial can be either a kerosene-type diluent or a high-molecular-weight aromatic asphalt extender oil. These materials are used with vulcanized and devulcanized products, respectively.

Although constituent analysis of rubber products is routine and the separation of various components includes identification of natural and synthetic rubber, ash content, carbon black, etc., only a requirement for the natural rubber content has been cited to date in specifications. This value has been set at 30 percent of total rubber weight.

Combining asphalt and rubber is the next step in the specification process after the materials description. This segment of the specification describes the method of bringing the three raw materials together. Two procedures are possible depending on whether vulcanized or devulcanized rubbers are used. The major differences between the two procedures include the proportions of asphalt and rubber, the introduction of viscosity-reducing agents, and the time and temperature of mixing. In general, however, the two procedures are alike regarding the product desired. This product is described as a homogeneous mixture, uniform throughout, that will not separate into constituent components when mixed or applied.

Much of the difficulty in producing any satisfactory paving product appears during construction. This is not necessarily the fault of the contractor. It is this stage of a project in which the greatest number of variables are introduced that can thwart success. In most paving construction, weather is generally the single variable that can make or break a given project. In asphalt-rubber construction, this is also true. Unlike conventional seal coats, asphalt-rubber membranes provide no room for obvious errors made during construction. Where moist or wet aggregate would be marginally acceptable in a conventional seal coat, this condition will lead to debonding with an asphalt-rubber membrane almost without exception. This may not be a problem if the membrane is to be used as an interlayer, but as a surface course, it should be avoided. The distance between the aggregate chip spreader and asphalt distributor truck is critical in asphalt-rubber construction. It has been suggested that the chip spreader be physically attached to the asphalt-rubber distributor. This may not be practical, but the implication is clear. The chips should be embedded in the hot binder before too much cooling occurs in the asphalt-rubber binder. The closer the chip spreader is to the asphalt-rubber distributor, the better the aggregate retention will be. Application rates for asphalt-rubber mixtures are approximately twice those of conventional seal coats. At 0.55-0.60 gal/yd² of hot asphalt-rubber, the quantity of aggregate chips required to cover this membrane is proportionately higher than conventional amounts. This increase in quantities of materials sometimes leads conscientious or well-

meaning employees to reduce specified application rates on site. Obviously, this situation should be guarded against, since reduction in asphalt-rubber application rates will jeopardize the effectiveness of the membrane.

The roller train used in membrane construction usually numbers three pneumatic types. Tire pressures are required to be 100 lbf/in² with a minimum 4000 lbf/wheel. Rollers must follow the chip spreader closely to achieve proper embedment. Various methods to ensure chip retention have been used with much success. On occasion, when maximum retention is required, precoated chips have been used with much success. Other times, when initial retention has been marginal, a light application of emulsified asphalt (5) applied to the compacted surface is often effective in reducing chip loss. Preheated chips may also be included in specifications when cool weather threatens or ensurance of low moisture content is desired.

SUMMARY

An outline of some of the factors involved in producing an effective specification for asphalt-rubber has been presented. A description of the constituents involved and the methods of combining and applying asphalt-rubber to pavement surfaces were discussed. The component and procedure type of specification currently in use may be replaced with a specification based on performance attributes in the future. Research and development efforts now under way are aimed at producing such a specification but complexities associated with composite and hybrid materials make identification of desirable materials properties difficult. A performance specification is a desirable method for identifying this product. Complex combinations of potential materials make the recipe specification inefficient and potentially wasteful. By calling for unique performance attributes, materials or methods heretofore overlooked might prove useful. In addition, performance specifying should simplify the process of description in asphalt-rubber specifications and should potentially eliminate misunderstandings regarding materials and procedures.

The use of asphalt-rubber membranes offers much to the pavement maintenance engineer and manager. However, construction of these systems has proved that diligence is required by the inspector. With conventional surface treatments, specified methods may be relaxed or modified when necessary and adequate results may be obtained. Asphalt-rubber membranes allow no such deviation from outlined practice. Procedures identified in specifications must be followed if successful results are to be achieved. Many dramatic successes have been documented for such a new paving material; however, when practices in variance to specifications are adopted, equally dramatic failures may also be documented.

ACKNOWLEDGMENT

I would like to acknowledge the U.S. Air Force Engineering Services Center, Tyndall Air Force Base, for support of on-going experimentation in asphalt-rubber research conducted at the University of New Mexico Engineering Research Institute, Albuquerque, New Mexico. I would also like to acknowledge the Texas State Department of Highways and Public Transportation for support of current research and development efforts at the Texas Transportation Institute, College Station, Texas.

REFERENCES

1. C.H. McDonald. New Patching Material for Pave-

- ment Failures. HRB, Highway Research Record 146, 1966, pp. 1-16.
2. G.R. Morris and C.H. McDonald. Asphalt-Rubber Stress-Absorbing Membranes: Field Performance and State of the Art. TRB, Transportation Research Record 595, 1976, pp. 52-58.
 3. Draft ASTM Specifications for Asphalt-Rubber. ASTM Task Force D04.37, Detroit, MI, 1981.
 4. B.D. LaGrone. Rubber Used in Asphalt-Rubber Applications. Proc., National Seminar on Asphalt-Rubber, San Antonio, TX, Federal Highway Administration, Oct. 1981.
 5. W.O. Ford and H.G. Lansdon. Development and Construction of Asphalt-Rubber Stress-Absorbing Membranes. Western Association of State Highway and Transportation Officials, 55th Annual Conference, June 14-18, 1976.

Publication of this paper sponsored by Committee on Characteristics of Non-bituminous Components of Bituminous Paving Mixtures.

Laboratory Measurements of Asphalt-Rubber Concrete Mixtures

R.A. JIMENEZ

The objective of this study was to develop procedures for making and testing specimens made with asphalt-rubber and aggregates. The investigation was aimed at finding a method or methods for (a) mixing the high-viscosity asphalt-rubber with aggregates, (b) forming test specimens made with this resilient material, and (c) testing the compacted specimens for characterization by using some common procedures. The above factors are discussed along with results obtained for Hveem stability, cohesiometer value, axial tension, double-punch tension and dynamic modulus of elasticity, and resistance to debonding under a dynamic repeated pore-water pressure exposure. In general, it was found that good aggregate coating can be obtained with a common laboratory mixer at the usual mixing temperatures, that California-tamping-foot compaction was not possible and that vibratory compaction yielded higher densities than static compaction, that compacted specimens required a storage period of three days in the mold at room temperature, that testing for strength had to be performed at room temperature or lower, and that expected low strength and durability are attributed to high air-void content.

Asphalt-rubber (A-R) is a blend of asphalt and fine grindings from rubber tires. The amount of rubber in the blend has been a relatively high value, about 25 percent by weight, and the rubber has been either vulcanized or devulcanized. An A-R with vulcanized rubber was investigated by C.H. McDonald and a specific formulation was patented by him in the 1960s. A review of the development and use of A-R has been given both by Jimenez, Morris, and DaDeppo (1) and by Morris and McDonald (2).

In Arizona, the main use of A-R blends has been as a binder in chip-seal construction. The chip seal has been placed as a surface course or as a strain-attenuating interlayer to minimize reflection cracking of a bituminous overlay.

Of particular concern to this study was the use of a strain-attenuating layer (SAL) constructed by using a mixture of A-R and aggregate. With chip-seal construction, there are difficulties with uniformity of application and with provision of a consistently good performance. The use of kerosene in the vulcanized rubber and asphalt blends would seem to present additional problems in the A-R SAL construction. The solution to the construction problems of chip seals would appear to be its replacement with a hot-mix A-R concrete. This would control the proportioning of materials and construction.

The objective of this study was to develop procedures for making and testing A-R concrete specimens; these mixtures were then characterized by using

common asphaltic concrete (AC) measurements.

MATERIALS

Asphalts

The majority of spray applications of vulcanized rubber and asphalt has used a soft asphalt of the AR-1000 grade that meets the specifications described by the Arizona Department of Transportation (3). The A-R blends containing devulcanized rubber generally use an AR-4000 asphalt and an extender oil. Both these asphalt grades were used in the work presented.

Rubbers

The two types of rubber granules that have been used in the construction of A-R were mixed with the above asphalts to make the A-R blends. The two types are vulcanized and devulcanized rubber. The vulcanized rubber granules came from the grinding of passenger tire treads. These are a styrene-butadiene rubber that is of one size passing a No. 16 sieve and being retained on the No. 25 sieve. The second type of rubber granules was a mixture of natural and devulcanized rubber. The particle sizes were graded from the No. 8 sieve to the No. 200 sieve.

Aggregates

Two aggregate gradations of 9.5-mm (3/8-in) maximum size were used for making the AC. Their gradations and other physical properties are listed in Table 1. The open gradation was chosen because of its possible use as a hot plant seal for replacing a chip seal. Also, it was anticipated that coating difficulties might be overcome by using an aggregate of low surface area. The maximum size of aggregate was limited to 9.5 mm since in its use as an SAL the thickness of the layer would not be greater than 12.7 mm (0.5 in).

A-R Blends

The A-R blends were made according to procedures described in a report by Jimenez (4). A brief description follows.

Table 1. Characteristics of aggregates.

Physical Characteristic	Dense Aggregate ^a (% passing)		Open Aggregate ^a (% passing)
	Original	Modified	
Sieve size			
3/8-in	100	100	100
No. 4	87	92	55
No. 8	68	76	12
No. 16	49	56	4
No. 30	34	34	1
No. 50	21	16	1
No. 100	13	7	
No. 200	8	4	
Surface area (ft ² /lb)	42		2
Sand equivalent	28	82	
Centrifuge kerosene equivalent oil ratio (%)	4.3		2.6
Effective specific gravity	2.614		2.588

Note: 1 in = 25.4 mm; 1 ft²/lb = 0.09 m²/kg.

^aSize: 3/8 in.

The batch size was held to about 1000 g (35 oz) [250 g (9 oz) of rubber and 750 g (26 oz) of AR-1000 asphalt]. The asphalt was brought to a temperature of 191°C (375°F) in a stainless-steel saucepan. The hot asphalt was stirred with an electric mixer that had a three-bladed propeller 76 mm (3 in) in diameter. The rubber grindings were added to the asphalt within a 5-min period and then the mixture was stirred and held to the 191°C temperature for 30 min. It was then ready to be added to the hot aggregate for mixing.

The blend containing the devulcanized rubber (AR-S) was made of 78.4 percent AR-4000, 1.6 percent extender oil, and 20.0 percent rubber granules. The reacting temperature of 204°C (400°F) was held for 1 h.

MIXING A-R AND AGGREGATE

In regular AC design it is customary to heat the aggregate and AC to specified temperatures prior to mixing and compacting. This is to provide good coating of the aggregate and compaction of the mixture. For example, test method T245-78 of the American Association of State Highway and Transportation Officials (AASHTO) (5) specifies that, for mixing, the asphalt temperature should correspond to that at which the asphalt has a viscosity of about 170 cSt and that, for compacting, the hot mixture should be at a temperature corresponding to that at which the asphalt has a viscosity of about 280 cSt. For most paving AC, the laboratory mixing temperature is usually less than 163°C (325°F) and the compaction temperature of the mixture is more than 110°C (230°).

Earlier work with making A-R blends showed that the mixture had high viscosity at a mixing temperature of 191°C. In the report by Jimenez, Morris, and DaDeppo (1), temperature-viscosity relationships were presented for an AR-1000 and an A-R blend that had a weight ratio of vulcanized rubber to AR-1000 asphalt of 25/75. These equations were as shown below:

$$\text{AR-1000} \quad \eta = 6.767 \times 10^{25} F^{-10.69} \quad (1)$$

$$\text{A-R blend} \quad \eta = 5.768 \times 10^{14} F^{-4.494} \quad (2)$$

where η is viscosity in poises at a shear rate of 0.05 sec⁻¹ and F is temperature in degrees Fahrenheit.

Equation 1 indicates that at a temperature of 121°C (250°F) the viscosity of the asphalt would be

1.6 poises or approximately 180 cSt if it is assumed that the hot asphalt has a density of 0.9 g/cm³ (0.03 oz/in³). If one enters in the A-R equation with a value of 1.53 poises (170 cSt), one will obtain a temperature of 938°C (1721°F), which is certainly not a reasonable temperature for mixing with aggregates. It is not necessary to explain the discrepancies involved in obtaining the unreasonable mixing temperature for A-R mixtures; however, at the initiation of this study it was considered reasonable to assume that a relatively high mixing temperature and/or high shear rate (much greater than 0.05 sec⁻¹) would be necessary to coat an aggregate with the A-R blend.

The results of experimentation with variations in mixing temperature and compaction are given in a report by Jimenez (6). The outcome of that work led to the use of our standard mixing procedure, in which the aggregate was heated to 149°C (300°F), the A-R was at 121°C, and a 4000-g (140-oz) batch was mixed in an 11-L (10-gt) Hobart C-10 mixer by using a type-D wire whip and a speed setting of 2, which gave a maximum free tangential speed of 2.56 m/s (8.35 ft/s).

Mixing time was approximately 2 min or less if the aggregate appeared to be completely coated. After machine mixing, the material was transferred to a large hot metal pan to check for completeness of coating and to facilitate the scooping of individual test samples for weighing.

TEST SPECIMEN COMPACTION

The concept of a strain-attenuating layer for minimizing reflection cracking in an overlay is based on the fact that the layer has a relatively low resistance to elongation but also the elasticity to recover on the release of stress. The rubber granules in an A-R and aggregate mixture enhance the necessary functions of a strain-attenuating layer; however, these characteristics would work against compaction efforts expended for making test specimens of the mixture. In addition, because of the intermediate-to-high value of shear rate or strain rate during most standard compaction procedures, the A-R blend would have much higher viscosity than that of the asphalt to resist the compaction effort.

From the work described by Jimenez (6), the compaction temperature for the A-R and aggregate mixtures was set at 121°C even though there was no prior knowledge of probable density values achievable under construction of an SAL. Specimen size was to be standard--102 mm (4 in) in diameter by 63 mm (2.5 in) in height--and two compaction procedures were used.

Static Compaction

The Triaxial Institute (T.I.) compactor, also known as the California kneading compactor, is used for making specimens to be tested by the Hveem procedure [AASHTO T246-74 and T247-74 (5)]. After the effects of variations in compaction temperature, tamping pressure, and static loading had been determined, the following procedure was established for making specimens to be evaluated for Hveem stability and cohesiometer value.

The hot mixture was placed in a hot mold and rodded according to AASHTO T247, a double-plunger load of 178 kN (40 000 lbf) was applied, and then the specimen was left in the mold and stored at 25°C (77°F) for three days prior to being extruded. It had been noted that specimens extruded soon after compaction became swollen and cracked. For the specimens that did not crack, radial swelling was such that the specimens could not be placed inside the stabilometer shell.

The storing of a specimen in a mold for three days eliminated the radial swelling but not the swelling in the axial direction. It was noted by feel that the ends of a specimen were less dense than the middle portion was. No measurements were made to determine a density gradient along the length of a specimen.

Vibratory Kneading Compaction

With the intent of lessening the elastic resistance to compaction of an A-R mixture by a static force, vibratory compaction was used to form specimens for the Hveem tests. Our vibratory kneading compactor (VKC) has been described elsewhere (7); however, it is appropriate to present a brief discussion here.

A mold 102 mm (4 in) in diameter that contains the hot mixture is mounted on the compactor's turntable. The turntable can be tilted as well as rotated; the standard angle for compaction is 0.02 rad (1 degree) and the rotation is 25 rpm. A vibratory and impact loading is applied through a steel foot 102 mm (4 in) in diameter placed in the mold and at a frequency of 20 Hz. The impacting force is applied in a vertical direction while the tilted mold is rotating; the mold is free to slide along the surface of the turntable. The combined vertical and rotational displacements cause a kneading action during the compaction period. After the period of kneading action, the turntable is leveled and 30 s of vibratory loading are given while the mold is being rotated. This final period of vibratory loading is for the purpose of squaring the specimen.

The resilient nature of the A-R mixtures required that a modification be made to the standard VKC procedure. The modification consisted of an additional double-plunger load of 2.0 MPa (300 psi) immediately following vibratory compaction.

TESTS AND MEASUREMENTS OF COMPACTED SPECIMENS

The tests and measurements made were selected for ease of implementation and for characterization of the specimens by using recognizable units.

Density and Air-Void Content

After the three days of curing in the mold, a specimen was extruded and measured for both height and diameter. Then the specimen was weighed in air and submerged in water. The density of the open-graded specimens was determined by using the dry weight and the volume calculated from the dimensions of the specimen.

The density of the compacted dense-graded specimen was determined by using the air and the submerged weight. This procedure yields a higher density value than the one that obtains the specimen's volume from its height and diameter.

It was expected that these A-R mixtures would be used in layers not thicker than 12.7 mm (0.5 in), and, as such, density values would not have as much significance as air-void content. Air-void content value was calculated by using the effective specific gravity (ESG) of the aggregate. The ESG accounts for the asphalt absorbed by the aggregate and was calculated from the Rice specific gravity of mixtures at various binder contents and the specific gravity of the binder. For a completely coated aggregate the ESG is constant regardless of asphalt content.

Hveem Tests

The frictional and cohesive components of shear strength of the compacted mixtures were charac-

terized with the Hveem stability and cohesiometer values. The tests for the value were performed at 25°C by using the basic procedures of test methods ARIZ 803 and 804 (8). The performance of the stability test had to be slightly modified since the initial confining pressure of 34.5 kPa (5 psi) could not be held without preloading the specimen. Modification of the test consisted of preloading the specimen in the stabilometer with 0.44 kN (100 lbf) and then obtaining the required confining pressure before the preload fell to 0.27 kN (60 lbf). Following the setting of the initial confining pressure, the specimen was loaded at the prescribed rate up to 26.7 kN (6000 lbf). The cohesiometer test was performed following that for stability, and the test temperature was also 25°C.

The double-punch test (9) is an indirect tensile test in which a specimen is loaded in compression by using an axial steel punch 25.4 mm (1 in) in diameter centered on each flat face. This test followed the cohesion test.

Tensile Strength and Elasticity

Tensile strength and elasticity were measured on specimens made with the 9.5-mm dense-graded aggregate and formed by VKC. The measurements were made under static axial tension and also under a new procedure in which a repeated load was applied to a specimen by using the double-punch concept.

Static Axial Tension

Specimens were formed by VKC to a height of 82.5 mm (3.25 in) so that 9.5 mm could be cut from each end. The ends of the specimens were glued to steel discs with Devcon plastic steel B and allowed to cure for one day.

The test setup was made of ball joints and chain so that an axial load was applied by using the testing machine. Tests were performed at temperatures of 4°C and 25°C (39.2°F and 77°F) and at a crosshead speed of 25.4 mm/min. A recorder drew a graph of load versus crosshead movement.

Dynamic Modulus of Elasticity

The dynamic modulus of elasticity (E_D) was obtained by using a repeated-load double-punch procedure (10), which has shown excellent correlation with resilient-modulus (M_R) values obtained by Chevron, U.S.A. (11). In this test, the specimen is subjected to a repeated double-punch load and radial displacements are obtained at mid-height and at three points 2.1 rad (120 degrees) apart. For a standard-sized specimen the loading fluctuates sinusoidally to yield tensile stresses that range from 34.5 to 131.1 kPa (5-20 psi) at a frequency of 11.5 Hz. The low tensile strength of the A-R specimens required that the stress level be reduced and ranged from 20.0 to 82.8 kPa (2.9-12.0 psi).

Resistance to Debonding

The strength of a paving mixture is as important as its durability. A measure of this property is given by its resistance to debonding of the asphalt from the aggregate caused by the action of water and traffic on the pavement.

Our test procedure for evaluating a mixture's resistance to debonding was described by Jimenez (9). The test concept is similar to the AASHTO immersion-compression test T165-77 (5) in that a retained strength after exposure to a test environment is a measure of durability. In our debonding test, the exposure environment is a repeated pore-

Table 2. Effects of vulcanized rubber granules on physical characteristics of compacted AC specimens at 25°C.

Compaction and Binder	AC Content (%)	A-R Content (%)	Void Content (%)	CV ^a (%)	Hveem Stability (%)	CV (%)	Cohesimeter Value	CV (%)	Double-Punch Tension(psi)	CV (%)
Dense-Graded Aggregate										
Static										
AR-1000	6.0		11.0	3	39	1	340	13	51	8
	7.5		8.0	4	44	1	330	5	57	1
	9.0		5.5	6	49	4	370	3	58	5
A-R	6.0	8.0	23.0	9	19	8	160	19	11	18
	7.5	10.0	21.0	13	13	4	240	16	11	13
	9.0	12.0	21.0	14	9	13	290	7	14	19
Vibratory										
AR-1000	6.0		9.5	8	53	7	340	7	75	10
	7.5		4.0	9	49	7	360	8	87	2
	9.0		2.0	35	35	2	320	13	65	4
A-R	6.0	8.0	18.0	2	35	7	200	8	31	10
	7.5	10.0	15.0	8	23	6	220	13	33	9
	9.0	12.0	14.5	4	16	4	200	5	29	7
Open-Graded Aggregate										
Static										
AR-1000	6.0		17.5	3	23	3	230	11	22	9
	7.5		14.5	3	24	9	240	18	26	7
	9.0		12.0	9	24	7	250	11	28	4
A-R	6.0	8.0	16.5	4	22	6	170	17	10	10
	7.5	10.0	14.5	13	20	16	190	11	14	29
	9.0	12.0	11.5	14	15	13	190	8	17	6
Vibratory										
AR-1000	6.0		24.5	3	28	39	200	20	19	29
	7.5		19.5	1	33	6	240	9	25	11
	9.0		18.5	17	26	36	240	8	19	37
A-R	6.0	8.0	23.5	8	22	22	130	24	12	6
	7.5	10.0	19.0	4	25	7	160	16	14	5
	9.0	12.0	15.0	0	22	12	170	4	18	4

Note: 1 psi = 6.89 kPa.

^aCV = coefficient of variation.

water pressure that fluctuates between 34.7 and 207.0 kPa (5-30 psi) for 10 min at a frequency of 9.7 Hz and at a temperature of 50°C (122°F). The strength value is from the double-punch test performed at 25°C.

TEST RESULTS AND DISCUSSION

Results of the tests performed are given in Tables 2-5. The reader is reminded that the mixtures under investigation are not intended to have strength properties comparable with those of standard AC that would impart structural strength to a pavement. The tests used and the values obtained serve to identify or characterize these materials, whose function would be to serve as strain attenuators to minimize reflection cracking.

Hveem and Double-Punch Tests

The results of the Hveem and double-punch tests are listed in Table 2 for the dense-graded and open-graded aggregates.

Dense Gradation

The values of test results listed in Table 2 show that the vibratory compaction was more effective than the static compaction in that all corresponding values were higher for the specimens formed by VKC. For both types of compaction the addition of vulcanized rubber reduced the strength values; apparently this was because of the reduced density caused by the rubber granules. The reduction in strength values due to the addition of rubber was less under vibratory compaction. The actual values of Hveem stability and cohesimeter should not be compared with standard results since these tests

were performed at 25°C rather than at 60°C (140°F). It is significant to note that the void content of the A-R specimens was quite high--generally more than 15 percent--and that the variability of replicates was seemingly not affected by the addition of rubber.

Open Gradation

It is noted that the compaction method did not have so great an effect on density as it did for the dense-graded specimens. As noted earlier, density was calculated from linear measurements for determining volume and thus resulted in lower-than-actual density. The differences in density between the specimens of the two aggregates would not be so great as that indicated. The relatively small differences in void content indicated for the four types of specimens are attributed to the filling of voids in the open-graded aggregate by the binder and in this location its effects on compaction would be minimized. As expected, all strength values were lower than those for the dense-graded specimens. A visual summary of these data is shown in Figure 1.

Tensile Properties of Vulcanized- and Devulcanized-Rubber Mixtures

The results obtained from tensile testing of two types of rubber mixture are shown in Tables 3 and 4. Table 3 presents data for specimens tested under static and dynamic conditions. The specimens were made with the 9.5-mm dense gradation with vulcanized rubber and formed by using vibratory compaction. Table 4 presents the data obtained for specimens made with devulcanized rubber and the two aggregate blends.

Vulcanized-Rubber Mixtures

Table 3 presents data for comparing effects of temperature, method of testing, and rubber granules on tensile properties of the dense-graded aggregate. In general, the addition of rubber reduced the modulus of elasticity and tensile strength as compared with the soft (AR-1000) specimens. Under the static loading, the modulus of elasticity for the straight asphalt was 75.8 MPa (11 000 psi) at 4°C (39.2°F) and was reduced to about 51.7 MPa (7500 psi) for the rubberized mixture. Both mixtures were too soft to obtain a measure of modulus at 25°C.

Under dynamic loadings, the modulus of elasticity increased for both mixtures; however, the percentage

increase was greater for the rubberized specimens. The asphalt specimens showed an increase by a factor of about 20 and the rubberized ones by a factor of about 26 to yield an E_D of about 1380 MPa (200 000 psi) at 4°C.

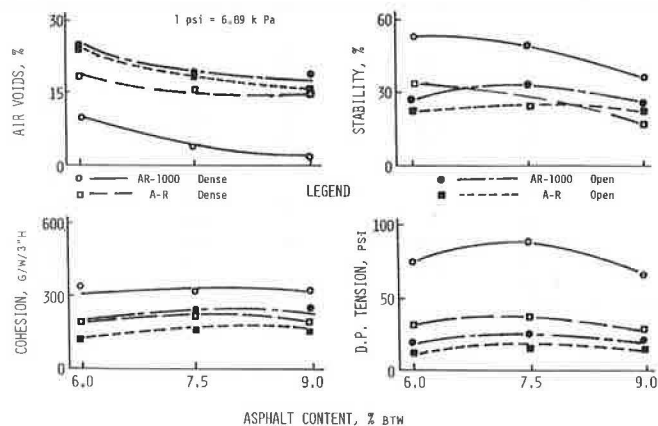
Although the effects of temperature on strength and modulus were greater for the rubberized mixtures, the absolute values for the A-R specimens were all less than those for the mixture containing the very soft AR-1000 asphalt.

The data in Table 3 are presented in Figure 2 for ease of comparison.

Devulcanized-Rubber Mixtures

The data in Table 4 show that the static tension test was not used with these mixtures. However, the open-graded aggregate was included for these tensile measurements.

Figure 1. Effects of asphalt content and vulcanized rubber on Hveem specimens by using dense and open gradation by vibratory compaction.



Air Voids

Examination of the air-void data shows that approximately equal density was obtained with the AR-4000 asphalt as was the case for the AR-1000 specimens compacted by VKC; this was so for both aggregates. However, it is noted that the density of the devulcanized-rubber (AR-S) specimens was much higher than that of the A-R specimens; again, this was so for both aggregates. Note that the void content for the dense-graded specimen was comparable with that of regular AC. The reader is reminded that the AR-S binder contains 1.6 percent of an extender oil, which would serve to reduce the viscosity of the AR-4000 asphalt.

Modulus of Elasticity

The dynamic modulus of elasticity (E_D) values

Table 3. Effects of vulcanized rubber granules on tensile characteristics of dense-graded AC specimens formed by using vibratory compaction.

Binder and Temperature	AC Content (%)	A-R Content (%)	Void Content (%)	CV ^a (%)	Static $E_D^b \cdot 10^{-3}$ (psi)	CV (%)	Dynamic $E_D^b \cdot 10^{-5}$ (psi)	CV (%)	Strength (psi)	CV (%)	
Static Axial Tension											
AR-1000 4°C	6.0		10.0	8	11.1	1			425	1	
	7.5		5.0	5	11.3	7			483	3	
	9.0		2.0	13	11.1	3			519	3	
	25°C	6.0		10.5	3	Too weak	Too weak			62	7
		7.5		5.0	12	Too weak	Too weak			92	8
		9.0		3.0	17	Too weak	Too weak			78	4
AR 4°C	6.0	8.0	14.5	9	7.4	22			145	6	
	7.5	10.0	15.5	13	7.9	6			126	3	
	9.0	12.0	15.0	5	7.2	6			132	14	
	25°C	6.0	8.0	15.0	20	Too weak	Too weak			11	33
		7.5	10.0	14.5	3	Too weak	Too weak			11	20
		9.0	12.0	14.5	12	Too weak	Too weak			15	13
Dynamic Double-Punch Tension											
AR-1000 4°C	6.0		12.0	0			2.3	10	296	2	
	7.5		5.5	5			2.2	13	397	2	
	9.0		4.0	8			1.5	12	300	4	
	25°C	6.0		12.5	7			2.1	20	79	20
		7.5		5.5	20			2.1	13	100	12
		9.0		4.0	27			1.6	14	93	20
A-R 4°C	6.0	8.0	18.0	6			1.8	21	47	15	
	7.5	10.0	15.0	2			2.9	15	69	4	
	9.0	12.0	15.0	7			1.4	15	74	7	
	25°C	6.0	8.0	19.0	5			1.2	13	27	2
		7.5	10.0	15.0	8			1.1	23	25	16
		9.0	12.0	18.0	6			1.7	32	17	32

Note: 1 psi = 6.89 kPa; $t^{\circ}C = (t^{\circ}F - 32)/1.8$.

^aCV = coefficient of variation.

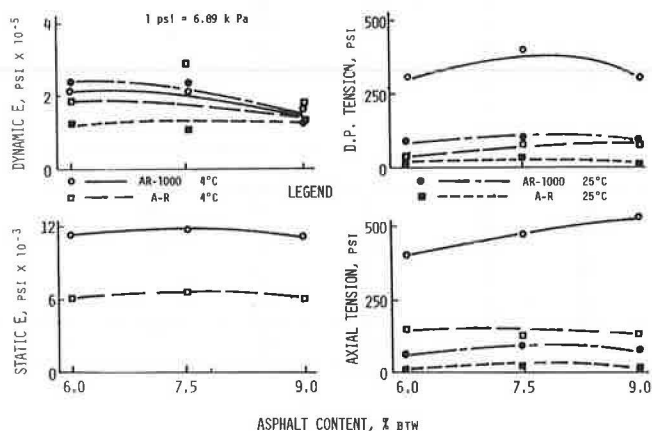
^b E_D = modulus of elasticity.

Table 4. Effects of devulcanized rubber granules on tensile characteristics of vibratory compacted AC specimens.

Binder and Temperature	AC Content (%)	A-R Content (%)	Void Content (%)	CV ^a (%)	Dynamic E _D ^b - 10 ⁻⁵ (psi)	CV (%)	Strength (psi)	CV (%)
Dense-Graded Aggregate								
AR-4000								
4° C	6.3		13.0	42	1.7	43	344	11
	7.8		5.0	22	2.2	34	404	12
	9.4		2.5	20	2.1	16	409	3
25° C	6.3		9.5	6	0.6	10	157	10
	7.8		5.5	5	1.4	24	138	6
	9.4		2.5	35	1.8	31	119	9
AR-S								
4° C	6.3	8.0	8.0	10	1.2	38	317	0
	7.8	10.0	4.0	8	0.8	13	353	13
	9.4	12.0	1.0	0	1.3	16	367	6
25° C	6.3	8.0	8.0	6	0.5	47	129	23
	7.8	10.8	3.0	10	0.9	22	137	1
	9.4	12.0	1.0	0	0.8	20	89	10
Open-Graded Aggregate								
AR-4000								
4° C	6.3		25.0	8	2.3	29	97	21
	7.8		16.0	11	2.9	11	202	12
	9.4		12.5	39	2.2	14	216	25
25° C	6.3		24.0	21	1.4	46	50	18
	7.8		16.5	11	1.8	33	66	5
	9.4		16.5	26	1.3	26	48	45
AR-S								
4° C	6.3	8.0	18.5	5	1.8	18	155	4
	7.8	10.0	12.0	12	1.7	9	215	13
	9.4	12.0	11.5	4	2.3	4	191	14
25° C	6.3	8.0	16.0	3	1.5	10	53	8
	7.8	10.8	15.0	0	2.3	18	42	9
	9.4	12.0	10.5	24	1.0	13	51	9

Note: 1 psi = 6.89 kPa; $t^{\circ}C = (t^{\circ}F - 32)/1.8$.
^aCV = coefficient of variation. ^bE_D = modulus of elasticity.

Figure 2. Effects of asphalt content, vulcanized rubber, and temperature on tensile properties of dense-graded specimens by vibratory compaction.



obtained for the AR-S specimens were lower than those for the A-R specimen. The values shown in Table 4 were obtained from a modification of our double-punch procedure. A study of our method for determining E_D showed the following relationship with values obtained by using the Chevron, U.S.A. (11) method for obtaining modulus of resiliency M_R:

$$M_R = 2.29E_D + 168(\text{ksi}) \quad (3)$$

By using Equation 3, E_D-values for the AR-S specimens are transformed to M_R-values that range from 1946 MPa (282 500 psi) at 25°C to 3209 MPa (465 700 psi) at 4°C for the dense-graded aggregate.

A close inspection of the E_D-values in Table 4 shows that the open-graded specimens with AR-S were

stiffer than the dense-graded ones. This is an interesting finding, since one would expect a less-dense specimen to have a lower modulus. Perhaps this behavior can be attributed to the supposition that because of the larger air voids in the open-graded aggregate, the rubber particles filled air-void space and the aggregate particles were coated with asphalt only. It is noted that the modulus values for the asphalt-only, open-graded specimens were comparable with those for the AR-S specimens at 25°C.

Tensile Strength

The results of the double-punch tensile strength listed in Table 4 show that the AR-S specimens had fairly high values. For the dense-graded aggregate the strength ranged from 620 kPa (90 psi) at 25°C to 2480 kPa (360 psi) at 4°C. In comparison with these strengths, the A-R specimens had corresponding values of 117-510 kPa (17-74 psi), which were even lower than the strength obtained for the open-graded specimens made with AR-S.

The following listing serves to compare the elastic modulus of these rubberized materials with that of AC determined at 4°C (1 MPa = 145 psi):

Material	M _R (MPa)	E _D (MPa)
AC	5257	1791
Dense-graded 9.5-mm A-R	4313	1378
Dense-graded 9.5-mm AR-S	3209	896
Open-graded 9.5-mm AR-S	4786	1585

Some of the data in Tables 3 and 4 are summarized and presented graphically in Figure 3.

Resistance to Debonding of Vulcanized-Rubber Specimens

The results of the debonding test on both the dense-

Figure 3. Effects of asphalt content, rubber type, and temperature on tensile properties of dense-graded specimens by vibratory compaction.

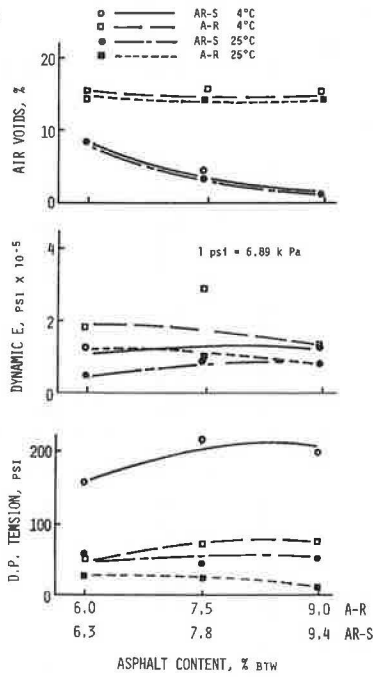


Figure 4. Effects of asphalt content on debonding test on dense- and open-graded A-R specimens by vibratory compaction.

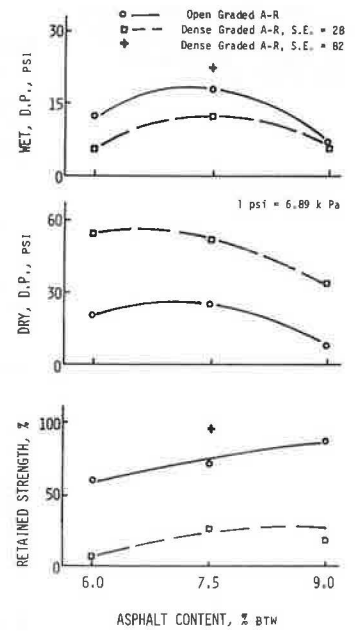


Table 5. Effects of vulcanized rubber granules on resistance to debonding of vibratory compacted AC specimens.

Binder	AC Content (%)	A-R Content (%)	Void Content (%)	CV ^a (%)	Double-Punch Failure Stress				Retained Strength (%)	Sand Equivalence Value
					Wet (psi)	CV (%)	Dry (psi)	CV (%)		
Open-Graded Aggregate										
AR-1000	6.0	-	25.5	7	21	15	28	7	75	
	7.5	-	20.0	6	23	4	33	12	70	
	9.0	-	25.5	7	8	22	6	0	100	
A-R	6.0	8.0	22.5	4	12	22	20	12	60	
	7.5	10.0	18.5	3	17	17	24	7	71	
	9.0	12.0	23.5	8	7	22	8	7	87	
AR-1000	7.5	-	-	-	-	-	-	-	-	
A-R	7.5	10.0	-	-	-	-	-	-	-	
Dense-Graded Aggregate										
AR-1000	6.0	-	10.0	16	3	19	73	9	4	28
	7.5	-	4.0	7	6	25	80	21	8	28
	9.0	-	2.0	14	31	5	49	10	63	28
A-R	6.0	8.0	15.5	9	5	11	54	9	9	28
	7.5	10.0	12.5	16	12	8	52	3	23	28
	9.0	12.0	11.0	30	6	33	32	17	19	28
AR-1000	7.5	-	5.5	9	51	6	66	5	17	82
A-R	7.5	10.0	14.0	11	22	11	24	11	96	82

Note: 1 psi = 6.89 kPa.
^aCV = coefficient of variation.

and the open-graded specimens are shown in Table 5. It is quite apparent that high void content and low sand equivalent value resulted in very low retained strength for the dense-graded specimens. It appears that the relative lack of dirty fines in the open-graded specimens resulted in high values of retained strength, although the dry strengths were much lower than those for the comparable dense-graded specimens (Figure 4).

Table 5 also shows that when the dense-graded aggregate was modified to have less fines and a sand equivalent value of 82, the retained strength was increased tremendously (to 96 percent) for the A-R specimens. The dry strengths were reduced because the reduction in fines yielded higher air-void values; the wet strength increased because of the cleanness of the aggregate. The data indicate that

because of the resulting high air-void content of A-R mixtures, the aggregates must be quite clean if the mixtures are to serve effectively as an SAL. It was noted that the 9.5-mm dense-graded specimens made with AR-S had a much lower air-void content than those made with A-R. As a consequence, we would expect the AR-S mixture to have better resistance to debonding than the A-R specimens.

CONCLUSIONS

The aim of the study was to make and test laboratory-prepared specimens in order to characterize A-R and aggregate mixtures by using results from standard test procedures. The A-R concrete is intended to be used as an SAL interlayer to minimize reflection cracking in an overlay and, as a consequence,

it was not expected to have strength values comparable with those of normal AC. Within the bounds of the experimentation and the materials used, the following conclusions are warranted for the making and testing of A-R concrete specimens:

1. Good coating of the 9.5-mm dense-graded and open-graded aggregates was obtained when the aggregate was at a temperature of 149°C (300°F) and the A-R was at 121°C. A Hobart C-10 food mixer was used with a type-D wire whip and medium speed. This type of mixer is commonly used in many laboratories.

2. Compaction of standard-sized specimens could not be effected by using the tamping-foot procedure of the California kneading compactor at a temperature of 121°C. Specimens could be formed by using static double-plunger compaction and also by using our VKC, which gave higher densities. The Marshall compaction procedure was not attempted.

3. It was necessary to leave the hot-compacted specimen in the mold for three days at ambient temperature prior to extrusion in order to eliminate swelling of the unconfined specimen that would cause cracking.

4. The air-void content for the 9.5-mm dense-graded specimens had a much higher value when the specimens were mixed with the vulcanized rubber than when they contained devulcanized rubber.

5. Specimens made by using both aggregates and the vulcanized rubber had Hveem stability values that ranged between 25 and 35 and cohesiometer values between 170 and 200 when tested at 25°C.

6. Dynamic modulus of elasticity values for both aggregates and both types of rubber were approximately 70 percent of that for regular AC. The lowest modulus at 4°C was 896 MPa (130 000 psi).

7. Due to the high void content of the 9.5-mm dense-graded specimen made with the vulcanized rubber, it will be necessary for the aggregate to be very clean in order to obtain good resistance to debonding from the action of water.

ACKNOWLEDGMENT

The initiation and support given to this study by personnel of the Arizona Department of Transportation are acknowledged. Special mention should be made of university students David Bell and Armando

Murrieta, who worked so carefully in an area so new and unpredictable. I acknowledge the sponsorship of the program by the Arizona Department of Transportation and the Federal Highway Administration.

REFERENCES

1. R.A. Jimenez, G.R. Morris, and D.A. DaDeppo. Tests for a Strain-Attenuating Asphaltic Material. Proc., AAPT, Vol. 48, 1979, pp. 163-187.
2. G.R. Morris and C.H. McDonald. Asphalt-Rubber Stress-Absorbing Membranes: Field Performance and State of the Art. TRB, Transportation Research Record 595, 1976, pp. 52-58.
3. Supplemental Specifications to the Arizona Highway Department Standard Specifications for Road and Bridge Construction. Arizona Department of Transportation, Phoenix, 1978.
4. R.A. Jimenez. Testing Methods for Asphalt-Rubber. Arizona Department of Transportation, Phoenix, Rept. ADOT-RS-15(164), 1978.
5. Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II. AASHTO, Washington, DC, 1978.
6. R.A. Jimenez. Testing of Asphalt-Rubber and Aggregate Mixtures. Arizona Department of Transportation, Phoenix, Rept. FHWA/AZ-79/111, May 1979.
7. R.A. Jimenez. Fatigue Testing of Asphaltic Concrete Slabs. In Fatigue of Compacted Bituminous Aggregate Mixtures, ASTM, Philadelphia, Special Tech. Publ. 508, 1972, pp. 3-17.
8. Materials Testing Manual. Materials Division, Arizona Department of Transportation, Phoenix, 1975.
9. R.A. Jimenez. Testing for Debonding of Asphalt from Aggregates. TRB, Transportation Research Record 515, 1974, pp. 1-17.
10. R.A. Jimenez. Structural Design of Asphalt Pavements. Arizona Department of Transportation, Phoenix, Rept. ADOT-RS-13(142), May 1975.
11. W. White. Relationship Between Values of Dynamic and Resilient Modulus for Asphaltic Concrete. Univ. of Arizona, Tucson, C.E. 200 Rept., 1978.

Publication of this paper sponsored by Committee on Characteristics of Non-bituminous Components of Bituminous Paving Mixtures.

Field Evaluation of Rubber-Modified Bituminous Concrete

JACK E. STEPHENS

The flexibility and, as a result, the durability of bituminous concrete placed as an overlay over old pavement can be increased by the addition of reclaimed rubber. The conditions under which the modified characteristics are most beneficial have not been well defined. Within the maintenance program of the Connecticut Department of Transportation, reclaimed-rubber test sections were placed at nine locations. The locations were selected to include three levels of traffic from a low of 1300 average daily traffic (ADT) to a high of 10 400 ADT and three levels of pavement condition (low, medium, and high). At each location, mixes in which the rubber content was 0, 1, and 2 percent of the mix were placed. Comparisons were made of permeability, density, skid number, and crack development over three years.

The planning for this study was formally started by

a group that consisted of the Office of Research of the Connecticut Department of Transportation (ConnDOT), the Civil Engineering Department of the University of Connecticut (UConn), the Solid Waste Section of the Department of Environmental Protection, and the Reclaiming Division of Uniroyal, Inc. ConnDOT, UConn, and the Rubber Reclaimers Association had carried out a laboratory study in which reclaimed rubber was added to asphalt paving mixes (1). The results from that work indicated that reclaimed rubber added to mixes in the laboratory significantly improved the properties of pavement

Table 1. Overlay test sections.

Pavement Distress Level	Traffic Level								
	Low			Medium			High		
	Route No.	Location	1978 ADT	Route No.	Location	1978 ADT	Route No.	Location	1978 ADT
Low	354	Colchester	1300	66	Marlborough	7600	44A	Coventry	9 500
Medium	85	Bolton	3700	2	Preston	4600	44	Barkhamsted	9 600
High	354	Salem	1300	69	Woodbridge	5000	101	Dayville	10 400

Figure 1. Temperature cracks in sections with low distress level.

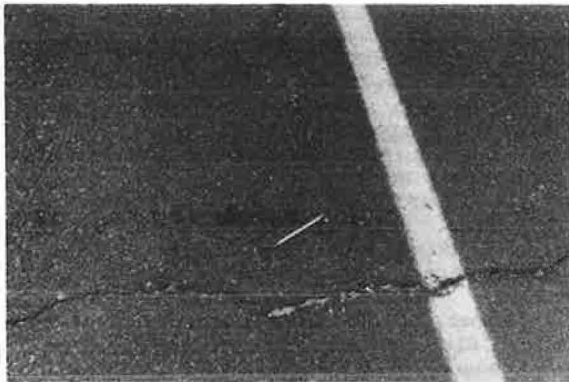


Figure 2. Temperature and age cracks.



mixes. Subsequent discussions resulted in an agreement between UConn and ConnDOT to undertake full-scale evaluations of rubber-modified mixtures. The Civil Engineering Department and the Office of Research would develop the program and the Civil Engineering Department would carry out the evaluations. The materials would be placed within the maintenance program of the Bureau of Highways. Mix design would be carried out by the Civil Engineering Department and quality control by the ConnDOT

materials laboratory. The Office of Research would obtain skid numbers. The Federal Highway Administration (FHWA) would provide partial funding to ConnDOT for the project.

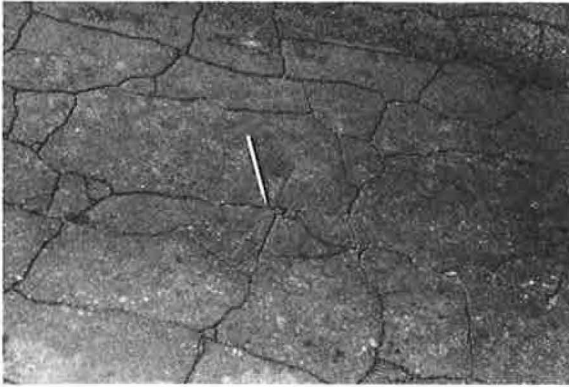
A bituminous concrete overlay over a cracked pavement is subjected to high stress concentrated in the volume of material immediately over each crack. If the strength of an overlay cannot be made great enough to resist these stresses, then sufficient flexibility of the mix is critical for long life of the overlay. The extent to which the overlay can be altered by the use of a softer rubber is limited by the stability of the resulting mix at summer temperatures.

The addition of reclaimed rubber increases the flexibility of the mix through two different mechanisms. The modified asphalt is less sensitive to temperature and a softer grade can be used; equally important, however, is the increased binder-film thickness. The potential for improvement of overlays and seal coats is apparent. However, there is a limit to the crack motion that can be accommodated by even the rubber-modified asphalt. Thus, a full-scale field-test program was needed to determine the effective improvement.

The first phase of placing test sections of rubber-modified bituminous concrete consisted of selecting test sites. Sections planned for overlays were reviewed for condition, volume of traffic, and construction problems. The plan called for selection of locations that had three levels of average daily traffic (ADT) and three levels of pavement condition (Table 1). A pavement of adequate structural strength cracks due to temperature stresses. Such cracks tend to create the effect of dividing the pavement into large blocks. Substantial movement occurs at these cracks and reflection cracks occur in the overlay. A typical pavement that exhibits a block pattern of cracks is shown in Figure 1. The width of these cracks changes sharply with temperature; the cracks expand and contract. Such pavements were used for good-condition sections. Further aging results in progressive formation of cracks, each at right angles to an existing crack or edge (Figure 2). Such cracks do not show much orientation with the wheel paths. These represented the record level of deterioration. Pavements similar to those in Figure 3 were selected as badly cracked test sections. Due to the frequency of such cracks, temperature does not cause significant changes in crack width.

Locations were selected in which conditions were constant for several hundred feet and from lane to lane. On most of the secondary roads used, the cross section had been modified repeatedly through the addition of widening strips and overlays. The support then varied across the width of the road and changes occurred within the current lanes. Where practical, the control and test sections were in the same lane. Since the precise location of test sections was not known in advance of the paver, precise crack surveys of prior condition could not be made and comparisons are therefore between the condition

Figure 3. Sections with high distress level.



of the control and that of the test sections at the same age.

MIX DESIGN

Since the rubber-modified mix was not a bid item and required maximum cooperation from the contractor, the changes were held to a minimum. The approved mix for each contractor became the starting point for developing the rubber-modified mix. The work done several years earlier in the University laboratories had resulted in criteria for such mixes. For most local aggregates, an optimum mix used 1-2 percent rubber and 0.25 percent more binder for each 1 percent rubber. Marshall tests were run on the approved mix, 1 percent reclaimed rubber and 0.25 percent more asphalt were added, and the tests were repeated. A third trial was made with 2 percent rubber and 0.50 percent additional asphalt. For most mixes, the Marshall results were within ConnDOT specifications, and the mixes were used without further modifications.

When it was recognized that the addition of both rubber and asphalt increased the possibility of overfilling the voids, the standard Marshall computations for the voids in the mineral aggregate and the percentage of such voids filled were not considered adequate. The accepted criteria are based on experience with a normal binder. The rubber-modified mix that used reclaimed rubber is quite different. If the rubber is only partly dissolved in the asphalt, it creates a stringy web within the asphalt, which stabilizes the binder and appears to permit the use of thicker films. Past experience with limits on the voids and percentage filled may not be applicable here. For this reason, each mix was placed in a gyratory compactor and subjected to 500-600 cycles as a test for long-term stability (2). A small number of the mixes did prove unstable under this test. Since the ConnDOT master specification for binder content of class 1 mixes ranges from 5.8 to 8 percent, the addition of 1 percent rubber and 0.25 percent asphalt to a mix already at the high end was significant. This problem was overcome for those mixes by not adding more asphalt. Of course, the same possibilities existed for the 2 percent rubber mixes and the extra asphalt was reduced by increments of 0.25 percent as indicated by the gyratory test. In no instance was the asphalt reduced below the content of the ConnDOT-approved mix (3).

In general, the mixes used for any one location were the producer's ConnDOT-approved mix, this same mix with 1 percent rubber and 0.25 percent binder added, and this same mix with 2 percent rubber and

0.50 percent binder added. The characteristics of the mixes used are listed in Table 2.

MIXING

Since at least 7 of the 41 plants operating in Connecticut would be supplying material for test sections, procedures that require significant plant modification could not be used. Uniroyal had successfully placed a number of rubber-modified running tracks by using reclaimed rubber. The same procedure of adding the reclaimed rubber to the aggregate and asphalt in the pug mill was used in this program. The presoftened character of the reclaimed rubber makes possible a degree of solution in the binder during the short time in the mixer. Most plants had inspection ports through which the rubber could be introduced into the pug mill, and owners obligingly cut such openings in plants that did not have them.

The reclaimed rubber was packed in 50-lb plastic bags. Batch sizes were adjusted to use an even number of bags. The pug mill was charged with aggregate and binder, and the bags of rubber were added. The heat in the mixer completely destroyed the bags by the time the rubber was uniformly distributed through the mix. Hauling and spreading was carried out as for any other mix. When the paving machine reached the test area, the preceding standard material was used up as much as was practical before the test mix was started, thus keeping transitions short. Rolling was accomplished by using a steel breakdown roller and then a rubber-tired roller.

EVALUATIONS OF OVERLAYS

The locations of the test sections were not intended to be representative of each roadway but were selected as having consistent conditions of traffic volume and pavement distress. In an area of shallow cuts and low fills, the relative elevation of grade line and land profile can change several times in the 1500 ft required for three sections. Road widths have gradually increased and alignments improved over the years. Current pavements are not always centered over old pavements. Where widening strips have been used, support conditions can vary across the width of the pavement. Where such undesired variables became apparent, the control section was adjusted to assure maximum usefulness of the comparisons. Tables 3-5 summarize the crack data. Since the length of the different sections varied, crack frequency has been tabulated in terms of feet of crack per foot of paver pass; transverse and longitudinal values are reported separately. By August 1980, no general deterioration had occurred and so none was included in the evaluation.

Since there were no replicate sections, the number of data points for any one set of conditions (traffic volume, pavement distress, rubber level) is too small for significant statistical analysis of individual sections. After each thick overlay had been reviewed, overall comparisons were made for different rubber levels.

Route 2, Preston

At the test site, the old bituminous concrete pavement had been widened at an earlier date to provide a third (climbing) lane. The longitudinal crack developing during the three-year evaluation of the 1.5-in overlay is a reflection of the edge of the old pavement and can be expected to appear over the full length of the old test sections. The transverse cracking is also a reflection of underlying

Table 2. Preliminary laboratory tests of thick overlays.

Route No. and Location	Anticipated Rubber Content (%)	Advance Laboratory Tests							Control Tests	
		Marshall			Flow (0.01 in)	Gyratory Compaction Index	Stability Index		Bulk Specific Gravity	Density (lb/ft ³)
		Asphalt (%)	Stability (lbf)	200 Rev.			600 Rev.			
2, Preston	0	5.3	2400	8	0.983	1.67	3.1	2.410	150.4	
	1	5.75	2340	9	0.986	1.91	1.9	2.409	150.3	
	2	6.0	1320	9	0.989	1.76	2.2	2.374	148.1	
44A, Coventry	0	5.5	3010	10.5	0.981	1.33	3.1	2.528	157.7	
	1	5.75	2340	13.5	0.941	1.24	1.9	2.458	153.3	
	2	6.0	2350	13.5	0.988	1.26	2.2	2.397	149.6	
66, Marlborough	0	5.5	3090	10.5	0.992	1.69	1.98	2.517	157.1	
	1	5.75	2340	13.5	0.979	1.24	1.5	2.430	151.6	
	2	6.0	2930	13.5	0.988	1.26	1.5	2.418	150.9	
85, Bolton	0	5.5	3010	10.5	0.981	1.33	3.1	2.572	152.9	
	1	5.75	2340	13.5	0.941	1.24	1.9	2.427	149.9	
	2	6.0	2350	13.5	0.988	1.26	2.2	2.377	152.2	
44, Barkhamsted	0	6.0						2.450	152.9	
	1	6.0						2.402	149.9	
	2	6.0						2.439	152.2	
354, Colchester	0	6.0	2330	13	0.975	1.45	1.6	2.611	162.9	
	1	6.0	2390	12	0.987	1.26	2.0	2.491	155.4	
	2	6.0	2040	13.5	0.989	1.33	1.4	2.334	145.6	
69, Woodbridge	0	5.5	3670	13.5	0.983	1.76	2.9	2.426	151.4	
	1	5.75	2790	12.0	0.986	1.70	1.5	2.463	153.7	
	2	6.0	2320	12.0	0.984	1.92	1.8	2.409	150.3	
101, Killingly	0	5.5	2600	8	0.948	1.86	0.24	2.375	148.2	
	1	5.75	2590	10	0.949	1.45	1.8	2.378	148.4	
	2	6.0	1220	12	0.986	1.97	72.5	2.355	147.0	
354, Salem	0	5.5	2790	5.5	0.982	1.30	1.7	2.467	135.3	
	1	5.75	2170	10.5	0.993	2.29	2.4	2.351	135.0	
	2	6.0	1510	10	0.986	2.11	73.4	2.300	133.6	

Table 3. Cracks in 1 percent and control sections.

Route No. and Location	Rubber Content (%)	1979			1980		
		Section Length (ft)	Crack Length (ft/ft of pass)		Section Length (ft)	Crack Length (ft/ft of pass)	
			Longitudinal	Transverse		Longitudinal	Transverse
2, Preston	1	490	0.0163	0.1469	490	0.0653	0.2796
	0	213	0.0084	0.2864	500	0.492	0.530
44, Barkhamsted	1	383	0.274	0.334	383	0.316	0.360
	0	240	0.358	0.467	435	0.524	0.423
44A, Coventry	1	607	0	0	612	0.008	0
	0	600	0	0	600	0.142	0
66, Marlborough	1	790	0.270	0.287	790	0.299	0.297
	0	0	0	0	646	0	0.272
69, Woodbridge	1	403	0.062	0.206	402	0.0597	0.2114
	0	403	0.2804	0.3648	402	0.2761	0.3781
85, Bolton	1	436	0.0160	0.0917	436	0.0367	0.0963
	0	436	0.0711	0.0092	436	0.2294	0
101, Killingly	1	630	0.0127	0.0063	630	0.0127	0.0412
	0	663	0	0.0618	663	0	0.0769
354, Colchester	1	515	0.0447	0.2000	509	0.1984	0.2043
	0	425	0.2188	0.1247	645	0.2822	0.1225
354, Salem	1	540	0.222	0.0370	540	0.400	0.048
	0	300	0.1267	0.0433	500	0.134	0.1180

cracks. These cracks are much more frequent in that portion of the lane over the old pavement. The portion over the widening strip is nearly free of cracks. There was a distinctive difference in the performance of the sections that contained 0, 1, and 2 percent rubber during the period of observation.

At two years, the pavement with 0 percent rubber was performing as well as or better than the rubber sections. During the third year, the cracks in the section with 0 percent rubber adjacent to the section with 1 percent rubber increased sharply and by the end of the year exceeded several times the quantity in the section with 1 percent rubber (Table 3). The cracks in the section with 2 percent rubber and in the corresponding area with 0 percent rubber also increased but more erratically. Considering

the 1 percent rubber, the cracks at two years are only two-thirds of those in the controls and the rate of increase during the third year is two-thirds that of the controls. During the first two years, the section with 1 percent rubber had sustained one-third less cracks than the average of the two controls. Equally significant is the fact that the rate of cracking also averaged one-third less during the third year.

Route 44, Barkhamsted

This section, 1.5 in thick, was placed on an old overlay over a two-lane concrete pavement. The resurfacing was carried out in three passes of the paver. As a result, the degree of reflection crack-

Stability (Ibf)	Flow (0.01 in)	Asphalt (%)	Rubber (%)	Gradation (% passing)				No. 4	No. 8	No. 50	No. 200
				3/4	1/2	3/8					
2880	8.5	5.6	0	100	97.2	70.9	50.2	40.8	16.2	3.7	
2480	11	6.5	1.3	100	100.0	85.8	63.1	52.2	22.2	6.4	
2140	8.5	6.4	1.6	100	92.6	70.2	53.6	45.0	20.0	5.7	
1575	12	5.4	0	98	93	74	57	43	16	4	
2180	8	6.2	1.8	100	93.8	69.1	56.1	45.5	17.5	3.9	
2330	11.5	5.7	1.7	100	90.2	65.3	51.7	42.0	17.8	4.0	
3010	12.5	6.8	0	100	100.0	74.5	50.4	39.4	15.8	5.4	
3120	10.5	6.7	1.8	100	98.4	75.4	63.4	51.5	19.6	4.5	
3300	11.5	7.1	2.2	100	98.1	71.9	55.4	43.8	17.0	4.6	
2400	12.5	5.5	0	100	95	77	56	45	18	5	
1760	10.5	5.8	1.5	100	87.3	63.8	53.3	42.3	16.8	3.7	
2190	10.5	6.4	1.7	100	98.9	69.2	58.4	45	17	3.8	
2400	12.5	5.5	0	100	95	73	53	43	16	4.4	
1760	10.5	7.2	0.9	100	95.4	71.9	52.4	44.9	19.9	4.4	
2190	10.5	5.6	1.6	100	96.3	66.4	43.5	36.4	14.9	2.7	
1707	11	6.0	0	100	92	74	53	43	17	4	
2250	10.5	6.6	1.3	100	98.5	78.9	58	48.2	22.8	6.6	
1200	15	6.7	1.9	100	92.5	84.0	58.2	44.2	16.4	4.7	
2590	9.5	6.0	0	100	100	81.9	62.9	52.5	16.9	3.7	
2250	10.5	7.5	1.1	100	97	88.1	68.1	56.2	18.7	5.1	
1770	11	7.8	2.0	100	98.2	87.3	64.6	52.4	18.4	4.7	
2880	8.5	5.7	0	100	100	80.1	52.9	42.7	15.8	4.0	
2500	12	6.9	1.2	100	97	72.6	54.7	45.3	18.3	5.9	
2500	11	8.3	1.7	100	98.7	85.5	63.2	52.4	19.2	4.8	
2754	10	6.8	0	100	100	74.5	50.4	39.4	15.8	5.4	
2870	9.5	6.7	1.8	100	98.4	75.4	63.4	51.5	19.6	4.5	
2080	11	7.1	2.2	100	98.1	71.9	55.4	43.8	17.0	4.6	

Table 4. Cracks in 2 percent and control sections.

Route No. and Location	Rubber Content (%)	Section Length (ft)	1979		1980		
			Crack Length (ft/ft of pass)		Crack Length (ft/ft of pass)		
			Longitudinal	Transverse	Longitudinal	Transverse	
2, Preston	2	500	0.134	0.1148	506	0.2253	0.3559
	0	500	0	0.186	427	0.1710	0.2579
44, Barkhamsted	2	456	0.508	0.314	456	0.492	0.314
	0	400	0.800	0.335	500	0.855	0.356
44A, Coventry	2	680	0.153	0.044	680	0.168	0.075
	0	680	0.003	0.022	680	0.010	0.063
66, Marlborough	2	616	0	0.203	616	0	0.0235
	0	297	0	0.23	646	0	0.272
69, Woodbridge	2	412	0.0121	0.2233	412	0.051	0.2937
	0	412	0.0510	0.2573	412	0.1141	0.4126
85, Bolton	2	586	0.3038	0.1348	595	0.3899	0.0924
	0	586	0.3127	0.1999	595	0.4370	0.2017
101, Killingly	2	690	0	0	690	0.0406	0.0145
	0	300	0	0.027	300	0	0.033
354, Colchester	2	578	0.2247	0.3685	578	0.2820	0.3927
	0	350	0	0.2029	658	0.0182	0.2342
354, Salem	2	528	0.4527	0.0322	528	0.6572	0.1212
	0	300	0.1267	0.0433	500	0.134	0.1180

ing must be compared within one pass. The center pass, in which the rubber was used, contains the center-joint reflection crack. Compared with cracking in overlays on bituminous concrete pavements, the cracking occurred early and there was little increase from 1979 to 1980. The maximum reflection cracks expected are 1 ft/ft of lane along the center line and 15 ft transversely at 40-ft intervals. This is 1.0 ft longitudinally and 0.337 ft transversely per foot of lane. For the control, 70 percent of the center joint and 100 percent of the transverse cracks have reflected through. For the section with 1 percent rubber, the amounts are 32 percent longitudinally and 100 percent transversely and for the section with 2 percent rubber, 49 percent longitudinally and 100 percent transversely. Both are better than the control sections. Due to

terrain variations, control sections have been included before and after the rubber sections.

Route 44A, Coventry

The existing pavement consisted of several inches of bituminous concrete. The predominant distress was an occasional temperature crack. Both the mixtures that contained rubber were mixed late in the afternoon and stored overnight in heated bins. Cracking since the application of the 1.5-in overlay has been slow to develop, which makes this one of the most successful of the test sections. Cracking amounts to only a foot or so per 100 ft of lane, and any differences are probably due to local differences independent of rubber level. Both sections reach from curb to center line of the westbound lane.

Table 5. Total cracks, 1980.

Route No. and Location	Rubber Content (%)	1 Percent Rubber		2 Percent Rubber		
		Total Cracks (ft/ft of pass)	Change from 0 Percent (ft/ft of pass)	Rubber Content (%)	Total Cracks (ft/ft of pass)	Change from 1 Percent (ft/ft of pass)
2, Preston	1	0.344 9	0.667 1	2	0.581 2	-0.157 3
	0	1.022		0	0.423 9	
44, Barkhamsted	1	0.670	0.277	2	0.806	0.405
	0	0.947		0	1.211	
44A, Coventry	1	0.008	0.134	2	0.243	-0.17
	0	0.142		0	0.073	
66, Marlborough	1	0.596	-0.324	2	0.023 5	0.248 5
	0	0.272		0	0.272	
69, Woodbridge	1	0.271 1	0.383 1	2	0.344 7	0.182
	0	0.654 2		0	0.526 7	
85, Bolton	1	0.133 0	0.096 4	2	0.482 3	0.156 4
	0	0.229 4		0	0.638 7	
101, Killingly	1	0.053 9	0.023	2	0.055 1	-0.022 1
	0	0.076 9		0	0.033	
354, Colchester	1	0.402 7	0.002	2	0.674 7	-0.422 3
	0	0.404 7		0	0.252 4	
354, Salem	1	0.448	-0.196	2	0.778 4	-0.526 4
	0	0.252		0	0.252	

Note: The minus signs indicate more cracking in the rubber section than in the control.

Most of the longitudinal cracking occurs near the outer edge, which suggests lack of lateral support.

Route 66, Marlborough

Like the Barkhamsted section, the 12-ft sections of 1.5-in overlay are in the center pass over an old overlay on concrete. In contrast to Route 44, Barkhamsted, the roadway here is not an embankment through the sections that contain 2 and 0 percent rubber, and the center-line joint has not opened. The section with 1 percent rubber includes a small fill and the center line has reflected through for a portion of the length. Comparison of longitudinal cracking is not realistic under these conditions. The transverse crack frequency is lower in the section that contains 2 percent rubber. More interestingly, 75 percent of the 1980 transverse feet of crack in 2 percent rubber consists of full-width cracks. Only 55 percent of the transverse feet of crack in the sections with 0 percent rubber are part of full-width cracks. That is, the cracks in the section with 2 percent rubber are longer but occur less frequently than those in the section with 0 percent rubber. No explanation of this difference has developed.

Route 69, Woodbridge

This 1.5-in overlay covers an older bituminous concrete pavement. Frequent well-defined cracks were present, but no rutting or surface degrading had occurred. The condition of the old pavement was in all probability due to age-hardening of the mix. To avoid placing spreader joints in the new layers above those in the existing pavement, the overlay was placed in four passes. It was anticipated that the two outer or the two inner passes would be generally comparable. Since a large portion of the outer passes was over areas originally constructed as shoulders, the two inner passes were selected as test and control sections. Both the section with 1

percent rubber and that with 2 percent rubber performed sharply better than the material with 0 percent rubber did. At two years, the section with 2 percent rubber had one-fifth as much longitudinal cracking as the section with 1 percent rubber but 10 percent more transverse cracking.

At three years, the cracking in the section with 2 percent rubber had increased to five-sixths of the longitudinal cracking of the section with 1 percent rubber and 39 percent more transverse cracking. Considering the two types of cracking together, the section that contained 2 percent rubber had 20 percent more cracking than the section with 1 percent rubber. Projecting the changes ahead would indicate that after another year, the section with 1 percent rubber would be slightly superior to that with 2 percent rubber for both types of cracks and that both rubberized sections would be 50 percent better than the untreated sections.

Route 85, Bolton

During the three years of observation, the sections on this roadway have shown more deterioration than did most of the thick-overlay test sections. Traffic does not include heavy wheel loads. Conditions vary locally along the road and so comparisons must be made between lanes. Several areas of insufficient strength have resulted in alligator cracking. Since these areas are not evenly distributed between the sections, they cannot be included effectively in comparisons between the sections. The fact that all occur on this one route indicates a roadway that has inadequate section design for current traffic. At two and three years, the sections with 1 percent rubber and that with 2 percent rubber showed less longitudinal cracking than the section with 0 percent rubber did. The section with 2 percent rubber also had less transverse cracking. Due to the local variations in conditions along this route, comparisons for the section with 1 percent rubber are not so clear. The total length of transverse cracks is

low in the section with 1 percent rubber but even lower in the adjacent area with 0 percent rubber. If a larger sample of lane with 0 percent rubber is considered (that opposite both the section with 1 percent rubber and that with 2 percent rubber), the section with 1 percent rubber is superior. Overall, at this test site the rubberized material is superior.

Route 101, Killingly

This test section is the main street of the village of Dayville. The 1.5-in test sections were placed in the center pass of a three-pass resurfacing. More recently, a utility trench has been dug longitudinally in the center of one outer pass. Slumping of the trench sides has caused some longitudinal cracking in the center test pass. Allowing for this source of damage, the Route 101 test has essentially no longitudinal cracks. Of the few transverse cracks that have appeared, approximately twice as many occur in the nonrubberized material.

Route 354, Colchester

The pavement under the sections 1.5 in thick is a bituminous concrete section with an occasional full-width, full-depth crack. The pavement had much the appearance of an overlay over a jointed-concrete pavement. Due to the large crack width, nearly all the cracks have reflected through in the test sections. There are large differences in the crack frequency between the test sections. Three factors contribute to this problem. The tests with 0 and 1 percent rubber are in shallow cuts. The section with 2 percent rubber is in a zone of transition from cut to fill. The section with 0 percent rubber is level. The grade increases through the section with 1 percent rubber to a maximum in the section with 2 percent rubber. In addition, heavy woods shade the sections with 0 and 1 percent rubber during most of the day. The resulting differences in daily temperature cycles may affect the spacing of cracks. The frequencies of transverse cracks in the rubber sections were greater than those in the controls in both 1979 and 1980. However, the reverse is true for the longitudinal cracks. The numerical differences are large enough to be impressive, but the reversal of effect between transverse and longitudinal cracking is not readily explained by the rubber level.

Route 354, Salem

Traffic for the two locations on Route 354 is similar since there are no major intersections between. However, the pavement conditions prior to overlaying were noticeably different. Cracking was much more extensive in the Salem area. The old pavement was a penetration treatment, and the cracks were smaller but occurred much more frequently. Only a small number of the cracks have reflected through the test overlay. At the end of two and three years, more cracks have appeared in the rubber sections and longitudinal cracks predominate. The comparison is complicated by a very different frequency of cracks in the two lanes. The road has been widened in the past, and the pavement conditions under the overlay vary from side to side. The comparison with a control section in the same lane reduces the difference, but the level of performance of the rubber sections appears lower.

ALL RUBBER-MODIFIED SECTIONS VERSUS CONTROLS

There are enough sections to justify statistical

comparisons of rubber levels. Since overlays should last several years, statistical comparisons have been made that use the three-year (1980) data. In anticipation of the possibility that performance measured by longitudinal crack frequency might be different from that found by using transverse cracks, probabilities were computed first for each crack type separately. In recognition of the differences between locations, the data were paired by putting the proper control with each test section and by grouping pairs for common rubber level. By using the Student's t-test, there is then a 90 percent probability that the sections that contain 1 percent rubber have fewer longitudinal cracks and a 35 percent probability that they have fewer transverse cracks. There is an 86 percent probability that the sections with 2 percent rubber contain more longitudinal cracks than the controls do and 70 percent probability that the sections with 2 percent rubber have more than the sections with 1 percent rubber do. The transverse cracking shows the same trend. There is a 90 percent probability of more cracks than in the controls and a 98 percent probability of more cracks than in the sections with 1 percent rubber.

Failure stresses in a pavement are the sum of flexure and temperature stresses. Cracks result when the tensile stress exceeds the strength of the mix. Under a wheel load, the radius of curvature is smallest at right angles to the direction of traffic. Since stress level is inversely proportional to the radius of curvature, the highest flexural tensile stress would be perpendicular to the wheel paths and would result in longitudinal cracks.

If base deflections are small, large tensile stress will not occur from flexural stress and the effect of temperature becomes more important. The proximity of free edges limits the magnitude of transverse temperature stresses. On the other hand, the continuity of the pavement in the direction of traffic induces high-temperature stresses that result in transverse cracks.

Thus, the demands on the overlay material over a crack are different, depending on the direction of the crack. The material with 1 percent rubber is more effective at resisting load-induced flexure tensile stresses than direct tension. This difference may be due to the magnitude of the strain. For flexure, the tensile strain is limited by the curvature. The strain due to temperature is limited only by the coefficient of expansion of the material. If cracks are far apart, the temperature strain can be large. The addition of rubber to the mix makes the material plastic enough to accommodate the strain associated with flexing but not that of shrinkage. The transverse cracks would then reflect through more quickly than the longitudinal cracks.

Although the longitudinal and the transverse cracking had different frequencies, considering them together gives further insight into the relative performance. Considering all cracking, there is a 96 percent probability that the sections with 1 percent rubber have less cracking than the control sections do but only a 30 percent probability that the sections with 2 percent rubber have less than the controls do.

Two sections were underlaid by concrete pavement. Both had high traffic levels. On Route 44, Barkhamsted, both the section with 1 percent and that with 2 percent rubber performed better than the control did; the section with 1 percent rubber was superior. On Route 66, Marlborough, the comparison reverses and the controls are better than the rubber-modified sections. Thus, no conclusions are possible about the effect over concrete.

The relative crack frequency does correlate with

Table 6. Effect of 1 percent and 2 percent rubber.

Level of Distress	Level of Traffic			All Levels of Traffic (avg)
	Low	Medium	High	
Sections with 1 Percent Rubber				
Low	+0.0027	-0.324	0.134	-0.0624
Medium	0.0964	0.6771	0.271	0.3482
High	-0.196	0.3831	0.0230	0.0700
All levels of distress (avg)	-0.0356	0.2454	0.1427	
Sections with 2 Percent Rubber				
Low	-0.4223	-0.2485	-0.170	-0.0813
Medium	0.1564	-0.1573	0.405	0.1347
High	-0.5264	0.182	-0.0221	-0.1222
All levels of distress (avg)	-0.2643	0.0911	0.0710	

Note: Values are decrease in feet of crack per foot of paver pass compared with 0 percent controls.

Figure 4. Transverse cracking in sections with 0-1 percent rubber.

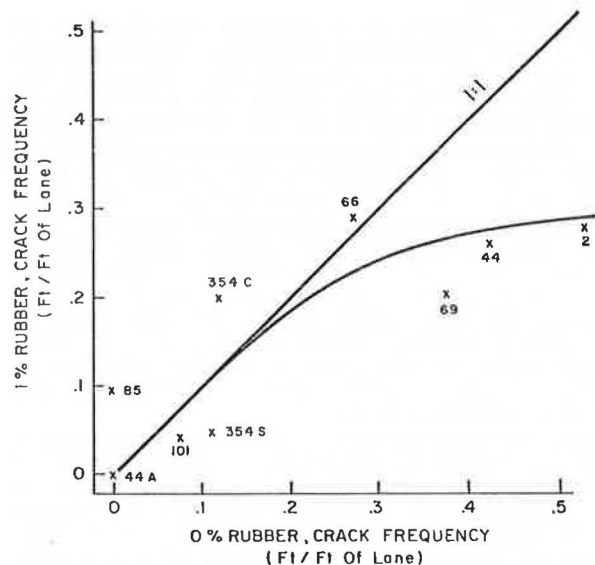
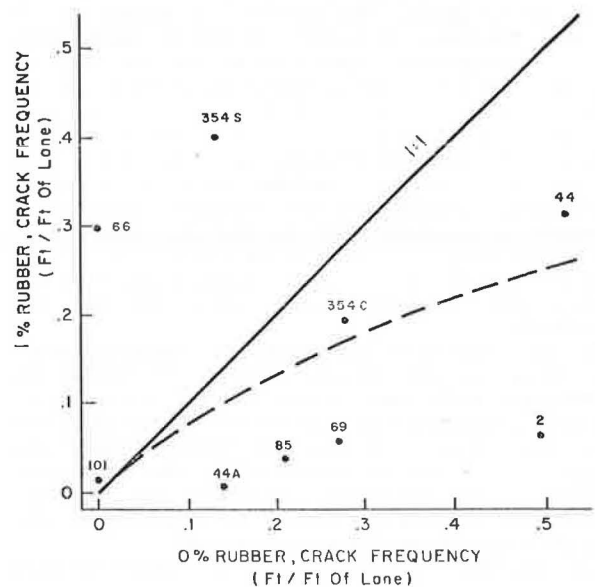


Figure 5. Longitudinal cracking in sections with 0-1 percent rubber.



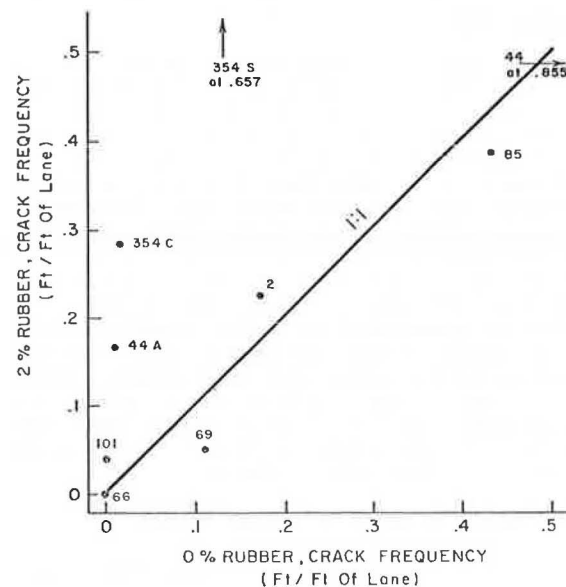
traffic and distress. Table 6 shows the differences in feet of crack per foot of lane for the rubberized test sections and the controls. The data have been arranged in the same relative position of traffic and distress levels as the original selections of test sections. That is, in Table 6 the entry corresponding to low traffic and low deterioration was the section on Route 354, Colchester, for which the feet of crack (transverse and longitudinal totaled) per foot of paving pass for material with 1 percent rubber is 0.0027 greater than that for the material with 0 percent rubber or the control. Throughout Table 6, positive values denote improvement of the section with 1 percent rubber compared with the section that had 0 percent rubber. Negative values indicate comparisons in which the section with 0 percent rubber has less cracks. Pavements originally classified as of medium distress benefited most from the addition of 1 percent rubber. Roadways with medium traffic levels demonstrated more benefits than did low-volume locations. The effect of traffic was great enough that when both low- and high-level distressed pavements were combined with high traffic, they showed a benefit from the addition of rubber.

Some rationalization of these trends is possible. A new overlay over low-distress pavement can be expected to perform adequately, and in the short period of three years, large differences might not be apparent. A new overlay over a high-distress pavement may also show little improvement. Frequently, the new overlay placed is not adequate for long life under the poor foundation and/or high loading conditions that had caused the original distress. The substitution of a better material, which is still less than adequate, would delay cracking, but by the end of three years it also would have cracked. Since the rubber-modified mix retains a greater degree of plasticity at all ages, increased traffic kneads the material and impedes the formation of reflection cracks.

By using the differences between the section with 2 percent rubber and the control sections, similar trends were found, except that at high distress levels the differences indicate better performance for the controls. This is caused by the poor performance of the section on Route 354 in Salem.

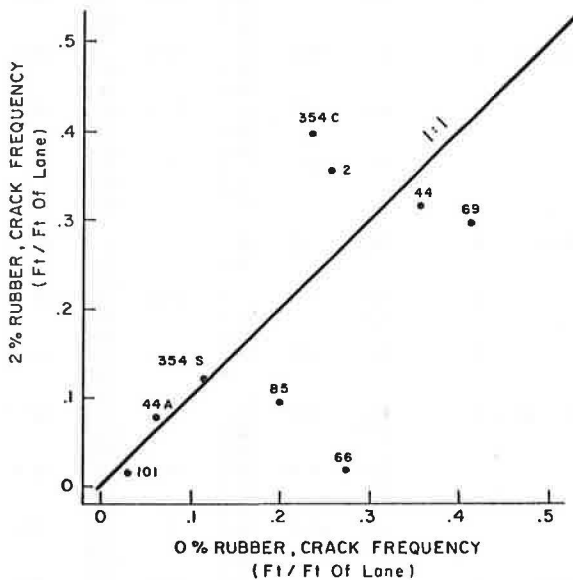
Figures 4-7 were prepared to provide a basis for

Figure 6. Longitudinal cracking in sections with 0-2 percent rubber.



further comparison of the performance of the different levels of rubber. In both Figures 4 and 5, the points tend to fall below a 1:1 correlation, which indicates that the material with 1 percent rubber performed better than the control did. In Figures 6 and 7 the points are along or above the 1:1 correlation line. The latter condition indicates no improvement by using 2 percent rubber.

Figure 7. Transverse cracking in sections with 0-2 percent rubber.



Figures 4 and 5 may give an indication of the degree of distress for which 1 percent rubber would be effective.

An approximate curve through the limited data (line A in Figure 4) could be useful for estimating the benefit to be obtained by adding 1 percent rubber to the mix. For example, a 1.5-in overlay is to be placed over a pavement with 0.9 ft of transverse and 0.6 ft of longitudinal crack per lane foot. Assuming that one-third of the cracks present at the time of placing an overlay 1.5 in thick reflect through a 0 percent overlay in three years, what reduction in cracks can be brought about by adding 1 percent rubber? Entering Figure 4 with one-third of 0.9 ft/ft of lane for 0 percent rubber gives 0.23 ft of transverse crack per foot of lane for 1 percent rubber or a reduction of 23 percent at three years compared with 0 percent rubber. To estimate longitudinal cracks, enter Figure 5 with one-third of 0.6 or 0.2 ft/ft of lane and read 0.13 or a 35 percent reduction. Since this use of the data was not anticipated, the assumption of one-third reflection cracks in three years must be refined with experience.

Permeability is one of the properties that has been useful in predicting the rate of aging of pavements. It has been postulated that air circulation through the pavement voids hastens hardening. Recognizing that three years of observation might not adequately show the differences in performance of the rubberized class 1 pavement, permeability tests were carried out. To a degree, density measurements serve the same purpose. Values for air permeability and nuclear density were determined in the wheel paths when the pavements were one year old (Table 7).

Table 7. Permeability and nuclear density of overlays.

Route No. and Location	Rubber Content (%)	Date Performed	Permeability (L/S, wheel path)					Nuclear Density (lb/ft ³ , wheel path)						
			Test 1	Test 2	Test 3	Avg	SD	ADT	Test 1	Test 2	Test 3	Avg	SD	ADT
2, Preston	0	6/14/78	0.028 7	0.031 1	0.029 9	0.029 9	0.001 200	4 600	138.6	138.7	141.2	139.5	1.473	4 600
	1		0.006 22	0.007 56	0.007 78	0.007 19	0.000 844 4		143.6	141.5	138.7	141.3	2.458	
	2		0.016 6	0.014 4	0.013 6	0.014 8	0.001 553		138.5	136.8	136.6	137.3	1.044	
44, Barkhamsted	0	7/5/78	0.010 0	0.009 07		0.009 54	0.005 146	9 600	151.4	151.4	151.7	151.5	0.173	9 600
	1		0.008 83	0.007 96		0.008 39	0.000 615		151.8	152.0	151.4	151.7	0.308	
	2		0.007 41	0.007 71		0.007 56	0.000 212 1		150.6	149.8	147.8	149.4	0.964	
44A, Coventry	0	5/31/78	0.005 04	0.004 98		0.005 01	0.000 042	9 500	142.7	142.6	142.0	142.4	0.379	9 500
	1		0.006 16	0.006 27		0.006 21	0.000 078		145.8	146.8	142.7	145.1	2.139	
	2		0.012 4	0.013 5		0.012 95	0.000 778		143.7	143.6	143.0	143.5	0.379	
66, Marlborough	0	6/6/78	0.012 1	0.012 6		0.012 4	0.000 354	7 600	146.1	149.6	144.32	146.7	2.689	7 600
	1		0.006 53	0.007 90		0.007 21	0.000 969		143.2	143.7	144.4	143.8	0.603	
	2		0.010 4	0.015 2	0.013 9	0.013 2	0.002 48		148.5	144.1	146.3	146.3	2.200	
69, Woodbridge	0	6/20/78	0.013 5	0.009 95	0.013 49	0.012 31	0.002 046 7	5 000	142.7	140.2	140.4	141.1	1.389	5 000
	1		0.002 29	0.005 10	0.004 30	0.003 90	0.001 448		144.7	146.2	145.1	145.4	0.789	
	2		0.003 47	0.003 57	0.004 50	0.003 84	0.000 568		141.5	141.8	139.0	140.8	2.175	
85, Bolton	0	6/1/78	0.017 4	0.012 69		0.015 03	0.003 33	3 700	142.7	142.6	142.0	142.4	0.379	3 700
	1		0.006 37	0.005 86		0.006 12	0.000 361		144.2	142.0	142.6	142.9	1.137	
	2		0.025 7	0.020 2		0.022 92	0.003 89		140.2	139.3	138.5	139.3	0.850	
101, Killingly	0	7/5/78	0.011 5	0.018 0		0.014 7	0.004 596	10 400	139.7	138.0	135.7	137.8	1.622	10 400
	1		0.009 99	0.009 46		0.009 73	0.000 375		138.4	139.1	138.0	138.5	0.556	
	2		0.008 30	0.001 54		0.004 92	0.004 78		138.1	138.1	140.1	138.8	1.197	
354, Colchester	0	6/15/78	0.003 59	0.003 53		0.003 56	0.000 042 4	1 300	139.0	139.1	140.3	139.5	0.723	1 300
	1		0.014 1	0.008 67	0.008 78	0.010 52	0.003 104		138.4	139.6	136.9	138.3	1.353	
	2		0.034 7	0.033 0	0.034 7	0.034 1	0.000 982		132.4	128.5	133.6	131.5	2.666	
354, Salem	0	6/14/78	0.040 1	0.022 1	0.023 9	0.028 7	0.009 91	1 300	135.6	137.6	139.1	137.4	1.756	1 300
	1		0.007 81	0.007 42	0.006 13	0.007 12	0.000 879		131.5	134.3	133.5	133.1	1.442	
	2		0.067 4	0.052 5	0.049 8	0.056 5	0.009 4		134.3	131.0	129.3	131.6	2.542	
Avg, class 1	0					0.014 57	0.009 23					142.03	4.543	
	1					0.007 38	0.002 08					142.23	5.286	
	2					0.018 98	0.010 62					139.83	6.072	

The average air permeability for the class 1 mix with no rubber was 0.014 57 L/s. Sections with 1 percent rubber averaged one-half as much, 0.007 38 L/s. However, 2 percent rubber increased the average permeability to 0.018 98. The relationship between permeability and traffic volume is shown in Figure 8. The air permeability decreases with in-

Figure 8. Overlay permeability versus traffic volume.

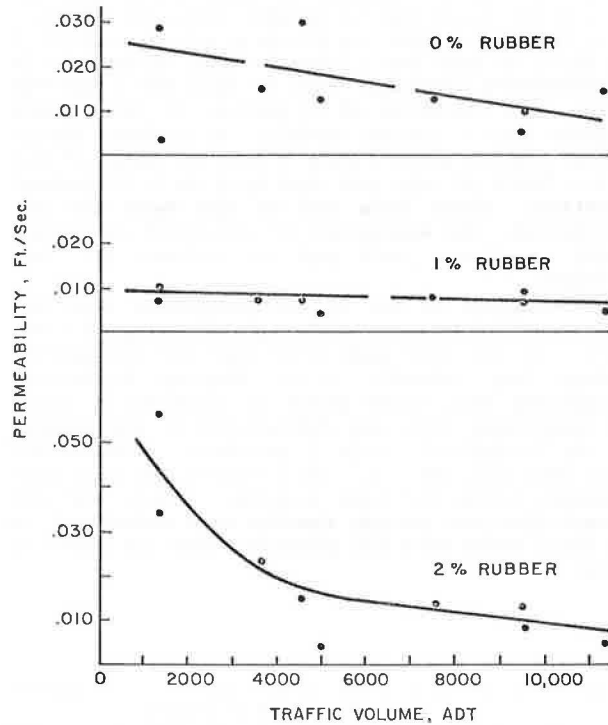


Figure 9. Overlay density.

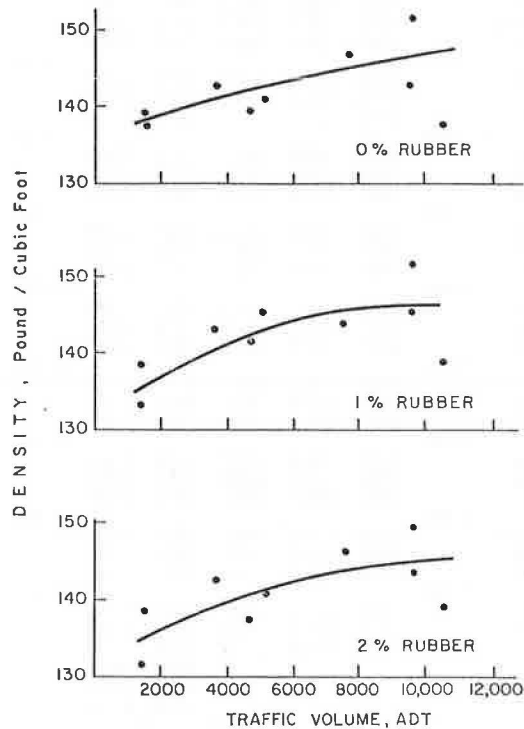


Table 8. Skid numbers, class 1 thick overlays.

Route No. and Location	Rubber Content (%)	Nov. 1977		Nov. 1978		Nov. 1980	
		SN	Avg SN	SN	Avg SN	SN	Avg SN
2, Preston	0	41.8	46.0	48.9	48.7	50.4	51.03
		50.2		45.8		50.9	
	1	46.6	46.6	51.5	47.7	51.8	51.57
				44.2		50.6	
				49.8		52.3	
				49.1		55.3	
2	50.2	50.2	50.3	49.8	53.9	54.5	
			49.9		54.3		
44, Barkhamsted	0	48.3	48.3	39.3	38.9	39.8	42.3
				39.5		46.1	
	1	43.4	43.4	38.1	39.3	41.3	41.8
				40.1		40.2	
				39.6		44.0	
				38.8	39.7	41.8	41.7
2	44.6	44.6	41.7	39.7	41.6	41.7	
			38.7		43.7		
44A, Coventry	0	33.2	33.2	35.3	35.8	38.9	40.2
				36.3		38.1	
	1	33.2	34.9	42.7	42.4	42.4	41.97
		36.6		41.9		39.0	
				42.6		44.5	
				38.4	40.5	43.1	43.17
2	39.8	39.8	42.5		42.7		
					43.7		
66, Marlborough	0	36.6	38.2	46.3	43.5	38.6	42.67
		39.8		44.0		42.0	
	1	36.3	38.85	44.4	44.9	41.3	45.47
		41.4		47.3		47.7	
				43.1		47.4	
				36.4	38.5	34.9	39.6
2	39.9	39.9	41.8		42.8		
	39.9		37.4		41.1		
69, Woodbridge	0	38.2	38.2	46.8	46.0	45.4	46.3
				44.9		44.3	
	1	33.2	33.2	46.2	42.0	49.2	44.8
				46.1		45.2	
				42.2		44.9	
				40.5		44.3	
2	38.9	38.9			43.8	46.1	
					46.1	48.3	
85, Bolton	0	43.0	43.0	43.4	43.7	43.8	46.0
				43.9		47.2	
	1	39.9	39.9	43.7	46.8	47.0	46.57
				48.1		46.4	
				45.4		47.4	
				46.8		45.9	
2	39.9	39.9	50.7	49.5	46.0	46.23	
			50.1		45.4		
101, Killingly	0	38.2	36.5	48.1	46.4	46.8	44.9
		34.8		45.0		44.5	
	1	34.8	36.5	46.1	38.1	43.4	40.6
		38.2		41.5		39.2	
				36.3		41.8	
				36.6		41.8	
2	34.8	35.6	37.9	36.7	36.7	38.4	
	36.3		34.3		40.5		
354, Colchester	0	43.4	47.6	51.4	52.9	53.0	53.33
		51.8		52.7		54.5	
	1	56.7	56.7	54.7	53.1	52.5	55.43
				52.0		54.4	
				54.4		55.6	
				53.0		56.3	
2	53.4	53.4	56.6	56.6	56.8	52.57	
			56.6		53.6		
354, Salem	0	54.4	54.4	48.8	47.9	54.1	54.3
				49.1		54.0	
	1	50.2	50.2	45.8	47.4	54.8	54.47
				46.3		54.5	
				49.3		54.9	
				46.6		54.9	
2	55.0	55.0	49.5	49.4	55.8	55.4	
			50.5		53.9		
		48.1		56.5			

Note: SN = skid number.

creasing traffic. At low traffic levels, the mixture with 2 percent rubber has significantly higher permeability but is similar to the mixture with 0 percent rubber at higher volumes. The air permeability of the mixture with 1 percent rubber is low and nearly constant regardless of traffic. This implies that the mixture with 1 percent rubber compacts more completely under the roller and that no further compaction occurs under traffic.

The average densities of the pavement with 0 percent and that with 1 percent rubber were the same at 142 lb/ft³. Since the voidless unit weight of the mix with 1 percent rubber is lower, this indicates a greater degree of compaction. The material with 2 percent rubber averaged 1.5 percent less at 139.8 lb/ft³. The relationship between density and traffic volume (Figure 9) is that of increasing density with increasing traffic level. The two locations where the original pavements were portland cement concrete recorded the highest nuclear densities. Either the gauge is influenced by the concrete or the more rigid base aids in compaction. Both had been overlaid previously by using bituminous concrete.

A comparison of the average nuclear density at one year with the average Marshall density at the time of construction provides an insight into the characteristics of the materials. At one year, the average wheel-track nuclear density for the sections with 0 percent rubber was 92 percent of the original Marshall density. For the material with 1 percent rubber this comparison was 95 percent and for that with 2 percent rubber, 94.6 percent.

Based on the average nuclear densities, the void content of the three types of pavement at one year would be 10.4 percent for the material with 0 percent rubber, 9.2 percent for that with 1 percent rubber, and 8.1 percent for that with 2 percent rubber.

Changes in paving materials always raise concerns about skid resistance. Skid-resistance numbers determined by using equipment that meets current federal highway standards provide realistic comparisons between surfaces. Those for the rubber-modified overlays are listed in Table 8. Extensive statistical computations by using the Student's *t*-test show few significant differences. The only

comparisons that resulted in confidence levels about 95 percent were those in 1977 when, at the 95 percent level, it could be said that those mixes that contained rubber were superior to those without and that 2 percent rubber mixes were better than mixes with either 0 or 1 percent rubber. At an 85 percent level, the mixes that had 2 percent rubber were higher than the mixes with 1 percent rubber in 1978. There is a weak trend for all materials to improve with age. For all test sections, the skid numbers compare well with the nonrubber control, and skid resistance should not be a problem.

CONCLUSIONS

The rubber-modified mixes are performing as a class better than the nonrubberized mixes do. Cracking has been reduced and there has been no reduction in skid resistance. The lower permeability and higher density of the rubber-modified mixture imply a probability of longer life.

ACKNOWLEDGMENT

The views expressed are mine and may not represent the official position of the University of Connecticut or the supporting agencies, ConnDOT and the Federal Highway Administration.

REFERENCES

1. J.E. Stephens and S. Mokrzewski. The Effect of Reclaimed Rubber on Bituminous Paving Mixtures. Univ. of Connecticut, Storrs, Civil Engineering Department Rept. CE 74-75, Feb. 1974.
2. J.L. McRae and B.D. Lagrone. Effect of Modified Reclaimed Rubber and Ground Vulcanized Rubber on the Physical Properties of Bituminous Pavements as Evaluated by the Gyratory Testing Machines. HRB, Highway Research Record 361, 1971, pp. 14-19.
3. Standard Specifications for Roads, Bridges, and Incidental Construction--Form 811. Connecticut Department of Transportation, Wethersfield, 1974.

Publication of this paper sponsored by Committee on Characteristics of Nonbituminous Components of Bituminous Paving Mixtures.

Methods of Increasing Fracture Toughness of Asphalt Concrete

R.T. WOODHAMS

Experience with rubber-modified asphalt concrete in the Canadian environment during the past seven years has yielded only marginal improvement with respect to cracking. The most effective rubbers for reducing the brittle temperature of asphalt concrete are those that have low glass transition temperatures, although reclaim tire rubber is preferred since it costs the least. Chopped nylon or polyester tire cord or integrated rovings can impart large increases in fracture toughness at temperatures below freezing. For maximum efficiency, the chopped cord should be near its critical pullout length. Calculations indicate that fracture toughness can be increased 20-fold under ideal conditions with the addition of only 1 percent fiber. Moisture damage can be minimized by the addition of a minor proportion of ferric oxide or iron naphthenate as an adhesion promoter.

The combined use of rubber, chopped fibers, and an adhesion promoter should help to improve the durability of most asphalt concretes in cold climates. Paving trials are needed to indicate the long-term economic benefit of these modifications.

Despite several serious deficiencies, bitumen continues to be favored as a low-cost binder in road paving formulations. The temperature susceptibility of bitumen is so pronounced that at elevated temperatures, creep and distortion of asphalt pavements

become excessive, whereas at temperatures below freezing, the pavement becomes so brittle that numerous cracks develop as a consequence of thermal shrinkage forces. It is difficult to tailor a bitumen that performs equally well at such environmental extremes and simultaneously resists oxidative hardening, moisture damage, stress fatigue, freeze-thaw cycles, thermal cycling, excessive loading, and unstable foundations. Therefore, in order to improve the performance of bitumen as a pavement binder it would be desirable to make the following changes:

1. Reduce the brittle temperature through the use of rubber additives and at the same time increase the resistance to flow at elevated temperatures;
2. Promote better adhesion to the aggregate, particularly in the presence of moisture, by incorporating suitable adhesion promoters; and
3. Increase the fracture toughness of asphalt concrete by adding a minor proportion of chopped fibers.

Previous investigators have demonstrated that the brittle temperature of bitumen and its mixtures can be lowered by the incorporation of rubber; the rubber is added to the bitumen as an uncured gum, thermoplastic elastomer, liquid latex, or powdered reclaim (1). The reduction in the brittle temperature will depend on the concentration of the added rubber and its glass transition temperature (T_g). Therefore, one of the most effective elastomeric materials should be cis-polybutadiene, which has a reported glass transition temperature of -120°C . For economic reasons, reclaim rubber powder obtained by the mechanical or cryogenic grinding of discarded tires has become the preferred additive even though it is not so efficient as other elastomers. Commercial methods have now been developed for paving rubber-modified asphalt concrete by the hot-mix process (2) or by seal coating (3). (Methods have also been reported by L.S. Brake, Sahuaro Petroleum and Asphalt Co., P.O. Box 6536, Phoenix, AZ 85005, and by E. Thompson, Pounder Emulsions, 1474 Wall Street, Winnipeg, Manitoba R3E 2S4, Canada.) These two methods have been used in Canada during the past several years with limited success. Despite rubber modification, cracking of the surface persists. Surface texture remains good after four years and there is negligible ravelling or stripping observable with Metro Toronto RA-3 rubberized asphalt (4,5).

The addition of rubber to an asphalt concrete does reduce the brittle temperature but has a minor effect on the tensile strength or toughness of the composite. Toughness can be increased through the addition of chopped cord or rovings of sufficient length and size to impart a large increase in fracture energy (6). The maximum toughness is obtained when the chopped cord or roving is near its critical length, a length that depends on the temperature, interfacial shear strength, and cord diameter. Chopped fibers have been used in the past to toughen portland concrete (7,8) and sulfur concrete (9). They often produce 20-fold increases in fracture toughness at fiber concentrations near 1 percent by weight (e.g., steel wires or glass, polypropylene, nylon, or polyester fibers). The availability of large quantities of waste tire cord makes this method of toughening especially attractive. This increase in fracture toughness is retained even at freezing temperatures when the asphalt concrete has become very brittle. With proper design, the fibers can induce multiple cracking so that large cracks are less likely to occur.

Various publications have drawn attention to the loss of adhesion at the interface between the bitumen binder and the aggregate when moisture is present. This phenomenon is sometimes referred to as water stripping, disbonding, or water damage. The loss of adhesion may occur after long immersion in water or prolonged exposure to a humid environment, after which the asphaltic concrete may be seriously weakened and thus become more susceptible to damage. Under moist conditions, asphalt concrete can lose up to 80 percent of its strength (10). Although adhesion promoters or antistripping agents (such as amines and lime) have been suggested as additives to help prevent this loss of adhesion, Scott (11) concluded that "cationic adhesion additives in the asphalt can delay but not prevent loss of adhesion," whereas "anionic types are only likely to be successful where there is an opportunity for water insoluble salt formation with tightly bound surface metal ions." Fromm (12) found that three commercial antistripping additives delayed the onset of stripping but actually caused accelerated stripping in the later stages, which also promoted increased emulsion formation in the asphalt. The most effective reagent found by Fromm for preventing stripping was an asphalt-soluble iron compound, iron naphthenate, which is a commercially available paint drier (Nuodex DMR Iron 6%, Nuodex Canada, Ltd., 34 Industrial Street, Toronto, M4G 1T9, Canada).

The following investigation describes attempts to evaluate the effectiveness of these three types of additives--rubber, fibers, and iron naphthenate--on the performance of asphaltic paving materials by using diagnostic laboratory tests. A more complete evaluation awaits further tests and paving trials.

EXPERIMENTAL

The dynamic mechanical properties of the bitumen-rubber mixtures were measured on a Rheovibron visco-elastometer (Model DDV-II) in a shear mode. The sample holder and measurement technique have been described previously (1). Viscosity measurements were conducted with a Brookfield viscometer while the samples were maintained at constant temperature in a heated bath.

The measurement of contact angles was carried out by the technique of Neumann and Good (13, Chapter 2, p. 31). The sessile drops of each bitumen sample were allowed to equilibrate in an air oven at 100°C for 15 min and were then removed for measurement of the contact angle by using a calibrated goniometer.

The asphalt concrete test beams were prepared in the laboratory by using a special roller equipped with a hydraulic stage (2). Rectangular slabs 30 cm long, 19 cm wide, and 5.2 cm thick could be prepared with a void content of less than 1 percent. A standard paving formulation (Metro Toronto HL-1 mix) was used as a control. Each compressed slab was cut into several rectangular beams for flexural testing. The full procedure has already been described (2).

Peel forces were measured at a 90° angle by using glass fiber tapes adhered by various bitumen mixtures to horizontal glass plates. Hot bitumen mixtures were spread onto clean glass plates and the glass fiber tapes (15x6.5 cm) were embedded into the liquid bitumen. A sheet of paper was placed over the glass tape so that a second glass plate could be positioned over the assembly without adhering to the bitumen. Thin aluminum shims (0.6 mm thick) placed between the parallel glass plates ensured a uniform reproducible film thickness. A weight was placed on top of the assembly and each specimen was allowed to equilibrate for one week before the assembly was subjected to complete immersion in distilled water or exposed to the atmosphere. [Details of this pro-

Figure 1. Setup for 90°-angle peel testing.

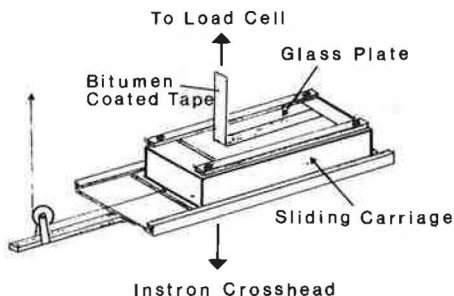
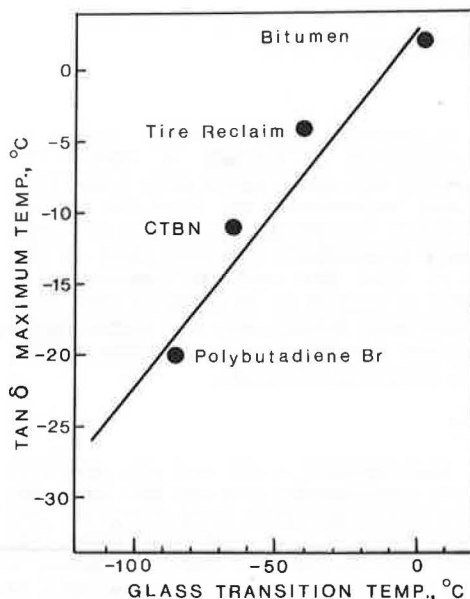


Figure 2. Effect of addition of 20 parts of rubber to 100 parts of bitumen on brittle temperature of mixture.



cedure may be obtained from the thesis of Vavrovakul (14); see ASTM B533-70.] Figure 1 shows the experimental rig for 90°-angle peel testing, which attaches to an Instron tester. All peel tests in this series were conducted at a temperature of 16.5°C and a peel rate of 1 cm/min. The sliding carriage may be heated or cooled to provide data over a wide range of temperatures if necessary.

RUBBER-MODIFICATION RESULTS

The addition of rubber (whether in particulate form or as a liquid) to bitumen lowers the brittle temperature of the mixture. The extent of lowering was proportional to the amount of rubber and its glass transition temperature. This trend is indicated in Figure 2 in which, from dynamic mechanical tests (1), the tan δ maximum for these rubber-bitumen blends shows a linear proportionality to the glass transition temperatures of the added rubbers at 20 parts of rubber. It is likely that cis-polybutadiene (not shown), which has a glass transition temperature of -102°C, would be the most effective rubber for reducing the brittle temperature of asphalt concrete at this concentration.

The extent of reduction of the brittle temperature ΔT may be predicted from the rule of mixtures for any concentration by using the following simple formula:

$$\Delta T = (T_a - T_r)\phi \tag{1}$$

where $T_a - T_r$ is the difference between the glass transition or brittle transition T_a of the asphalt cement and the glass transition T_r of the added rubber. The symbol ϕ is the volume fraction of added rubber in the mixture. The glass transitions of most elastomers are known or can be estimated from the tables (15). The brittle temperature of bitumen is near 0°C at 30 Hz.

The addition of rubber to an asphalt mix also increases the viscosity so that, in practice, longer mixing times are required to achieve the same degree of aggregate dispersion. In the case of powdered reclaim rubber, an additional 10 s in a production-sized pug mill was necessary for proper mixing. When the temperature of the pug mill was increased by 20°C (about 1°C for each part of added rubber), no additional mixing time was required. Since the time in the pug mill is usually quite short (less than 1 min), no appreciable swelling or reaction can take place between the particulate rubber and the bitumen. There is evidence to suggest, however, that over a longer period of time (about one year) at ambient temperatures, a chemical reaction (probably initiated by air oxidation) takes place after the mixture has been applied to the road surface. This chemical interaction may lead to improved bonding (adhesion) of the bitumen onto the rubber phase and a concordant increase in the toughness of the rubber-modified concrete as it ages. According to Fromm and McDougall of the Ministry of Transport and Communications in Downsview, Ontario, samples of rubberized asphalt concrete removed from three-year-old pavements in Metro Toronto after re-compaction gave Marshall flow values (0.01 in) between 16 and 40 that correspond to Marshall stability values of 2765 and 1350 lbf. This surface texture of the road was rated excellent and there was negligible evidence of stripping or ravelling. Good adhesion to the portland concrete base was noted although the usual number of cracks (both longitudinal and transverse) was apparent. The experience with reclaim rubber in hot-mix paving in the Metro Toronto area now extends more than six years, and although all pavements have been rated in good condition, the improvement gained through the addition of rubber thus far is considered marginal. The incorporation of reclaim rubber into a paving mixture does require a corresponding increase in the asphalt concentration, which in view of the increasing price of asphalt has tended to deter further trials.

Laboratory samples of rubberized asphalt concrete containing vulcanized tire rubber (minus-30 mesh size) showed a slight gain in strength and toughness after one year's immersion in tap water. It is not known to what extent the other elements and compounds usually present in tire compounds (sulfur, zinc oxide, accelerators, antioxidants, anti-ozonants, oil extenders, stearic acid, processing aids, carbon black, etc.) influence the long-term performance of asphalt paving materials, but they could well have a beneficial effect.

In a series of experiments, chopped nylon and chopped polyester tire cords were incorporated into a standard hot-mix formula as shown below:

Material	Percent by Weight
Trap rock (1 cm coarse)	42.0
Trap-rock screenings	12.8
Sand	38.6
Asphalt	6.0
Chopped cord (1.2 cm)	0.6

Figure 3. Stress-strain response of sample that contained 1.2 volume percent chopped nylon tire cord 1.3 cm long.

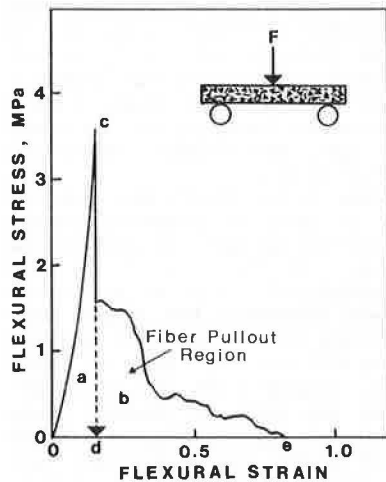
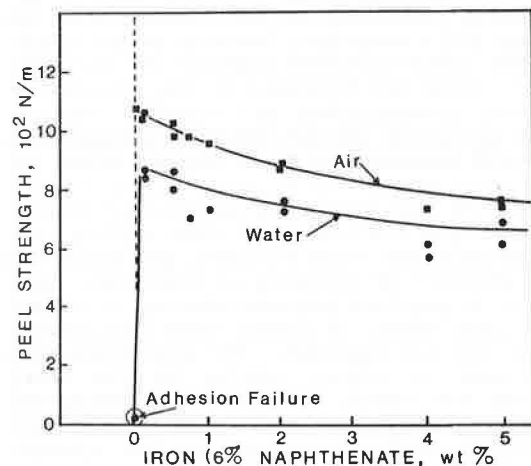
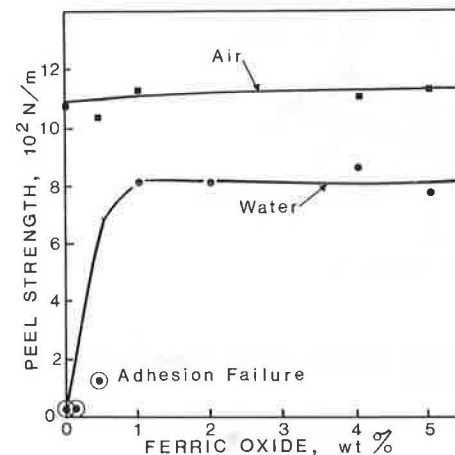


Figure 4. Peel strength when iron naphthenate is added to asphalt mixture at 16.5°C.



In a three-point flexural test, the stress-strain response shown in Figure 3 was obtained at -20°C (which is well below the brittle temperature of the bitumen binder). The dotted line shows the path of the recorder pen when the chopped tire cord was absent. The influence of the tire cord on the stress-strain behavior is negligible up to the appearance of the first crack at *c*, after which the ultimate strain is increased from *d* to *e*. The area under the stress-strain curve is divided into two sections--that represented by *a* is the energy absorbed up to the first crack and that labeled *b* is attributable to debonding and fiber pullout energies. As the fiber length and fiber concentration increase, area *b* also increases without greatly influencing the first portion of the stress-strain curve. In other studies the optimum cord length was found to be 5 cm at fiber concentrations between 1 and 2 percent. When chopped roving is used instead of tire cord, the roving must be treated by using a binder to prevent dispersion of the individual filaments during the mixing stage. Attempts to incorporate loose reclaim tire cord filaments into the bitumen produced such large increases in viscosity that mixing with the aggregate became extremely difficult and would present problems in a pug mill. Even short fila-

Figure 5. Peel strength when ferric oxide is added to asphalt mixture at 16.5°C.



ments (a few millimeters long) produced large increases in the viscosity of bitumen at 120°C . The extent of toughening was quite small with such short filaments and this emphasizes the need for longer multifilament strands (or cord) that act as a single unit. Preliminary experiments indicate that in the brittle region at -20° the critical length for a typical tire cord is near 5 cm in asphalt concrete. Such chopped integrated fibers are now commercially available in Canada (Hartford Fibre Company, Ltd., Progress Avenue, Kingston, Ontario).

ADHESION RESULTS

The peel-test measurements with asphalt mixtures that contain iron naphthenate (Nuodex DMR Iron 6%) substantiated the observations made by Fromm in 1974 (12). After long immersion in distilled water (25 days), the bond between asphalt that contained a small percentage of iron naphthenate (0.1 percent by weight) failed in a cohesive manner rather than adhesively (at 16.5°C) when applied to a borosilicate glass plate. Without the iron naphthenate, the peel strength was almost zero with adhesive failure. In a few experiments, ferric oxide powder (red, anhydrous) was added to the bitumen. Although not quite so effective as the soluble iron naphthenate, the iron oxide did confer good adhesion when immersed in water. These two additives are compared in Figures 4 and 5. It is apparent that the iron naphthenate is more effective at low concentrations. However, ferric oxide can be obtained as an inexpensive waste from iron foundries, so this additive may be more economically attractive. Since most bitumens are slightly acidic, it is possible that some of the ferric oxide reacted to form soluble iron salts.

The greater adhesion to a glass surface of bitumen that contained iron naphthenate was also confirmed from contact-angle measurements. Bitumen containing 1 percent iron naphthenate gave a contact angle of 11° compared with 17° without naphthenate. This simple diagnostic test may be used to check the relative performance of other additives and substrates by using very small samples, if preferred.

DISCUSSION OF RESULTS

Although liquid or solid rubbers that have low glass transition temperatures are more efficient for reducing the brittle temperature of asphalt concrete, powdered reclaim rubber from discarded tires repre-

sents the only economically viable source of rubber for general use in road surfacing. Simple mixing in a conventional pug mill is adequate in order to disperse the rubber particles, which are preferably about 1 mm in size (2). Smaller particles are more difficult to disperse and do not appear to offer any advantages. Powdered tire rubber also contains antioxidants, antiozonants, sulfur accelerators, zinc oxide, carbon black, and other additives that can be beneficial for the long-term protection of the road surface by retarding embrittlement and increasing toughness. Some evidence suggests that long-term aging causes the bitumen molecules to interact with the rubber so that the rubber particles become more strongly bound to the bitumen. This slow reaction may be due to air oxidation or continued sulfur cross-linking during hot summer periods. This interaction may help to promote toughening in a manner analogous to the toughening of polystyrene and acrylics with rubber (16).

The fracture toughness G for a composite that contains randomly dispersed fibers wherein the fibers are all at their critical length L_c and with a volume fraction V_f and an ultimate tensile strength σ_{fu} will be given approximately by the following simple relationship:

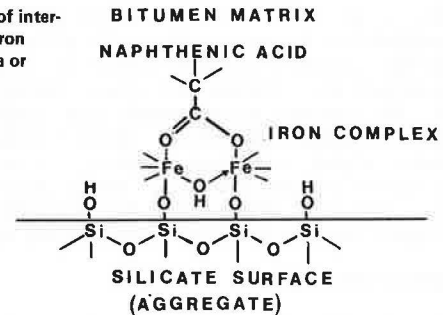
$$G \approx kV_f\sigma_{fu}L_c \quad (2)$$

where k is a proportionality factor that has a value near 0.1 for chopped nylon or polyester fibers. Thus for a volume fraction of 0.02 nylon cord that has a critical length of 5 cm and an ultimate tensile strength of 1 GPa, the total fracture energy is approximately 100 kJ/m² compared with an apparent fracture energy (including plastic deformation at the crack tip) for a typical asphalt concrete of 5 kJ/m² at -10°C. Thus, a relatively small proportion of either organic or inorganic fibers (that have an appropriate critical length) can increase the fracture toughness of asphalt concrete by a factor of 20 at low temperatures when the system is brittle. This toughening has been observed in experiments with other brittle materials such as portland concrete (7,8) and sulfur concrete (9). The above example would require about 20 lb of chopped fiber per ton of asphalt concrete mix, which would cost somewhere between \$10 and \$30/ton of mix. Therefore, the addition of even such a small proportion of fibrous material can add substantially to the total materials cost and may double the cost of a standard hot-mix formulation. Waste tire cord, if it can be recovered in the appropriate length, may represent a potential source of cheaper fibers for use in road construction.

Coupling agents are commonly used in the plastics industry to establish permanent bonds between inorganic fillers (such as glass) and polymeric organic molecules (17). The adverse effect of moisture on the mechanical properties of glass fiber composites is well known. Consequently, it is common practice to coat glass fibers with special organosilanes or chromium complexes (18) to establish strong covalent bonds that are less susceptible to the harmful effects of moisture. It is likely that such chemical reagents would be equally effective in asphalt concrete, except that cost prohibits their use.

Fortunately, there are cheap alternatives that are commercially available for this purpose. First observed by Fromm, the addition of iron oxide or iron derivatives to asphalt in minute quantities helps to prevent the loss of adhesion between silica and silicate-type aggregates in the presence of moisture. Iron naphthenates, commonly employed as driers in paints, are effective at concentrations of

Figure 6. Schematic of interfacial bond between iron naphthenate and silica or silicate surface.



less than 0.1 percent in asphaltic bitumen. Waste iron oxides from iron smelters, which are recovered as finely divided powders, are also effective. The latter are available at prices near \$50/ton and, at the small concentrations required, would help to ensure pavement stability without incurring significant added costs. The influence of iron compounds on limestone aggregates is unknown. However, contact-angle measurements may help to anticipate the influence of water on limestone and other aggregates.

The unusual effect of iron compounds on the adhesion of bitumen to silicate or glass materials is attributed to strong coordinate or covalent bonding, as suggested in Figure 6. The cohesive bond is largely made up of van der Waals forces. Similar interfacial structures have been identified in chromium derivatives that are used in the treatment of glass fibers (18).

A cautionary note should be added concerning the long-term effect of iron naphthenate on the oxidative hardening of asphalt. Since iron naphthenate is a commercial paint drier, it therefore promotes air oxidation of unsaturated hydrocarbons and may cause premature embrittlement of paving concretes. On the other hand, Fromm observed that asphalt pavements that contain a reddish-brown aggregate were particularly outstanding with respect to water stripping and deterioration, an observation that prompted a further study of the phenomenon and the discovery of soluble iron compounds as adhesion promoters (12).

CONCLUSIONS

Additives can help to alleviate some of the problems associated with the deterioration of asphalt concrete paving materials. It has been shown that, although the incorporation of reclaim rubber into asphalt concrete reduces the brittle temperature by as much as 10°C, it cannot eliminate cracking entirely. Under favorable conditions, the addition of chopped tire cord or other fibrous material to asphalt concrete can increase fracture toughness by a factor of 20. If possible, the process for the recovery of powdered reclaim from discarded tires should be modified to permit the recovery of tire cord in short lengths of about 5 cm.

The incorporation of iron oxide or iron naphthenate into a standard paving mix can help to improve adhesion of the bitumen to the aggregate and perhaps reduce the incidence of stripping, cracking, spalling, ravelling, and swelling due to attack by moisture. Iron oxide of suitable quality is available as a by-product from most steel foundries.

The combined use of rubber, fibers, and adhesion promoters should help to prolong the lifetime of asphalt concrete paving materials and reduce maintenance and thereby justify the additional cost.

ACKNOWLEDGMENT

Sincere appreciation is expressed to Harold J. Fromm

and Joseph D. George, whose helpful criticisms and suggestions based on their lifetimes of experience in the paving industry helped to motivate this investigation. Thanks are also accorded Tony French of the Toronto Metro Roads and Traffic Department and to Robert Hunter of Stelco Canada, Ltd., for their views and suggestions. Anwar Merchant of Hartford Fibers Company, Ltd., Kingston, Ontario, was primarily responsible for suggesting the use of chopped organic fibers as toughening agents.

REFERENCES

1. J. Shim Ton, K.A. Kennedy, M.R. Piggott, and R.T. Woodhams. Rubber Chemistry and Technology, Vol. 53, No. 1, 1980, pp. 88-106.
2. M.R. Piggott and R.T. Woodhams. Recycling of Rubber Tires in Asphalt Paving Materials. Environment Canada, Toronto, 1979.
3. J.L.M. Scott. Use of Rubber Asphalt Binder with Graded Aggregate for Seal Coats. Surfacing Branch, Saskatchewan Highways and Transportation, Regina, Canada, 1979.
4. A. French. RA-3 Rubberized Asphalt. Metro Toronto Roads and Traffic Department, Toronto, 1978.
5. J.I. McDougall. Rubberized Asphalt Concrete Memorandum Ref. 3162-2-1-10. Bituminous Section, Ontario Ministry of Transportation and Communications, Downsview, 1979.
6. M.R. Piggott. Methods of Making Composites More Tough. Centre for the Study of Materials, Univ. of Toronto, Canada, 1981.
7. G.C. Hoff and others. Chemical, Polymer, and Fiber Additives for Low Maintenance Highways. Noyes Data Corp., Park Ridge, NJ, 1979.
8. A. Neville and others, eds. Fibre Reinforced Cement and Concrete. Construction Press, Lunsdale House, Hornby, Lancaster, Ontario, 1975.
9. J. Ho and R.T. Woodhams. Fiber Reinforced Sulphur Concrete. Proc., Sulphur 81 Conference, Calgary, Alberta, Canada, May 25-28, 1981.
10. L. Ciplijauskas, M.R. Piggott, and R.T. Woodhams. Industrial and Engineering Chemistry, Vol. 18, 1979, pp. 86-91.
11. J.A.N. Scott. Adhesion and Disbonding Mechanisms of Asphalt Used in Highway Construction and Maintenance. Proc., AAPT, 1978.
12. H.J. Fromm. The Mechanisms of Asphalt Stripping from Aggregate Surfaces. Research and Development Division, Ministry of Transportation and Communication, Downsview, Ontario, Rept. RR 190, 1974.
13. R.J. Good and R.R. Stromberg, eds. Surface and Colloid Science: Experimental Methods. Plenum Press, New York, 1979.
14. S. Varevorakul. Adhesion Promoters in Bitumen. Department of Chemical Engineering and Applied Chemistry, Univ. of Toronto, B.A.Sc. thesis, 1981.
15. D.W. van Krevelen and P.J. Hoftyzer. Properties of Polymers, 2nd ed. Elsevier, New York, 1976.
16. R.D. Deanin and A.M. Crugnola, eds. Toughness and Brittleness of Plastics. American Chemical Society, Washington, DC, Advances in Chemistry Series 154, 1976.
17. Interfaces in Composites. American Society for Testing and Materials. Philadelphia, ASTM Special Tech. Publ. 452, 1968.
18. Quilon and Volan Chromium Complexes. E.I. du Pont de Nemours Tech. Brochure, Wilmington, DE, 1979.

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

Notice: The Transportation Research Board does not endorse products of manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Evaluation of Filler Effect of Sulfur in Asphalt Binder

AMIR F. BISSADA

The effectiveness of both dissolved and dispersed crystalline parts of sulfur in asphalt binders used in road-paving mixtures was evaluated. The role of sulfur as a binder extender and filler material was compared with that of conventionally used limestone filler. For this purpose, a series of viscosity measurements were performed on sulfur-extended asphalt (SEA) binders and limestone filler-extended asphalt (FEA). Ratios vary up to 50 percent by weight at a temperature range from 25°C to 140°C. Rotational and cone-plate viscometers were used in this study to evaluate the relative viscosity, temperature susceptibility, and shear susceptibility of these binders as a function of the volume concentration. The research indicates that (a) the effect of the dissolved part of sulfur in asphalt cement on the decrease in viscosity is balanced by the filler effect of a certain part of dispersed sulfur particles, which varies in its effective volume concentration according to the temperature of the SEA binder; (b) at temperatures below the softening point of asphalt cement, values of effective volume concentration for both SEA and FEA are found to be similar; and (c) the crystallization process of sulfur in terms of dynamic growth of the sulfur particles seems to be effective if the SEA is mixed with mineral aggregates and represents the major part of stiffness improvement of finished pavement with age. Increase in viscosity of SEA due to increase of effective volume

concentration of dispersed sulfur particles represents only a minor part of the stiffening effect.

During the past seven years, sulfur-extended asphalt (SEA) materials have been used to pave experimental road sections in many parts of the United States and in the Persian Gulf states in the Middle East. Investigations of the engineering properties of sulfur asphalt-paving mixtures and their structural response against both fatigue and permanent deformation have suggested that additional comprehensive evaluation of the SEA used and of its role as a binder extender as well as a filler material were warranted (1-4).

SEA is produced by high-shear-rate mixing of liquid sulfur with liquid asphalt. A part of the added sulfur by the dispersion technique ends up as dissolved sulfur. It is believed that the remainder

of the added sulfur, which is finely dispersed in a crystalline form, may have a kind of filler action that provides the observed increased strength of the paving mixtures.

Therefore, this study was aimed at the evaluation of the stiffening filler action of the added sulfur by analyzing the volume concentration of the dispersed sulfur crystals compared with limestone filler particles of equal amounts. For this purpose, a series of viscosity measurements were performed on asphalt binders extended with sulfur and limestone filler-extended asphalt (FEA) that had ratios varying up to 1:1 by weight to the asphalt. The analysis included the temperature as well as shear susceptibility of these binders and the effect of heating.

MATERIAL CHARACTERIZATION AND SAMPLE PREPARATION

The asphalt cement (AC) considered in this study is the only one produced from the local crude oil by the blowing process, graded as 60/70 penetration, and used in asphalt-paving mixtures in Kuwait. The sulfur used throughout the investigation of SEA is also a local commercial-grade product of 99.8 percent purity. The mineral filler used for comparison purposes is a locally produced limestone powder. A hydrometer analysis conducted on the filler passing sieve No. 200 (ASTM D422-63) showed that about 70 percent is finer than 10 μm and about 25 percent is finer than 1.5 μm in diameter. Photomicrographs of both elemental sulfur and limestone filler were taken with the scanning electron microscope and used to analyze particle shape and size. Sulfur and limestone agglomerates were found to be about 4 μm long on the average and the largest individual particle was about 1 μm long. These agglomerates are held together by surface forces, which cause the agglomerates to act as single particles. Grain form is found to be cubic with rounded corners for both sulfur crystals and limestone filler.

Both SEA and FEA were prepared with normal laboratory dispersing equipment operating at a constant speed of 1500 rpm. Prior to emulsification, both sulfur or filler and asphalt were proportioned by weight and preheated at about 150°C. The melted sulfur and the asphalt were added simultaneously to the receiving hopper of the mixer while it was running at its constant speed. For FEA, samples were prepared by slowly mixing the filler into the asphalt by hand and then by mixing at the same shear rate in the mixer. Shearing time was kept to 8 min for all samples, during which the mill clearance varied between 1.0 mm and 3.0 mm. To prevent coagulation and settlement of dispersed particles, a sample of the emulsified binder was used immediately after mixing for viscosity measurement. Samples were kept in the sensors of the viscometer in rotation at relatively high shear rates while viscosities were measured at temperatures above 80°C. As the binder temperature dropped below 80°C, the settling rate was found to approach zero.

TESTS AND TEST METHODS

The viscosity of SEA material was measured and compared with those of conventional AC and FEA materials used locally at a range of temperature between 25°C and 140°C, which covers operation and prevailing pavement service temperature conditions. At temperatures between 60°C and 140°C, a rotational viscometer with different measuring cylinder sensor system is used, which allows the use of different shear rates that range from 10^{-1} to 10^3 s^{-1} and the application of shear stress values up to 5×10^3 Pa. At temperatures between 25°C and

60°C, viscosities were measured by using a cone-plate viscometer that has a measuring head of 49 N-cm maximum torque and 1:100 reduction gear, which allows for determining flow behavior of the binder materials at a starting shear value of 10^{-3} s^{-1} and at shear stress values up to 10^6 Pa.

A viscosity value measured by using a given shear stress and an arbitrarily selected value of shear rate is termed "apparent" viscosity. At relatively low shear rates that induce low shear stress, the apparent viscosities appear to be constant. Such a constant, relatively shear-independent viscosity is designated as "initial" viscosity. In all relative low-temperature viscosity measurements, an attempt was made to establish such initial viscosities. To accomplish this, it was necessary to exceed the shearing time allowed in ASTM D3205 by using gradually increasing shearing loads (alternate method 1) from 10 to 20 min (5).

The temperature was maintained by circulating a light oil in a jacket outside the cup of the sensor for the rotoviscometer and underneath the plate for the cone-plate viscometer in which the sensor system is located inside a temperature-control chamber. The reading on the control thermometer is an indication of the cup or plate temperature. Before each test, the sample was kept for 30 min at the fixed temperature. The instruments allow for setting the shear rate either at 40 fixed-speed values or at a programmed steplessly variable speed setting and speed-change direction (increasing, decreasing, or constant).

Viscosity η is determined as a function of shear rate D or shear stress S . These are given by $\eta = S/D$. For the rotor-cylinder sensor used at 60°C to 140°C, the following equations hold:

$$S = (T \cdot a) / (2\pi \cdot L^* \cdot R_i^2) \quad (\text{Pa}) \quad (1)$$

$$D = [(n \cdot \pi) / 15] \cdot [R_a^2 / (R_a^2 - R_i^2)] \quad (\text{s}^{-1}) \quad (2)$$

For the cone-plate sensor used at 25°C to 60°C, the following equations hold:

$$S = (3T \cdot a) / (2\pi \cdot R_c^3) \quad (\text{Pa}) \quad (3)$$

$$D = (n \cdot \pi) / (30 \cdot \alpha) \quad (\text{s}^{-1}) \quad (4)$$

where

- T = torque (N·cm),
- a = constant correlation factor (to correlate torque applied at shaft of measuring head to indicated scale value),
- n = speed in revolutions per minute of the rotor (min^{-1}),
- L^* = corrected height of rotor sensor (cm),
- R_i = radius of rotor sensor (cm),
- R_a = radius of cup (cm),
- R_c = radius of cone (cm), and
- α = opening angle of cone (rad) ($1^\circ = \pi/180$ rad).

Viscosity measurements were carried out on freshly prepared binder samples as well as on aged ones held in the sensor of the viscometer at 25°C for 1, 24, and 72 h before viscosity measurements were run at 25°C and at different shear-rate values. These tests are supposed to study the phenomenon of the slow development of internal structure due to the formation of the crystallization process in SEA.

SEA, like other asphalt binders, during their application and use are subjected to heating for variable time periods and at a wide range of temperatures. It is well known that such heating affects properties of asphalt binders and that this

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

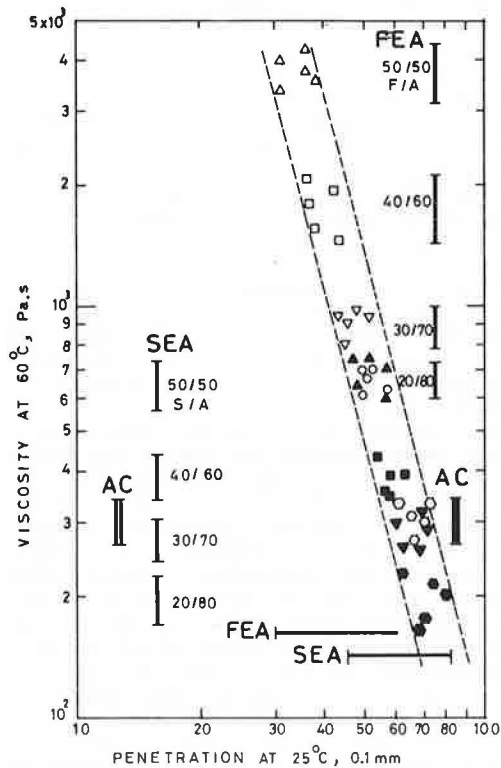
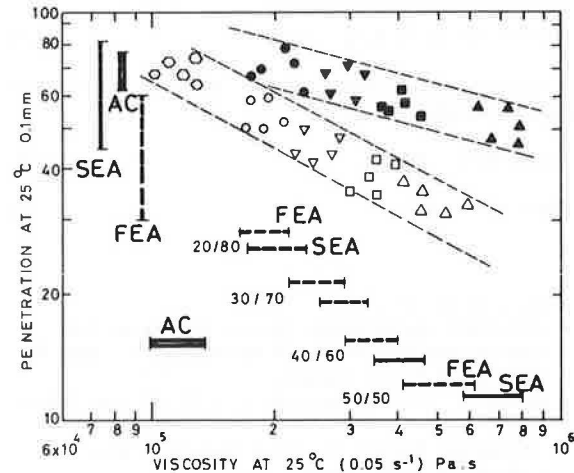


Figure 2. Relationship between viscosity at 25°C and penetration at 25°C.



in turn influences the behavior of the paving mixtures. For the evaluation of the heating effects, the thin-film oven test (TFOT) was applied on an SEA film about 3 mm thick at a temperature of 140°C for 7 h instead of 163°C for 5 h as described by AASHTO T240178. These changes were made to avoid any sulfur-asphalt chemical reaction by dehydrogenation and formation of H_2S during the process of heating the SEA. Viscosities were measured at several temperatures after heating and compared with viscosities measured for AC and FEA heated under the same testing conditions.

DISCUSSION OF RESULTS

Relationship Among Different Rheological Properties

A comparison among three properties currently used

as a basis for grading asphalt binders is given. The properties are viscosities at 25°C and 60°C, penetration at 25°C, and softening point. Figures 1 and 2 illustrate the relationship among viscosities at 25°C and 60°C of fresh prepared samples of AC, SEA, and FEA and compare such viscosities with penetration measured at 25°C. Figure 1 is a comparison of the two properties currently used as a basis for grading AC. Both viscosity and penetration ranges for AC, SEA, and FEA are indicated by bars drawn in the proximity of the appropriate axis. It may be seen readily that, through the extension of AC 60-70 penetration grade with sulfur in ratios up to 50/50 by weight, the viscosity varied between 160 and 800 Pa·s, a fivefold variation. This range of SEA viscosities that belong to a range of penetration grades varies between 80 and 45. The comparison of SEA with FEA for the same ratios up to 50/50 by weight indicates that the viscosity varied between 600 and 4000 Pa·s (about sixfold variation) and belonged to a range of penetration grades that varied between 60 and 30.

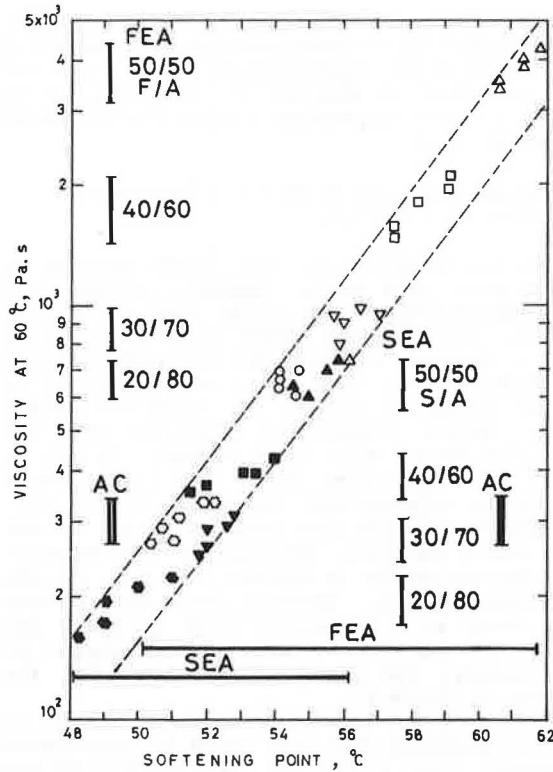
The linear relationship between log viscosity at 60°C and log penetration at 25°C illustrated in Figure 1 for both SEA and FEA indicates the relatively narrow range of penetration values compared with the wide variation of viscosities; this is a major reason for proposing viscosity grading to evaluate the behavior of these binders. It is rather obvious that two binders that belong to the same penetration grade but exhibit about threefold variation in their viscosities would behave quite differently when exposed to variable construction operation temperatures or pavement service temperatures.

The comparison of data points in Figure 1 indicates that there exists an overlap of viscosity and penetration grades for SEA with S/A ratios between 40/60 and 50/50 and FEA with F/A ratios between 20/80 and 30/70 by weight. Thus, it may be assumed that the relationship between viscosities at 60°C and penetration at 25°C is very similar for SEA and FEA with these concentration ratios.

Figure 2, which relates viscosity and penetration at 25°C, is included to provide a comparison of the low-temperature consistencies of SEA and FEA binders. Since viscosities of these binders are affected by the shear rate applied, apparent viscosities at a shear rate of 0.05 s^{-1} were used. Figure 2 indicates a general broad correlation between the two properties in question. As can be judged from the data symbols, no substantial differences can be observed between the SEA and FEA viscosity values at 25°C at each concentration value. A pronounced overlap in the viscosity grades measured for both SEA and FEA is indicated. Viscosities vary between 2×10^5 and $8 \times 10^5 \text{ Pa}\cdot\text{s}$ for the range of concentrations considered in this analysis. This means that both sulfur and limestone filler additives within these percentages provide a similar effectiveness on the viscosity values of the binder at 25°C. Figure 2 also indicates that the relationships between viscosity at 25°C and penetration at 25°C for SEA and FEA are not similar as in the case of viscosity at 60°C and penetration at 25°C, shown in Figure 1. This could be due to the susceptibility of these binders at lower temperatures to variable shear rates applied during the penetration and viscosity tests.

The softening point of the AC 60-70 penetration grade used was found to range between 48°C and 51°C. Over the concentration range in this study of sulfur and limestone filler in both SEA and FEA, the softening point was found to change between 48°C and 62°C. A linear relationship between log viscosity at 60°C and softening point for both SEA and FEA is

Figure 3. Relationship between viscosity at 60°C and softening point.



illustrated in Figure 3. The comparison of data points in Figure 3 indicates that an overlap of viscosity and softening point exists for SEA with an S/A ratio of 50/50 by weight and for FEA with F/A ratios of 20/80 and 30/70 by weight. Thus, it may be also assumed that the relationships between viscosities at 60°C and the softening point are similar for SEA and FEA with these concentration values.

Viscosity as Function of Volume Concentration

The volume concentration of the dispersed particles is an important variable that influences the viscosity of asphalt paving materials. Since the SEA normally used has a low sulfur concentration, the effect of the shear rate may not become an overriding factor and the relationship among the relative viscosity η_{rel} , the volume concentration C_v , and the occupation ratio V_p proposed in the work of Eilers (6) and of Heukelom (7) could be applied:

$$\eta_{rel} = \left\{ 1 + [1.25C_v / (1 - V_p \cdot C_v)] \right\}^{3.6} V_p^{-2.8} \quad (5)$$

where $\eta_{rel} = \frac{\eta_{SEA}}{\eta_{AC}}$ or $\frac{\eta_{FEA}}{\eta_{AC}}$. An infinite value of the relative viscosity was found at $C_v(max) = 0.78$ in correspondence with an occupation ratio $V_p = 1.35$ for dispersed graded spheres.

A nearly linear relationship between $1/\eta_{rel}$ and C_v exists up to a relative viscosity of about 100. This can be demonstrated by rewriting Equation 5 as follows:

$$1/\eta_{rel} \approx 1 - 1.25C_v \quad (6)$$

By introducing constant k and m in the above equation, which accounts for the effect of shear dependence of dispersion and the shape of the dispersed

particles (mC_v is the virtual increase in concentration), respectively, Equation 6 can be written in the following more general form:

$$\begin{aligned} 1/\eta_{rel} &= 1 - [(1+k)(1+m)C_v] \\ &= 1 - (C_v/C_{max}) \end{aligned} \quad (7)$$

where $C_{max} = 1/[(1+k)(1+m)] =$ the value of C_v at which the viscosity tends to become infinite. For the case of fully peptized spherical particles, $m = 0$, and for low shear rates, $k = k_0 = 1/C_{max} - 1 = 0.28$.

The results for the relative viscosity, derived from the rotor and cone-plate viscometers at a low shear rate (0.05 s^{-1}) and at temperatures that range from 25°C to 140°C, were plotted in Figures 4 and 5 for SEA and FEA, respectively. For both sulfur and limestone filler ratios that vary up to 50 percent by weight of total binder, the relative viscosity of both binders is found to be temperature dependent.

At temperatures below the softening point, relative viscosity values of both SEA and FEA are similar. Above the softening point of these binders and up to the melting point of sulfur, the stiffening effect of the sulfur additive was found to be much lower than that of the limestone filler. At 50 percent by weight of total binder, relative viscosity values measured were 1.6 and 12.5 for SEA and FEA, respectively. Above the melting point of sulfur and below the dehydrogenation temperature, relative viscosity values of SEA ranged between 0.45 and 0.55.

Figure 6 shows the relative viscosities in terms of $1/\eta_{rel}$ of both SEA and FEA as a function of volume concentration. From the resulting linear relationship shown in Figure 6, the values of C_{max} for both SEA and FEA can be compared and used to compute the corresponding values of the correction factor $(1+m)$, introduced in Equation 7, as $(1+m) = 0.78/C_{max}$ for the actual dispersed particles.

At 25°C, relative viscosity values of both SEA and FEA were found to be similar. As the temperature increases, the volume concentration of sulfur needed to maintain a relative viscosity equal to 1 increases. For SEA, volume concentrations at $\eta_{rel} = 1$ were found to be equal to 0.08, 0.21, 0.27, and 0.43 (equivalent to 15, 35, 43, and 61 percent by weight) at temperatures of 40, 60, 100, and 120°C, respectively. It can be assumed that at $\eta_{rel} = 1$, the effect of the dissolved part of the sulfur in asphalt cement on the decrease in viscosity has been balanced by the effect of a certain part of dispersed sulfur crystals that varies in its volume concentration according to the temperature of the SEA binder.

From the test data illustrated in Figure 6, it can be observed that values of effective volume concentration of both SEA and FEA at temperatures below the softening point are similar ($C_{max} = 0.51$). At a temperature range above the softening point ($\approx 50^\circ\text{C}$) and below the melting point of sulfur ($\approx 116^\circ\text{C}$), the peptization state of particles in both SEA and FEA changed so that the values of effective volume concentration C_{max} are much higher than those at temperatures below the softening point. The maximum value of volume concentration C_{max} for both limestone filler and dispersed sulfur particles in the asphalt binder was found to be 0.37.

However, for SEA, the type of interaction between the sulfur particles and the asphalt medium changes as soon as the temperature reaches the melting point of sulfur ($\approx 116^\circ\text{C}$). At this stage, a case of quasi-complete peptization exists. However, an increase in viscosity still occurs, depending on the

forces of repulsion between the sulfur and the asphalt particles, which causes an increase in size of each particle and therefore in effective volume concentration.

Figure 4. Relative viscosity of SEA versus sulfur content.

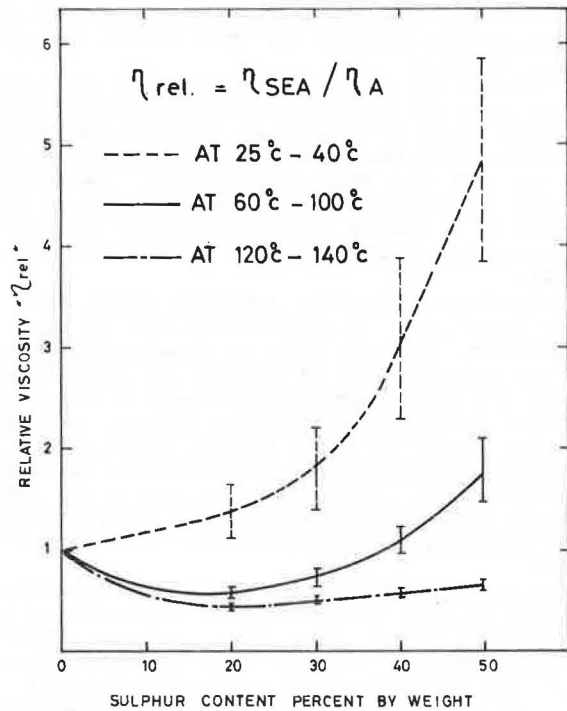
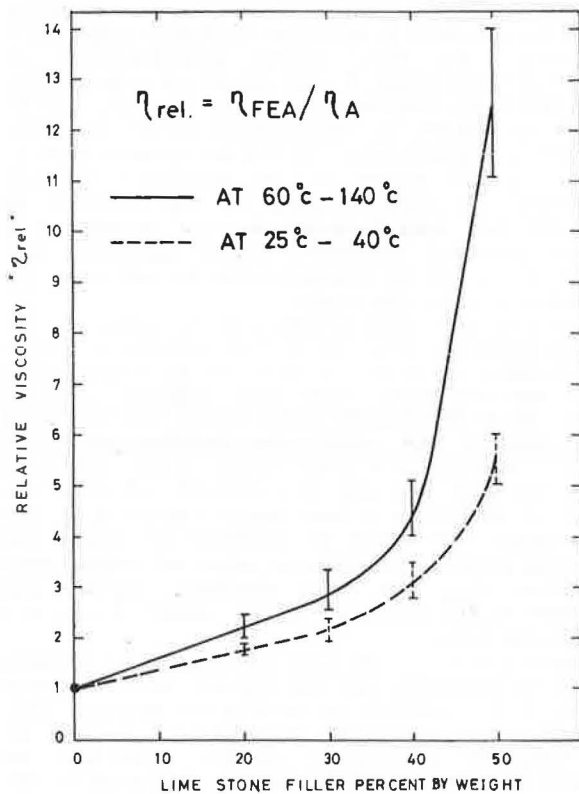


Figure 5. Relative viscosity of FEA versus limestone filler content.



Temperature Susceptibility

For SEA, the change in viscosity of the binder with changing temperature is described here by means of a viscosity-temperature susceptibility (VTS) index and compared with that for FEA and AC. For calculating VTS within a given temperature range, the following expression is used:

$$VTS = (\log \log \text{viscosity at } T_2 - \log \log \text{viscosity at } T_1) / (\log T_1 - \log T_2).$$

In most VTS calculations, viscosities measured at 135°C and 60°C were used. However, because SEA gives viscosity-temperature curves that consist of two straight lines, which differ from those of AC and FEA, viscosities at 100°C and 40°C were considered. Figure 7 shows that for SEA composed of 50 percent by weight of sulfur, the curve of log-log viscosity versus temperature consists of two branches that have nearly equal slopes. It can be seen that at temperatures lower than 100°C, where the sulfur is crystallized, the slope of the viscosity temperature line goes above that for AC and they are nearly parallel. However, at high temperatures, where the sulfur is molten, the curve for SEA is considerably lower. Between these two straight branches there is a transition range in which the test data are scattered because of differences in thermal history; the range is represented by the dashed line in Figure 7.

For SEA composed of 20 percent by weight of sulfur, the viscosity test data could be represented by a straight line, within the repeatability of the tests, as shown in Figure 7. However, the line has a steeper slope than that of AC, which indicates that the temperature susceptibility of SEA (of 20 percent by weight of sulfur) is much higher than that of SEA of 50 percent by weight of sulfur. Figure 8 provides a comparison of temperature susceptibilities of FEA composed of 20 and 50 percent by weight of limestone filler. From the VTS index calculated, it is found that SEA binders are more temperature-susceptible than asphalt cement. VTS values decrease from an average of 3.96 to 3.59 and the increase of sulfur content is from 20 to 50 percent by weight. However, lower VTS values calcu-

Figure 6. Relative viscosities of SEA and FEA as a function of volume concentration.

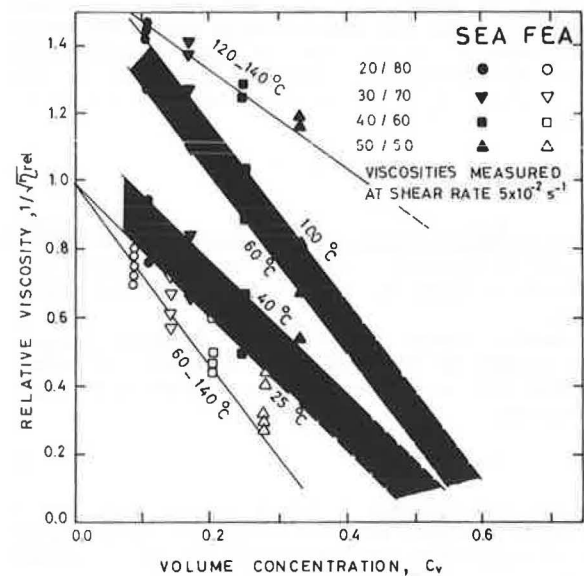


Figure 7. Viscosity-temperature relationship for SEA.

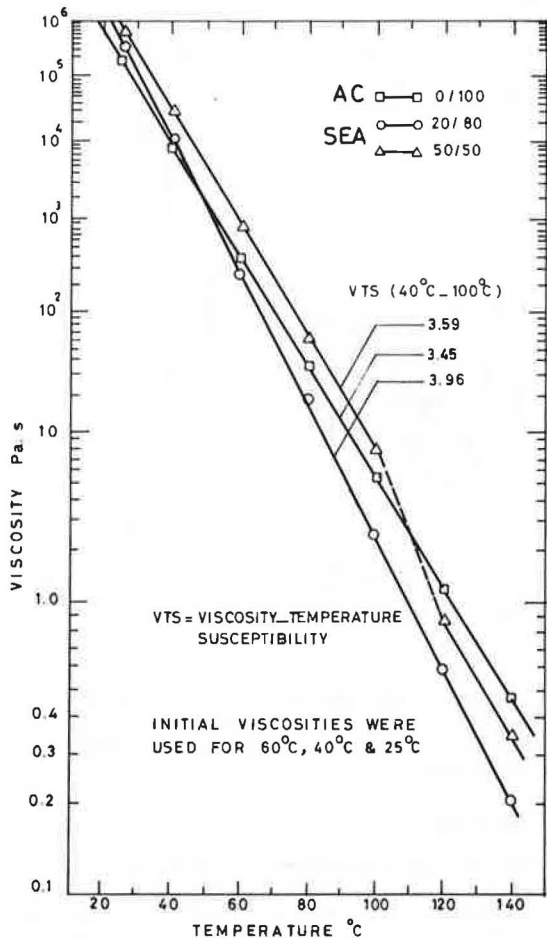
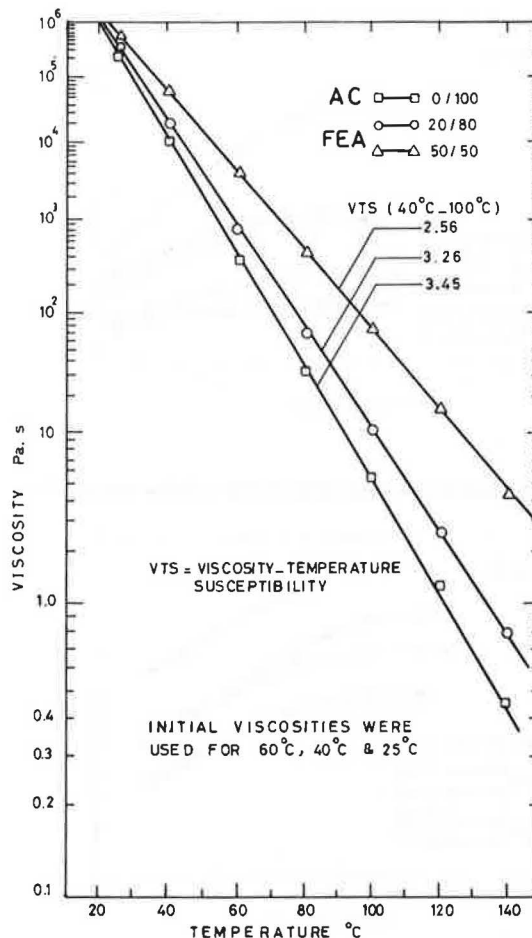


Figure 8. Viscosity-temperature relationship for FEA.



lated for limestone FEA (3.26 and 2.56 for 20 and 50 percent by weight, respectively) indicate a less-pronounced viscosity change of FEA with changing temperature compared with that of SEA binders.

At temperatures higher than 120°C, viscosities measured for SEA were found to be much less than those for AC and FEA of the same volume concentration. As may be seen in Figures 7 and 8, at 120°C, the viscosity of AC is about twice that of SEA (20/80) and about 1.5 that of SEA (50/50). However, viscosities measured for FEA at the same temperature were found to be five times higher than that of SEA for a 20/80 ratio by weight and they increase to about 15 times that of SEA for a 50/50 ratio by weight. It should be noted that these relatively low viscosities of SEA at a temperature range between, say, 120°C and 140°C lead to questions related to the mixing, laying, and compaction-temperature requirements of sulfur paving mixtures, which should be included in the specifications of SEA, and to find out the benefits that can be derived or expected from such requirements in specifications.

Shear Susceptibility

Figures 9-11 depict viscosity versus shear rate for SEA compared with that of AC and FEA, each at three temperatures that vary from the ambient room temperature of 25°C to 60°C. Figures 9-11 were prepared by plotting the logarithm of apparent viscosity in pascal seconds against the logarithm of shear rate in reciprocal seconds. It should be noted that at 60°C, within the indicated range of

shear rate, all asphalt binders tested except FEA made up of 50 percent by weight of limestone filler behave approximately as a simple Newtonian liquid; their viscosities are not affected much by the varying shear rate. Shear susceptibility calculated at 60°C (tangent at an angle of the plot of log viscosity versus log shear rate at a shear rate of 5×10^{-1}) was found to range from about 0.25 for SEA and AC to about 0.5 for FEA (Figure 9).

However, the non-Newtonian behavior becomes progressively more apparent as test temperature decreases. As illustrated by the gradually increasing slopes of the curves in Figures 10 and 11, the viscosities of SEA, AC, and FEA become more dependent on shear rate. Shear susceptibility calculated at 25°C was found to vary between 1.12 and 1.18 for SEA and AC. For FEA it ranged between 0.7 and 0.85. The curves for low shear rate are horizontal and parallel to the shear-rate axis. This part of the curve, which is not dependent on shear rate, is called the initial viscosity.

It was difficult to evaluate the effect of volume concentration on shear susceptibility due to factors other than shear rate, which influences viscosity measurements, specifically at 25°C. Factors such as the effects of loading sequence and sample conditioning were not considered in this work. However, it has been found during the shear test that the reversal of the loading sequence resulted in a difference in the apparent viscosity values. Measurements with gradually increasing shear stress generally resulted in higher apparent viscosity values.

Figure 9. Relationship between apparent viscosity and shear rate at 60°C.

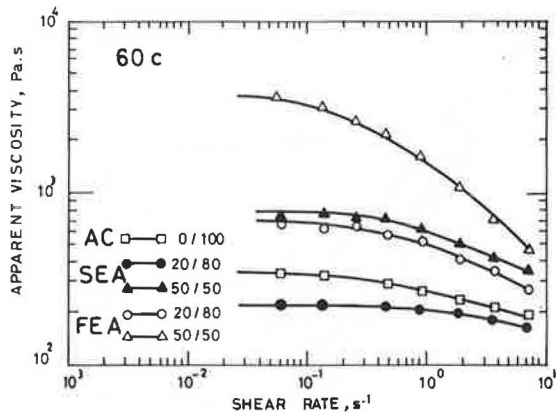


Figure 10. Relationship between apparent viscosity and shear rate at 40°C.

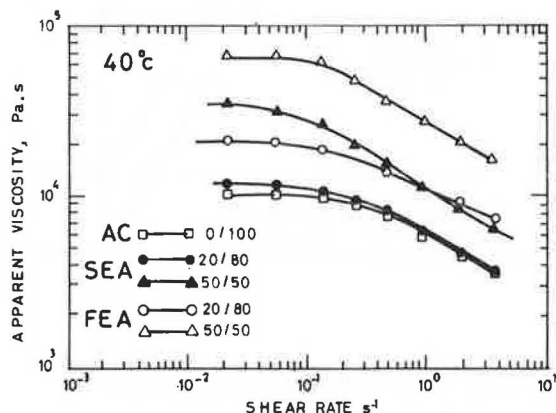
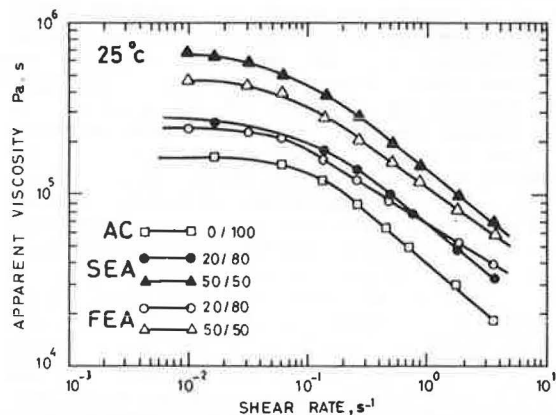
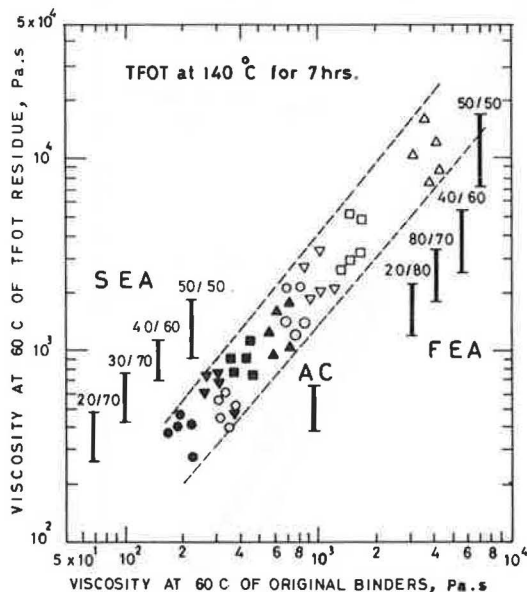


Figure 11. Relationship between apparent viscosity and shear rate at 25°C.



Also, the conditioning of samples of SEA, FEA, and AC held in the viscometer at 25°C for 24 and 72 h before the shear test was run at the same temperature resulted in higher apparent viscosity values in the range of low shear rates (less than 0.1 s^{-1}). The measured increase in viscosity values after three days' aging for both SEA and AC at a shear rate of 5×10^{-2} was found to be about 150 percent of the fresh tested sample. This could be due to the internal structure developed in these asphalt binders but it is not an indication of the crystallization process of sulfur in SEA, which was

Figure 12. Relationship between viscosity at 60°C of original and of heated SEA, FEA, and AC.



found to increase the stability of the sulfur-asphalt mixture. The crystallization process of sulfur seems to be effective if SEA is mixed with mineral aggregates.

Effect of Heating

A comparison of the viscosities of SEA, AC, and FEA at 60°C before and after TFOT heating (7 h at 140°C) is shown in Figure 12. The vertical bars indicate the ranges of viscosity after heating. It should be noted that the straight lines are used to envelop the data points plotted on the logarithmic coordinates. This suggests a certain regularity in the increase of viscosity at 60°C for SEA, AC, and FEA due to the TFOT heating condition prescribed. It is evident that, based on the original viscosity of the tested asphalt binders, aging-index values were found to vary between a minimum of 1.5 and a maximum of 4.5. The heat sensitivity of SEA does not vary substantially from that of AC and FEA under the TFOT heating conditions considered. It is assumed that if the test temperature is raised to 163°C (according to ASTM D2872) and the exposure time is changed, expected different aging-index values could result due to the effect of sulfurization, which may occur at temperatures above 150°C. Since SEA binders are not exposed during the mixing and paving operations to temperatures of more than 150°C, it was decided to look at the effects of heat and air at 140°C.

CONCLUSIONS

This study presents an evaluation of the physical properties of SEAs made with currently produced asphalt cement (60-70 penetration grade) used in Kuwait for asphalt pavement mixes. The main study objective was an evaluation of the physical properties of SEA with the emphasis on such properties as locally high pavement service temperatures compared with conventional limestone FEAs with same volume concentrations.

Test data presented in this study led to the following observations and conclusions:

1. Viscosity grading of SEA at 60°C provides a

fundamental measure of consistency for use at relatively high pavement service temperatures. It is less subject to test-imposed variables such as shear-susceptibility variations and less subject to binder-improved variables such as variations in inherent complex flow. On the other hand, penetration measurements as a control of the consistency of SEA would not be adequate to evaluate the temperature susceptibility of these binders.

2. At the average maximum service pavement temperature in Kuwait ($\approx 60^{\circ}\text{C}$), the volume concentration of sulfur needed to maintain a relative viscosity of $\eta_{\text{SEA}}/\eta_{\text{AC}} = 1$ was found to be equal to 0.21, which is equivalent to 35 percent by weight. However, at 25°C , relative viscosity values of both SEA and FEA were found to be similar. These results are only valid for the materials used locally. Asphalt cements from different sources or of other grades may have a different effect on the rheological properties of SEA.

3. The dissolved sulfur in SEA (16-18 percent) results in lowering the viscosity of asphalt cement at a range of temperature between 140°C and 40°C and in an increase of the temperature-viscosity susceptibility. However, the suspended sulfur particles in SEA showed a similar effective volume concentration compared with that of limestone filler particles with reference to the increase in viscosity at temperatures below the melting point of sulfur. It is recommended that the filler effect of suspended sulfur particles in SEA be compared with that of other types of filler materials used in asphalt mixes.

4. Viscosities of SEA at temperatures above the melting point of sulfur were found to exist at all sulfur ratios lower than that of asphalt cement and much lower than that of FEA. This characteristic of SEA has a significant effect on improving the workability of the paving mix.

5. The crystallization process of sulfur in terms of dynamic growth of the sulfur crystals seems to be effective if SEA is mixed with mineral aggregates and represents the major part of the stiffness improvement of finished pavement with age. Increase in viscosity of SEA due to increase of effective volume concentration of dispersed sulfur particles represents only a minor part of the stiffening effect.

6. In paving mixtures, at temperatures of pavement use, SEA is admixed with a variety of mineral

aggregates and forms binders varying greatly in properties from the original in bulk. The response of SEA to heating in the preparation of the hot paving mixture differs from that in the laboratory. Therefore, measurement of paving-mixture properties, rather than properties of SEA binder, appears to be a more rational approach. Generally, the responses for SEA, AC, and FEA to TFOT heating at 140°C are almost the same. The viscosity at 25°C is affected by heating more than that at higher temperatures.

ACKNOWLEDGMENT

This research was conducted by the Civil Engineering Department of the College of Engineering and Petroleum of Kuwait University. Appreciation is expressed to all individuals who gave assistance in the laboratory work and in typing the drafts. The opinions, findings, and conclusions in this paper are solely mine and do not reflect the views of the Highway Department in Kuwait.

REFERENCES

1. G.J.A. Kennephol, A. Logan, and D.C. Bean. Conventional Paving Mixes with Sulfur-Asphalt Binder. Proc., AAPT, Vol. 44, 1975.
2. D. Saylak, B.M. Gallaway, and J.S. Noel. Evaluation of Sulfur-Asphalt Emulsion Binder System for Road-Building Purposes. Texas A&M Research Foundation, College Station, Project RF3146, Final Rept., Jan. 1976.
3. C. Garrigues and O. Vincent. Sulfur-Asphalt Binder for Road Construction. Advances in Chemistry Series, Vol. 140, 1975, pp. 130-153.
4. T.W. Kennedy, R. Haas, P. Smith, G.A. Kennephol, and E.T. Hignell. Engineering Evaluation of Sulfur-Asphalt Mixtures. TRB, Transportation Research Record 659, 1977, pp. 12-17.
5. V.P. Puzinauskas. Properties of Asphalt Cements. Proc., AAPT, Vol. 48, 1979.
6. H. Eilers. Kolloidzeitschrift. Vol. 97, 1941, p. 313; Vol. 102, 1943, p. 154.
7. W. Heukelom and P.W. Wijga. Viscosity of Dispersions as Governed by Concentration and Rate of Shear. Proc., AAPT, Vol. 40, 1971, pp. 418-437.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Predicting Surface Friction from Laboratory Tests

W.H. PARCELLS, JR., T.M. METHENY, AND R.G. MAAG

The objective was to develop and refine methods for preevaluating aggregates and paving mixtures so that predictions can be made of properties of proposed and in-service pavement types. A usable correlation was established between the field testing by using the data from the British portable tester and the locked-wheel pavement friction trailer at speeds of 40 and 55 mph. To extend this correlation, core samples were extracted from the locked-wheel tester skid path and were subjected to wear on the small-wheel circular track with periodic surface friction testing by using the British portable tester. The final step was to remix and remold the cored pavement samples or make samples with new materials to obtain an "as-new" surface and to subject these samples again to wear on the small-wheel circular track with

periodic testing by using the British portable tester to find the British pendulum number (BPN). Other segments of the project included efforts to correlate (a) the stereophotography number (SPN) with the locked-wheel pavement friction tester skid number, (b) the SPN with BPN, and (c) the linear traverse number with BPN on the wear and polish machine. Research with various chat (chert) and limestone mixtures shows that the blend offers good skid resistance.

The objective was to develop and refine methods for preevaluating aggregates and paving mixtures so that predictions could be made of the skid-resistance

Figure 1. British portable tester versus locked-wheel tester for 17 HM-R/BM-1 test locations.

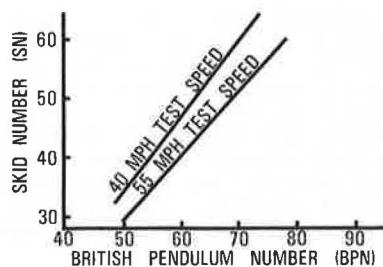


Figure 2. British portable tester versus locked-wheel tester for 18 HM-3/BM-2 test locations.

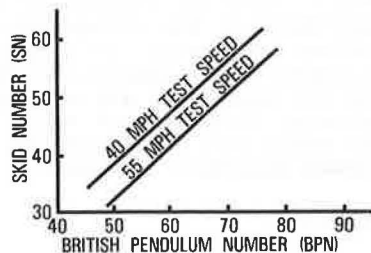


Figure 3. Test speed versus locked-wheel tester with constant BPN for 17 HM-R/BM-1 test locations.

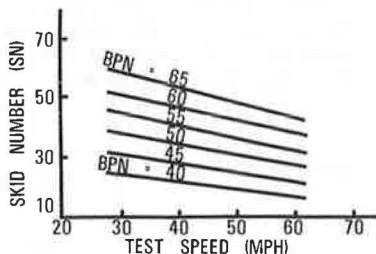
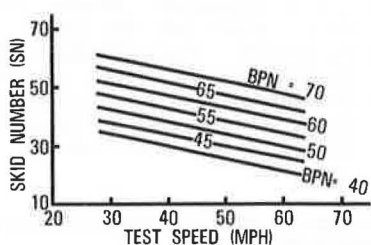


Figure 4. Test speed versus locked-wheel tester with constant BPN for 18 HM-3/BM-2 test locations.



properties of proposed and in-service pavement types. The procedure used in conducting the test was patterned after the work done under the direction of Mullen (1). The small-wheel circular-track wear and polish machine was built by Kansas Department of Transportation (KsDOT) personnel according to plans obtained from Mullen. The locked-wheel pavement-surface friction tester (skid trailer) was built by K.J. Law Engineers, Inc., and conforms to ASTM E-303-69.

PRELIMINARY FIELD-CORRELATION RESEARCH

The testing was conducted on two dense-graded bituminous pavement types in Kansas. One is now designated BM-2, but in the past it was known as HM-3. Eighteen sections of HM-3/BM-2 pavement were selected across the state. The age of the pavement sections varied from zero to 17 years. The second pavement type tested is currently designated BM-1 and in the past it was called HM-R. Seventeen sections of HM-R/BM-1 pavement were chosen and the age of these pavement sections ranged from zero to nine

years. At each location the pavement friction property was tested by using the locked-wheel trailer to determine the skid number (SN) at 40 mph (SN_{40}) and at 55 mph (SN_{55}) both in the left wheel path and between the wheel paths.

This procedure resulted in 10 skid paths (five in the wheel path and five between wheel paths) at each of the 35 locations and both an SN_{40} and an SN_{55} for every skid path. The usual length of such a test location is 2 miles if the pavement surface being tested is long enough but may be shorter if necessary in order to contain the entire series of tests on a specific pavement surface.

The British portable tester, also known as the British pendulum, was used in determining all 350 skid paths and was cycled seven times in each path. The resulting SNs were averaged to provide a single British pendulum number (BPN) for each of the 350 skid paths. The data collected by locked-wheel testing and British pendulum testing were used to establish a set of correlation curves (Figures 1-4). A Hewlett-Packard Model 65 programmable calculator and statistical program 1-22A for linear regression was used to make the calculations to establish the best-fit lines shown in Figures 1-4. For the test locations of BM-1 pavement, the calculated correlation coefficient was 0.66 at 55 mph and 0.73 at 40 mph. The calculated correlation coefficient for the BM-2 test locations was 0.65 at 55 mph and 0.68 at 40 mph. It should be remembered, however, that SN reflects both macrotexture and microtexture, whereas BPN primarily reflects only the microtexture (2).

After completion of these two tests, a series of core samples 6 in in diameter was obtained from the pavement. Eight samples were taken from one of the five spots where the trailer tire had been skidded in the wheel path and seven samples were taken from one of the five spots where the trailer tire had been skidded between the wheel paths at each of the 35 locations. There was a total of 525 field samples for laboratory testing.

SELECTION OF COMPACTION METHOD

Three methods of compaction were tried on laboratory samples by using a 6-in mold. They were static-load compaction of 2000 psi for 2 min; electric-vibratory-hammer compaction of 15 s, 30 s, and 60 s; and 50 blows to one side of the sample by using the Marshall hammer, which weighs 22.5 lb. Laboratory samples made by using each compaction method were exposed to the North Carolina wear and polish machine concurrently with a core sample from the field and the results indicated that the sample made by using the Marshall hammer most closely paralleled the field sample in BPN versus time of exposure on the wear and polish machine. The asphalt was extracted from samples made by each method and aggregate gradation was performed. The results indicated that the Marshall-hammer method caused less aggregate fracturing. The Marshall-hammer method also resulted in higher compaction levels and greater stability.

LABORATORY TESTING OF FIELD CORES

The next procedure was to prepare the core samples by sawing them to a thickness of 1.87 in and mounting them in the small-wheel circular track (Figure 5) for exposure to wear and polish (Figure 6). Three samples from each location were used, which allowed four skid-path locations to be tested simultaneously. Three other samples from each location were heated, remixed, and remolded before exposure to the wear and polish machine in an effort to ob-

Figure 5. Pavement core samples mounted on circular track.

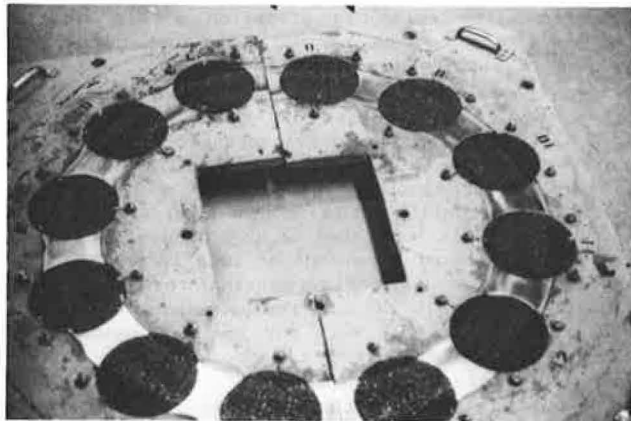


Figure 6. North Carolina small-wheel circular-track wear and polish machine.



Figure 7. British portable tester on wear and polish samples.

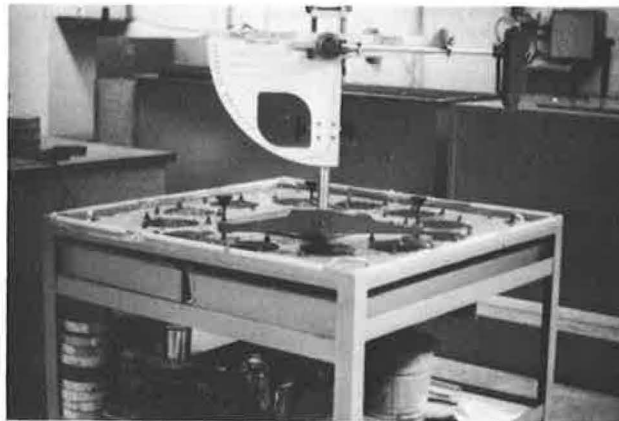


Figure 8. British portable tester readings versus time during wear and polish testing of HM-R/BM-1 field core samples.

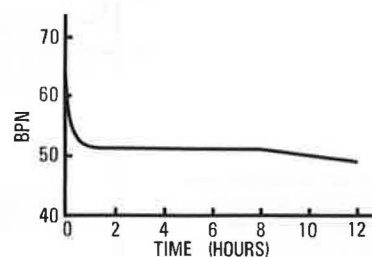
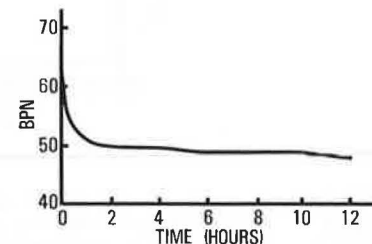


Figure 9. British portable tester readings versus time during wear and polish testing of HM-3/BM-2 field core samples.



tain an "as-built" surface. At least one sample was used for asphalt extraction and aggregate gradation tests.

British pendulum readings (Figure 7) were taken at 0, 10, 20, and 30 min and 1, 1.5, 2, 2.5, 3, 4, 6, 8, 10, and 12 h of exposure on the wear and polish machine. At each time interval, the pendulum was cycled eight times and the last seven readings were recorded and averaged. This yielded more than 21 000 recorded BPN readings, which were used to establish the graphs of BPN versus time of exposure (T) on the small-wheel circular track (Figures 8 and 9). As had been noted in the study by North Carolina University (1), the samples reach terminal polish very early in the cycle and are near the BPN low-point reading within the first hour. The rapid change in BPN indicates that BPN readings should be recorded at shorter intervals during the first 30 min of exposure to the small-wheel circular track so a more sensitive curve could be generated. All the data gathered for this report were obtained by using a tire 5.2 in wide on the wear and polish machine. Use of the narrower 3.5-in tire was discontinued because of the small wear surface on the sample, which made operation of the British portable tester difficult, and because the narrow tire raveled and rutted the pavement samples more severely. Since all testing was subsequently done by using the wider tire, no correlation was attempted with the narrow-tire data.

Testing of Two BM-1 Mix Designs

A set of 12 laboratory samples of a recent BM-1 mix design was made by using 50 blows from a 22.5-lb Marshall hammer on one side of the 6-in mold. After aging at room temperature for one week, the samples were tested on the North Carolina small-wheel circular-track wear and polish machine. Readings were taken with the British portable tester at 1-min intervals for the first 10 min and then at 5-min intervals for up to 30 min total time and thereafter at 1, 1.5, 2, 3, 4, and 6 h of wear.

After 6 h, six of the samples were removed, reheated, remixed, remolded, aged for one week, and put back in their respective locations in the wear and polish machine. The process was repeated as described above for an additional 6 h. The six original samples were removed and wear was continued on the six remixed samples up to 12 h total time with a final reading at 12 h. This last step was done to again verify that little or no change in BPN occurs with extended exposure to wear and polish (Figure 10).

Another set of six laboratory samples of a different recent BM-1 mix design was made and exposed to the North Carolina wear and polish machine. Readings were again taken with the British portable tester at 1-min intervals for the first 10 min, then

Figure 10. British portable tester readings versus time during first wear and polish testing of BM-1 laboratory samples.

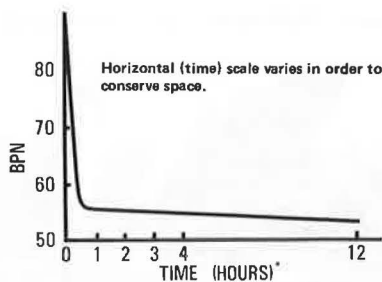


Figure 11. British portable tester readings versus time during another wear and polish testing of BM-1 laboratory samples.

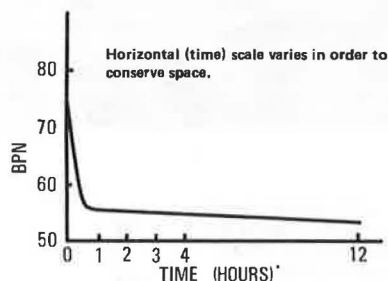
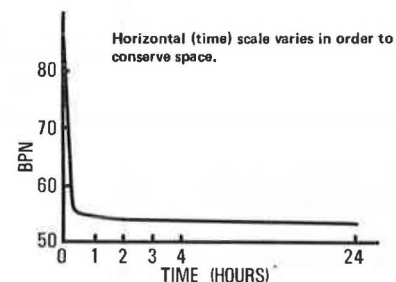


Figure 12. British portable tester readings versus time during wear and polish testing of BM-2 laboratory samples.



at 5-min intervals for up to 30 min total time, and then at 1, 6, and 12 h. The resulting curve of BPN versus time very closely resembles those of previous groups of samples (Figure 11). No remixing or extractions were performed on this test group.

Testing of One BM-2 Mix Design

A set of 12 laboratory samples of a recent BM-2 mix design was made and exposed for 12 h on the small-wheel circular track, and BPN readings were taken at 0, 5, 10, 15, 20, 30, and 60 min and at 1.5, 2, 3, 4, 6, 8, 10, and 12 h. Six of the samples were then removed from the small-wheel wear and polish machine and were heated, remixed, remolded, and aged for one week as before. The 12 samples (six remixed and six with 12 h wear) were again exposed for 12 h more on the wear and polish machine. BPN readings were taken each minute for the first 5 min and then on the same schedule as that for the first 12 h. On completion of the second 12 h, the six original samples, which had a total of 24 h exposure, were removed and the six remixed samples were run an additional 12 h for a total of 24 h (Figure 12).

ASPHALT EXTRACTION AND AGGREGATE GRADATION

Asphalt extraction and aggregate gradation were performed on one sample of the original group and on one sample of the remixed group in the BM-1 and the BM-2 test groups. In the BM-1 group the results showed less than 1 percent variation between the washed-gradation results of the remixed sample com-

pared with those of the original sample. There was a slight loss of all sizes when washed gradation was compared with design-mix gradation. This loss is primarily due to the removal of fines during the process of washing the extracted samples.

When compared with the initial BM-2 mix design, there was a 2.5 percent loss of the largest size (3/8 in), probably due to fracturing of large stones during compaction, and a loss of 2-2.5 percent on sieves smaller than No. 30, primarily because the extraction/gradation is a washed analysis, whereas the original mix was based on dry gradation. There was a variation of 1 percent or less in the percentage retained on each sieve during the extraction/gradation analysis of the remixed samples compared with the samples that were not remixed, which indicated minor changes in mix composition due to remolding. There was also a slight (0.4 percent) loss of asphalt, probably through adherence to mixer, bowl, mold, compaction head, etc. (Table 1).

CORRELATION OF STEREOPHOTOGRAPHY NUMBER WITH SN AND BPN

A method of classifying pavement surface texture and determining skid resistance from stereophotographs was developed by Schonfeld (3) of the Ministry of Transportation and Communications in Ontario. This method, which is designated ASTM E557-75T, was studied and compared with skid-resistance values obtained by the Law friction tester as part of this study and an earlier study of skid resistance made by the KSDOT Research Section (4).

The stereophotographs of the pavement surface were obtained by the use of a camera box that had self-contained illumination. The camera box was constructed by the KSDOT Research Section from information in publications by Schonfeld. Stereophotographs were obtained of a variety of surface textures from throughout the State of Kansas. The pictures were taken of the pavement in place and in the track of the skid test by the locked-wheel trailer. It is essential that the pavement be dry and that the pictures be carefully focused for clear, sharp image reproduction. These stereophotographs were interpreted according to the instructions in Skid Resistance Photo-Interpreter's Guide by Ma and Musgrove (5). Some variation from their instructions was necessary because the mirror-stereoscope used was not equipped for higher magnifications. Approximately 300 pairs of stereophotographs of existing pavement were analyzed.

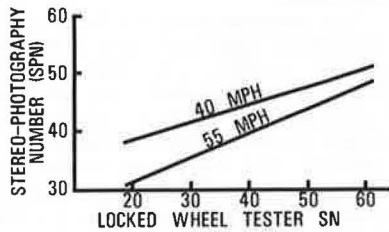
A Hewlett-Packard Model 65 programmable calculator and statistical program 1-22A for linear regression were used to make the calculations to establish the best-fit lines of stereophotography number (SPN) versus locked-wheel tester SN for 40 and 55 mph (Figure 13). The calculated correlation coefficients are 0.66 for 40 mph and 0.72 for 55 mph.

An attempt was also made to correlate the SPN with the BPN of samples of pavement mounted in the North Carolina small-wheel circular-track wear and polish machine. The 6-in samples tested were of two types--core samples of existing pavement and samples made in the laboratory. About 120 pairs of photographs were analyzed at various time intervals during the wear and polish procedure. The resulting data did not formulate a satisfactory correlation. The BPN data indicated a rapid decrease during the first few minutes of wear and polish testing. Conversely, the SPN appeared to increase as time increased on the wear and polish, primarily because the spalling and raveling of aggregate made the surface appear rougher, which would indicate a higher SPN. This divergence possibly is because BPN is more sensitive to microtexture (2) and SPN is a mea-

Table 1. Asphalt-extraction and aggregate-gradation data.

Test Group	Sieve Size (% retained)								Asphalt (%)
	3/8 in	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	
BM-1 mix design	1.0	24.0	42.0	56	69	85	90	92	6.0
Washed uncompact sample	0.3	23.3	40.1	52.7	65.8	81.4	86.7	88.5	-
Original sample	0.5	23.0	40.8	53.1	65.8	80.8	86.2	88.1	6.1
Remixed sample	0.2	23.0	40.3	52.7	65.2	80.2	85.6	87.5	5.8
BM-2 mix design	24	39	53	64	75	89	92	94	5.5
Washed uncompact sample	22.9	39.7	52	62.5	74.1	87.5	90.7	92.4	-
Original sample	21	39	52	62	73	86	90	91.3	5.1
Remixed sample	22	40	52	62	73	86	89	90.1	5.1

Figure 13. Correlation between locked-wheel tester SN at 40 and 55 mph and SPN.



sure of macrotexture and microtexture. Another difficulty experienced was that the 6-in sample provides a restricted surface area to analyze by stereophotographs. It was also noted that the large aggregate in the laboratory samples was often fractured during compaction, which affected the stereophotograph analysis and contributed to increased spalling when compared with field cores.

A fourth factor that may have had some influence on the correlation was the axis of the pair of photographs. In the field the long axis of the stereo pair was perpendicular to the flow of traffic. Because of physical limitations of equipment, the long axis of the pair of stereophotographs in the laboratory was parallel to the direction of movement of the wear and polish tires. The influence of this last item, however, was felt to be minor when it was compared with the influence of the raveling discussed earlier.

CORRELATION OF LINEAR TRAVERSE NUMBER WITH BPN

During the project, some effort was made to determine whether there existed any correlation between surface roughness as measured by a linear traverse device (6,7) and readings obtained with the British portable tester during the wear and polish testing discussed earlier. The device used allowed the sample to move horizontally while it was being scanned with a microscope. The vertical deformations were registered by changes in the focus knob of the microscope, and both horizontal and vertical movements were recorded on a continuous-line graph that gave a cross section of the sample.

This cross section was then enlarged three times by using a pantograph to facilitate measuring the length of the line. The central 9-in horizontal-measure portion of the enlarged graph was then measured by using an irregular-line rolling-wheel measuring device called a map measure. The measured length of the irregular line within the 9-in horizontal boundaries was recorded in an effort to obtain an objective measure of surface roughness and then correlate that with the BPN.

Two types of samples were tested. The first type were core samples from existing pavement that had some wear from exposure to traffic before being exposed to the wear and polish machine. The second type were new or remixed samples made in the laboratory that had had no previous wear before exposure

to the wear and polish machine. The initial traverse was made of each sample before polishing. The samples were then mounted in the wear and polish machine and BPN readings were taken as discussed above. After 12 h of wear, the samples were removed from the wear and polish machine and the second linear traverse reading was made. In some cases the samples had raveled so badly that the 12-h reading could not be made. In all, more than 560 samples were traversed; they represent approximately 46 test locations.

Efforts to calculate a correlation between the linear traverse number (LTN) and the BPN were not successful. The situation was similar to that involving the stereophotography method discussed earlier. It is now evident that the length-of-line method used in this study does not provide differentiation between one large hole or piece of protruding aggregate and a series of small holes or small pieces of protruding aggregate, which could result in traverse lines of the same length. The same problem existed when the area below the line was used instead of the line length. A visual analysis of the irregularities of the profile appears necessary. Also, as the samples raveled and became rougher the length of the traverse line tended to increase, which indicated an upward trend in skid resistance, whereas the BPN indicated a drop between the zero-time and the 12-h exposure on the wear and polish machine.

Another cause of the divergence could be that the LTN, at least at the magnification used in this study, was primarily sensitive to macrotexture, whereas BPN is sensitive to microtexture (2). Based on previous research studies (2), other profiling techniques have been shown to yield a usable correlation with BPN.

CHAT AND LIMESTONE-MIX EXPERIMENT

In an effort to provide an asphalt concrete that maintains its high skid-resistance property, some experimentation was conducted mixing various percentages of chat aggregate with limestone aggregate. Chat, as used here, is a waste product of the lead and zinc mining operations of southeast Kansas and is composed primarily of the rock chert. It was anticipated that the very angular, sharp, hard chat aggregate would provide initial surface friction due to its angular and sharp properties. It should also provide prolonged surface friction through differential wear and the softer limestone and asphaltic medium compared with the harder chat as well as through its angular and sharp properties.

Initial testing by using 6-in samples and the North Carolina small-wheel circular-track wear and polish machine with periodic testing by using the British portable tester did not indicate any appreciable advantage of the mixture. The initial BPN before exposure to the wear and polish machine was approximately 77. As was characteristic of other mix design samples, the BPN dropped rapidly, and at

20 min exposure on the wear and polish machine, the BPN was 57. After 4 h of exposure to the wear and polish machine, the BPN was approximately 56. At that point the samples had each begun to ravel, so BPN readings were stopped. The wear and polish was continued until 12 h of exposure and all samples were severely raveled.

A test area of field exposure was constructed in 1980, and tests were run with the locked-wheel surface-friction tester. The results of the field testing indicate that initially the frictional property of the chat/limestone mixture is essentially the same as that of the standard BM-1 surface-course mixture. Tests run in 1981 indicated that the chat/limestone mix was providing excellent skid resistance after one year's exposure to Interstate traffic. Subsequent testing after extended time and exposure to traffic will indicate whether the chat/limestone mixture results in prolonged retention of surface friction.

TESTING OF OPEN-GRADED MIXTURE

In 1974, seven different semi-open-graded pavement surface mixtures were applied as test overlay sections in various areas throughout the state. Subsequent outflow-meter testing indicated a densification of all samples on exposure to weather and traffic. This densification somewhat defeated the subsurface drainage characteristics of the various mixtures. The higher asphalt cement content plus the densification also lowered the surface friction characteristic of one section after exposure to traffic. Only two of the initial test sections remain in service. The other five locations were overlaid during the normal resurfacing cycle for the road but not because of any surface deterioration in the open-graded test locations. The latest tests on the remaining two sites indicated that the SN had increased as much as 20 numbers since installation and was well above average for the bituminous concrete surfaces in Kansas.

INFORMATION ESTABLISHED BY TESTING

Several items of information were verified or established by this testing. First, all samples had reached the practical low-point BPN within 2 h of exposure on the North Carolina small-wheel circular-track wear and polish machine, and extended running on the machine is not necessary on asphaltic-concrete types of pavement.

Second, the most critical time zone is the first 30 min, and readings should be taken each minute for the first 10 min, each 5 min thereafter to 30 min total elapsed time, and then each 30 min to 2 h total time. After 2 h, wear and polish testing could be stopped or readings could be taken each 2 h or only at the end of 6 or 12 h since there is very little change in BPN after 2 h.

Third, the wear and polish machine and the British portable tester will give consistent results regardless of the location of the sample in the mounting platform on the wear and polish machine. This was indicated by the similarity of the curves for BPN versus time for the original samples of a group and for the remixed samples of a group. The beginning points (0 min) for the remixed samples were lower than those for the original samples but the curves converged within the first 20-30 min. The data are not sufficient to determine what caused the initial variation.

Another interesting phenomenon was that the remixed samples appeared to be more resistant to wear and raveling than the original samples. This was characteristic of both the BM-1 and BM-2 mix design

sample groups. Additional testing and exposure to actual traffic of recycled asphaltic pavement may provide some indication of whether this apparent increase in durability for remixed samples can be a prediction of improved behavior of recycled pavement as well.

The data collected with the locked-wheel pavement surface friction tester for use during this study were considered reasonable. Since field BPN readings were taken at approximately the same time as the SN data were, neither temperature variations nor seasonal variations were considered a significant influence in the field correlation study as it was conducted.

The considerable variation between field BPN and laboratory BPN for the same sample remains unexplained except for the following factors, which possibly contributed: (a) washing action while core drilling, (b) washing action while sawing the core to the proper height to mount on the wear and polish machine, (c) temperature and humidity change from field to laboratory, and (d) orientation of the sample to traffic flow in the field and to the path of the wear and polish wheels.

DESIGN PREDICTION RESULTS

On completion of the laboratory testing, an effort was made to predict what the SN_{40} would be in a field test that used the locked-wheel pavement friction test trailer for both of the BM-1 sample mix designs. To accomplish this, the average field BPN of 56 for the field core samples of the 17 HM-R/BM-1 locations was projected onto the plot of BPN versus time in Figure 14. The corresponding time of 5.5 min was then projected onto the plot of BPN versus time for the mix design to establish the predicted BPN of 58, as shown in Figure 15. The data for Figures 14 and 15 were the same as those for Figures 8 and 10, respectively, but they were expanded to show only the first 30 min, since that appears to be the critical portion of the graph. The predicted BPN can then be entered on either the plot of SN versus BPN (Figure 16) or the plot of SN versus test speed (Figure 17) to establish the predicted SN for the desired speed.

The two field projects that correspond to the BM-1 mix designs used in the lab were tested with the locked-wheel pavement friction test trailer. The result of the first correlation was a predicted SN_{40} of 45, whereas the average SN_{40} in the field was 42.

The result of the second attempt to predict SN was about the same. The predicted SN_{40} was 43 and the locked-wheel test results ranged from 44 for the portion of the project surfaced in 1979 when the prediction and field testing was done to 35 for the portion of the project constructed a year earlier. The lower SN was believed to be the result of a slightly higher percentage of asphalt in the mix used on the highway.

No prediction attempt was made on the BM-2 mix design because in 1978 and 1979 that particular mix was used as a surface material only on some paved shoulders, which made testing with the locked-wheel tester unsafe and impractical.

As a sidelight, it should be mentioned that worn skid-trailer tires no longer suitable for testing purposes are transferred to KsDOT maintenance forces to be used as front tires on tractors and mowing equipment. Thus the many miles of life remaining in the tire are not wasted. When the tires are no longer serviceable on the maintenance equipment, they along with other tires are sold to bulk-rubber dealers for recycling.

Figure 14. Expansion of British portable tester readings versus time during wear and polish testing of HM-R/BM-1 field core samples.

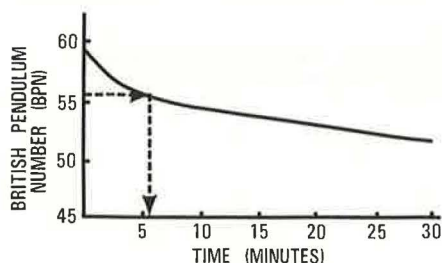
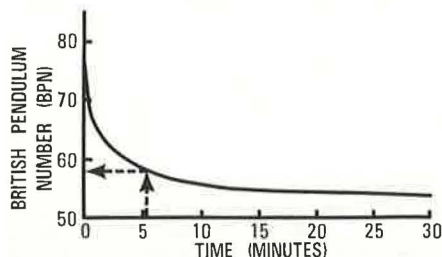


Figure 15. Expansion of British portable tester readings versus time during wear and polish testing of BM-1 laboratory samples.



CONCLUSIONS

A field correlation between SN and BPN for an asphalt concrete pavement type can be established, as indicated by the correlation established in this study for BM-1 and BM-2 mixes. A graph of BPN versus time can be drawn from data provided by wear and polish testing. A prediction of field SN can be made by using the above information and the graph of the wear and polish BPN versus time established by testing samples of the proposed mix of asphalt concrete. If the field mix is allowed to vary from the tested laboratory mix, however, the predictions will be subject to considerable potential for error. An increase of 0.5-1 percent in asphalt concrete content can result in SNs 20-30 points below a predicted level. Variations in aggregate gradation do not appear to be so critical in their effect on the SN.

In view of the effect that variation in asphalt content has on SN, it appears that educating field construction inspectors on the importance of adhering to designed mix proportions would be advisable so that tighter control might be exercised, especially on the percentage of asphalt concrete.

A satisfactory correlation was calculated between the SPN of pavement photographed in place and the SN obtained with the locked-wheel tester. However, efforts that used the data obtained during the testing of samples on the North Carolina small-wheel wear and polish machine to correlate SPN with BPN were not successful. Similarly, efforts that used data collected during this study to correlate LTN with BPN were not successful. Extended operation of the wear and polish machine beyond 2 h on a set of bituminous pavement samples does not provide any additional significant data.

If more research is conducted, it should start with more trial implementation of the procedure to determine whether acceptable predictions can be made repeatedly. Further study might also produce a correlation between time on the wear and polish device and number of wheel passes on the pavement in the field, which might indicate a prediction of pavement life via laboratory testing.

Figure 16. British portable tester versus locked-wheel tester for 17 HM-R/BM-1 test locations.

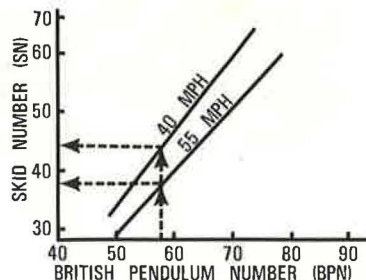
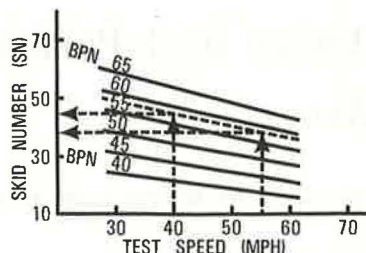


Figure 17. Test speed versus locked-wheel tester with constant BPN for 17 HM-R/BM-1 test locations.



ACKNOWLEDGMENT

The research documented in this report has been coordinated and conducted by the Kansas Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration, under the Highway Planning and Research Program, FCP Category LH.

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

We, the State of Kansas, and the U.S. Government do not endorse products or manufacturers. Trade or manufacturers' names appear here solely to identify equipment or materials used.

REFERENCES

1. W.G. Mullen. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures. Highway Research Program, North Carolina State University at Raleigh, Rept. ERSD 110-69-1, Sept. 1972.
2. J.J. Henry and S.H. Dahir. Effects of Textures and the Aggregates that Produce Them on the Performance of Bituminous Surfaces. TRB, Transportation Research Record 712, 1979, pp. 44-50.
3. R. Schonfeld. Photo-Interpretation of Pavement Skid Resistance. Research and Development Division, Ontario Ministry of Transportation and Communications, Downsview, Rept. RR 188, June 1974.
4. H.E. Worley, T.M. Metheny, and F.W. Stratton. Pavement Surface Dynamics Friction Measurement and Analysis in Kansas. Planning and Development Department Research Section, Kansas Department of Transportation, Topeka, Rept. FHWA-KS-RD 73-3, June 1976.
5. A. Ma and G. Musgrove. Skid Resistance Photo-Interpreter's Guide. Engineering Research and Development Branch, Ontario Ministry of Transportation and Communications, Downsview, March 1974.

6. C. Crumpton and G. McCaskill. Paint Stripe and Glass Bead Study: Report 2. Planning and Development Department, Research Division, State Highway Commission of Kansas, Topeka, 1970, pp. 21-23.
7. J. Dale. Development of Improved Pavement Mark-

ing Materials: Laboratory Phase. NCHRP, Rept. 45, 1967.

Publication of this paper sponsored by Committee on Characteristics of Bituminous-Aggregate Combinations to Meet Surface Requirements.

Design and Performance of Bituminous Friction-Course Mixes

NABIL KAMEL, G.R. MUSGROVE, AND A. RUTKA

Performance data from two major experimental field projects carried out in Ontario to develop bituminous friction-course mixes with improved texture and friction characteristics are presented. The new mixes maintain excellent surface texture and provide longer-lasting skid-resistance characteristics. Design principles, construction, and subsequent performance characteristics of these skid-resistant pavement surfaces are discussed. Aggregate properties, gradations, and mixture characteristics that produce and maintain optimum texture levels with Ontario materials are identified. Mixes within such gradation boundaries were found not only to maintain superior texture qualities and friction levels but also to require less asphalt cement when compared with conventional asphalt surfaces. The new friction-course mixes use normal paving-grade asphalts and require no special additives or fillers. Use of the new friction-course mixes for rehabilitation of pavements that have low friction levels and experience a high rate of wet-weather collisions has produced an average reduction of 54 percent in wet-pavement collisions and a 29 percent reduction in total collisions at eight black-spot freeway locations. Treatment at five black-spot signalized highway intersections produced an average reduction of 71 percent in wet-pavement collisions and 46 percent reduction in total accidents. The Ontario Ministry of Transportation and Communications has implemented a policy that specifies the use of the new friction-course mixes for new construction and resurfacing projects for all main highways. The new surface mixes are also used for rehabilitation at locations other than main highways at which excessive wet-pavement collisions occur.

Pavement skid-resistance research in Ontario started in 1962 by using the British portable skid tester. Early efforts were primarily directed toward developing high-speed friction measuring capabilities and developing techniques to evaluate pavement textures. In 1967, organized high-speed skid testing started with a Ministry-built brake-force trailer that met the requirements of the American Society for Testing and Materials (ASTM 274). In 1970, Schonfeld's photointerpretation technique for pavement-texture classification (1,2) was introduced.

In the mid-1970s, considerable attention was given to the construction and maintenance of skid-resistant pavements so that wet-weather accidents would be reduced. An extensive program of transverse grooving on slippery concrete pavements was introduced (3) as were procedures for the posting of Slippery When Wet signs and wet-pavement advisory speed limits at highway locations at which more than one-third of the accidents were occurring under wet conditions. In addition, two major experimental projects were carried out to develop alternative bituminous surface-course mixes with improved texture and friction characteristics for new construction. In 1974, 17 test mixes were constructed on a section of Canada's Highway 401 (Toronto By-Pass) to evaluate improved surface mixes for heavily trafficked freeways and main highways (4). In 1978, 17

other test mixes were constructed on Highway 7 near Lindsay to develop improved surface mixes for highways that had a lower traffic volume (5). Each experimental project included both types of dense and open-graded mixes and evaluated a variety of aggregate types. Results from the test roads provided an excellent data base for examining the effects of traffic, mix properties, aggregates, etc., on skid resistance.

It is the purpose of this report to review Ontario's experience with the design, construction, and performance of these skid-resistance mixes over the past seven years and to present data on their effectiveness in terms of accident reductions observed after resurfacing at highway locations that had experienced excessive rates of wet-pavement accidents.

SKID-RESISTANT MIXES

A skid-resistant surface must have sufficient microtexture (harshness) and sufficient macrotexture (stone projections). Figure 1 shows a pavement surface profile and texture parameters as defined by Schonfeld (1,2). Pavement surface microtexture is a function of the harshness of the microprojections on matrix surfaces as well as the harshness of the macroprojection surfaces. Macrotexture is a function of such physical properties as height, width, and angularity as well as density of the macroprojections on the pavement surface.

The role of the macrotexture is to break up the water film and to provide drainage channels so that most of the water can be drained from the contact area between the rolling tire and the pavement surface. The microtexture allows penetration of the remaining thin film of water on the roadway surface. Good friction levels can only be obtained with adequate harshness or microtexture on the pavement surface. This is a desired property at all speeds. Adequate macrotexture will limit the drop in friction levels as vehicle speed and/or water thickness on the pavement surface increases.

Microtexture may be obtained by using aggregates with high polish resistance, which show differential polishing and/or microtexture regeneration characteristics (6,7). Attainment and maintenance of macrotexture stone projections on the pavement surface are influenced by aggregate size, gradation, type and composition, hardness, and resistance to wear (4-7).

SELECTION OF COARSE AGGREGATE IN FRICTION-COURSE MIXES

Both macrotexture and microtexture qualities required for good wet-surface friction characteristics are influenced by the quality, quantity, and gradation of the coarse aggregate in the mix. To ensure attainment of adequate texture and friction levels, many European countries and Japan use quality specifications or guidelines related to polished-stone value (PSV), aggregate-abrasion value (AAV), texture depth, and minimum skid-resistance levels (8).

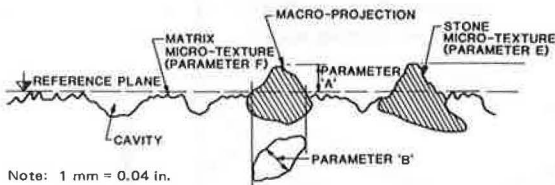
In Ontario, results from both the Highway 401 and the Highway 7 test sections confirm that coarse aggregates with higher PSVs produce mixes that maintain higher friction levels. This is demonstrated in Figure 2 for both heavy and moderate traffic. The two mixes from Highway 401 shown in Figure 2 have been exposed to extremely heavy traffic. Over a five-year testing period, accumulated total traffic amounted to approximately 24 million vehicles of which 7 million were commercial trucks. The Highway 7 test at Lindsay has an annual average daily traffic (AADT) of 5500, of which 12 percent are heavy commercial vehicles.

Of particular interest is the low PSV of the traprock aggregate (PSV = 46). This value indicates that the material will ultimately polish to a degree equivalent to, or only slightly better than, some limestone aggregates. The traprock aggregate is very hard, fine-grained, dark in color, and 100 percent crushed material. Mixes that have 100 percent traprock materials (such as that illustrated in Figure 2) have been used satisfactorily for a number of years in Ontario on highways that experience very heavy traffic. The satisfactory skid performance of the traprock mixes may be attributed to the excellent abrasion resistance of the traprock material (AAV = 2). The traprock aggregates tend to maintain angularity under severe traffic conditions and in turn provide and maintain sharp, angular macrotexture projections on the pavement surface and acceptable overall skid-resistance levels.

It is important that PSV be considered along with aggregate ability to withstand abrasion and maintain angularity. Maintenance of angular macrotexture stone projections on the pavement surface is essential in providing good friction levels for high-speed traffic. Stones with moderate PSV but high abrasion resistance such as the traprock materials appear to provide satisfactory skid-resistance performance under extremely heavy traffic.

Blending of a better friction-quality coarse aggregate, i.e., one that has a higher PSV, in a mix can yield significant improvement in skid resistance. Figure 3 provides a comparison between the skid-resistance performance of four mixes from the Highway 7 test. In the dense-graded friction-course (DFC) mix, the coarse aggregate is a 1:1 blend of local limestone and imported igneous materials. This mix provides superior skid-resistance levels in comparison with the two mixes that contain 100 percent limestone coarse aggregate. Its skid resistance is as good as the standard heavy-duty H11 mix, which incorporates 100 percent traprock coarse aggregate and local fines. This DFC mix with the coarse-aggregate blend is of particular interest because of its apparent economic advantages in comparison with the standard H11 mix; the DFC mix uses a higher percentage of local materials and requires less asphalt cement.

Figure 1. Pavement surface profile.



Note: 1 mm = 0.04 in.

- A - Height of macro-projections (mm)
- B - Width of macro-projections (mm)
- C - Angularity of macro-projections (scale 0.0-3.0)
- D - Density of distribution of macro-projections (%)
- E - Harshness of macro-projection surfaces (scale 0.0-6.0)
- F - Harshness of micro-projections on matrix surfaces (scale 0.0-6.0)

Figure 2. Effect of PSV on skid-resistance performance.

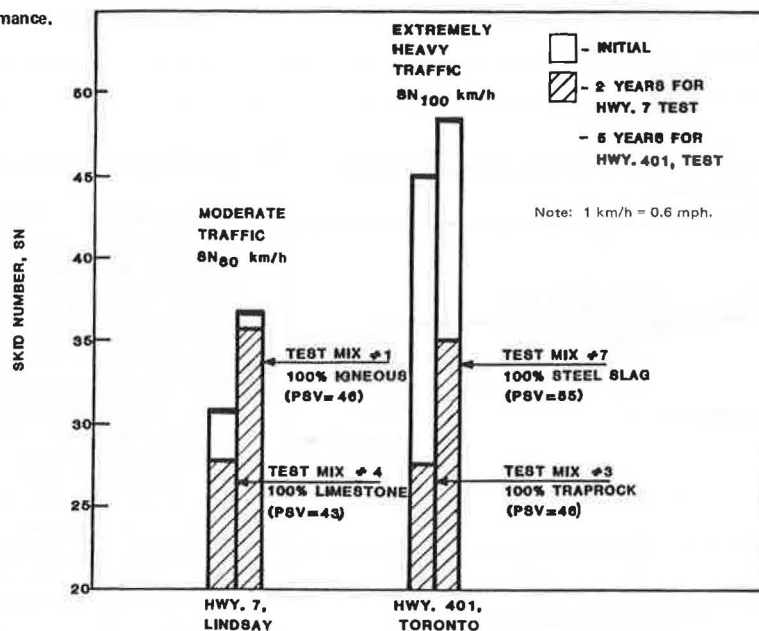


Figure 3. Skid-resistance performance of four mixes with and without coarse-aggregate blend.

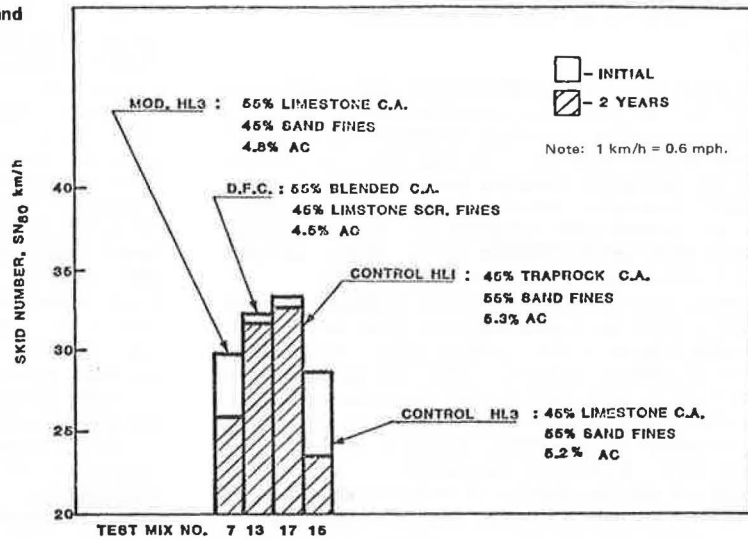


Figure 4. Change in height and density of macroprojections for two HL1 mixes with different coarse-aggregate content.

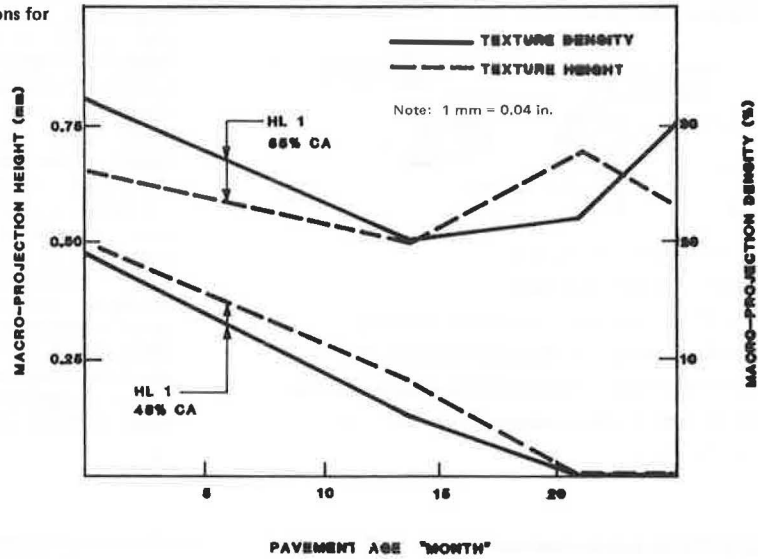
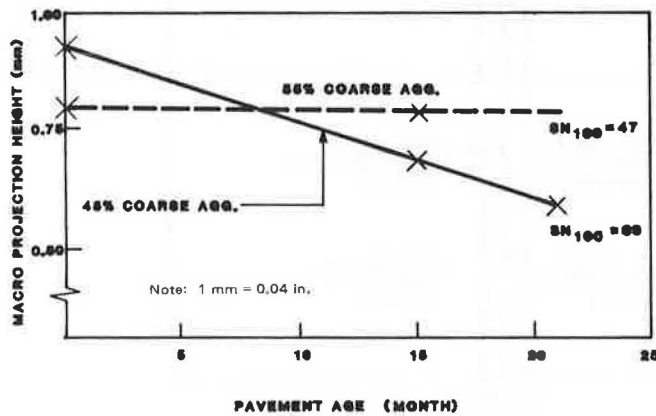


Figure 5. Change in macroprojection height for two steel-slag mixes with different coarse-aggregate content.



COARSE AGGREGATE IN MIX

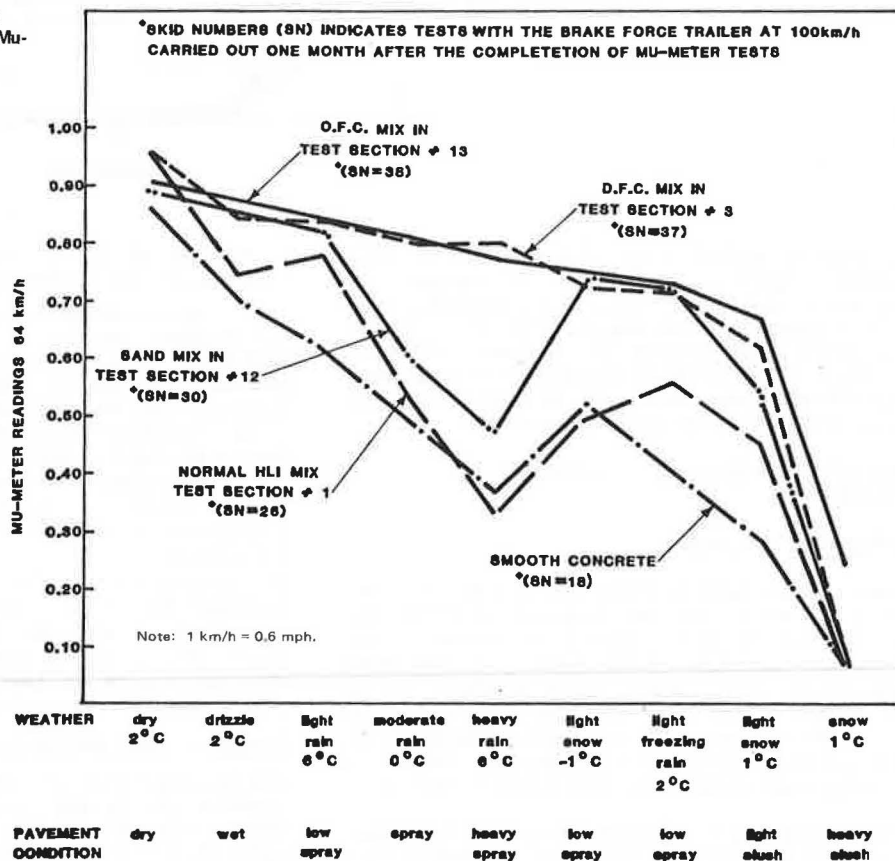
An important finding of the Highway 401 Toronto By-Pass test is that mixes that had a higher stone content maintained better surface textures than traditional mixes that had 45 percent stone. Figure 4 shows the change in macrotexture depth and density (measured by the photointerpretation method in the driving lane; AADT was 12 900 of which 3740 were commercial vehicles) with pavement age for two mixes that contained 45 percent and 55 percent traprock coarse aggregate (i.e., retained by No. 4 sieve). The fine aggregate in both mixes is a blend of sand and limestone screenings. The superiority of the mix that has 55 percent stone content is quite clear. It provided and maintained a better texture depth and density of stone projections on the pavement surface, whereas the macrotexture on the mix that had 45 percent coarse aggregate virtually disappeared after less than two years in service.

Table 1. Mix composition and asphalt content for conventional and stonier mixes.

Type of Mix	Aggregate Retained on 4.75-mm Sieve		Aggregate Passing 4.75-mm Sieve		Asphalt (percent by weight of mix)	Test Mix No.	Test Date ^a
	Type	Percent	Type	Percent			
HL1	Traprock	45	Sand and limestone screenings	55	5.4	1	1974
	Traprock	55	Sand and limestone screenings	45	4.8	4	1974
DFC	Steel slag	45	Steel slag screenings	55	5.3	7	1974
	Steel slag	55	Steel slag screenings	45	4.8	2	1978

^aAll tests were done on Highway 401.

Figure 6. Skid-resistance measurements in different weather conditions by using the Mu-meter (driving lane).



Overall, the mix that had 55 percent coarse aggregate provided skid numbers (SNs) approximately 4 points higher than traditional HL1 mix that had 45 percent stone when tested under standard test conditions at 100 km/h.

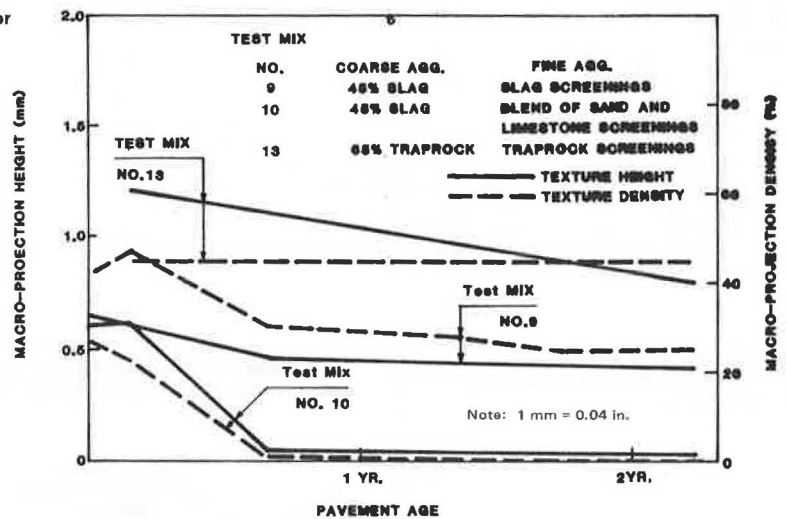
Similar observations may also be made for two steel-slag test mixes that contained 45 percent and 55 percent coarse aggregate (Figure 5). The skid resistance of the mix that had 45 percent stone (retained by No. 4 sieve) declined from SN₁₀₀ of 48 to 39 after 21 months in service. The skid resistance of the mix that contained 55 percent coarse aggregate has shown no significant decline in its 21 months of service.

It appears that mixes that have 55 percent coarse aggregate provide higher resistance to the immersion of stone projections into the matrix under heavy truck traffic; they maintain better and longer-lasting surface macrotexture. The use of mixes that have such a high stone content also has economic advantages, mainly due to the reduced asphalt content required. As shown in Table 1, the stonier mixes use approximately 10 percent less asphalt cement than is used in traditional mixes that contain 45 percent stone content.

Results of laboratory testing on mixes that have varying coarse-aggregate content in the United Kingdom appear to confirm our findings with the stonier mixes. Fabb and Heyes (9) found the optimum stone content for deformation resistance to be approximately 55 percent of the total mix when crushed coarse aggregate is used. In Japan (10), the use of such stonier asphalt mixes is prompted due to its excellent skid-resistance characteristics and the higher resistance to rutting, particularly on heavily trafficked routes.

One last observation concerns sand mixes that contain little or no coarse aggregate. The Highway 401 mixes included four sand mixes that had coarse aggregate percentages between 9 and 30. All the mixers contained traprock coarse aggregate and traprock screening fines. Such mixes produce good microtexture but little macrotexture on the pavement surface. They provide reasonably good levels of skid resistance when tested under standard conditions with the brake-force trailer, but tests with the Mu-meter showed these mixes to exhibit a significant decline in skid resistance under conditions of moderate or heavy rain [Figure 6 (4)]. Such mixes

Figure 7. Change in height and density of macroprojections for test sections 9, 10, and 13.



would provide a satisfactory surface texture only for low-speed traffic. In contrast, as indicated in Figure 6, mixes that have a high stone content, such as the open-graded friction-course (OFC) mix and the DFC mix, which also contain 100 percent traprock, retain a high level of skid resistance during particularly adverse conditions such as heavy rain and light snow or slush on the pavement surface.

SELECTION OF FINE AGGREGATE IN FRICTION-COURSE MIXES

Heavily Traveled Pavements

Maintenance of adequate texture levels and skid resistance on heavily traveled highways depends not only on the quality of the coarse aggregate in the asphalt mixture but also on the quality of the fine aggregate in the mix. Under heavy traffic, as on the Highway 401 Toronto By-Pass, the fine aggregate plays as important a role as coarse aggregate in achieving and maintaining skid resistance. This has been clearly demonstrated by the Highway 401 test mixes.

All test mixes in the Highway 401 test that contained a blend of sand and limestone screening fine aggregate showed a sharp decline in skid resistance in their early service life. In contrast, all successful mixes contained premium-quality crushed fine aggregate such as traprock and steel-slag screenings.

Figure 7 (4) presents texture measurements for three test mixes. Macrot texture depth for test mix 10, which contains a blend of sand and limestone-screening fine aggregate, declined rapidly and almost ceased to exist during the first year in service. This is mainly due to compaction of the bituminous mix under traffic, which resulted in the impression of the coarse aggregate particles into the matrix of the mix. Under the same testing conditions, mixes 9 and 13, which contained slag and traprock screening fine aggregate, provided and maintained reasonable macrot texture depth, macrot texture density, and good overall levels of skid resistance.

In 1976, the above finding triggered a skid-resistance survey for HLL mixes in service. More than 50 contract locations were tested on highways where AADT was 5000 vehicles or more per lane per day. It was found that the fine aggregate in 93 percent of the deficient pavements was predominantly sand and in 90 percent of the satisfactory pavements was predominantly crushed screenings.

The use of hard, harsh, and angular fine aggregate appears to provide a number of advantages:

1. Increased aggregate interlock, which causes a significant increase in Marshall stability for such mixtures; this provides high resistance to the impression of coarse aggregate, under heavy truck traffic, into the matrix of the mix;
2. Increased harshness of the pavement surface due to an angular and harsh pavement matrix; and
3. Increased macrot texture depth and density of stone projections on the pavement surface by the protrusion of the coarse aggregate particles from the matrix.

In summary, for heavily trafficked freeways and main highways that have a high volume of truck traffic, it is essential that surface-course mixes contain prime-quality (hard, harsh, and angular) coarse and fine aggregates so that adequate texture and skid-resistance levels can be maintained.

Pavements That Had Moderate and Low Traffic Volume

For highways that have a moderate volume of traffic, surface texture and skid resistance of asphalt mixes depend primarily on the quality of the coarse aggregate in the mix. Results from the Highway 7 test indicate that mixes that have similar coarse aggregate (i.e., similar in type and in percentage in the mix) provide essentially similar skid-resistance levels regardless of the type of fine aggregate in the mix. For example, the highest skid-resistance values were obtained with four test mixes that contained igneous coarse aggregate. Two of these mixes contained local limestone screenings and two contained igneous-screening fines (mixes 1 and 6 and 11 and 12 in Figure 8). The Lindsay mixes also indicated that replacing the sand fines with a blend of sand and limestone or sand and igneous screenings or with 100 percent limestone screenings failed to provide significant improvement in skid resistance (mixes 7, 8, 9, 14, and 15).

It appears that for moderate and low traffic conditions, skid-resistance levels (SN_{80}) of bituminous concrete mixes are predominantly influenced by the type of coarse aggregate in the mix. The role of fine aggregate is significantly less apparent than it is in the case of heavily traveled highways.

AGGREGATE GRADATIONS FOR FRICTION-COURSE MIXES

The gradation requirements of the Ontario Ministry

Figure 8. Initial and two-year skid-resistance measurements, westbound.

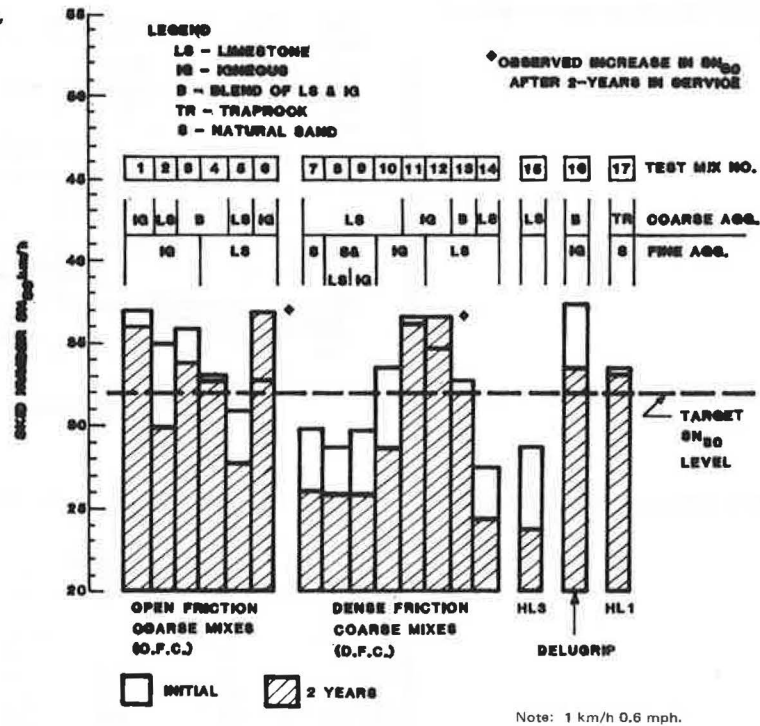


Table 2. MTC gradation requirements for coarse aggregates.

MTC Sieve Designation	Percentage Passing by Weight		
	HL1 and HL3	DFC	OFC
13.2 mm	100	100	100
9.5 mm	50-73	50-73	95-100
6.7 mm	-	-	20-45
4.75 mm	0-10	0-10	0-10
75 μm	0-2	0-2	0-2

Note: 1 mm = 0.04 in.

Table 3. MTC gradation requirements for fine aggregates.

MTC Sieve Designation	Percentage Passing by Weight		
	HL1 and HL3	PFC	OFC
9.5 mm		100	100
4.75 mm	100	85-100	85-100
2.36 mm	80-100	50-70	50-70
1.18 mm	55-90	25-50	25-45
600 μm	35-70	15-40	10-30
300 μm	15-40	10-30	0-20
150 μm	5-10	5-25	0-10
75 μm	0-5	0-17	0-3

Note: 1 mm = 0.04 in.

of Transportation and Communications (MTC) for coarse aggregate used in conventional surface mixes (HL1 and HL3), DFC mixes, and OFC mixes are shown in Table 2. The nominal size for coarse aggregate is 13.2 mm (1/2 in) for conventional and DFC mixes and 9.5 mm (3/8 in) for OFC mixes. Only crushed-screening fine aggregate is acceptable for friction-course mixes. Screenings are washed for use in the OFC mixes and used unwashed for the DFC mixes. Gradation requirements for fine aggregate in conventional, DFC, and OFC mixes are shown in Table 3.

Based on the performance of the various friction-course mixes on Highway 401 and Highway 7, recom-

mended target gradations for total aggregates in OFC and DFC mixes have been developed as shown in Figure 9. From 1978 to 1980, MTC constructed many pavement friction-course sections that met the above target gradations with excellent results. Mixes were placed on highways that had extremely heavy traffic volumes, e.g., the Highway 401 Toronto By-Pass and Highway 417 in Ottawa, as well as on highways that had moderate traffic, e.g., Highway 401 in Kingston and Highway 7 in Lindsay. Such mixes produce excellent surface textures and maintain superior overall friction levels. As previously shown, these stonier mixes demand less asphalt cement and thus have additional economic advantages over normal mixes. They also maintain high levels of skid resistance during adverse weather conditions such as heavy rain and light snow or slush on the pavement surface.

MIX DESIGN

The DFC mixes are designed in almost the same manner as conventional mixes. The MTC mix design method uses the Marshall equipment (11). The briquettes are compacted mechanically by using 60 blows of a Marshall hammer per side. The most significant difference is that particular emphasis is placed on visual evaluations of loose and compacted mix samples. The results of both the visual assessments and the Marshall mix properties determine the designed asphalt content in the mix.

DFC mixes that contain 100 percent crushed-screening fine aggregate are characterized by high Marshall stability values, usually a minimum of 9000-10 000 N (2000-2250 lbf). They are also characterized by relatively higher voids in the pavement that result from difficulties in compacting these dense, high-stone-content mixes in the field (5). With maximum effort, it is normal to obtain only 90 percent compaction in the field with these high-stability, high-stone-content mixes. For this reason, and to ensure durability, DFC mixes must be designed to have well-coated aggregate particles. MTC practice is to design these mixes with a rich to very rich asphalt appearance. In many instances

Figure 9. Target aggregate gradations for OFC and DFC mixes.

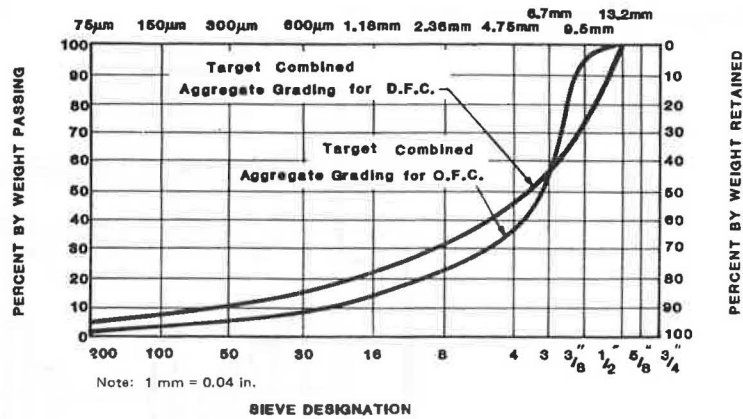


Table 4. Marshall design values for DFC and OFC mixes.

Aggregate Type in Mix	Percent Retained by 4.75-mm Sieve	Asphalt (%)	Voids (%)	Flow (0.25 mm)	Voids in Mineral Aggregate (%)	Stability (N)
DFC Mixes						
Steel slag	55	4.8	3.7	15.1	17.4	13 608
Igneous	55	5.0	3.0	16.2	13.7	10 564
Traprock	45	4.5	1.2	21.5	12.7	13 900
Sandy dolostone	60	4.8	2.0	15.1	11.9	10 453
Limestone	55	4.5	1.7	20.3	11.9	9 341
OFC Mixes						
Steel slag	65	5.2	5.6	17.9	16.0	8 227
Igneous	65	5.3	6.5	13.6	17.7	6 049
Traprock	65	5.0	8.4	11.7	20.1	8 139
Sandy dolostone	65	4.6	9.0	11.1	18.0	7 784
Limestone	65	4.1	5.5	13.5	13.7	7 784

Note: 1 mm = 0.04 in.; 1 N = 0.04 in.

visual assessment of loose and compacted mix samples becomes the decisive factor in selecting the designed asphalt level. Examples of Marshall design values for DFC mixes are shown in Table 4. For purposes of comparison, design specifications for traditional HL1 and HL3 surface-course mixes call for an asphalt content of 5-7 percent and design voids of 2-4 percent.

For design of OFC mixes, Marshall testing is carried out, but once again visual evaluations of loose and compacted mix samples would greatly influence the selection of the designed level of the asphalt binder. OFC mixes are also designed to have well-coated aggregates and a rich to very rich asphalt appearance. Examples of Marshall design values for various OFC mixes are given in Table 4.

CONSTRUCTION OF FRICTION-COURSE MIXES

Details on construction of DFC and OFC mixes have been given by Kamel, Corkill, and Musgrove (5). Mixing temperatures for OFC mixes range between 121 and 135°C (approximately 250-275°F). It is MTC practice to maintain the mixing temperature for OFC mixes below 149°C (300°F) to prevent asphalt runoff. Mixing temperatures for DFC mixes range from 150°C to 165°C (approximately 300-325°F).

Placing the OFC and the DFC mixes is carried out by using conventional equipment, i.e., a paver equipped with a vibratory screed. Compaction is carried out immediately after the mix has been placed by using only a 10-ton steel drum roller on the OFC mixes and both the steel and rubber-tired rollers on the DFC mixes. The OFC mixes are con-

structed to be 25 mm (1 in) thick and the DFC mixes are placed 38 mm (1.5 in) thick.

PERFORMANCE OF FRICTION-COURSE MIXES

This section provides additional information on field performance of both OFC and DFC mixes. Testing results on pavement core samples, pavement permeability, skid resistance, and noise characteristics as well as winter maintenance and performance are discussed.

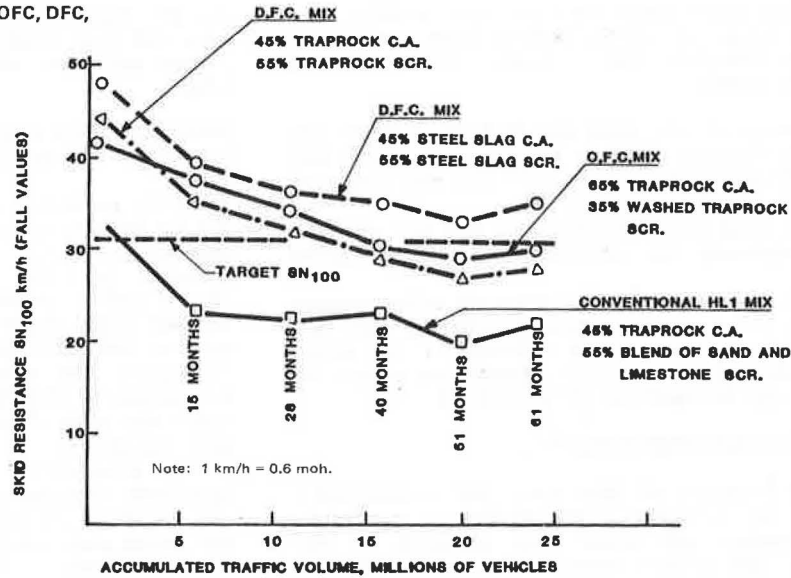
Pavement Core Samples

Pavement core samples taken from the wheel tracks in the driving lane from both the Highway 401 and the Highway 7 tests showed initial pavement voids for OFC mixes that ranged between 10 and 20 percent (4,5). Initial pavement voids of 15 percent in these open-graded mixes are desired.

On the Highway 401 Toronto By-Pass, further coring after approximately four years indicated that pavement voids declined on the average by 3.7 percent. There was no significant difference in results from cores taken from the wheel paths and from the center of the lane. The Highway 7 tests on the OFC mixes indicated that the voids in the pavement decreased by an average of 1.3 percent during the first 13 months in service.

For the DFC mixes, initial pavement voids were found to range between 9 and 15 percent. The high number of voids in the pavement illustrates the problem of compacting these high-stone-content/high-stability mixes in the field. Subsequent testing,

Figure 10. Five-year skid performance history of OFC, DFC, and HL1 mixes.



however, showed a reduction in the pavement voids in cases of both heavy and moderate traffic. On the Highway 401 Toronto By-Pass, the DFC mixes experienced an average reduction of 3.65 percent in the pavement voids during the first 18 months after construction. On Highway 7 the reduction in pavement voids averaged 0.9 percent during the first 13 months of service. It should be noted that initial pavement voids in conventional dense-graded surface mixes (HL1 and HL3) would normally range between 7 and 8 percent.

Pavement Permeability Tests

In situ permeability tests have also been carried out on the OFC and the DFC mixes on Highway 401 in Toronto and Highway 7 in Lindsay (4,5). Test results indicated high initial permeability obtained in all cases, but subsequent measurements after one, two, and four years showed substantial permeability reductions.

Under the heavy truck traffic on Highway 401 in Toronto, the OFC mixes showed high permeability in the first two years after construction. During the next two years the mixes became considerably less permeable; three of nine locations showed permeability readings in the impermeable range. The testing also showed a trend for the passing lane, in which a limited number of trucks travel, to retain its initial high permeability characteristics better than the center and driving lanes do. Under the moderate traffic conditions on Highway 7 at Lindsay, testing conducted one year after construction indicated that the OFC mixes maintained their original high permeability properties.

Test results on the various DFC test mixes on Highways 401 and 7 also indicated considerable reductions in pavement permeability within the first two years of service. All DFC mixes on the Highway 401 test provided four-year permeability readings in the impermeable range.

Pavement Skid Resistance and Noise Characteristics

Figure 10 shows the high-speed skid resistance (SN₁₀₀) versus accumulated traffic volumes for some selected mixes from the Highway 401 Toronto By-Pass test: two DFC mixes that contained traprock and steel-slag coarse and fine aggregates, one OFC mix that contained traprock coarse aggregate and

washed traprock screening fines, and a standard HL1 (control) mix that contained traprock coarse aggregates and a blend of sand and limestone screening fines. As can be seen, the initial SN₁₀₀ values obtained on the DFC and the OFC mixes have SNs 9 to 15 points higher than that observed on the standard HL1 mix. Over the five years in service, the OFC and the DFC mixes maintained significantly higher friction levels in comparison with the HL1 mix. With a total of approximately 24 million vehicle passes, 7 million of which were heavy commercial trucks, the skid resistance of the OFC and the DFC mixes was close to or above the identified target friction level, whereas the skid resistance of the standard mix declined sharply below target values during the first year in service.

Both the DFC and the OFC mixes that had high stone content are characterized by well-developed macrotexture of the surface. The texture and appearance of such mixes can be described as coarse, rugged, and open in comparison with the fine, level, and tight surface of conventional mixes.

As indicated previously, the OFC and the DFC mixes also retain a high level of skid resistance during particularly adverse conditions such as heavy rain and light snow or slush on the surface (Figure 6). The performance of two OFC and four DFC test mixes in the Highway 401 experimental sections has been excellent. During their seven years in service, these mixes have maintained high skid-resistance levels and excellent overall performance.

The OFC surfaces also provide reduced tire-pavement noise levels. An earlier study (12) found the OFC mixes 3 dB(A) quieter than adjacent smooth concrete and 4-5 dB(A) quieter than a typical conventional HL1 mix. This study measured near-tire noise levels by using a microphone mounted 152 mm from the road surface on a 1974 Ford LTD sedan equipped with summer radial tires and operated at 100 km/h.

Winter Performance and Maintenance

MTC also carried out a study of the OFC and DFC mixes on the Highway 401 Toronto By-Pass to observe pavement performance during various winter storms and to determine whether more deicing materials were required to clear these coarse open-textured mixes (13). Adjacent normal dense-graded asphalt surfaces and concrete pavements were included as control sections.

The observations extended over two winters and covered all types of winter storms with particular emphasis on freezing rain storms. The following results were found:

1. Spinning of the sand and salt in such a way to cover all traffic lanes is the most effective way to clear OFC pavements. More salt was not required to clear OFC and DFC pavements.
2. Under most storm conditions, the OFC and DFC pavements appeared to provide the best friction levels.
3. The timing of salt applications with respect to freezing rain storms appeared critical for all pavement surface types. With spinning of the deicing materials, freezing rain storms presented no problems on the OFC and the DFC pavements.

BITUMINOUS-SURFACE-COURSE POLICY

In 1978, as a result of this work, MTC introduced a new policy for bituminous-surface-course construction on freeways and other main highways in Ontario. The new policy specifies the use of OFC mixes for all urban freeway surfacing and the use of DFC mixes on other freeways and main highways that carry a traffic volume in excess of 5000 vehicles/lane/day. H11 mixes are used on facilities that have an AADT range between 2500-5000 vehicles/lane/day.

Since the introduction of the new surface policy in 1978, a total of approximately 400 lane-km (250 lane miles) of pavement has been constructed by us-

ing OFC and DFC mixes in the MTC system. For freeways and main highways only traprock and steel-slag coarse and fine aggregates are acceptable for DFC mixes.

FRICITION-COURSE MIXES FOR REHABILITATION AT BLACK-SPOT HIGHWAY LOCATIONS

In 1978, procedures were also introduced that identified highway locations that had high rates of wet-pavement accidents (black spots). It was found that although the total length of identified black spots on the system represented only a fraction of a percentage point (0.29 percent), they exhibited 26 percent of the total wet-pavement accidents on the system (14). The study also found that wet-pavement collisions on freeway and four-lane black spots account for more than 75 percent of the total black-spot accidents.

The black-spot improvement program provided an excellent opportunity to test the effectiveness of the new friction-course mixes. Both the OFC and the DFC mixes were used for rehabilitation at such highway locations. The tentative friction guidelines shown in Table 5 have been used in establishing priorities for treatment. Rehabilitation at some black spots included geometrics and/or other improvements as well as resurfacing as noted in the following sections.

Number of Accidents Before and After Treatments at Freeway Black Spots

Table 6 shows the number of wet-pavement and total accidents before and after resurfacing at eight freeway locations treated during the period 1976-1978. Six locations are on the Highway 401 Toronto By-Pass, one on Highway 401 near London, and one on Highway 417 (Ottawa Queensway). Two years' accidents were considered before resurfacing, and one, two, and three years' accidents were considered after treatment for work done in 1978, 1977, and 1976.

Reductions in wet-pavement accidents after resurfacing ranged between 17 and 73 percent. For all eight locations combined, an average of 54 percent reduction in wet-pavement accidents was obtained.

Table 5. Tentative guidelines for friction-classification system.

Facility Type	Speed Limit (km/h)	Friction Level (SN) at Speed Limit		
		Good	Borderline	Low
Freeway and main highway	100	>30	25-30	<25
Two-lane and four-lane road	80	>31	27-31	<27
Intersection	80	>39	31-39	<31
	60	>44	36-44	<36

Note: 1 km = 0.6 mile.

Table 6. Change in accidents before and after resurfacing at black-spot freeway locations.

Location	Year of Treatment	Wet-Pavement Accidents		Total Accidents		Remarks
		No.	Change (%)	No.	Change (%)	
Highway 401, Toronto: Interchanges 51A to 53, express lanes	1976	B 64	-53	B 220	-21	Eastbound and westbound, three lanes each direction
		A 30		A 173		
Collector lanes	1977	B 40	-40	B 155	-24	Eastbound and westbound, three to four lanes each direction
		A 23		A 118		
Interchanges 58 to 57, collector lanes	1978	B 7	-71	B 16	-69	Westbound, three lanes before and four lanes after treatment
		A 2		A 5		
Interchange 56 to 55, collector lanes	1978	B 22	-55	B 46	-24	Westbound, four lanes
		A 10		A 35		
Transfer lanes at Interchange 56	1978	B 15	-60	B 26	-42	Westbound, two lanes; AADT, 17 000
		A 6		A 15		
Interchanges 51A to 50, collector lanes	1978	B 30	-17	B 98	-6	Westbound, three to four lanes
		A 25		A 92		
Highway 401, London: Interchange 19 to 1.1 km west	1978	B 6	-67	B 13	-46	Only two westbound lanes treated; WB AADT, 9000
		A 2		A 7		
Highway 417, Ottawa: St. Laurent Boulevard to Hurdmans Bridge	1977	B 73	-73	B 168	-53	Eastbound and westbound, two lanes before and three lanes after treatment; AADT, 61 000 in both directions
		A 20		A 79		
Total		B 257	-54	B 742	-29	
		A 118		A 542		

Note: B and A designate number of accidents before and after resurfacing, respectively.

Table 7. Change in accidents before and after treatment at black-spot intersections.

Location	Year of Treatment	Wet-Pavement Accidents		Total Accidents		Rehabilitation Type	Remarks
		No.	Change (%)	No.	Change (%)		
Highway 7 at Islington Avenue	1978	B 17	-76	B 35	-69	DFC mix with 100 percent trap-rock aggregate plus warning traffic sign	Four-lane undivided, 6 percent grade; AADT, 27 000; only westbound lanes (downgrade) treated
		A 4		A 11			
Highway 2, Counter St., Kingston	1978	B 6	-33	B 13	None	DFC mix with 100 percent trap-rock aggregate	Four-lane divided, 5 percent grade, sharp curve; AADT, 17 000
		A 4		A 13			
Highway 40 at Plank Rd., Sarnia	1978	B 5	-80	B 7	-57	DFC mix with 100 percent steel-slag aggregate	Four-lane undivided; AADT, 7500; slight grade
Highways 2 and 4 at Wonderland Road, London	1978	B 4	-75	B 12	-25	DFC mix with 100 percent steel-slag aggregate	Four-lane undivided; AADT, 8750; slight grade
		A 1		A 9			
Highway 3 at Ridge Road	1977	B 3	-100	B 4	-50	Retexturing by cold planing	Four-lane undivided; 4 percent grade; AADT, 6000; only two westbound lanes downgrade treated
		A -		A 2			
Total		B 35	-71	B 71	-46		
		A 10		A 38			

Note: B and A designate number of accidents before and after treatment, respectively.

Overall reduction in total accidents (i.e., wet, dry, snow, and ice) was 29 percent.

An OFC mix was used on all the Highway 401 Toronto locations and a DFC mix was used at London and on Highway 417. In all projects the surface-course mix included traprock coarse aggregate and traprock screening fines, except on Highway 401 at London where steel-slag coarse and fine aggregates were used. At Highway 417, rehabilitation included widening the pavement from two to three lanes in each direction. In all cases the old pavement was concrete. Resurfacing with the OFC and DFC mixes has approximately doubled the skid-resistance levels.

It is interesting to compare the number of accidents on the treated portions of the Highway 401 Toronto By-Pass with the accidents on the total length of the by-pass [30 km (19 miles)]. During the period of observation, traffic volumes increased by approximately 25 percent, a number of geometric improvements were made, and in 1976 the speed limit was reduced from 112 km/h (70 mph) to 100 km/h (60 mph). In the five-year period between 1975 and 1979, the total number of accidents on the 30-km section remained at relatively constant annual levels. The wet-pavement accidents as a percentage of total accidents varied between 31 and 33 percent for the period between 1975 and 1977 and 24-26 percent for 1978 and 1979. It would appear that as more rehabilitation work is carried out at black-spot locations on the by-pass, further reductions in the number and the percentage of wet-pavement to total accidents may be achieved.

Number of Accidents Before and After Treatment at Signalized Intersections

Table 7 gives the number of wet-pavement and total accidents before and after rehabilitation at five black-spot signalized intersections treated in 1977 and 1978. An average of two years' accidents before and one year after treatment was considered.

The reductions in wet-pavement collisions after treatment ranged between 33 and 100 percent. Overall, for the five intersections combined, an average of 71 percent reduction in wet-pavement accidents and 46 percent in total accidents was obtained. Pavement skid-resistance levels observed before and after treatment at various sites showed improvements between 13 and 20 SNs as measured by the ASTM skid trailer.

With one notable exception, treatment at all

sites included resurfacing with a DFC mix. Traprock coarse and fine aggregates were used on Highway 7 and on Highway 2 and steel-slag coarse and fine aggregates were used on Highway 40 and on Highways 2 and 4. For Highway 3, treatment included retexturing of the pavement surface by cold planing by using a CMI rotomill (3).

The old pavement in all cases was bituminous concrete. Major deficiencies included excessive polishing of coarse aggregates on the surface, wheel-track rutting, and surface contaminations by oil deposits at the intersections.

Treatment of black-spot intersections appears most effective at the intersection of Highway 7 and Islington Avenue in Woodbridge. At this location, rehabilitation included resurfacing with a DFC mix plus installation of an electronic overhead warning traffic signal with flashing lights at the top of the grade approaching the intersection. The sign operates in conjunction with the traffic lights at the intersection and reads Be Prepared to Stop when the amber or red lights are on. During the time prior to rehabilitation, this location had consistently shown a high incidence of wet-pavement accidents. During the first year after treatment, rehabilitation resulted in a 76 percent reduction in wet-pavement accidents and a 69 percent reduction in the total accidents at the intersection.

CONCLUSIONS

- Both macrotexture and microtexture qualities required for good wet-pavement friction characteristics are influenced by the quality and gradation of the aggregate in the asphalt mix. Mixes that contain aggregates with higher PSVs produce higher friction levels.

- PSV should be considered along with aggregate abrasion characteristics, particularly for high-speed traffic in which maintenance of angular macrotexture stone projections on the pavement surface is essential in providing good friction levels. Stones that have moderate PSVs but high abrasion resistance, such as the traprock materials in southern Ontario, provide satisfactory skid-resistance performance under extremely heavy traffic.

- Blending a higher PSV coarse aggregate in a mix can yield significant skid-resistance improvements.

- The use of a high percentage of crushed coarse aggregate in bituminous mixes appears to

produce better and longer-lasting surface textures and skid-resistance levels. Such stonier mixes also have economic advantages, mainly because of less requirement for asphalt cement. Sand mixes containing traprock screenings that produce good microtexture but little macrotexture provide a reasonable level of skid resistance when tested under standard conditions with the brake-force trailer but exhibit a significant decline in skid resistance under conditions of moderate or heavy rain. Such mixes would provide a satisfactory surface texture only for low-speed traffic.

5. For very heavily trafficked pavements with large numbers of trucks, use of prime-quality (hard, harsh, and angular) crushed coarse and fine aggregates is essential so that adequate texture and skid-resistance levels can be maintained under such severe traffic conditions. For moderate traffic volumes, skid-resistance levels of bituminous mixes are predominantly influenced by the type of coarse aggregate in the mix. The role of the fine aggregate is significantly less apparent than that in the case of heavily trafficked highways.

6. Performance analysis of various test mixes on Highway 401 in Toronto and Highway 7 in Lindsay identified target total aggregate gradation limits for DFC and OFC mixes. Mixes within such gradations produce excellent surface textures and maintain superior overall friction levels. They also maintain high levels of skid resistance during adverse weather conditions such as heavy rain or light snow and slush on the pavement surface. The OFC mixes provide reduced noise levels.

7. Use of the new friction-course mixes at highway locations that have low friction levels and experience a high rate of wet-pavement collisions appears to produce substantial reduction in wet-pavement and total accidents.

ACKNOWLEDGMENT

We would like to acknowledge many MTC staff persons for their contributions. In particular, J. McDougall and J. Ryell of the Engineering Materials Office, P. Ksenych of the Central Region, and F. Field of B.P. Walker Associates, Ltd., formerly Bituminous Testing Engineer with MTC.

REFERENCES

1. R. Schonfeld. Skid Numbers from Stereo-Photographs. Department of Highways, Ontario, Rept. RR155, Jan. 1970.
2. R. Schonfeld. Photo-Interpretation of Pavement Skid Resistance. Engineering Research and Development Division, Ministry of Transportation and Communications, Downsview, Ontario, June 1974.
3. N. Kamel and D. Wong. Evaluation of Retexturing Methods for Improving Skid Resistance of Existing Slippery Pavements. Design Technical Sessions and Workshops, Roads and Transportation Association of Canada, Conf. Proc., 1978.
4. J. Ryell, J.T. Corkill, and G.R. Musgrove. Skid Resistance of Bituminous-Pavement Test Sections: Toronto By-Pass Project. Transportation Research Record 712, 1979, pp. 51-61.
5. N. Kamel, J.T. Corkill, and G.R. Musgrove. Bituminous Friction Course Sections at Lindsay, Ontario. Presented at 25th Annual Conference of the Canadian Technical Asphalt Association, Victoria, B.C., Nov. 1980.
6. S.H. Dahir and J.J. Henry. Alternatives for Optimization of Aggregate and Pavement Properties Related to Friction and Wear Resistance. FHWA, Rept. FHWA-RD-78-209, April 1978.
7. R.W. Miller and W.P. Chamberlin. Skid Resistance of Bituminous Pavements Built with Carbonate Aggregates. Engineering Research and Development Bureau, New York State Department of Transportation, State Campus, Albany, Res. Rept. 77, April 1980.
8. Skidding Accidents: Pavement Characteristics. Proc., 2nd International Skid Prevention Conference. TRB, Transportation Research Record 622, 1977.
9. T.R.J. Fabb and J.V. Heyes. Deformation of Asphalt Wearing Courses. Proc., 24th Annual Conference of Canadian Technical Asphalt Association, 1979.
10. K. Ichihara. Application of Knowledge of Pavement Surface Properties in Japan. TRB, Circular 192, Feb. 1978, pp. 21-27.
11. MTC Method of Mix Design for Asphalt Concrete Pavements. Engineering Materials Office, Ontario Ministry of Transportation and Communications, Downsview, 1979.
12. D.N. May and M.M. Osman. Noise From Retextured and New Concrete and Asphalt Road Surfaces. Presented at Inter-Noise 78, San Francisco, CA, May 1978.
13. F.B. Holt. Winter Maintenance of Open Friction Course Mixes. Engineering Research and Development Division, Ministry of Transportation and Communications, Downsview, Ontario, Internal Rept., May 1979.
14. N. Kamel and T. Gartshore. Ontario's Wet Pavement Accidents Reduction Program. Presented at the ASTM Symposium on Surface Characteristics and Materials, Dec. 1980, Orlando, FL, MTC Rept. MSR-80-001, 1980.

Performance Observations on Open-Graded Bituminous Concrete Overlays in Connecticut

CHARLES E. DOUGAN

Performance data are presented for open-graded overlays of different ages. The overlays investigated were all placed on Interstate highways that had average daily traffic volumes in excess of 20 000 vehicles/day. Associated with one project was the application of open-graded overlays of varying thickness and support layers. All overlays have exhibited high skid resistance, which has stabilized at skid numbers 40-50, irrespective of lane or traffic volume. Attempts to apply thermoplastic striping were unsuccessful because snowplows tended to remove not only the stripes but also a portion of the underlying pavement to which they adhered. Compared with conventional overlays, there appears to be no advantage gained in terms of resistance to reflective cracking by using open-graded overlays. Air-permeability tests indicate a certain choking of the void structure in open-graded pavements with time. The removal of snow, and especially ice, from open-graded pavements is somewhat more difficult than it is from conventional pavements because of the greater surface area presented for adhesion and the loss of chemical deicers into the void structure. In this respect, greater amounts of deicers are normally required for open-graded pavements than anticipated.

In the 1960s, transportation agencies completed large segments of the Interstate system in the United States. At the end of that decade, the need arose for surfaces with high skid resistance to provide increased safety for the motoring public. Work completed by the National Cooperative Highway Research Program (NCHRP), the Federal Highway Administration (FHWA) (1-4), and several states (5-7) led to the current standards for high-friction pavement surfaces. By the early 1970s, work by the British led FHWA to promulgate Notice HNG-23 (2), which encouraged the use of the open-graded friction course (OGFC). Since then, most states and the asphalt paving industry have used open-graded materials to improve the friction characteristics of pavement surfaces.

Connecticut's first experience with open-graded material was a limited test section placed on a temporary roadway in Groton, Connecticut. The locale was ideally suited to demonstrate the unique high-friction properties of the material, since the test area carried a substantial traffic load around a construction site. The pavement was evaluated for approximately two years. During this period, the section was structurally stable and the skid numbers (SNs) were fairly constant in the range of 45-55 (7). From this work, larger projects evolved; the first was on I-91 south of Hartford (7).

FIRST LARGE-SCALE PROJECT

In June 1975, a two-course overlay was placed on the original concrete pavement on I-91 in the towns of Cromwell, Rocky Hill, and Wethersfield (State Project 33-90). Plans for the project called for a 2-in lift of dense-graded material [Connecticut Department of Transportation (ConnDOT) class 1] topped by a 0.75-in OGFC (ConnDOT class 14) as a safety improvement (Tables 1 and 2). Incorporated into this project were two experimental sections; one was located at the north end of the project in the northbound roadway and the other was at the south end of the project in the southbound roadway.

The primary purpose of this study was to determine the optimum depth of the OGFC and the class-1 layers and their longevity. Each of these sections was divided into three sublots; each sublot received a different treatment. These were as follows:

1. Class-1 material, 1.5 in, topped by 0.75 in of open-graded mix;
2. Open-graded material, 1.5 in placed on the concrete; and
3. Open-graded material, 0.75 in placed on the concrete.

It should be stated here that a class-8 scratch coat was placed on the surface on the concrete as a bonding agent for the overlays over the entire project, including the experimental areas. The class-8 material is a dense sand-asphalt leveling course applied by a skid box. In this case, it served not only as a leveler but also as a primer to improve adhesion between the overlay and the old concrete. Current ConnDOT practice uses an emulsified asphalt tack coat applied at a rate of 0.05 g/yd².

The locations of the six sublots are presented in Figure 1. As is apparent from Figure 1, each of the experimental sublots has a replicate in the opposite roadway. The 25-ft gaps between the variable-depth experimental sections were employed for taper.

ConnDOT design for overlays on jointed-concrete pavements requires saw cuts 0.38 in wide by 0.5 in deep in the overlay over transverse joints to control reflection cracking. It was claimed that OGFC would inhibit this cracking due to its physical properties. To prove or disprove this claim, additional experimental sites were selected within the project to evaluate reflection cracking of the old portland cement concrete joints through the open-graded bituminous overlay. These sections and the treatment afforded them are as follows:

1. Southbound: The first three joints in experimental sublots 1S, 2S, and 3S were sawed in OGFC surface, whereas the last two joints were left unsawed; and
2. Southbound: The first 20 consecutive joints south of the sign "Wethersfield-Rocky Hill Town Line" were sawed in the dense-graded leveling course prior to placement of OGFC and were not sawed through OGFC.

OBSERVATIONS AND TESTS

During the period from June 1975 to September 1978, we performed condition surveys of the experimental and control sections placed on I-91. These inspections were generally conducted twice yearly and included an accounting of longitudinal and transverse cracking, random cracking, durability of line striping, and loss of material. Air-permeability and skid tests have also been conducted, rutting measurements have been obtained at various intervals since placement, and sand-patch and silly-putty tests have been performed once. To increase our data base, we also performed air-permeability, sand-patch, silly-putty, and skid tests and obtained rutting measurements on two open-graded pavements placed on I-95 in July 1977. Here, however, one of these pavements contained a 3/8-in top-size aggregate (Madison) and the other a 1/2-in top-size aggregate.

Table 1. Job-mix formula, 1975 and 1981: sieve analysis.

Sieve Size	Percent Passing by Weight				
	1975 Formula		1981 Formula		
	Class 14	Class 1	1/2 in	3/8 in	Class 1
1 in					100
3/4 in		95-100	100		90-100
1/2 in		70-100	90-100	100	70-100
3/8 in	100	60-82	-	90-100	60-82
No. 4	30-45	40-65	20-45	20-45	40-65
No. 8	11-20	28-50	5-19	5-19	28-50
No. 50	-	10-26	-	-	6-26
No. 200	0-5	2-8	2-5	2-5	2-8

Table 2. Job-mix formula, 1975 and 1981: other characteristics.

Characteristic	1975 Formula		1981 Formula		
	Class 14	Class 1	1/2 in	3/8 in	Class 1
Bitumen content (AC-20) (%)	5.0-6.5	5-8	5.5-7.5	5.5-7.5	5-8
Temperature (°F)					
Mix	225-250	265-325	325 max.	325 max.	325 max.
Aggregate	None	280-350	225-250	225-250	280-350
Void content ^a (%)	None	3-6	None	None	3-6
Stability (lbf)	None	1200 min.	None	None	1200 min.
Flow (in)	None	0.08-0.15	None	None	0.08-0.15
Aggregate ^b loss (%)	40 max.	40 max.	40 max.	40 max.	40 max.

^a Determined from 75-blow Marshall test.
^b Trap rock (American Association of State Highway and Transportation Officials T96).

SUMMARY OF RESULTS

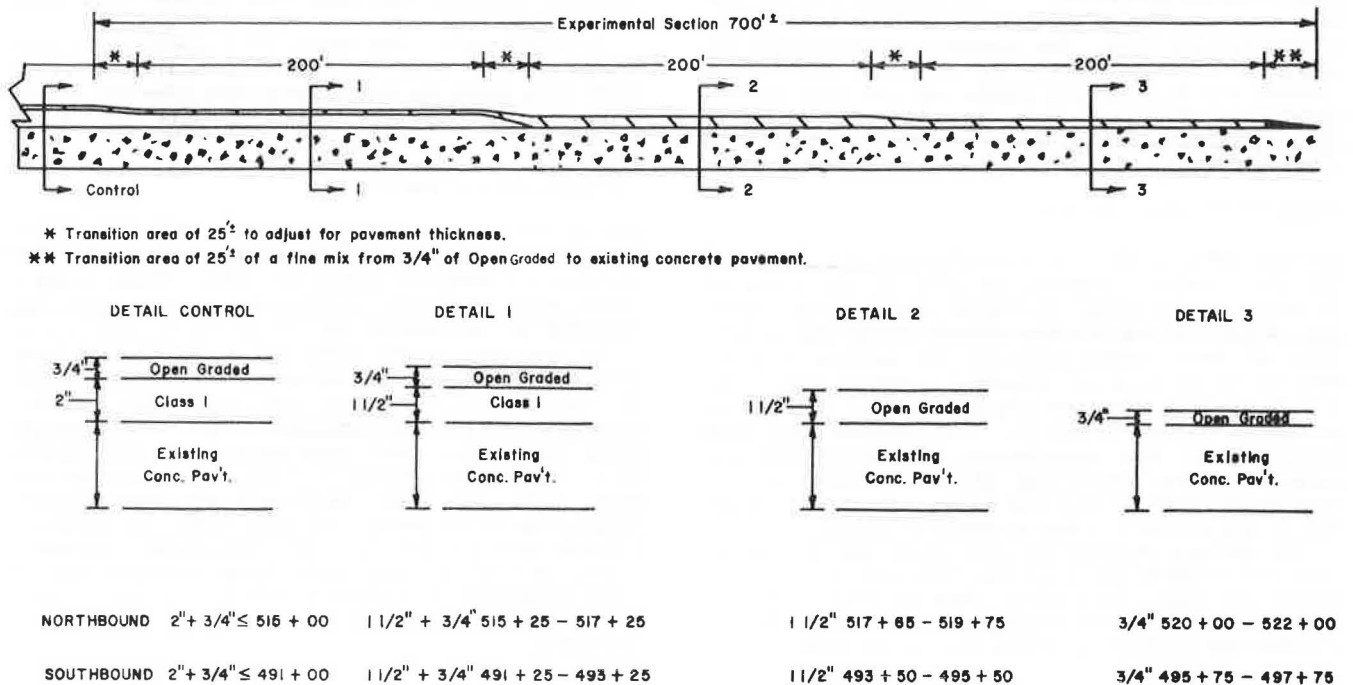
The data obtained from recent surveys and tests conducted on the various open-graded pavements not only reinforce but also complement the conclusions incorporated in an earlier report (6). The following is a summary of our latest findings:

1. OGFC overlays have for the most part performed well, not only in standard pavements (OGFC over 2 in of dense-graded material) but also in two of the three test sections placed on I-91. The test section with the thinnest overlay (0.75 in of open-graded material on concrete) failed in adhesion and lifted off in several large areas. Several smaller areas in the standard sections have also lifted off; one possible explanation for this failure is the inadequacy or poor application of the sand-asphalt scratch coat. Figures 2-13 show typical views of the experimental and control sections on I-91.

The gouge marks shown in Figure 3 and the peeling off of material under the thermoplastic paint (Figure 5) demonstrate the susceptibility of the open-graded material to impact loads or forces exerted by snowplows. Most of the type of damage was sustained early (up to six months after placement), when the asphalt was "young" and had not hardened. The thermoplastic striping originally placed on the open-graded material has since been modified because of the damage sustained by the pavement when the striping peels off under the action of snowplows. We now employ a thermoplastic stripe that has a thinner cross section and tapered ends.

2. The open-graded mat has failed to arrest longitudinal reflective cracking (Figure 2). Transverse reflective cracks have also appeared (Figure 13) where the crack-relief joints have been misaligned or the underlying transverse joints have opened to unusual widths. Spalling is virtually absent or minimal at reflective cracks to date. Of the 20 consecutive unsawed joints in the control section on I-91, 95 percent have reflected through

Figure 1. I-91 safety improvement experimental treatment.



the surface, whereas 100 percent of the unsawed joints in the experimental sections have reflected through.

We have not conducted any condition surveys on the I-95 pavements, but a report from another unit monitoring the surface condition of a whole series of pavements that incorporates both 3/8- and 1/2-in aggregates indicates that the 1/2-in top-size aggregate has superior crack resistance.

3. Although the permeameter tests that we conducted on I-91 and I-95 are by no means statistically valid, they do show a trend toward "choking" of the open-graded surface course and variation in permeability between the centerline and the edge of the shoulder. The results of the permeability tests on I-91 and I-95 are presented in Table 3. The I-95 permeability tests indicate that, initially, the

Figure 2. Propagation of longitudinal cracking beyond previously sealed portions (northbound, control section).



Figure 3. Gouge marks from hard object in surface of open-graded material (southbound, control section).

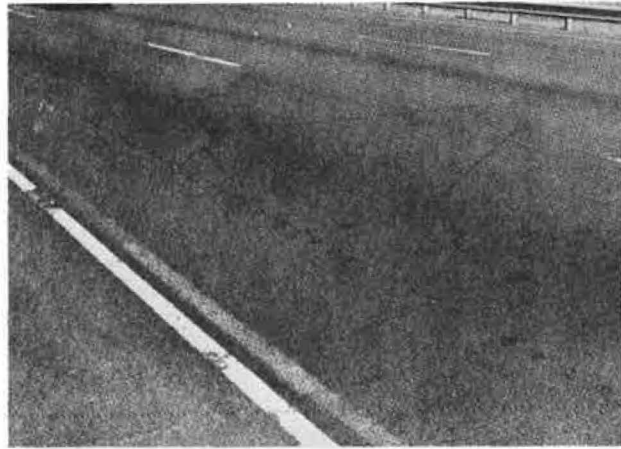


Figure 4. Typical gouge in open-graded surface (southbound, control section).

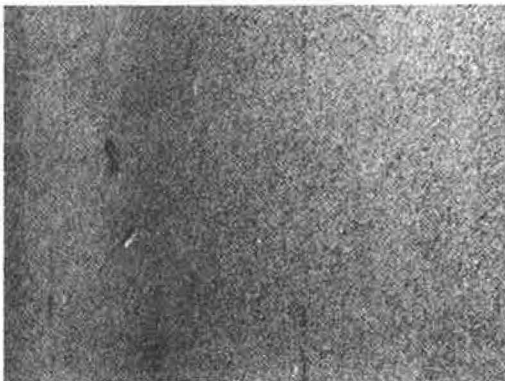


Figure 5. Loss of open-graded material over dense-graded course (southbound, control section).



Figure 6. Open-graded material lost beneath thermoplastic line striping during snowplow operations (southbound, control section).

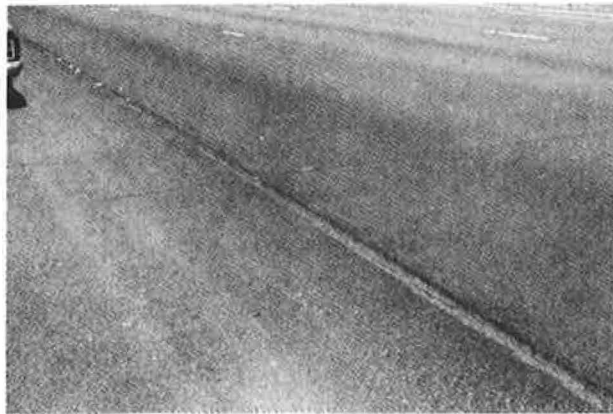
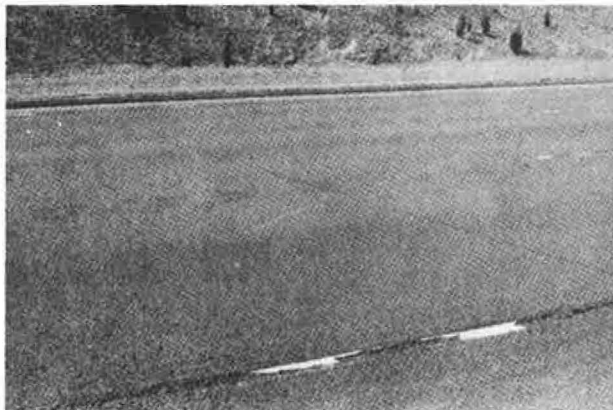


Figure 7. Overview of portion of test section 1N.



pavements (four to nine months) were extremely open and passed considerable volumes of air quite rapidly; this was true even of the shoulder area. At four years, the shoulders of the pavements that had both 3/8- and 1/2-in aggregate became somewhat plugged; the outer lane remained more open, particularly in the pavement that contained the 1/2-in aggregate. The I-91 pavements (all 3/8-in top-size aggregate) all show considerable lessening of voids,

not only in the shoulder areas but also in the low-speed lane. Here a pattern is developed in which the middle of the lane yielded a lower permeability than that of the wheel paths. As compared with the three-year tests, the six-year tests showed a surprising improvement in permeability in both the shoulder and the outer lane. This improvement could

Figure 8. Overview of portion of test section 2N.

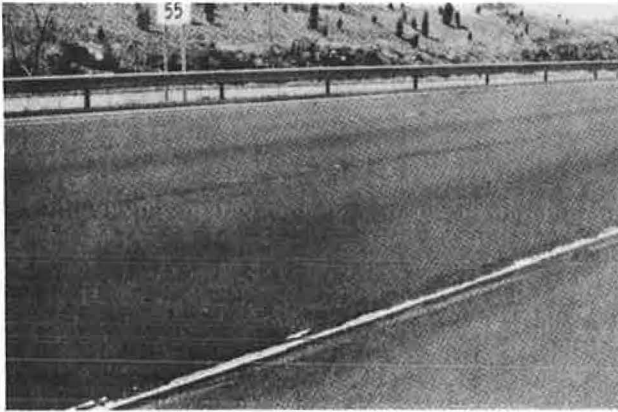


Figure 9. Overview of portion of test section 3N.



Figure 10. Overview of portion of test section 1S.

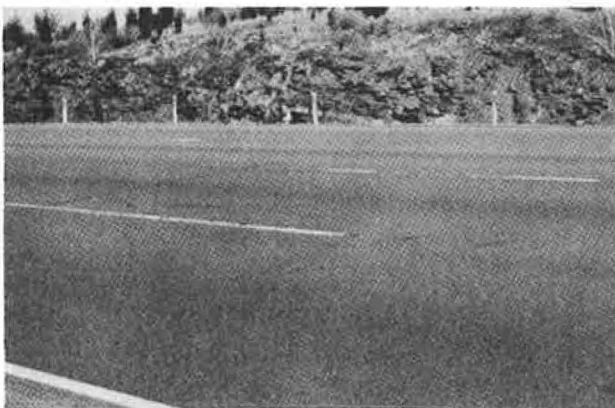


Figure 11. Overview of portion of test section 2S.

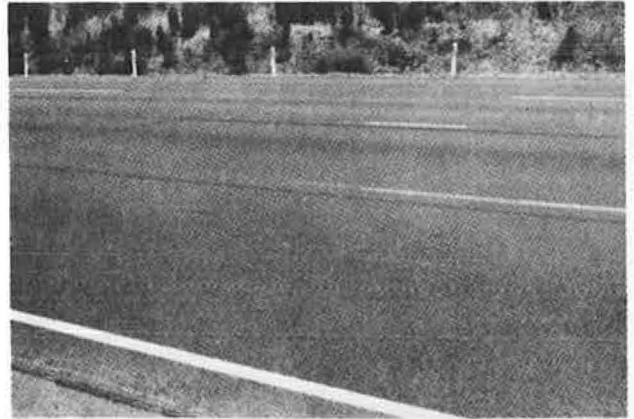


Figure 12. Overview of portion of test section 3S.

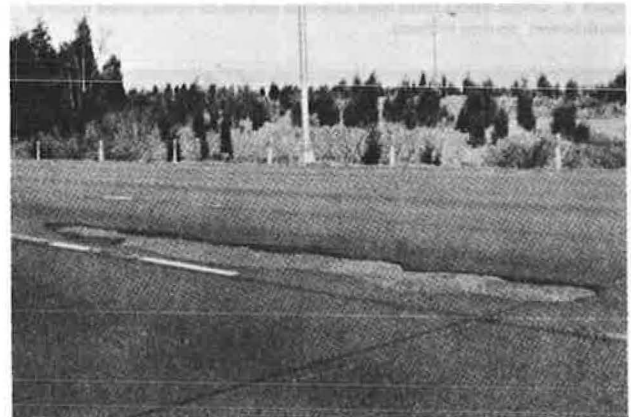
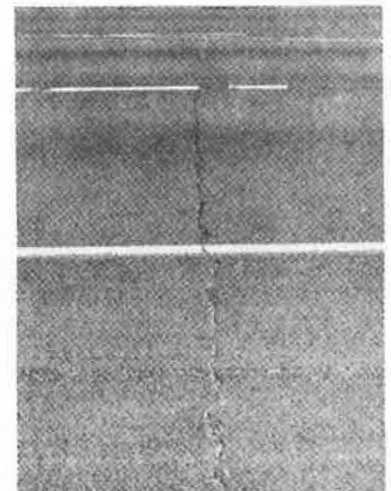


Figure 13. Typical reflection crack over transverse joint in concrete pavement.



be attributed to the extreme lack of snow and ice over the last two winters and, in turn, to a rather sparse application of sand and chemical deicers, which are prime contributors to choking.

4. High SNs have been maintained on all open-graded pavements with normal seasonal variations. Initial tests showed relatively low readings until the asphalt coating had worn away in one to two months and then an increase to a value in the range of SN40-50. The high friction has been maintained over the life of the pavements (up to six years on I-91). Figure 14 shows the history of skid resistance on all travel lanes of the I-91 pavement and the general behavior of the skid resistance of our conventional dense-graded pavements. As can be seen, there is a considerable difference between the

SNs of the open-graded and the dense-graded pavements after six years.

5. As compared with the new open-graded material placed on I-95, the 2.5-year-old I-91 pavement exhibited a much shallower texture as determined from sand-patch and silly-putty tests (Table 4). This can probably be related to the clogging of surface voids with foreign materials, which is in turn related to the reduced permeability of the I-91 pavement.

6. Rutting has been minimal in the open-graded surfacing; maximum depths approached 0.25 in after six years of heavy traffic on I-91 and 0.4 in after four years on I-95 (Figures 15 and 16). All sections tested on I-95 showed a definite trend toward wheel-path rutting. The section that contained the 1/2-in aggregate demonstrated what might appear to be shoving of material from the shoulder inward toward the centerline, although the roadway in this section was on tangent. As compared with the 0.75-in open-graded sections on I-91, the thicker 1.5-in section has experienced little rutting, a fact that would attest to the stability of the OGFC material under load.

7. Table 5 presents before and after accident statistics for the I-91 and I-95 projects. The sixth column of Table 5 lists accidents per million vehicles over each section investigated. This figure was arrived at by dividing the total number of reported accidents by the product of the average daily traffic (ADT) derived for the time period in question, the number of days per year, and the number of years in the period. It does not take into account the length of the various sections, since we are primarily interested in the before and after effects over the same sections.

It follows from Table 5 that accidents were reduced by 40 and 29 percent, respectively, in the northbound and southbound roadways. Conversely, accident rates actually increased on the I-95 sections after applications of the open-graded friction courses. The increase was 53 percent with the 3/8-in material, and 16 percent on the 1/2-in material. The fact that from 1972 to 1973 only police-reported accidents and no operator-reported accidents were considered in our accident statistics would tend to bias the results in favor of the "before" period. From 1974 on, all qualifying operator and police-reported accidents were included in the statistical data; this would result in a greater number of accidents in all "after" periods for each section.

Table 3. Air-permeability tests on open-graded pavements.

Section	Age (years)	Outer Lane			Shoulder	
		Left Wheel Path	Center of Lane	Right Wheel Path	Left Edge	Right Edge
I-95 Eastbound (Madison-Westbrook)						
3/8-in aggregate,	0.33	0:14	0:14	0:12	0:14	0:15
0.75-in OGFC,	0.75	0:13	0:13	0:10	0:08	0:08
2-in dense-graded	4 ^a	0:41	4:15	0:23	6:24	4:23
I-91 Northbound (Rocky Hill)						
Control (0.75-in OGFC, 2-in dense-graded)	2.5	2:21	8:52	3:15	7:00	3:34
	3	8:44		4:46	34:20	11:32
	6	5:25	6:53	6:53	10:15	6:25
1N (0.75-in OGFC, 1.5-in dense-graded)	2.5	60	>60	0:50	3:13	3:26
	3	19:16	>60	9:10	12:33	8:26
	6	4:25	5:25	2:15	6:50	5:50
2N (1.5-in OGFC, concrete)	2.5	>60	>60	1:53	11:09	3:23
	3	>60	>60	>60	21:08	11:53
	6	2:09	7:55	1:27	8:20	4:55

Notes: Figures denote time required to pass 1 L of air through 81 cm² of pavement surface (min:sec). Pavements containing 3/8- and 1/2-in top-size aggregate are located in Madison and Westbrook, respectively. Depth of open-graded course containing 1/2-in top-size aggregate was 1 in after compaction; current specifications now call for 0.75-in depth, irrespective of aggregate size.

^a Average of two readings taken 8 ft apart in longitudinal direction.

Figure 14. History of skid resistance on open-graded friction course, I-91, Rocky Hill.

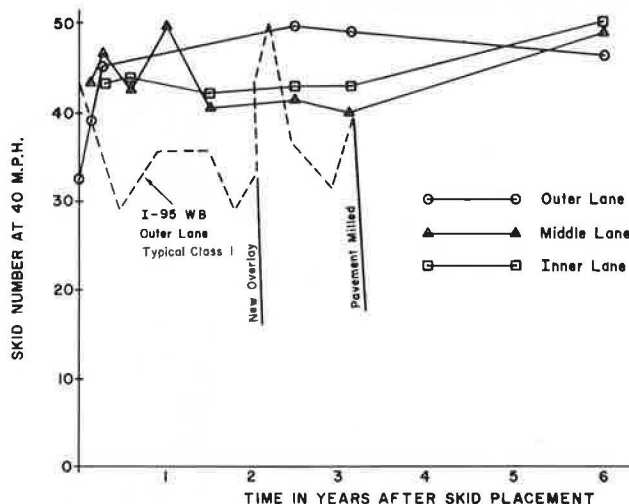


Table 4. Sand-patch and silly-putty test results, Oct. 1977.

Section	Sand-Patch Test (avg depth of texture, in)		Silly-Putty Test (avg depth of texture, in)	
	Middle of Outer Lane	Inner Wheel Path of Outer Lane	Middle of Outer Lane	Inner Wheel Path of Outer Lane
I-91 Northbound (Rocky Hill, placed June 1975)				
Control section (0.75-in OGFC, 2-in dense-graded)	0.034	0.044	0.021	0.038
Test section 1N (0.75-in OGFC, 1.5-in dense-graded)	0.037	0.050	0.027	0.045
Test section 2N (1.5-in OGFC)	0.039	0.053	0.030	0.037
Test section 3N (0.75-in OGFC)	0.028	0.039	NA	NA
I-95 Eastbound (Madison-Westbrook, placed July 1977)				
3/8-in top-size aggregate (Madison)	0.095	0.111	0.048	0.054
1/2-in top-size aggregate (Westbrook)	0.114	0.130	0.100	0.098

NA = not available.

ADDITIONAL INFORMATION OBTAINED

In conversations concerning snow and ice control with maintenance foremen in charge of both the I-91 and I-95 sections of open-graded material, it was acknowledged that the open-graded material required special treatment during winter storms. Chloride applications were reported to have been 10 and 30 percent higher on the open-graded sections than on adjacent conventional sections on I-91 and I-95, respectively. Since no salt-use records were maintained strictly for these open-graded sections, these figures are based on the opinions of the foremen involved.

Noise readings obtained by ConnDOT's Environmental Planning Division showed a reduction of 1 dBA in the open-graded section versus comparable readings for our standard pavement section. Obviously, this is not a significant reduction in noise such as has been reported by drivers who travel over open-graded surfaces. The notable reduction in sound perceived by the driver is associated with change in the frequency of sound generated by the course surface texture of the open-graded material.

Figure 15. Cross sections of I-91 in Rocky Hill showing rutting in outer lane of different design sections.

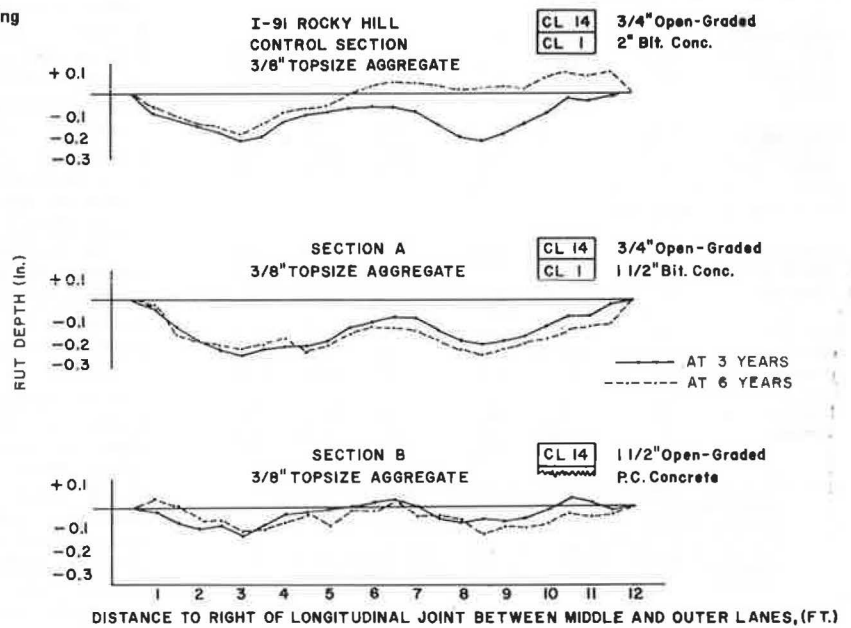


Figure 16. Cross sections of I-95 in Madison and Westbrook that show rutting in outer lane.

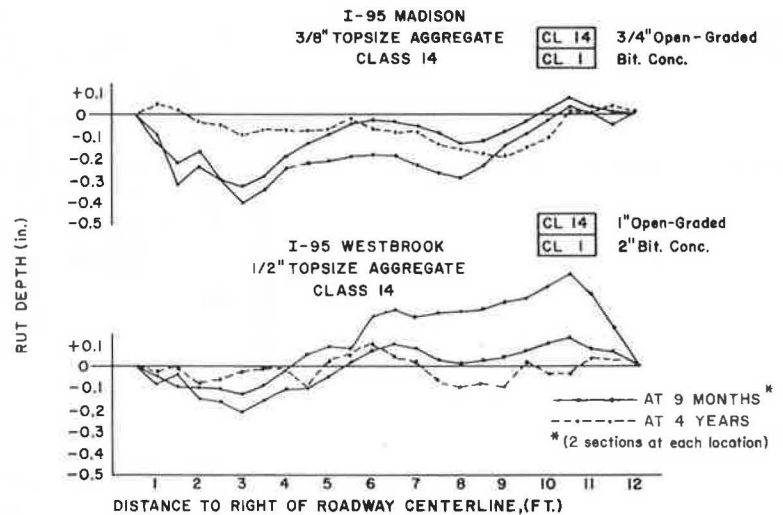


Table 5. Accident statistics for various roadways with open-graded applications.

Condition	Section	Total Accidents	Period (years)	ADT	Accidents per Million Vehicles	SN and Date
No overlay	NB I-91 (before)	69	2.5	24 500	3.09	38.7, 1974
	NB I-91 (after)	98	5	29 400	1.83	47.3, 1980
No overlay	SB I-91 (before)	98	2.5	27 000	3.98	38.5, 1975
	SB I-91 (after)	165	5	32 100	2.82	47.3, 1980
3/8-in OGFC	EB I-95 (before)	124	5 + 7 months	13 800	4.409	35.0, 1975
	EB I-95 (after)	104	2 + 10 months	14 900	6.750	47.7, 1981
1/2-in OGFC	EB I-95 (before)	69	5 + 7 months	13 600	2.490	35.0, 1975
	EB I-95 (after)	43	2 + 10 months	14 400	2.887	48.2, 1981

CONCLUSIONS

Based on the above discussion, the following general conclusions can be drawn. As a skid-resistant surface under heavy traffic, OGFCs have performed well in Connecticut for periods of up to six years. Compared with other ConnDOT mixes used for overlays, OGFC appears to be equal in terms of resistance to reflective cracking. The lateral-internal-drainage characteristics of OGFC have been shown to decrease with time. This decrease is primarily attributed to the use of winter deicing chemicals and abrasives. Due to the short lengths of the three test sections on I-91, no conclusive statements can be made regarding the effect of layer thicknesses on the performance of OGFC mats. In this connection, however, it should be pointed out that the standard ConnDOT design consisting of 0.75-in OGFC on a 2-in dense-graded surface has performed well to date.

ACKNOWLEDGMENT

The project reported in this paper was carried out with the cooperation of the Federal Highway Administration, U.S. Department of Transportation. The contents of this report reflect my views and I am responsible for the facts and accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of the Connecticut Department of Transportation or of the Federal Highway Administration. The report does not

constitute a standard, specification, or regulation.

REFERENCES

1. R.W. Smith and others. Design of Open-Graded Asphalt Friction Courses--Interim Report. FHWA, Rept. FHWA-RD-74-2, Jan. 1974.
2. Notice HNG-23: Open-Graded Plant Mix Seals. FHWA, May 1973.
3. R.A. Ray and J.K. Stephens. Porous Friction Courses and Roadway Surface Noise. FHWA Implementation Package 74-11, March 1975.
4. E.S. Richardson and W.A. Liddle. Experience in the Pacific Northwest with Open-Graded Emulsified Asphalt Pavements. FHWA Implementation Package 74-3, July 1974.
5. R.W. Miller and R.A. Brown. Performance of Open-Graded Friction Courses. FHWA, Rept. FHWA-NY-78-RR-58, March 1978.
6. D.G. Bowers. Performance of an Open-Graded Bituminous Concrete Overlay. Bureau of Planning and Research, Connecticut Department of Transportation, Wethersfield, Rept. 343-4-78-11, Oct. 1978.
7. G.A. Ganung. Friction Characteristics of Paving Materials in Connecticut. Bureau of Planning and Research, Connecticut Department of Transportation, Wethersfield, Final Rept., May 1975.

Publication of this paper sponsored by Committee on Characteristics of Bituminous-Aggregate Combinations to Meet Surface Requirements.

Effects of Baghouse Fines and Mineral Fillers on Properties of Asphalt Mixes

BASSAM A. ANANI AND HAMAD I. AL-ABDUL-WAHHAB

The objective of this study was to determine the effects of baghouse fines on asphalt mixes. The analysis included Marshall tests on mixes that had various ratios of filler to baghouse fines. Other tests to study these effects included stability loss, viscosity, penetration, shear-modulus, and softening point. The results of the study indicated that baghouse fines can greatly affect the properties of the mix, such as the optimum asphalt content, stability, and stability loss. Asphalt mortars that used different ratios of filler to baghouse fines exhibited varied viscosity and penetration. Stability loss, which is a main factor in the design of local mixes, was decreased drastically by the inclusion of baghouse fines. One factor that controls the effect of baghouse fines on asphalt mixes was the percentage of carbon. It is anticipated that the results of this study will be of great help in the improvement of mix properties by incorporating baghouse fines.

Pavement systems in the Eastern Province of the Kingdom of Saudi Arabia are exposed to a multitude of severe environmental factors, mainly the high temperature and the high humidity. Roads usually show excessive failures at an early stage of pavement life. Other factors that contribute to the early failure are the extremely heavy axle loads applied to the roads of this province. There is currently no enforcement of the maximum loads permitted on an axle or of the tire pressures. Another major contributor to failure is the low quality of local materials used for highway construction. For ex-

ample, aggregates used for the bituminous-concrete base course are known to be of low quality. Experience with such material has indicated excessive stripping and high absorption. Local aggregates were observed to expand greatly when soaked, which results in fracture of the aggregates. This is manifested in the roads by the excessive ravelling and existence of small cavities in the road surface caused by the deterioration of the aggregates.

Saudi Arabia is starting a new era of highway construction. Virtually thousands of miles of new, modern highways are under construction, and there will still be a great need for more highways in the near future. The asphalt industry has also begun to expand. Asphalt plants are more abundant now than ever, and the pollution potential from dust emitted from the hot-aggregate elevators, plant screens, bins, hoppers, and pug mills of these plants is becoming more eminent. This dust pollution has already caused major problems for farms located near asphalt plants. An asphalt plant that operates at a rate of 100-200 tons/h will generate about 3000 lb/h of dust, or 50 lb/min.

A major step in the improvement of the existing performance of roads is ensuring a proper mix design. It is anticipated that some of the failures discussed earlier may be attributed to the poor design of the asphalt mixes. The existence of varied properties for local materials requires different mix designs. Fillers used in the mix design are known to have an effect, especially on the optimum asphalt content. The amount of filler used in the plant mixes affects the properties of the mix produced. However, it is not possible to establish the exact amount of this filler due to the loss of fines in the form of dust from the plant.

The large quantities of dust in the air from the asphalt plants have been of great concern to local authorities and farmers. Dust collectors have been incorporated into certain asphalt plants to ensure collection of the dust. This collection dust (referred to as "baghouse fines" in this report) is produced in large quantities from every plant, which causes yet another problem--disposal of the baghouse fines.

A possible solution to the problem of disposal and regulation of the amount of fines in the mixes would be reintroduction of the baghouse fines into the mix. However, the amounts used should be determined in such quantities to ensure a properly designed mix.

The objectives of this study can be summarized as follows:

1. To evaluate the existing mix designs that use local aggregates,
2. To evaluate the effects of introducing baghouse fines into the asphalt mix,
3. To study various characteristics of the new asphalt mixes that incorporate baghouse fines, and
4. To study the effects of baghouse fines and filters on the stability loss of the asphalt mixes.

The results of this study are applicable to mix designs that use aggregates from the Damman, Abohadryyah, and Riyadh. The baghouse fines used were collected from two different sources, namely, Riyadh and Abohadryyah. The Marshall test was used exclusively for the mix design.

DUST COLLECTOR SYSTEMS

Dust collectors have been introduced in asphalt plants to reduce the amount of solids emitted into the air. The main function of such a collector is first to separate the dust from the gas stream in

the plant and then to collect this fine material. Certain rules have been reinforced in some countries to guarantee that the amounts emitted do not exceed maximum limits. Currently no such regulations exist in Saudi Arabia.

There are three general types of dust collectors in use: mechanical centrifugal separators, wet collection systems, and secondary collectors. The last consists of two different kinds: electrostatic filters and fabric filters. Asphalt plants in Saudi Arabia that incorporate such dust collectors use several different types. However, due to the shortage of water and electricity, most plants have adopted either the centrifugal or the fabric-filter types. The cyclonic separator was used to collect the baghouse fines used in this study from the Riyadh plant.

Dust collected from the Abohadryyah plant consists of two distinct types. The first, referred to here as AB.BH.1, is collected by using multicloner separators. These consist of a series of small cyclones arranged together to give an increased capacity. The second type of dust, referred to here as AB.BH.2, is collected by using the fabric-filter separators (1). These separators use fabric filters as the heart of the collection system. The exhaust gases are cleaned by passing the air through a fabric that captures the dust particles. The filters are narrow, long, and baglike, dressed on steel cages. These baghouses are arranged in several groups. The filtering process is conducted by sucking the air from the inside of the bags by using a high-power turbine. This will cause gases to go through the fabric and leave the dust on the outside of the bags.

Fuels are used in the asphalt plants to dry the aggregates before they are mixed with hot asphalt. The effect of the type of fuel used on the performance of the separator could be appreciable. The carbon that results from burning certain fuels will increase the amount of fines that the dust collectors must remove. This might result in clogging the bags in the fabric-filter separator.

Baghouse fines collected from Abohadryyah have shown that approximately 7.0 percent is carbon from the burning of diesel fuel, which is the fuel used in the plant. Excessive sulfur in the fuel can also cause damage to fabric filters (2).

MATERIAL CHARACTERIZATION

This section gives the results of tests to determine the characteristics of the material used in this study, namely, asphalt, aggregates, fillers, and baghouse fines.

Asphalt

Asphalt that had a 75-100 penetration grade was used for all mixes. It was brought from the Ras-Tanura refinery. A series of tests was performed to determine the basic properties of the asphalt. Results of asphalt characterization are given below. The properties measured indicate that the asphalt meets the specifications of the American Society for Testing and Materials (ASTM):

<u>Property</u>	<u>Amount</u>
Penetration (100/5) at 77°F	82
Ductility (cm)	100
Flash point (°F)	625
Specific gravity, 77°F	0.97
Softening point (ball and ring) (°F)	70

Aggregates

Three different types of aggregate were used in this study. The first type was gravel and natural sand obtained from Riyadh, the second was limestone and natural sand from Damman, and finally limestone and natural sand from Abohadryyah. The gradation of the three aggregates is shown in Table 1. As indicated, the gradation was not very different for the three sources. The maximum size for the Riyadh aggregate was 1/2 in, whereas it was 3/4 in for the others. The different sizes of each aggregate were blended to meet the ASTM specifications. Figure 1 gives the gradation of the blend of each aggregate as compared with ASTM specifications.

Specific gravity was determined for each aggregate type when no appreciable difference was observed. However, absorption tests indicated that Abohadryyah aggregates had high absorption values when compared with Riyadh aggregates, which clearly showed very low absorption. Abrasion tests indicated a similar trend. Results of these tests are given in Table 2.

Table 1. Job-mix formula: sieve analysis.

Sieve Size	Aggregate (% passing)			
	Damman	Riyadh	Abohadryyah 1	Abohadryyah 2
3/4 in	100	-	100	100
1/2 in	92	100	91.5	91.5
3/8 in	81	80.0	82.5	82.5
No. 4	54	56.0	59.5	59.5
No. 10	38	39.0	36.5	36.5
No. 40	23	25.0	24.0	24.0
No. 80	15	12.5	13.0	13.0
No. 100	4-15	10.0	10.0	10.0
No. 200	-	6.5	6.8	6.8

Figure 1. Gradation of aggregate mixes.

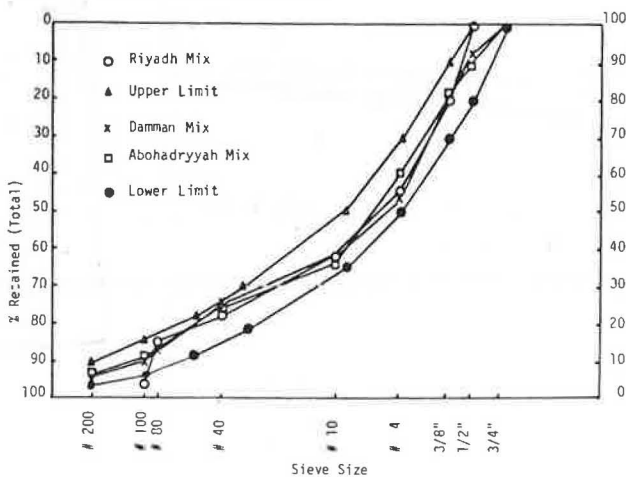


Table 2. Job-mix formula: other tests.

Property	Aggregate			
	Damman	Riyadh	Abohadryyah 1	Abohadryyah 2
Specific gravity				
Bulk	2.54	2.62	2.57	2.57
Apparent	2.79	2.69	2.68	2.68
Asphalt (%)	0.87	0.49	1.25	1.25
Absorption				
Water (%)	3.1	0.9	1.9	1.9
Abrasion (%)	31.2	23.1	28.8	28.8

Fillers

Two different fillers were used--one from the Damman area and one from Abohadryyah. Damman filler was used with the Damman aggregate, whereas Abohadryyah filler was used with both the Riyadh gravel and the Abohadryyah aggregate. Fillers used passed the No. 100 sieve.

Hydrometer analysis was conducted on the fillers. The specific gravity of the fillers was determined by using the vacuum method. The gradation based on the hydrometer test is given in Table 3 and Figure 2. Determination of the plasticity index showed that the fillers are nonplastic. The same conclusion was obtained from the chemical analysis to determine the amount of plastic material in the filler.

Baghouse Fines

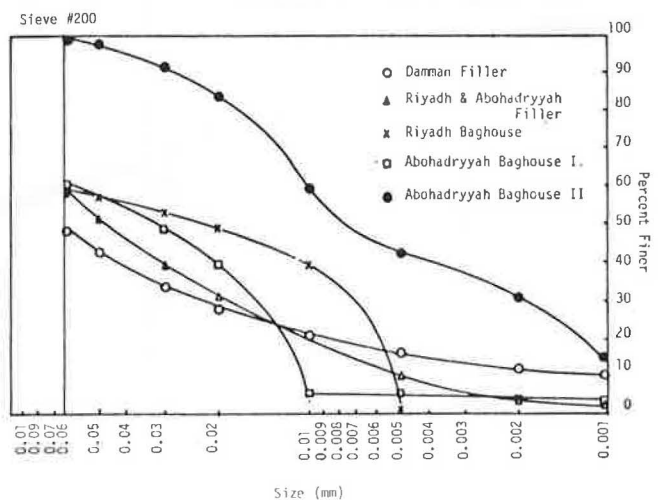
Baghouse fines are the airborne particles separated from the gas stream on a baglike filter. Hydrometer analysis was performed on the different types of baghouse fines; the results are given in Table 4. It is clear that AB.BH.2 was finer than the other baghouse fines, as shown in Figure 2. It should also be noted that Abohadryyah baghouse fines contain some carbon from oil used in the drying process. Specific gravities were determined for the three baghouse fines. AB.BH.2 has a slightly lower unit weight (by 1.1 lb/ft³).

Table 3. Hydrometer analysis of fillers.

Sieve and Grain Size	Filler (% finer)	
	Damman	Riyadh and Abohadryyah
No. 100	100	100
No. 200	48	59
0.05	43.3	51.0
0.04	38.9	48.3
0.03	34.5	40.7
0.02	28.8	33.0
0.01	24.0	19.4
0.009	23.0	18.2
0.007	19.2	15.9
0.005	16.3	11.2
0.003	12.4	5.3
0.001	10.5	4.1

Note: Specific gravity for both fillers was 2.73; both fillers were nonplastic.

Figure 2. Gradation analysis for fillers and baghouse fines.



Plasticity-index determinations on Riyadh baghouse fines and AB.BH.1 showed them to be nonplastic, whereas AB.BH.2 had a plasticity index of about 10. Carbon was separated from AB.BH.1 and AB.BH.2 by using a 2 percent diluted acid. It was found that AB.BH.1 contained about 0.4 percent carbon, whereas AB.BH.2 contained about 6 percent carbon. Separated carbon was washed by using sweet water, dried at 100°C, and then stored in a dry place for further analysis.

ASPHALT MORTARS

To study the effect of adding filler and baghouse fines to asphalt, a series of tests was performed on

the asphalt mortars. Baghouse fines (BH) were mixed with filler (F) at different ratios, namely, F/BH = 100/0, 50/50, 65/35, 80/20, and 0/100. Asphalt was added to each of these ratios at different percentages, and penetration tests were done on the mortars. Figure 3 indicates that penetration decreases as the percentage of baghouse fines increases when log penetration versus the ratio of filler to baghouse fines was plotted. The penetration decreases as the ratio of filler to baghouse fines increases. It was observed that using Abohadryyah baghouse fines will cause more reduction in penetration than using Riyadh baghouse fines (Figure 4).

A softening-point test was conducted on asphalt/baghouse fines and mortars. The results are given in Figure 5. It was found that there is a tremendous increase in the softening point of AB.BH.2 mortar, which could have been caused by carbon. To investigate this, carbon was mixed at different percentages with asphalt and the softening-point tests were done. The results affirmed the above assumption. The softening point of asphalt mixed with carbon was found to increase at a higher rate than that with AB.BH.2. This effect is again shown in Figure 5.

The effect of carbon was investigated further by using a sliding-plate rheometer. The same mortars were tested for shear modulus and viscosity. The spacing of the two plates for the test was 6 mm and the temperature for testing was 25°C. The results, given in Figures 6 and 7, indicate that shear modulus and viscosity for baghouse fines are higher than those for the fillers. Moreover, carbon causes an increase in the viscosity of asphalt. Carbons that have a tremendous surface area cause a rapid increase in asphalt viscosity. It is quite difficult

Table 4. Hydrometer analysis of baghouse fines.

Sieve and Grain Size	Baghouse Fines (% finer)		
	Riyadh	Abohadryyah 1	Abohadryyah 2
No. 100	90.3	86.8	100
No. 200	59.4	60.6	99.8
0.05	55.9	56.1	97.5
0.04	54.7	54.6	94.8
0.03	51.1	49.7	91.8
0.02	47.6	40.0	86.8
0.01	44.6	5.7	51.9
0.009	35.7	5.4	45.9
0.007	14.9	4.8	44.8
0.005	--	4.8	44.0
0.003	--	4.8	34.9
0.001	--	4.8	15.9

Note: Specific gravity was 2.72 for Riyadh BH, 2.7 for AB.BH.1, and 2.70 for AB.BH.2. The first two were nonplastic; AB.BH.2 had a plasticity index of 10.

Figure 3. Penetration test: log penetration versus ratio F/BH.

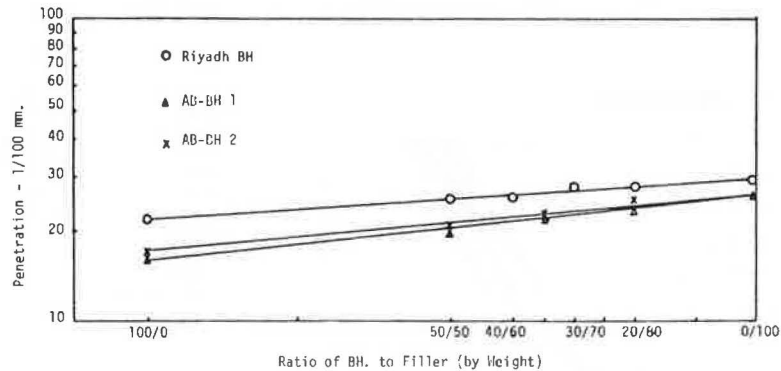


Figure 4. Penetration test: log penetration versus ratio F/asphalt.

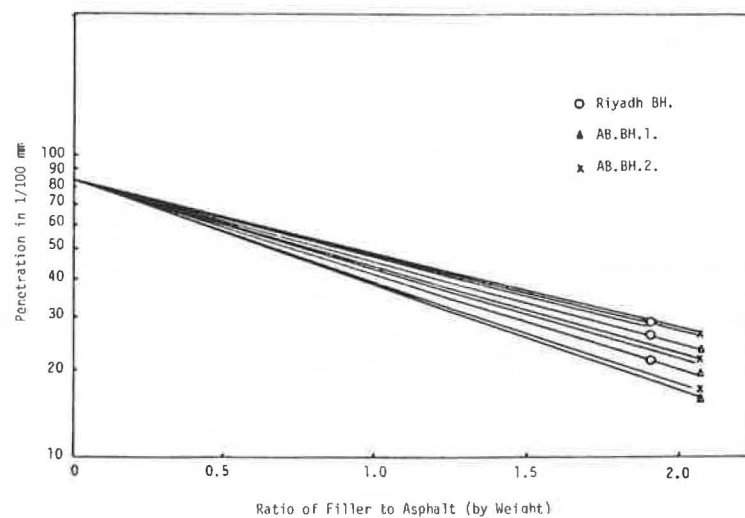


Figure 5. Softening points for asphalt/baghouse fines and filler mortars.

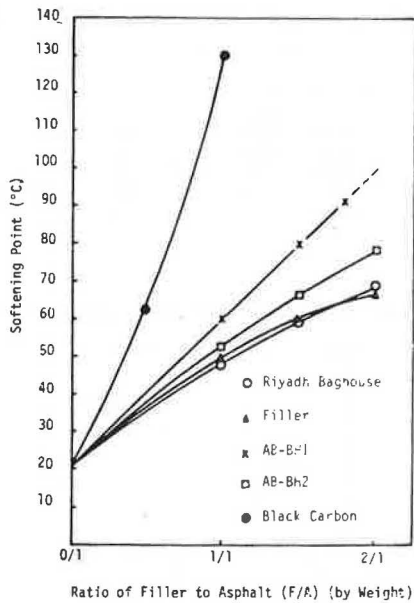
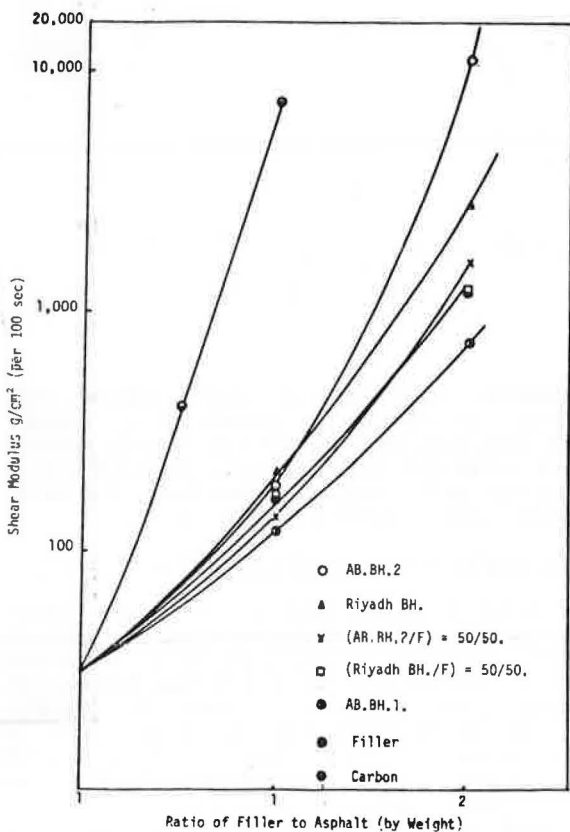


Figure 6. Shear modulus from rheometer analysis.

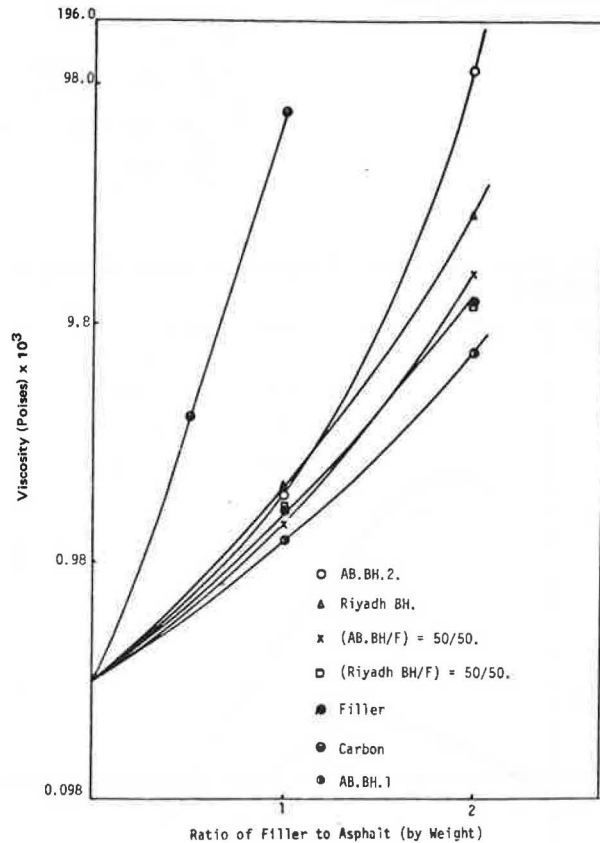


to mix asphalt with carbon at a ratio of filler to asphalt content of 1.5 by weight because this gives a very dry mix. The effect of filler on asphalt was also determined. Results for viscosity, shear modulus, and softening points are given in Figures 5-7.

MIX DESIGN

To investigate the effect of filler and baghouse fines on the mix design, the Marshall method was used because of its wide acceptability, simplicity,

Figure 7. Viscosity evaluation by rheometer analysis.



and correlation with the field performance. At least three samples were repeated for each mix to ensure reproducibility of results.

Two types of tests were used. First the Marshall test was done on specimens after they had soaked 30 min in a 60°C water bath. The second test was the evaluation of the water susceptibility of each mix. This test was conducted by immersing compacted specimens in a controlled water bath at 60°C for 24 h. Stability was measured and stability loss was determined. It was believed that this test was more severe than the conditions that existed in the field.

Effect of Filler on Mix Properties

Damman aggregate is known to cause some problems in the field because of its chalk content. To investigate what effect Damman filler has on mix properties, different percentages of filler were added to the mix. These corresponded to 4, 8, 10, 12, and 15 percent of the weight of the aggregates. Initially, specimens were compacted by using 50 blows of the Marshall hammer on each side.

Stability was determined for the different levels of filler content by two different methods. The Marshall stability results for samples that were soaked 30 min are given in Figure 8. They indicate that an increase in the percentage of filler in the mix will increase the stability. However, no results are given for the stability of mixes soaked 24 h. This is due to the total collapse of the specimens. Another group of specimens was compacted by using 75 blows and tested for 30-min and 24-h stability. After the 24-h soaking, samples were still compact. The stability loss S_L is defined as follows:

$$S_L = S_{30} - S_{24} \tag{1}$$

Table 5. Stability-loss analysis for Damman filler (compacted by 75 blows).

Item	Filler (%)							
	4	4	4	8	10	10	15	15
Aggregate asphalt content (%)	5.0	5.6	6.0	5.6	5.6	6.5	5.6	6.0
Marshall stability (lbf), 30 min at 60° C	2500	2900	2650	3337	3430	3200	4050	3800
Flow	10.0	12.0	14.0	14.0	14.5	16.0	13.0	15.0
Marshall stability (lbf), 24 h at 60° C	1364	1753	1250	613	355	1230	Collapse	500
Flow	22.0	16.0	22.0	25.0	34.0	25.0	Collapse	34.0
Stability loss (%)	45.0	39.5	52.0	81.0	89.0	61.0	100	86

Figure 8. Marshall stability versus asphalt content for different percentages of Damman filler.

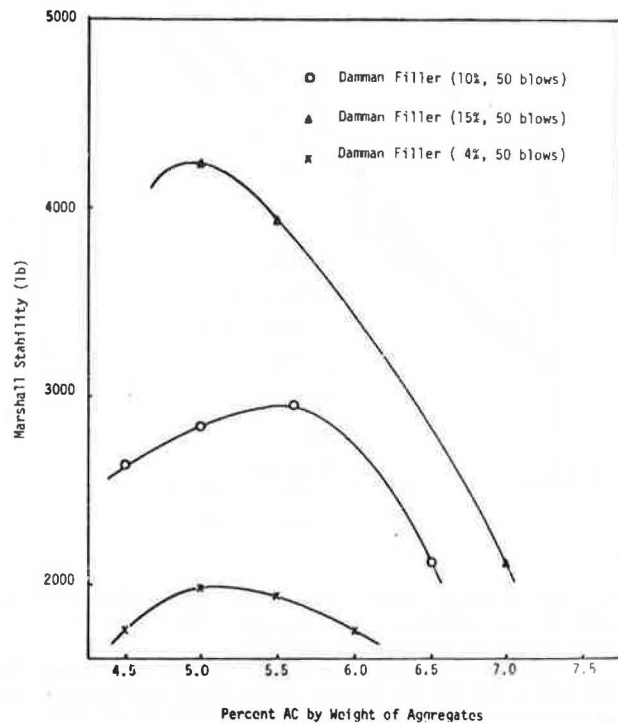
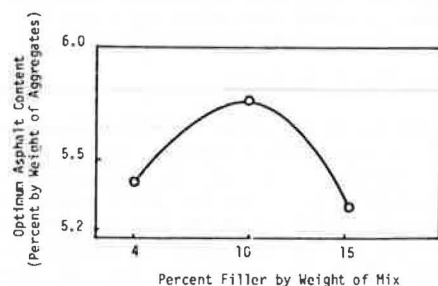


Figure 9. Optimum asphalt content versus percentage of Damman filler.

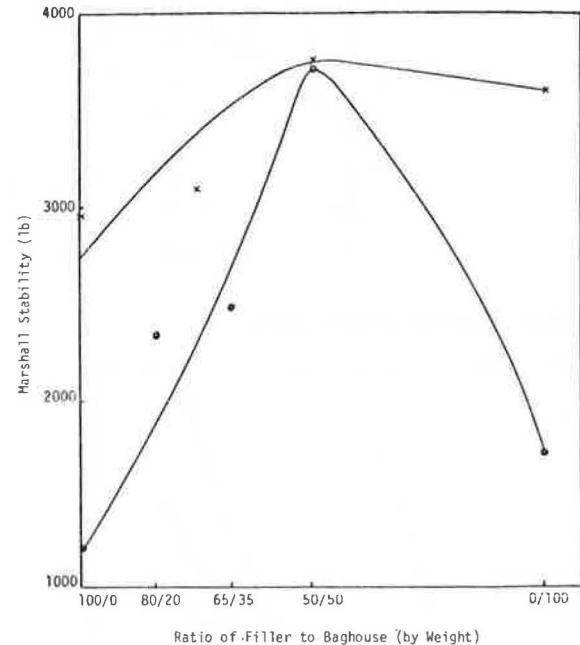


where S_{30} is the stability of the 30-min soaked sample in pounds, S_{24} is the stability of the 24-h soaked sample in pounds, and the percentage of stability loss is defined as follows:

$$\text{Percent } S_L = S_L/S_{30} \quad (2)$$

Stability results for Damman filler are shown in Table 5. It is clear that filler content has a direct relation to stability loss. By increasing filler from 4 to 15 percent, stability loss increases from 39 to 100 percent (for the same mix).

Figure 10. Marshall stability versus ratio F/BH for Riyadh mix.



Calculations show that the optimum asphalt content varies by varying the percentage of filler. Figure 9 indicates that the optimum asphalt content increases while the filler content increases to a maximum value of 10 percent. After that, the optimum asphalt content starts to decrease.

Effect of Baghouse Fines on Mix Properties

Riyadh baghouse fines affected stability positively. Increasing the amount of baghouse fines in the filler increased the stability up to a maximum value at an F/BH ratio of 50/50 and then the stability decreased, as illustrated in Figure 10. Optimum asphalt content increased slightly by the addition of baghouse fines and then decreased to a minimum ratio of F/BH of 0/100. Percentage of air voids varied in a short range.

When Abohadryyah baghouse fines were added to the mix, a slight reduction in stability resulted. As the amount of baghouse fines increased, stability decreased until it reached the minimum at a ratio of F/BH of 0/100 as shown in Figures 11 and 12. Air voids and percentage of air voids filled varied in a wide range (wider than that for Riyadh baghouse fines).

The effect of baghouse fines on the optimum asphalt content was determined. Figure 13 shows that optimum asphalt content was increased in AB.BH.1 up to the maximum at a ratio of F/BH of 50/50 and then decreased (the opposite of the effect from using Riyadh baghouse fines). Optimum asphalt content in-

Figure 11. Marshall stability versus ratio F/BH for AB-BH-1.

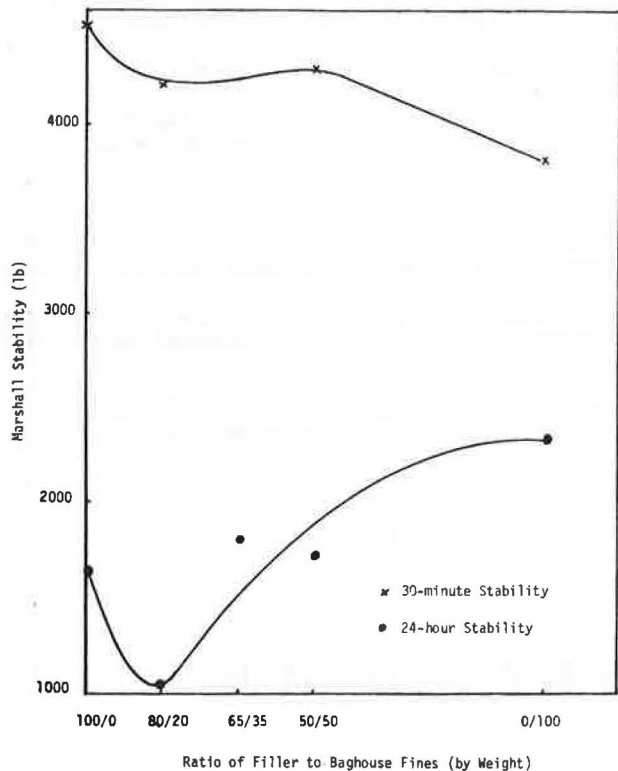


Figure 12. Marshall stability versus ratio F/BH-2 for AB-BH-2.

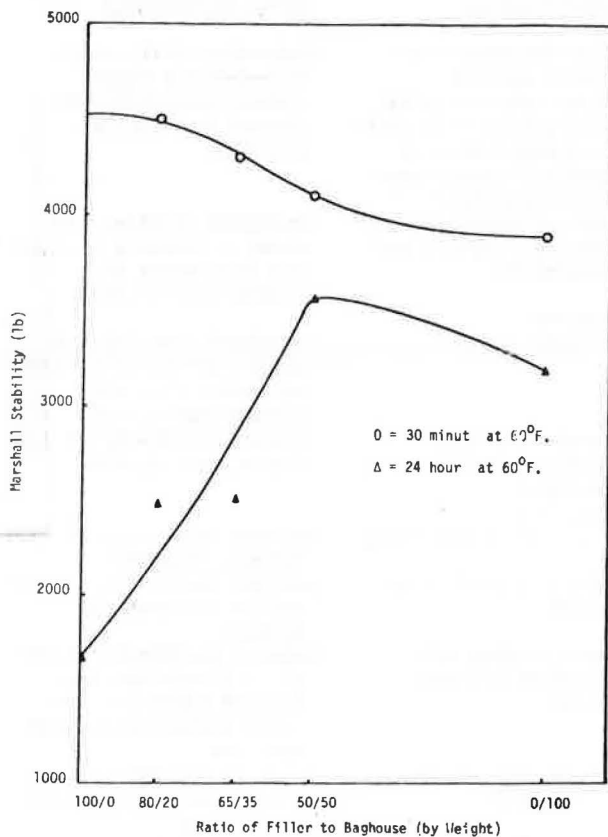


Figure 13. Optimum asphalt content versus ratio F/BH.

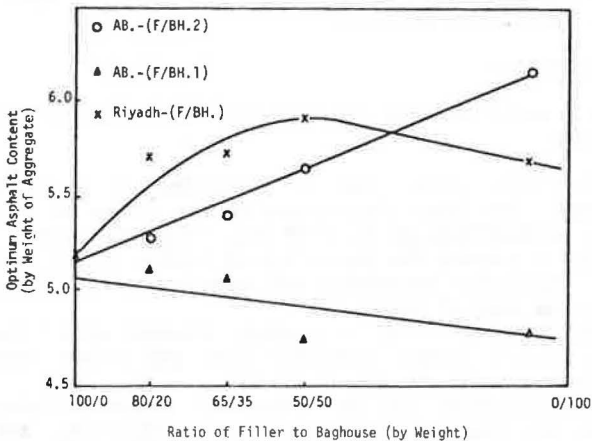
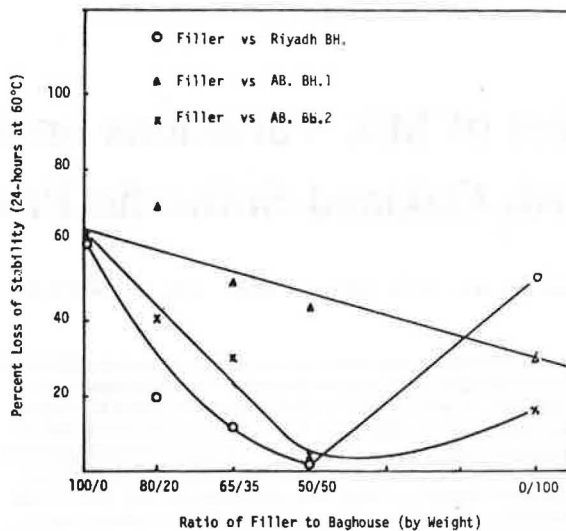


Figure 14. Stability loss versus ratio F/BH.



creased linearly by addition of AB.BH.2, which was due to the presence of carbon.

It was found that Riyadh baghouse fines can decrease the stability loss tremendously and can even prevent it. By increasing the amount of baghouse fines in the filler, stability loss decreased gradually to approximately 2.0 percent at a ratio of F/BH of 50/50. In fact, some of the specimens showed higher stability after 24 h than after 30 min. By adding more baghouse fines, stability loss increased. At a combined ratio of 50/50, an ideal filler is obtained that requires the minimum optimum asphalt content and gives the highest stability and lowest stability loss. Adding AB.BH.1 to the mix caused a linear reduction in stability loss from 62 to 30 percent. The same trend was noted for AB.BH.2, in which stability loss was decreased from 62 to 4 percent at a ratio of F/BH of 50/50 and then increased to 15 percent at a ratio of F/BH of 0/100.

Stability loss at a ratio of F/BH of 0/100 for AB.BH.2 was lower than that for AB.BH.1, which in turn was lower than that for Riyadh baghouse fines, as shown in Figure 14. This effect was caused by carbon fines, which exist in AB.BH.2 at 6 percent concentration, whereas the concentration is only 0.5 percent in AB.BH.1 and zero for Riyadh baghouse

fines. It is believed that carbon fines increase the resistance of asphalt to the effects of water and temperature variation.

CONCLUSIONS

The conclusions for this study are summarized as follows:

1. Filler affects the mix properties to a large extent. The lower percentages of the filler in the mix, as specified by the ASTM E11, should be used.
2. Decreasing the percentage of filler in the mix and increasing compaction and asphalt content will decrease loss of stability.
3. Baghouse fines, if properly blended with filler, should reduce stability loss and affect the optimum asphalt content of the mix.
4. Increasing the percentage of baghouse fines will increase the viscosity, shear strength, and

softening point of the mortar.

5. Presence of larger percentages of carbon in the baghouse fines will have a major effect on the performance of baghouse fines in the mix. This will decrease the stability loss of the mix and increase the optimum asphalt content.

6. Mixes that use approximately a 50/50 ratio of filler to baghouse fines are the optimum mixes.

REFERENCES

1. J.B. Collins. Dust Collection Systems for Asphalt Plants. Proc., Canadian Technical Asphalt Association, Vol. 7, 1962.
2. W. Schenk. The Impact of Baghouses on the Paving Industry. Proc., Canadian Technical Asphalt Association, Vol. 27, 1972.

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

Effect of Mix Variations on Asphalt Pavement Life: North Oakland-Sutherland Project

JEAN WALTER, R.G. HICKS, JOE P. MAHONEY, AND J.E. WILSON, JR.

An increase in construction and short-term pavement performance problems has been noted in the Pacific Northwest and throughout the United States during the past five years. Several reasons have been suggested by others to explain this sudden change, such as recent variations in asphalt properties and new developments in paving technology. By using the data and construction materials from projects built in 1978-1979, the Oregon Department of Transportation and Oregon State University initiated a laboratory study to determine the relationship between asphalt-concrete mix performance and mix compaction, asphalt content, and percent passing the 2-mm and 0.075-mm (Nos. 10 and 200) sieves. Conventional and dynamic tests were run on laboratory-compacted samples to determine mix stiffness, fatigue life, and permanent deformation characteristics. By using the fatigue data generated as an example, pay-adjustment factors were developed by comparing the performance of mixes prepared at the design optimum with that of mixes from specifications. It was found that fatigue life is primarily affected by the level of compaction of the mix. Test results indicate that there is an optimum asphalt content for fatigue and that gradation slightly affects fatigue life. The most critical pay-adjustment factors for fatigue are presented. Additional work is being completed to combine the fatigue and permanent deformation test results.

Several changes in highway materials and in asphalt paving technology have occurred in recent years. New asphalt sources have been developed that introduce changes in asphalt properties. New equipment has been developed that affects mixing (drum mixers, more efficient dust-collector systems), storage (mix storage silos), and compaction (vibratory compactors). In the same period, economic constraints have resulted in increased use of lower-quality aggregate. As a result, there has been an increase in construction or short-term performance problems throughout the Pacific Northwest (1). The impact of such changes on the mix properties is, however, difficult to evaluate. The main changes observed and their expected influence on the mix behavior are summarized below:

Change Observed

Asphalt:

Wide difference between asphalt temperature-viscosity curves from various suppliers, increased temperature susceptibility
Reduced compatibility between asphalt and aggregate

Aggregate:

Reduced quality

Single stockpile, elimination of plant screens

Equipment:

Use of dust collector

High mix production rate

Lower mixing and laydown temperatures

Use of vibratory compactors

Use of drum mixers

Impact on Pavement

Compaction difficulty, slow-setting mixes, reduced resistance to thermal and fatigue cracking

Increased raveling, reduced resistance to damage from water and freeze-thaw effects

Increased raveling, reduced resistance to damage from water and freeze-thaw effects
Reduced uniformity of gradation, segregation

Reduced uniformity of gradation, flushing
Reduced uniformity of gradation and asphalt content
Reduced uniformity of asphalt viscosity, increased moisture, reduced asphalt-aggregate adhesion

Breakage of aggregates, low compaction from improper use
Incomplete coating of aggregate

Figure 1. Flow chart of study.

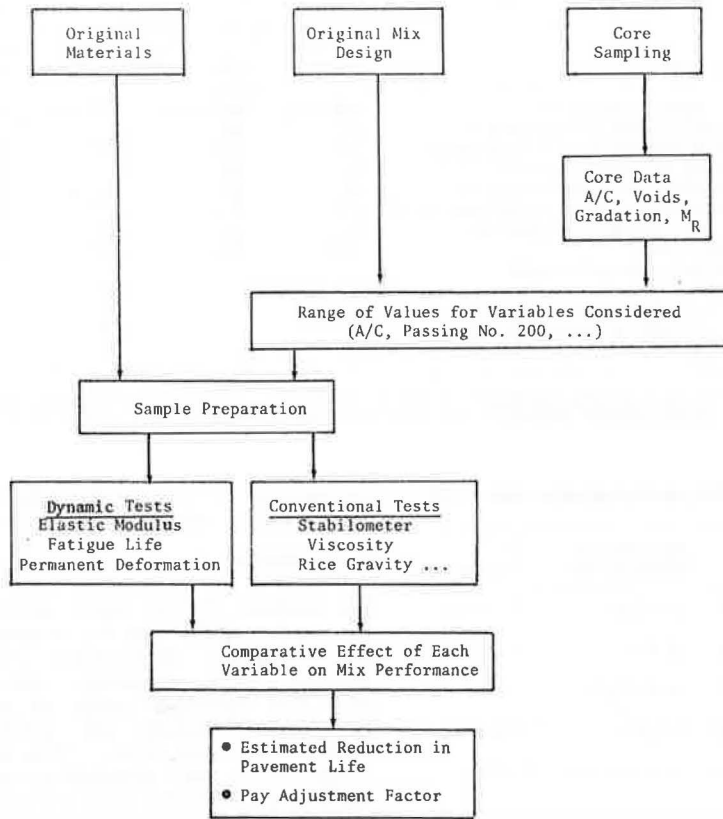
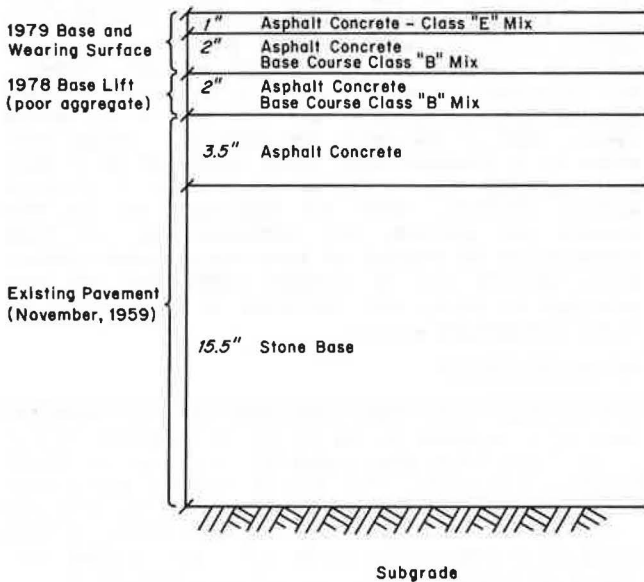


Figure 2. Pavement cross section.



Change Observed

Mix storage silos and belly dump hauling equipment

Impact on Pavement

Mix segregation from improper use

Three recently constructed paving projects in Oregon are being evaluated to develop an improved understanding of how mixture properties affect pavement performance. The paving project discussed in this paper is located on Interstate 5 between the North Oakland and Sutherlin Interchanges approxi-

mately 19 km (12 miles) north of Roseburg, Oregon. The overall project length is 5.1 km (3.2 miles).

The purpose of this paper is to obtain a better understanding of the causes of the pavement problems experienced in recent years and to develop relationships between mix performance and different mix variables. Such information should be useful in developing pay-adjustment factors for projects not complying fully with specifications. Figure 1 is a flowchart of the approach followed for the study of the North Oakland-Sutherlin Project.

PROJECT DESCRIPTION

Cross Section

The pavement cross section is shown in Figure 2. The original pavement, constructed in 1959, was composed of a 9-cm (3.5-in) asphalt concrete layer over a 39-cm (15.5-in) stone base layer. The new section of the pavement was built in 1978 and 1979. The 1978 lift was class B asphalt concrete base built with marginal-quality aggregate, which developed distress soon after construction. Patching of the first lift of base course was required prior to placement of the second 5-cm (2-in) lift of asphalt concrete in 1979. A 2.5-cm (1-in) class E open-graded friction course was placed as the final surface layer.

Mix Design

A summary of the mix design for the 1978 asphalt concrete base lift is presented in Table 1. To achieve an index of retained strength greater than 70 percent, 6.9 percent of an AR 8000 asphalt cement with 0.85 percent Pave Bond (antistrip agent) was recommended. The need for the antistrip agent was related to the quality of the aggregate. The aggregate soundness test (AASHTO T-104) indicated a per-

Table 1. Original mix design: results.

Property	Asphalt Content (AR 8000) (%)				
	5.5	6.0	6.5	7.0	7.5
Asphalt film thickness	Sufficient	Sufficient	Sufficient	Sufficient, thick	Thick
Stability value, first compaction	32	34	35	38	39
Bulk specific gravity, first compaction	2.23	2.25	2.27	2.29	2.31
Voids, first compaction (%)	11.2	9.6	8.1	6.5	4.9
Stability value, second compaction ^a	49	50	50	49	52
Bulk specific gravity, second compaction ^a	2.32	2.34	2.35	2.36	2.38
Voids, second compaction ^a (%)	7.6	6.0	4.9	3.7	2.1
Rice gravity	2.51	2.49	2.47	2.45	2.43
Index of retained strength					
AR 8000	11	—	20	—	45
AR 8000 + 1 percent Sucon	62	—	79	—	82
AR 8000 + 1 percent Pave Bond	64	—	81	—	84
AR 4000	16	—	25	—	53

^aAfter each sample has been tested for stability and specific gravity, it is recompact (by using the sample compaction effort) to simulate subsequent densification under traffic. The second compaction corresponds to the target density for field control.

Table 2. Summary of daily plant report, 1978 base lift, bituminous mix class B.

In-Place Mix Data	Avg Value	SD	Maximum and Minimum Values	Job-Mix Tolerance
Core bulk specific gravity (24 tests)	2.28	±0.07	2.15-2.36	2.16 min
Asphalt content (% of total mix) (72 tests)	7.17	±0.61	5.1-8.9	6.9 ± 0.5
Percent passing 2-mm sieve (72 tests)	25.66	±3.55	16.00-35.00	25 ± 4
Percent passing 0.075-mm sieve (72 tests)	6.72	±1.16	4.2-10.1	5.0 ± 2.0

Note: 1 mm = 0.04 in.

centage loss between 6.6 and 24.1 for the 10.1- to 6.4-mm (0.75- to 0.25-in) fraction and between 17.7 and 45.2 for the 6.4-mm minus fraction. The class B aggregate gradation for the asphalt concrete base course is given below (1 mm = 0.04 in):

Sieve Size (mm)	Percent Passing		
	Target Value	Mix Tolerance	Specification (broadband)
25	100	100	100
19	100	95-100	95-100
12.5	86	80-92	--
6.3	60	54-66	52-72
2	25	21-29	21-41
0.425	12	8-16	8-24
0.075	5.0	3.0-7.0	2-7

Construction Testing

Table 2 summarizes the results of field tests made during pavement construction. The variables considered are the mix bulk specific gravity, asphalt content, and percent passing the 2-mm and the 0.075-mm sieves (Nos. 10 and 200). It appears from the average field data that the asphalt content and the amount passing the 0.075-mm sieve were high and that the amount passing the 2-mm sieve and the specific gravity were within tolerance limits.

Table 2 also indicates that the mix variables ranged within a very wide band, which indicates quality-control variations during mixing (asphalt content, gradation) and during compaction (mix bulk specific gravity). Consequently, the mix quality was reduced. Oregon Department of Transportation (ODOT) field tests for production control of the mix indicated that the pavement deficiencies are mainly the result of an excess passing the 0.075-mm sieve and an excess amount of poor-quality aggregates. Based on past experience and specification requirements of various pavement structure components, ODOT

recommended a 15 percent price reduction in the total cost of the mix and asphalt cement.

TEST PROGRAM

The purpose of the tests performed at Oregon State University (OSU) was to determine the fatigue life and permanent deformation characteristics of the asphalt mix in question. All tests were performed over the selected range of variables on standard laboratory samples by using the repeated-load indirect-tensile test. The samples were prepared according to ODOT standard procedures (2). Only the modulus and fatigue data are reported in this paper.

A minimum of 16 samples were prepared for each condition. Eight samples were tested as compacted, and eight samples were tested after conditioning (3). Figure 3 is the flowchart for the test program. The principal variables studied included (a) percent level of compaction, (b) percent passing the 0.075-mm sieve, and (c) percent asphalt content. Two secondary variables were also studied: percent passing the 2-mm sieve and use of an antistripping agent. Each of the above variables was studied relative to a standard mix, which consisted of 6 percent passing the 0.075-mm sieve and 6 percent asphalt content. When the influence of the mix density was studied, the reference mix was that compacted at 96 percent of laboratory second compaction, whereas the 92 percent compaction mix was selected to study the influence of the amount of fines and asphalt content.

Resilient Modulus

All modulus tests were performed in the diametral mode (4). Samples 10 cm (4 in) in diameter by 6.3 cm (2.5 in) high were prepared by using the Hveem kneading compactor. One set of samples was tested for modulus in the as-compacted state, whereas the other set was tested after vacuum saturation followed by a freeze-thaw cycle (3). All samples were tested at a load duration of 0.1 s, a frequency of 1 Hz, and a test temperature of 21°C ± 2°C (70°F ± 2°F).

Fatigue Life

Fatigue life is characterized by the number of load applications required to cause failure of the sample. Attempts to relate the number of load applications to the sample state of stress and strain showed that a strong correlation exists between the tensile strain and the number of load applications, according to the following model (5-10):

$$N_f = K_1(\epsilon_t)^{K_2} \quad (1)$$

Figure 3. Experimental test program.

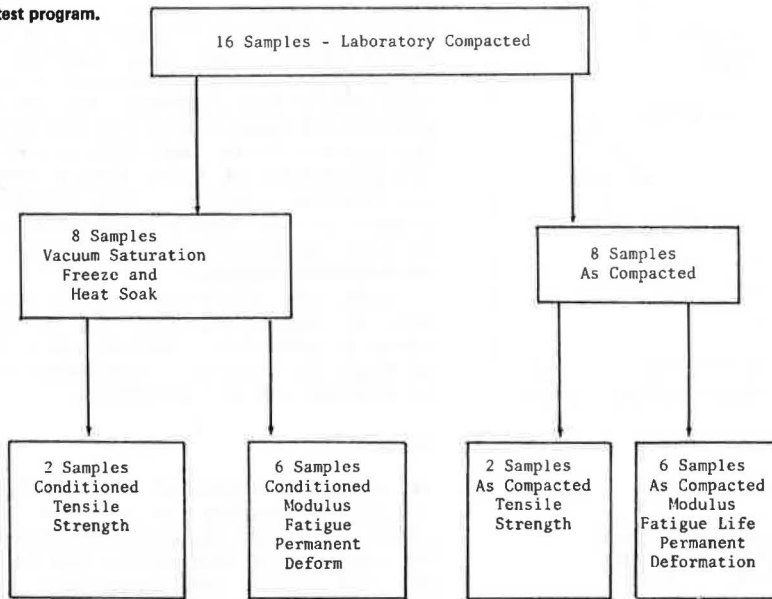
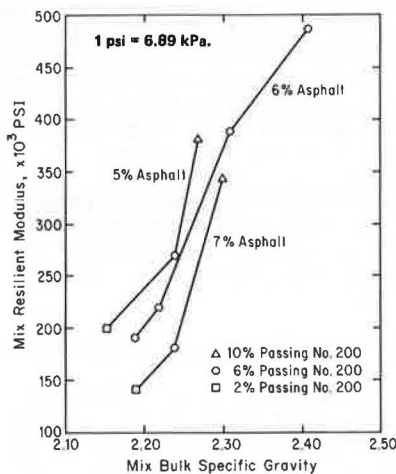


Table 3. Resilient-modulus data: as-compacted samples.

Level of Compaction (%)	Percent Passing 0.075-mm Sieve	Asphalt Content (%)	Resilient Modulus (psi)	Voids (%)	Bulk Specific Gravity
100	6	6	488 000	3.28	2.41
96	6	6	389 000	7.33	2.31
92	2	5	200 000	14.23	2.17
	2	7	143 000	10.46	2.19
	6	5	270 000	11.46	2.24
	6	6	220 000	10.85	2.22
	6	7	180 000	8.45	2.24
	10	5	381 000	10.47	2.27
	10	7	343 000	6.00	2.30
91	6	6	191 000	11.95	2.19

Note: 1 mm = 0.04 in.

Figure 4. Influence of bulk specific gravity on resilient modulus: as-compacted samples.



where

- N_f = number of load repetitions to failure,
- K_1, K_2 = regression constants, and
- ϵ_t = initial tensile strain.

The fatigue life of a specific mix is therefore defined by the constants K_1 and K_2 . For each set

of mix variables, six samples were tested at different values of the initial tensile strain in the diametral test mode (test conditions were identical to those used for the modulus test). The number of load repetitions to failure was then measured and recorded. The constants K_1 and K_2 were determined by using linear regression by the method of least squares. The tensile strain ϵ_t is calculated from the following equation (4):

$$\epsilon_t = [(0.03896 + 0.1185v)/(0.0673 + 0.2494v)] \tag{2}$$

where ϵ_t is the horizontal elastic tensile strain and v is Poisson's ratio. If we assume that Poisson's ratio is constant and equal to 0.35, Equation 2 becomes the following:

$$\epsilon_t = \Delta H \times 0.5203 \tag{3}$$

where H is the horizontal elastic tensile deformation in inches.

The number of load repetitions to fatigue failure was defined as that required to cause a vertical crack approximately 0.64 cm (0.25 in) wide in the sample. To stop the test at the specified level of deformation, a thin aluminum strip was attached to the sides of the samples along a plane perpendicular to that formed by the load platens. The aluminum strip was connected to a normally closed relay that controlled the dynamic-load system. As the sample deformed, the aluminum strip was stressed. When the sample deformation exceeded a certain level, the aluminum strip broke and opened the relay, which shut off the test. Proper calibration of the length of the aluminum strip caused the test to stop for a specific sample crack width.

TEST RESULTS

Resilient Modulus

The resilient-modulus data of the as-compacted samples are presented in Table 3. The influence of the bulk specific gravity on the mix resilient modulus, independent of the other variables, can be observed for the 6 percent asphalt content and the 6 percent passing the 0.075-mm sieve. Plotted in Figure 4, the relationship between modulus and bulk specific

Figure 5. Influence of amount of fines on resilient modulus: as-compacted samples.

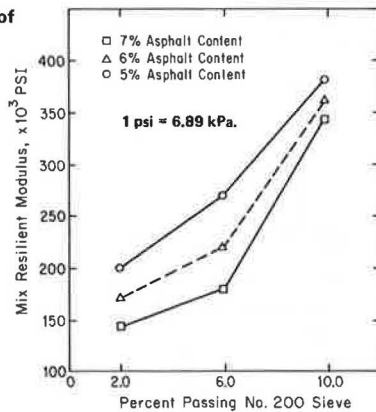


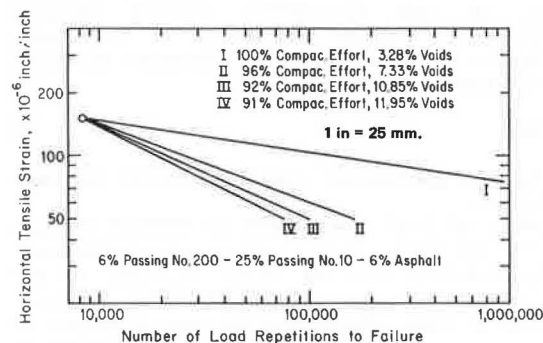
Table 4. Resilient-modulus data: conditioned samples.

Level of Compaction (%)	Percent Passing 0.075-mm Sieve	Asphalt Content (%)	Resilient Modulus (psi)	Retained Stiffness ^a (%)
100	6	6	435 000	89.1
96	6	6	214 000	55.0
92	2	5	93 000	46.5
	2	7	93 000	65.0
6	5	5	136 000	50.4
	6	6	126 000	57.3
	6	7	103 000	57.2
	10	5	176 000	46.2
	10	7	279 000	81.3
	91	6	6	109 000

Note: 1 mm = 0.04 in.

^a(Conditioned modulus/as-compacted modulus) x 100.

Figure 6. Influence of mix density on fatigue life: as-compacted samples.



gravity appears to be almost linear and slightly affected by changes in asphalt content and percent passing the 0.075-mm sieve. The nonlinearity of the curves joining the points of equal asphalt content indicates the interaction of the asphalt content and fines on the mix stiffness. The resilient modulus increases at a rate of approximately 1120 Pa (160 000 psi) for each 0.1 increase in bulk specific gravity (or 6.2 lb of density).

Figure 5 illustrates the relation between the mix resilient modulus and the amount of fines at a constant level of compaction and 5, 6, and 7 percent asphalt content. The rapid increase in stiffness for an increasing percentage of fines is particularly important at higher percentages of asphalt content. The resilient modulus increases at a rate of approximately 175 Pa (25 000 psi) for each 1 percent increase in the 0.075-mm material.

Modulus values of conditioned samples are pre-

sented in Table 4 together with their percentage of retained stiffness (compared with as-compacted sample moduli). These data indicate that higher retained strengths are obtained at high asphalt content and/or mix density. The relationship between conditioned modulus and the mix bulk specific gravity appears to be less affected by asphalt content and percentage of fines than it was in the case of as-compacted samples. The resilient modulus increases at approximately the same rate for increases in bulk specific gravity as that given for the as-compacted samples.

Also, mix stiffness appears relatively independent of asphalt content for a low percentage of fines (2 percent). However, at a higher percentage of fines (10 percent), increasing amounts of asphalt do increase the mix stiffness.

Fatigue

The effect of degree of compaction, asphalt content, and percent passing the 2-mm and 0.075-mm sieves on fatigue life can be estimated directly by plotting, for each set of conditions, mix tensile strain versus the number of repetitions to failure. The level-of-compaction fatigue curves for 6 percent asphalt content and 6 percent passing the 0.075-mm sieve are presented in Figure 6 for the as-compacted samples. The as-compacted mix shows a substantial decrease in fatigue life as the mix level of compaction drops from 100 to 91 percent (a similar trend was found for the conditioned samples). The influence of the asphalt content on the as-compacted samples is illustrated in Figure 7. As indicated, asphalt content for unconditioned samples has very little influence on the fatigue life. Conditioned samples were more sensitive to changes in mix asphalt content in that the fatigue life increased with increases in asphalt content.

The influence of the percent passing the 0.075-mm sieve on the mix fatigue life is shown in Figures 8 and 9 for 5 and 7 percent asphalt content. These figures show the importance of the percentage of fines in the mix on fatigue performance. The fatigue life increases with increasing percent passing the 0.075-mm sieve, independent of the asphalt content. Conditioning the samples emphasizes the importance of the fines and indicates that a mix with 10 percent passing the 0.075-mm sieve performs better in the fatigue mode than a mix with 6 percent passing the 0.075-mm sieve (the higher percentage of fines, in this case, also results in a lower amount of permanent deformation, however, is just the reverse. Figure 10 gives the effects of percent passing the 2-mm sieve. The as-compacted series clearly shows an optimum mix fatigue life for 25 percent passing the 2-mm sieve, as did the conditioned series.

DEVELOPMENT OF PAY-ADJUSTMENT FACTORS: FATIGUE BASIS

The testing program covered a wide range of mix variables. From this, it is possible to evaluate the reduction in pavement life when the design requirements are not satisfied. By using the mix that fulfills the design requirements as a reference mix, the fatigue life of mixes that do not meet specifications has been determined and compared with the standard mix fatigue life. The resulting ratios of fatigue lives have been used as an estimate of the corresponding pay factor.

The calculations were accomplished at three strain levels: 125, 100, and 50 $\mu\epsilon$. Table 5 presents the estimated reduction in pavement life when the design mix density is not achieved. The

reference mix is composed of 6 percent passing the 0.075-mm sieve, 25 percent passing the 2-mm sieve, and 6 percent asphalt content and is compacted at 96 percent of the laboratory second-compaction density. This standard mix is compared in Table 5 with mixes compacted at different levels (100, 92, and 91 percent compaction). The pay factors (defined as the ratio of the repetitions to failure for a mix when compared with the reference mix) shown in Table 5 indicate that the variations in mix density are extremely important for low strain values and less important for high strain values.

Table 6 shows pay factors for mixes with low and high asphalt contents. The reference mix is composed of 6 percent passing the 0.075-mm sieve, 25 percent passing the 2-mm sieve, and 6 percent asphalt and is compacted at 92 percent. The reference mix is fixed at 92 percent compaction because this is the minimum specified density required by ODOT. As the fatigue results indicate, the effects of a change in the asphalt content on fatigue life are less than that for density over the range of asphalt content studied.

The impact of the percentage of fines on fatigue

life is shown in Table 7 for mixes containing 5 percent asphalt and for mixes containing 7 percent asphalt. Compared at 100 $\mu\epsilon$, the pay factors from Table 7 are relatively close, which tends to corroborate the fact that ± 1 percent asphalt content does not make a substantial difference on the fatigue performance of the mix. However, the complex interaction between asphalt and fines substantially affects the fatigue life of the mix. Increasing the amount of fines from 2 to 10 percent increases the pay factor by approximately 3 points. This is due in part to the decrease in voids associated with the higher fines constant. However, when permanent deformation is considered, an increase in fines content will result in increased problems due to deformation.

Pay factors corresponding to different percentages passing the 2-mm sieve are presented in Table 8. Since compacted and conditioned results both indicate a reduction in pavement life when the percentage passing the 2-mm sieve is increased above optimum, results for 30 and 35 percent passing the 2-mm sieve are very similar.

Figure 7. Influence of asphalt content on fatigue life: as-compacted samples.

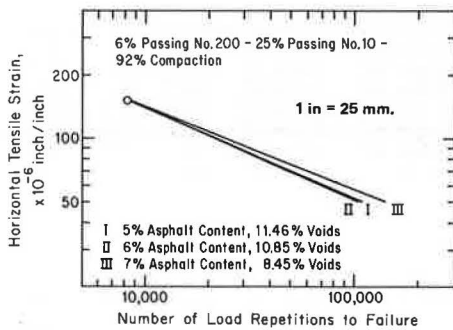


Figure 8. Influence of percent passing 0.075-mm sieve on fatigue life: as-compacted samples (5 percent asphalt content, 92 percent compaction).

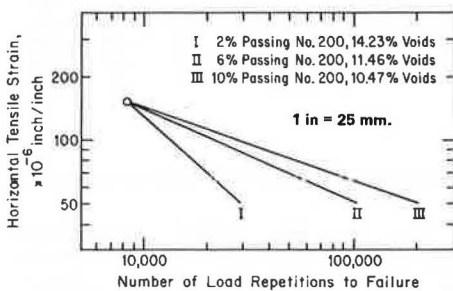


Figure 9. Influence of percent passing 0.075-mm sieve on fatigue life: as-compacted samples (7 percent asphalt content, 92 percent compaction).

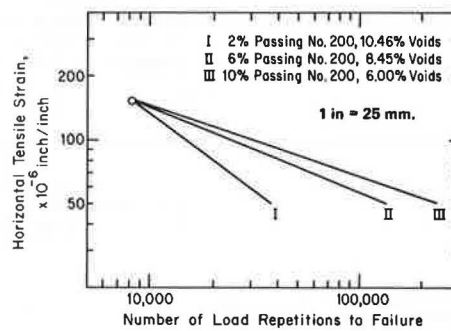


Figure 10. Influence of percent passing 2-mm sieve on fatigue life: as-compacted samples.

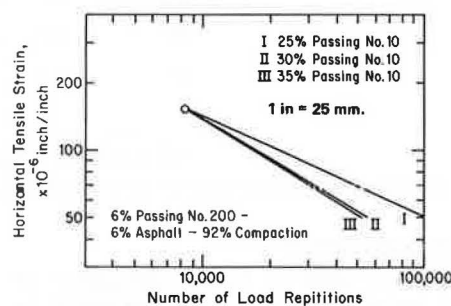


Table 5. Estimated pavement life and associated pay factors: four levels of mix density.

Level of Compaction (%)	Mix BSG	Test Condition ^a	Pavement Life			Pay Factor		
			Strain Level ($\mu\epsilon$)			Strain Level ($\mu\epsilon$)		
			50	100	125	50	100	125
Standard 96	2.31	B.C.	1.68×10^5	2.55×10^4	1.39×10^4	1.0	1.0	1.0
		A.C.	1.59×10^5	3.50×10^4	2.15×10^4	1.0	1.0	1.0
100	2.41	B.C.	1.37×10^7	1.29×10^5	2.86×10^4	8.5	5.06	2.06
		A.C.	6.80×10^5	7.75×10^4	3.97×10^4	3.91	2.21	1.84
92	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^3	0.601	0.812	0.892
		A.C.	1.34×10^5	3.11×10^4	1.95×10^4	0.842	0.890	0.906
91	2.19	B.C.	7.91×10^4	1.94×10^4	1.23×10^4	0.471	0.761	0.885
		A.C.	2.80×10^4	1.26×10^4	9.76×10^3	0.177	0.361	0.456

^aB.C. = before conditioning; A.C. = after conditioning.

Table 6. Estimated pavement life and associated pay factors: three levels of asphalt content.

Asphalt Content (%)	Mix BSG	Test Condition ^a	Pavement Life			Pay Factor		
			Strain Level ($\mu\epsilon$)			Strain Level ($\mu\epsilon$)		
			50	100	125	50	100	125
Standard 6	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^4	1.0	1.0	1.0
		A.C.	1.34×10^5	3.11×10^4	1.95×10^4	1.0	1.0	1.0
5	2.24	B.C.	1.07×10^5	2.13×10^4	1.27×10^4	1.06	1.03	1.02
		A.C.	8.14×10^4	2.31×10^4	1.54×10^4	0.610	0.740	0.788
7	2.24	B.C.	1.40×10^5	2.39×10^4	1.35×10^4	1.39	1.16	1.09
		A.C.	1.53×10^5	3.34×10^4	2.05×10^4	1.14	1.07	1.05

^aB.C. = before conditioning; A.C. = after conditioning.

Table 7. Estimated pavement life and associated pay factors for three levels of percent passing 0.075-mm sieve: 5 and 7 percent asphalt content.

Percent Passing 0.075-mm Sieve	Mix BSG	Test Condition ^a	Pavement Life			Pay Factor		
			Strain Level ($\mu\epsilon$)			Strain Level ($\mu\epsilon$)		
			50	100	125	50	100	125
5 Percent Asphalt Content								
Standard 6	2.24	B.C.	1.07×10^5	2.13×10^4	1.27×10^4	1	1	1
		A.C.	8.14×10^4	2.31×10^4	1.54×10^4	1	1	1
2	2.17	B.C.	2.90×10^4	1.31×10^4	1.01×10^4	0.272	0.612	0.794
		A.C.	1.08×10^4	7.16×10^3	6.27×10^3	0.133	0.311	0.493
10	2.27	B.C.	2.06×10^5	2.73×10^4	1.42×10^4	1.94	1.28	1.12
		A.C.	2.10×10^5	4.12×10^4	2.44×10^4	2.54	1.79	1.58
7 Percent Asphalt Content								
Standard 6	2.24	B.C.	1.40×10^5	2.39×10^4	1.35×10^4	1	1	1
		A.C.	1.53×10^5	3.34×10^4	2.05×10^4	1	1	1
2	2.19	B.C.	3.87×10^5	1.50×10^4	1.10×10^4	0.276	0.626	0.814
		A.C.	2.39×10^4	1.13×10^4	8.87×10^3	0.156	0.33	0.432
10	2.30	B.C.	3.94×10^5	4.66×10^4	2.34×10^4	2.81	1.95	1.73
		A.C.	4.38×10^5	6.24×10^4	3.33×10^4	2.87	1.87	1.62

Note: 1 mm = 0.04 in.

^aB.C. = before conditioning; A.C. = after conditioning.

Table 8. Estimated pavement life and associated pay factors: three levels of percent passing 2-mm sieve.

Percent Passing 2-mm Sieve	Mix BSG	Test Condition ^a	Pavement Life			Pay Factor		
			Strain Level ($\mu\epsilon$)			Strain Level ($\mu\epsilon$)		
			50	100	125	50	100	125
Standard 25	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^4	1.0	1.0	1.0
		A.C.	1.34×10^5	3.11×10^4	1.95×10^4	1.0	1.0	1.0
30	2.23	B.C.	5.50×10^4	1.67×10^4	1.14×10^4	0.55	0.81	0.92
		A.C.	2.23×10^4	1.09×10^4	8.67×10^3	0.17	0.35	0.45
35	2.21	B.C.	5.57×10^4	1.70×10^4	1.16×10^4	0.55	0.82	0.93
		A.C.	4.64×10^4	1.71×10^4	1.24×10^4	0.35	0.55	0.64

Note: 1 mm = 0.04 in.

^aB.C. = before conditioning; A.C. = after conditioning.

CONCLUSIONS AND RECOMMENDATIONS

Performance of the mix used in the construction of the North Oakland-Sutherland project was evaluated by dynamic testing of laboratory-compacted samples (11). Mix resilient modulus, fatigue life, and permanent deformation characteristics were determined for samples prepared within the following range of variables:

1. Mix level of compaction: 100, 96, 92, and 91 percent;
2. Asphalt content: 5, 6, and 7 percent;
3. Percentage of fines: 2, 6, and 10 percent; and
4. Percentage passing the 2-mm sieve: 25, 30, and 35 percent.

Conclusions

It was found that the mix level of compaction is the

dominant factor for all mix dynamic properties. Increasing the mix density increases the mix stiffness and fatigue life. High mix density substantially reduces the damaging action of water and other environmental factors. A 1 percent change in asphalt content from the design optimum did not change the fatigue life of the mix significantly, but a slight increase in fatigue life was noted when the asphalt content was increased to 7 percent. Fatigue life improved substantially when the amount of fines was increased to 10 percent. The improved fatigue performance due to the higher percentage of fines may be related to the fact that the primary evaluation was conducted at a low level of compaction. Thus, the higher percentage of fines resulted in a lower amount of air voids. Increasing the amount passing the 2-mm sieve decreased the mix fatigue life slightly.

Based on fatigue curves, pay factors have been developed to show variations in mix performance re-

sulting from changes in mix density, asphalt content, and percentage of fines. These data, shown in detail earlier, are summarized below. [The values presented were calculated for a mix tensile strain of 100 $\mu\epsilon$. Laboratory test results indicated that fatigue life was generally shorter than permanent-deformation life. Therefore, the permanent-deformation pay factors are not included in this summary. Only the conditioned data have been considered in this summary, since conditioned data are assumed to more closely duplicate a cured-pavement condition. Pay factors developed at 2 and 10 percent passing the 0.075-mm sieve are the average pay factors calculated at 5 and 7 percent asphalt. The results corroborate earlier remarks that (a) lowering the mix density decreases the fatigue life and (b) fatigue life improves with increasing asphalt content.]

<u>Variable</u>	<u>Pay Factor</u>
Level of compaction (%)	
96	1.0
100	2.21
92	0.89
91	0.36
Asphalt content (%)	
6	1.0
5	0.74
7	1.07
Percentage of fines	
6	1.0
2	0.32
10	1.83
Percent passing 2-mm sieve	
25	1.0
30	0.35
35	0.55

Recommendations

The pay reduction used for this project was 15 percent based on ODOT experience. The results of the fatigue test should allow for a more realistic determination of the pay-adjustment factor. The results also indicate that control of density (or voids) should receive the highest priority. Work is now under way to finalize pay-adjustment factors by using the approach reported in this paper and by considering both fatigue and permanent-deformation data.

ACKNOWLEDGMENT

The study described here was supported by the Federal Highway Administration. The Federal Highway Administration has not reviewed the findings pre-

sented. The contents of this report reflect my views and I am responsible for the facts and accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of either the Oregon Department of Transportation or the Federal Highway Administration.

REFERENCES

1. J.E. Wilson and R.G. Hicks. Evaluation of Construction and Short-Term Performance Problems for Asphalt Pavements in Oregon. Proc., AAPT, 1979, pp. 1-33.
2. Laboratory Manual of Test Procedures. Highway Division, Material and Research Section, Oregon Department of Transportation, Salem, March 1978.
3. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete. Univ. of Idaho, Moscow, U.I. Project 677-K297, Feb. 1979.
4. G. Gonzales, T.W. Kennedy, and J.N. Anagnos. Evaluation of the Resilient Elastic Characteristics of Asphalt Mixtures Using the Indirect Tensile Test. Center for Highway Research, Univ. of Texas, Austin, Res. Rept. 183-6, Nov. 1975.
5. P.S. Pell. Characterization of Fatigue Behavior. HRB, Special Rept. 140, 1973, pp. 49-64.
6. M.W. Witczak. Fatigue Subsystem Solution for Asphalt Concrete Airfield Pavements. HRB, Special Rept. 140, 1973, pp. 112-129.
7. J.H. Havens, R.C. Deen, and H.F. Southgate. Pavement Design Scheme. HRB, Special Rept. 140, 1973, pp. 130-142.
8. F. Finn, C.L. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah. Development of Pavement Structural Subsystems. NCHRP, Project 1-10B, Final Rept., Feb. 1977.
9. P.S. Pell and K.E. Cooper. The Effect of Testing and Mix Variables on the Fatigue Performance of Bituminous Materials. Proc., AAPT, Vol. 44, 1975.
10. J. Verstraeten. Moduli and Critical Strains in Repeated Bending of Bituminous Mixes--Application to Pavement Design. Proc., 3rd International Conference on Structural Design of Asphalt Pavements, London, 1972.
11. J. Walter and others. Evaluation of North Oakland-Sutherlin Project for Pay Adjustment Factors. Oregon Department of Transportation, Salem, Interim Rept., Sept. 1981.

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

Dune Sand-Aggregate Mixes and Dune Sand-Sulfur Mixes for Asphalt Concrete Pavements

M.N. FATANI AND H.A. SULTAN

Results are presented of a study to determine the feasibility of using dune sand in asphalt concrete pavement in hot, desertlike climates through the use of one-size crushed aggregates, dense-graded aggregates, and powdered sulfur in the sand-asphalt mixes. Engineering properties, including Marshall design parameters, compressive strength, tensile strength, modulus of rupture, and dynamic modulus of elasticity, of the various mixes are given and discussed. Results indicate that a mixture of dune sand and asphalt is weak, unstable, easily deformed under light loads, and therefore unacceptable for pavement construction in a desertlike environment. Introducing a one-size crushed gravel at various ratios improved the mix but not sufficiently for it to pass the required standards. Introducing a dense-graded aggregate to the mix raised its properties to the required standards; however, such improvement was only achieved when the ratio of dense aggregate to dune sand was at least 60 percent to 40 percent. The results also demonstrate that the use of powdered sulfur in the sand-asphalt mixes reduces the optimum asphalt content, considerably increases the qualities of the mix even under severe environmental conditions, and reduces the pavement thickness required to protect the subgrade from deflections and the surface layer from fatigue cracking. A tentative thickness design chart of a pavement for typical traffic and environmental conditions found in desertlike areas is also presented.

In Saudi Arabia an extensive road-building program is currently under way. During the second five-year plan (which ended in 1980), about 7460 miles of asphalt roads and 6200 miles of rural roads were completed (1). Although a good percentage of these roads and of others currently planned go through sand-dune areas, dune sand had not been used as a pavement material. Instead, good-quality aggregates, which are in scarce supply, are being imported from other localities at a considerably high cost. Therefore, it seems appropriate to consider the potential of large-scale use of dune sand as a pavement material to reduce cost and save the good-quality aggregates for other construction uses.

Dune sand is a natural material that exists in abundant quantities on every continent and under almost every climatic condition. Although the existence of dune sand is characteristic of desert areas, sand is also commonly found along the shores of seas, lakes, and large streams (2).

This paper presents some data from a comprehensive investigation aimed at determining the feasibility of using dune sand in asphalt concrete pavement through the use of one-size crushed aggregates, a dense-graded aggregate, and powdered sulfur in the sand-asphalt mixes (3).

MATERIALS USED

Dune Sand

The dune sand selected was obtained from the dunes area west of Yuma, Arizona. The sand is composed of subrounded to subangular grains with very fine texture. It is essentially a one-size material with a uniformity coefficient of about 2 and a specific gravity of 2.65. The grain-size distribution for this sand is included in Figure 1. Quartz is the major constituent of this material. Other minerals such as plagioclase feldspars, orthoclase feldspars, and micas are also present. It is classified as an A-3(o) material.

One-Size Crushed Gravel

This aggregate is primarily a mixture of crushed gravel and limestone and was obtained from the

Tanner Pit southeast of Tucson, Arizona. The maximum particle size is 3/4 in, with only 2 percent retained on a 3/8-in sieve, as shown in Figure 1. The specific gravity is 2.65, the Los Angeles abrasion loss (AASHTO T96-65) is 20 percent, and the sodium sulfate soundness test (AASHTO T104-65) gave a loss of 6 percent. More than 90 percent of this gravel had at least one mechanically fractured face.

Dense-Graded Aggregate

The dense-graded aggregate is a mixture of four materials: 15 percent 3/4-in crushed gravel, 25 percent 3/8-in crushed gravel, 53 percent Pantano Wash sand, and 7 percent fly ash.

1. Crushed gravel, 3/4-in: This is the one-size crushed gravel discussed above.

2. Crushed gravel, 3/8-in: This material, as shown in Figure 1, has a maximum size of 3/8 in with about 20 percent retained on a No. 4 sieve. The specific gravity is 2.63, the Los Angeles abrasion loss is 20 percent, and the sodium sulfate soundness test loss is 6 percent. More than 90 percent of this gravel had at least one mechanically fractured face.

3. Pantano Wash sand: This sand was obtained from the Pantano Wash, a dry river in the Tucson area. The unwashed sand had a sand equivalent (AASHTO T176-6) of about 31 and 8 percent passing a No. 200 sieve. The sand used in this investigation was washed thoroughly until all the fines were washed out. The clean sand gave a sand equivalent of 83. The grain-size distribution for the washed sand is given in Figure 1.

4. Fly ash: The fly ash (Navajo fly ash) used in this investigation was obtained from the Western Ash Company. Its major constituents include 52.7 percent silicon dioxide, 20.5 percent aluminum oxide, and 4.9 percent ferric oxide (4). The grain-size distribution is given in Figure 1.

These four materials were combined in the proportions indicated above to form a dense-graded aggregate that met the Asphalt Institute gradation limits (5, Specification IVb).

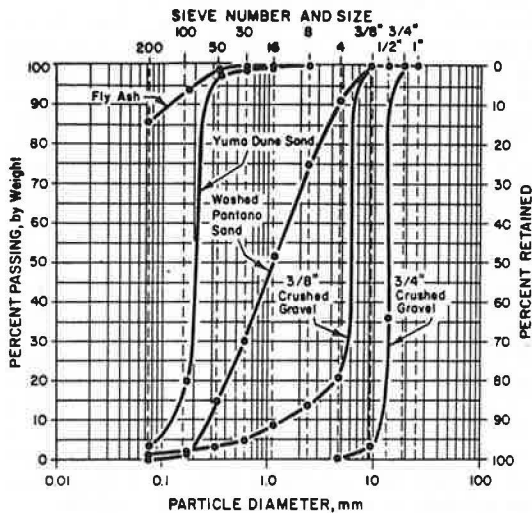
Asphalt

The asphalt cement used throughout this investigation was an AR-4000 (6-70 penetration), which is widely used for hot mixes in road construction in Arizona. Its physical properties are given elsewhere (1,3).

Sulfur

The sulfur used in preparing the sand-asphalt-sulfur mixes was a bright yellow elemental sulfur in a powdered form that had 99.5 percent purity. It is a commercial-grade sulfur known as Ortho Sulfur and manufactured by the Chevron Ortho-Division Company. Additional data on this sulfur are given elsewhere (3).

Figure 1. Grain-size distribution curves for aggregates used.



SAMPLE PREPARATION

Mixes of Dune Sand, Aggregate, and Asphalt

The dune sand, one-size crushed gravel, and components of the dense-graded aggregate were dried to a constant weight at 230°F. The desired portions of these materials for each type of mix were combined, mixed dry, and then placed in a forced-draft oven at 300 ± 5°F for at least 10 h. The asphalt was heated to 250 ± 5°F in an electric oven for not more than 1 h. The heated aggregate mix (sand and aggregate) was dumped into a preheated mixing bowl, a crater was formed in the aggregate, and the heated asphalt was weighed into the crater. Mixing was done with a Hobart C-10 mixer at medium speed for 90 s. This speed and mixing time produced a uniformly coated aggregate-asphalt mix. Next, the mix was placed in an oven at 250 ± 5°F until it was ready for compaction.

The first combination of sand, aggregate, and asphalt (Test Series A) consisted of dune sand, 3/4-in one-size crushed gravel in various proportions, and asphalt. The proportions of sand to one-size crushed gravel were 100/0, 90/10, 80/20, 70/30, and 60/40.

The second combination (Test Series B) consisted of dune sand, the dense-graded aggregates in various proportions, and asphalt. The proportions of dune sand to dense-graded aggregates were 100/0, 80/20, 60/40, 50/50, 40/60, 20/80, and 0/100.

Mixes of Dune Sand, Asphalt, and Sulfur

Sufficient sand was weighed and placed in a forced-draft oven at 350 ± 5°F overnight. Asphalt was heated to 300 ± 5°F. Sufficient powdered sulfur was weighed and maintained at a room temperature of 72 ± 2°F. The order of mixing the three components resulted in either a sulfur-coated mix of dune sand and asphalt (S-A) or an asphalt-coated mix of dune sand and sulfur (A-S).

For the (S-A) mix, the preheated sand and the sulfur (at room temperature) were placed in a preheated mixing bowl and mixed at a medium speed in the Hobart C-10 mixer for about 60 s. The mixture was then hand mixed with a trowel four or five times, which took about 10 s. A crater was formed in the mix and the required amount of preheated asphalt was weighed into the sulfur-coated sand mix.

Additional mixing at medium speed for about 60 s was required to coat the particles uniformly with the asphalt. This combination was designated Test Series C.

For the (A-S) mix, the heated asphalt was added to the preheated sand and the two components were mixed together in the preheated bowl of the Hobart C-10 mixer for 60 s. The sulfur (at room temperature) was then added and the entire batch was remixed for an additional 60 s. This combination was designated Test Series D.

After mixing, both mixtures were placed in an oven at 280 ± 5°F until compaction.

TEST RESULTS AND DISCUSSION

Test Series A

The Marshall test procedure for evaluating the performance of hot asphalt concrete mixes (ASTM.D1559) was used to evaluate the properties of these mixes. The Marshall test was performed on specimens prepared for the medium-traffic category (50 blows per end). The Marshall design criteria (6) were used to evaluate the results obtained.

The Marshall stability results for these mixes are given in Figure 2, and they indicate that with an asphalt content up to 7 percent by total weight, the Marshall stability generally increases as the ratio of dune sand to crushed gravel decreases. The highest stability value obtained was about 200 lbf for the 60/40 mix. This stability value is well below the minimum 500-lbf value required for roads in the medium-traffic category (6), and therefore no further testing was conducted on the Test Series A combinations. The test results also indicated that increasing the amount of crushed gravel increased the unit weight from 112 to 130 lb/ft³, reduced the Marshall flow from 25 to 6, reduced the percentage of air voids from 25 to 12, decreased the percentage of voids in mineral aggregate (VMA) from 37 to 26, and increased the percentage of filled voids (3).

Test Series B

The Marshall test procedures were also used to evaluate the properties of these mixes. Specimens were compacted with 50 blows per end (medium-traffic category).

The Marshall stability results for these mixes are given in Figure 3; they indicate that as the ratio of dune sand to dense-graded aggregate decreased (less sand and more aggregate), the Marshall stability increased slightly up to a ratio of 50/50 and then increased considerably as the amount of sand decreased further. The combination of dune sand and asphalt (100/0) gave a stability value of 82 lbf at an optimum asphalt content of 6.5 percent. The combination of dense-graded aggregate and asphalt with no dune sand (0/100) gave a maximum stability value of 1860 lbf at an optimum asphalt content of 5.4 percent.

To meet the stability requirement for medium-traffic conditions (500 lbf minimum) the ratio of dune sand to dense-graded aggregate should not be lower than 40/60 (i.e., a minimum of 60 percent dense-graded aggregate) at the corresponding optimum asphalt content. Although a mix of this ratio met the minimum required stability and flow values, it failed to meet the durability requirements such as percentage of air voids and percentage of filled voids (3). Furthermore, such a mix would not be economical due to the large percentage of imported dense-graded aggregates. Accordingly, no further testing was conducted on Test Series B.

Test Series C and D

All specimens used in these series were from mixes that had an asphalt content of 5 percent by total weight. This was based on various reported results on mixes of sand, asphalt, and sulfur (7-9). The sulfur content in the mixes varied from zero to 20 percent by total weight.

For the Marshall stability test on these mixes, each specimen was compacted by applying 10 blows to each end of the specimen. It was decided this would be the optimum compaction effort after a series of test specimens (80 percent sand, 5 percent asphalt,

and 15 percent sulfur) had been compacted by using 2, 5, 10, 15, 25, and 50 blows per face (3).

The Marshall stability results for the (S-A) mixes and the (A-S) mixes are given in Figure 4 with data for a similar mix that used a binder of molten sulfur and asphalt added to the same dune sand in a one-wet-mixing process (7). The results indicate that both the (S-A) and (A-S) mixes have their maximum Marshall stability values at 15 percent powdered sulfur. The maximum Marshall stability value for the (A-S) mix was 6500 lbf and that for the (S-A) mix was 5983 lbf. A sulfur content greater than 15 percent reduced the stability of both mixes.

The sharp increase in Marshall stability from a weak, unstable sand and asphalt mix (82 lbf) to a strong, stable mix of sand, asphalt, and sulfur is attributed to the filling of the voids by the recrystallized sulfur. The sulfur acts as a filler in addition to bonding the sand particles and thus increasing their interlocking. The (S-A) mix gave a slightly lower stability value because the particle-to-particle contact was asphalt, a viscous material, rather than sulfur, a solid material below 235°F.

Aboaziza (7) used a premixed blend of molten sulfur and asphalt mixed in a high-shear mixer at 300°F as the binder, and he mixed it with the dune sand in a one-wet cycle. Similar results to those shown in Figure 4 were reported but with slightly lower Marshall stability values.

Figure 5 presents the effect of increasing the percentage of sulfur in the mix on its unit weight. The increase of sulfur between 0 and 15 percent significantly increases the unit weight. Sulfur additions beyond 15 percent do not increase the unit weight very much, if at all. The main reason for the increase in unit weight is the high specific gravity of sulfur (about 2.0), which moves in to fill the voids between the sand particles. It appears that at sulfur additions above 15 percent the sulfur tends to start displacing equal volumes of sand, the specific gravity of which is 2.65.

The effect of sulfur addition on the Marshall flow is given in Figure 6, which indicates sharp reductions in the flow as the percentage of sulfur increases. This reduction in flow is attributed to

Figure 2. Relation between Marshall stability and ratios of dune sand to one-size crushed gravel (Test Series A).

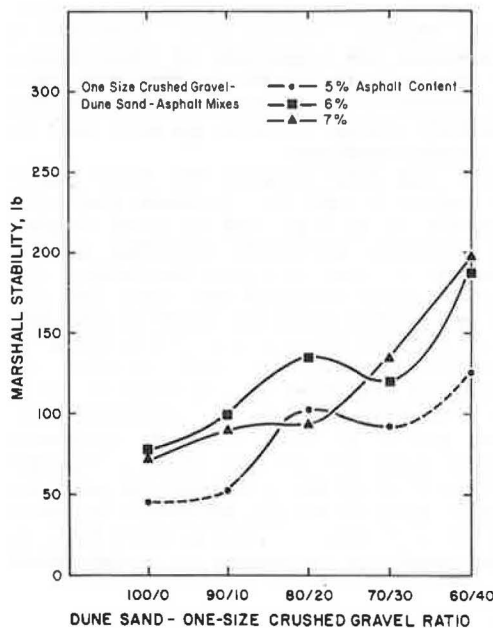


Figure 3. Relation between Marshall stability and ratios of dune sand to dense-graded aggregate (Test Series B).

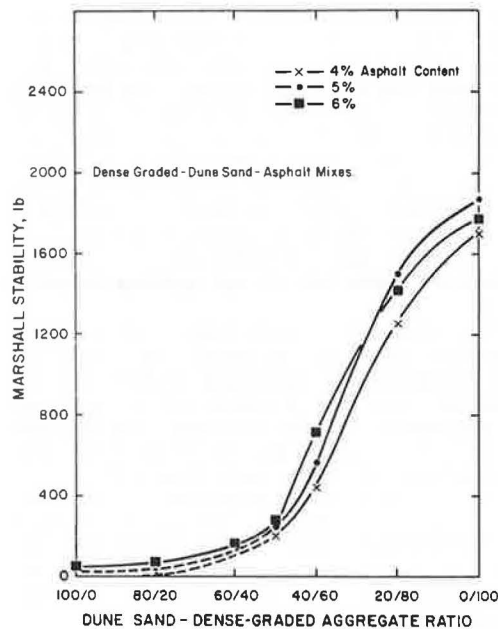
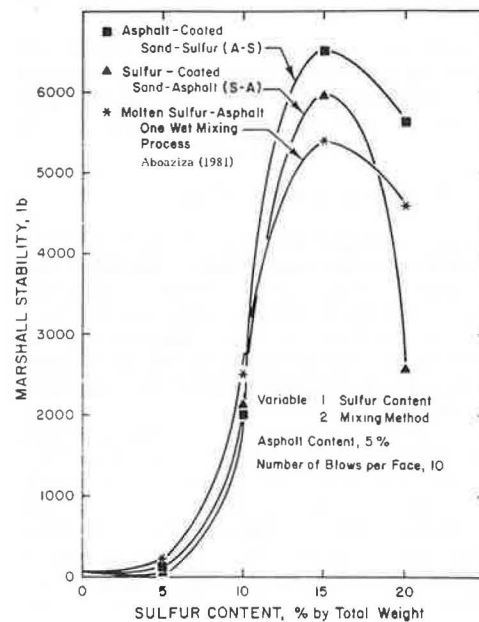


Figure 4. Relation between Marshall stability and sulfur content in mixtures of dune sand, asphalt, and sulfur.



the brittle nature of the recrystallized sulfur and the increased unit weight of the mix.

Figure 7 shows the effect of the sulfur addition on the percentage of air voids in the mix, which indicates sharp reductions of the air voids and results in more stable mixes. The mix of dune sand and asphalt without sulfur has a very high amount of air voids due to the uniformity of the sand-grain size. When sulfur is added to the mix, it fills the voids and results in a denser mix on recrystallization. It may be pointed out that although the percentage of air voids in the mixes exceeds the recommended range for the Marshall design criteria (6), it had been shown that the permeability of mixtures

of sand, asphalt, and sulfur is much lower than that of conventional asphalt concrete mixes for a given content of air voids (10). Burgess and Deme (10) reported that at 15 percent content of air voids, sand-asphalt-sulfur mixes were considered impervious with a permeability coefficient of about 1.0×10^{-8} cm². Conventional dense-graded asphalt concrete mixes reached similar permeability values at about 4-5 percent content of air voids (10).

Similar results were obtained by Aboaziza (7) by using molten sulfur. These results are also given in Figures 5 through 7 for purposes of comparison.

Based on the above test results, the (A-S) mixes in Test Series D appeared to give the overall best performance for the mixes of sand, asphalt, and sulfur. The (A-S) mix with dune sand, asphalt, and sulfur contents of 80, 5, and 15 percent, respectively, was therefore considered for further testing. Additional tests included tensile strength, compressive strength, soaked compressive strength, flexural strength, and dynamic modulus of elasticity.

Figure 5. Relation between unit weight and sulfur content of mixes of dune sand, asphalt, and sulfur.

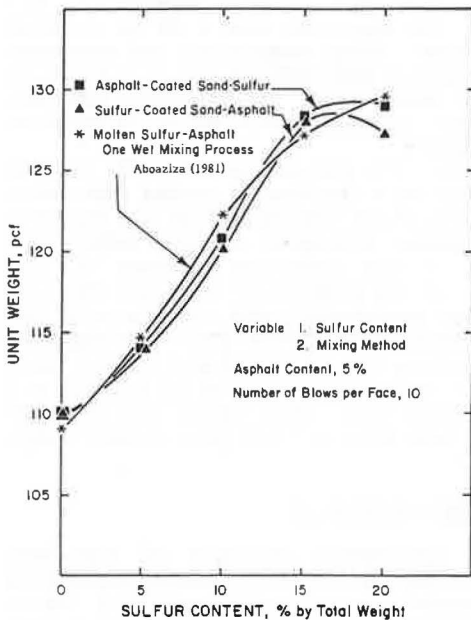
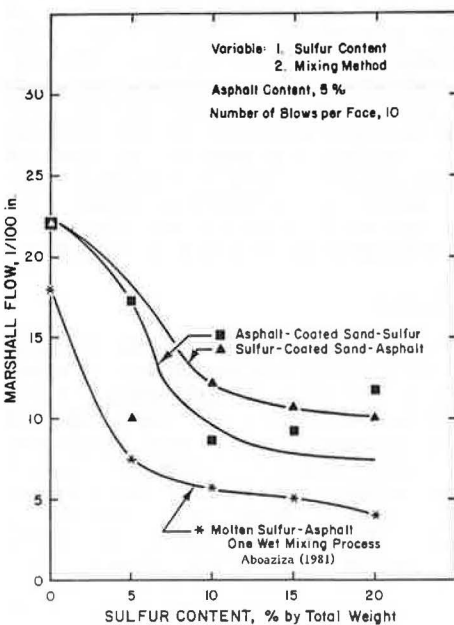


Figure 6. Relation between Marshall flow and sulfur content of mixes of dune sand, asphalt, and sulfur.



Tensile-Strength Test

The static double-punch test (11) was proposed as a simple indirect tension test for determining the tensile strength of concrete. Fang and Chen (12) developed, both theoretically and experimentally, the applications of the double-punch test to cohesive soils. Jimenez (13) extended the use of the double-punch method to test asphaltic mixtures for tensile strength, indicating its better repeatability than the split-cylinder (Brazilian) test.

The static double-punch test is conducted by using two steel disks (punches) centered on both flat ends of a cylindrical specimen. The vertical load is then applied slowly on the punches until the specimen reaches failure. The tensile strength of the specimen is calculated from the maximum load by a simple equation based on the theory of perfect plasticity (12).

In this investigation, specimens for the static double-punch test were 4 in in diameter and 2.5 in high. The mixes of sand, asphalt, and sulfur were compacted by the Marshall compactor (10 blows per

Figure 7. Relation between percentage of air voids and sulfur content of mixes of dune sand, asphalt, and sulfur.

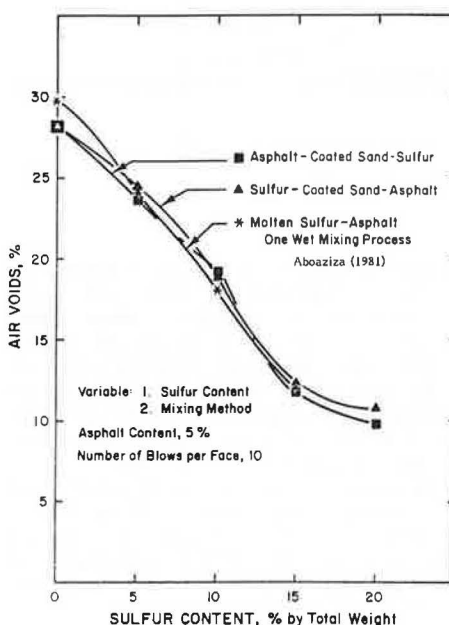


Figure 8. Effect of temperature on tensile strength of various mixes.

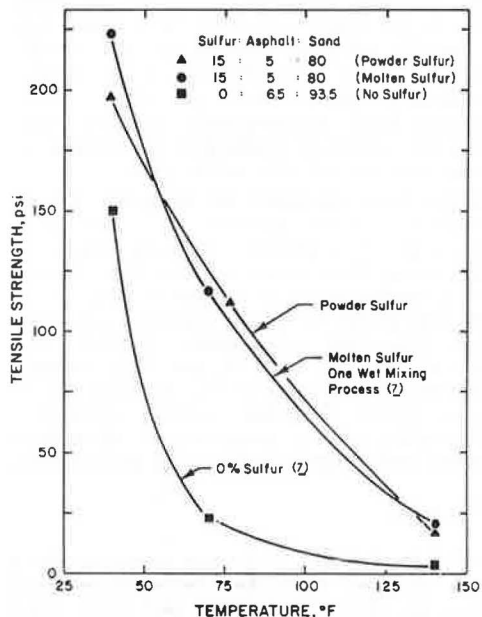
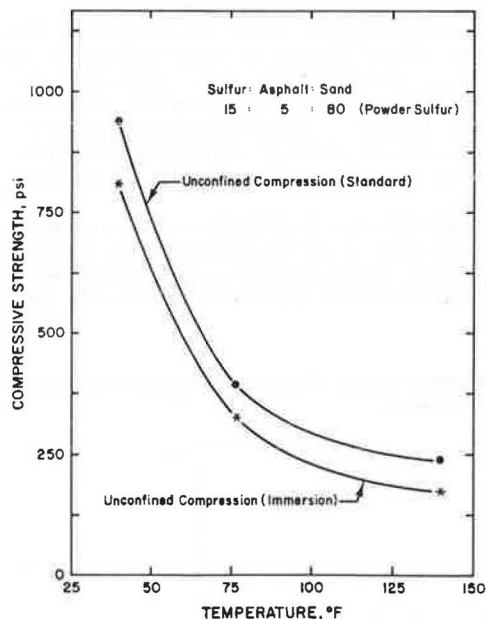


Figure 9. Effect of temperature on compressive and immersion compressive strength of mixes of dune sand, asphalt, and sulfur.



end). Specimens were tested in an Instron Universal Testing Machine at a deformation rate of 0.05 in/min. After compaction, the specimens were cured at various temperatures for 24 h prior to being tested. The curing (and testing) temperatures of $40 \pm 5^\circ\text{F}$, $77 \pm 5^\circ\text{F}$, and $140 \pm 5^\circ\text{F}$ were selected to represent the average low temperature, the average mean temperature, and the average high temperature in a desert environment, respectively (3,14).

Tensile-strength data for these (A-S) mixes are given in Figure 8 along with strength data for a sand-asphalt mix without sulfur and data for a mix of sand, asphalt, and molten sulfur obtained by Aboaziza (7). The results indicate significant improvement in the tensile strength of the sand-as-

phalt mix on addition of 15 percent sulfur in either molten or powdered form. The mix without sulfur had very low tensile-strength values at temperatures above 70°F . Similar tensile-strength values were obtained for the powdered-sulfur (A-S) mix and the molten-sulfur (one-wet-cycle) mix, with significant reductions in strength values as the curing (and testing) temperature increased from 40°F to 140°F . This reduction in tensile strength is attributed to the fact that the binder, and thus the mix, becomes softer and more flexible, which results in a progressive decrease in tensile strength as temperature increases.

Unconfined-Compression Test

The compressive strength of the (A-S) mix was determined according to the ASTM D1074 (AASHTO T167) test procedure. The specimens were compacted statically to their respective unit weights obtained in the Marshall test. The specimens were 4 in in diameter and 4 in in height. After compaction, the specimens were cured at various temperatures for 24 h prior to testing. After being cured, the specimens were immediately tested in compression at a deformation rate of 0.05 in/min.

Figure 9 shows the compressive-strength results for the (A-S) mix as a function of curing (and testing) temperatures, which indicates that the compressive-strength values decreased as the temperature increased due to the softening effects of the binder. It may also be pointed out that the addition of sulfur to the sand-asphalt mix significantly increased the compressive strength (1). The compressive-strength values for the sand-asphalt mix were 950, 180, and 5 psi at temperatures of 40, 77, and 140°F , respectively (1); these values are compared with values of 940, 370, and 235 psi, respectively, for the (A-S) mix.

Immersion Compressive Strength

The immersion compressive strength of specimens similar to those tested for the standard compressive strength was tested after an additional 24-h immersion in water at the same temperatures at which they were cured. The immersion compression test results at the three temperature levels, shown in Figure 9, are lower than the respective standard compressive-strength values.

If we take the ratio of the immersed compressive strength to the respective standard compressive strength as an index for retained strength, the results indicate a minimum retained strength of about 73 percent for the temperature range tested. The highest index of retained strength for the sand-asphalt mix (without sulfur) was reported by Aboaziza (7) to be only 45 percent. This demonstrates that the addition of sulfur in sand-asphalt mixes significantly increases their strength and durability and hence their resistance to stripping or debonding.

Flexural Strength Test

The modulus of rupture, which measures the flexural strength of the (A-S) mixes, was determined by using simple beams with third-point loading tests. These tests were conducted according to the ASTM C78 (AASHTO T97) test procedure. The beam specimens, $3 \times 3 \times 11.25$ in, were statically compacted to the corresponding unit weights obtained in the Marshall test. After compaction, the beams were cured at the various temperatures for 24 h. After being cured, the beams were tested at a deformation rate of 0.05 in/min.

Flexural strength data for these (A-S) mixes are

Figure 10. Effect of temperature on flexural strength of various mixes.

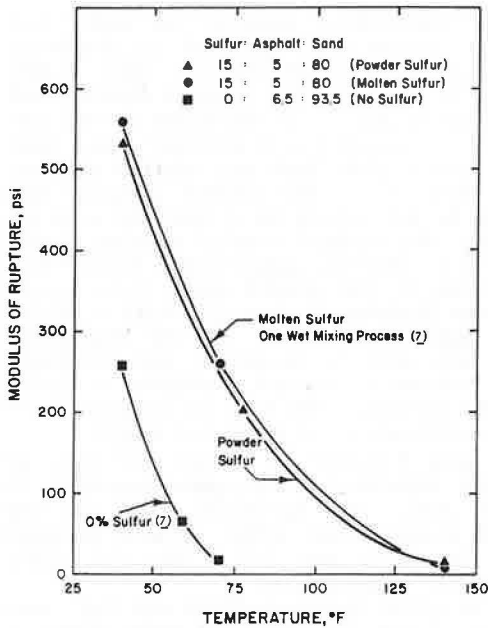


Figure 11. Effect of temperature on dynamic modulus of elasticity of various mixes.

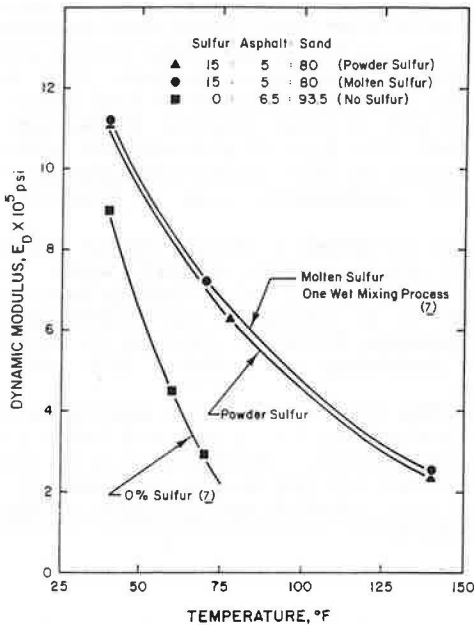


Table 1. Comparison of dynamic moduli of elasticity obtained by different methods.

Method	Aggregate	Asphalt (%)	Sulfur (%)	E _D (psi x 10 ⁵)			Reference
				40° F	70° F	100° F	
Schmidt	Beach sand	6	13.5 ^a	8.9	4.4	2.3	(16)
Schmidt	Beach sand	4	13.5 ^a	10.5	11.9	3.8	(16)
Deflectometer	Dune sand	6	0	10.0	1.5	0.3	(17)
Double punch	Dune sand	6.5	0	9.0	3.0	0.0	(7)
Double punch	Dune sand	5	15 ^a	11.2	7.1	4.6	(7)
Double punch	Dune sand	5	15 ^b	11.1	7.0	4.0	(3)
Typical values	Dense mix						(18)
Mean			0	16.0	5.0	1.0	
Range			0	9-27	4-9	0.7-2.2	

^aMolten sulfur. ^bPowdered sulfur.

given in Figure 10 along with strength data for a sand-asphalt mix without sulfur and data for a mix of sand, asphalt, and molten sulfur obtained by Aboaziza (7). The results indicate significant improvement in the flexural strength on addition of sulfur, in either powdered or molten form, to the sand-asphalt mix. It may be pointed out that at 77°F, the mix without sulfur had no flexural strength and could not support its own weight, whereas with 15 percent sulfur, the mixes of sand, asphalt, and sulfur had a flexural strength of about 200 psi at that temperature.

Dynamic Modulus of Elasticity

The dynamic or resilient modulus of elasticity is considered one of the most important characteristics of a pavement material. Use of the elastic theory (multilayered systems) is essentially dependent on application of this modulus. The dynamic modulus of elasticity is defined as the ratio of applied stress to the recoverable strain as obtained by dynamic measurements. In this investigation, the dynamic modulus of elasticity for the (A-S) mixes was obtained by using the dynamic double-punch test (15).

Jimenez (15) extended the use of the double-punch test to determine the dynamic modulus E_D of asphaltic concrete. Fatani (3), Aboaziza (7), and Jimenez (15) have discussed this procedure and the theory used to formulate the equation for calculating E_D in more detail.

The test data for dynamic modulus E_D-values for the (A-S) mixes are given in Figure 11 along with data for a sand-asphalt mix without sulfur and data for a mix of sand, asphalt, and molten sulfur obtained by Aboaziza (7). The data indicate the superiority of the mixes of asphalt, sand, and sulfur over the sand-asphalt mixes (without sulfur), particularly at high temperatures. All mixes indicate a reduction in E_D-values with increases in temperature.

Table 1 shows a comparison between the values of the dynamic modulus of elasticity at the three temperature ranges reported here and those from other investigations (7, 16-18) by using different techniques with or without sulfur. It can be noted that the dynamic modulus results from the double-punch test are in general agreement with those obtained by using the Schmidt method (16), especially at the low testing temperature. The weak, unstable sand-asphalt mix gave E_D-values either below or at the lower range of the typical values of dynamic modulus obtained for a dense-graded aggregate-asphalt mix along the entire temperature range under consideration (18).

At low temperatures, the addition of sulfur to sand-asphalt mixes improved the E_D-values to the middle ranges of E_D for the dense-graded aggregate-asphalt mix. The improvement approached the

higher ranges of E_D at intermediate temperatures, and it exceeded the given ranges at the high temperatures (which are critical for arid and semiarid climates).

Figure 12. Design chart (18 000-lbf single-axle load) for mix of dune sand, asphalt, and sulfur.

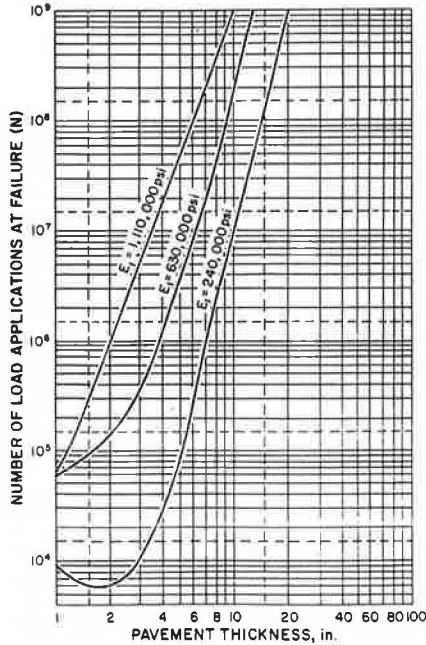
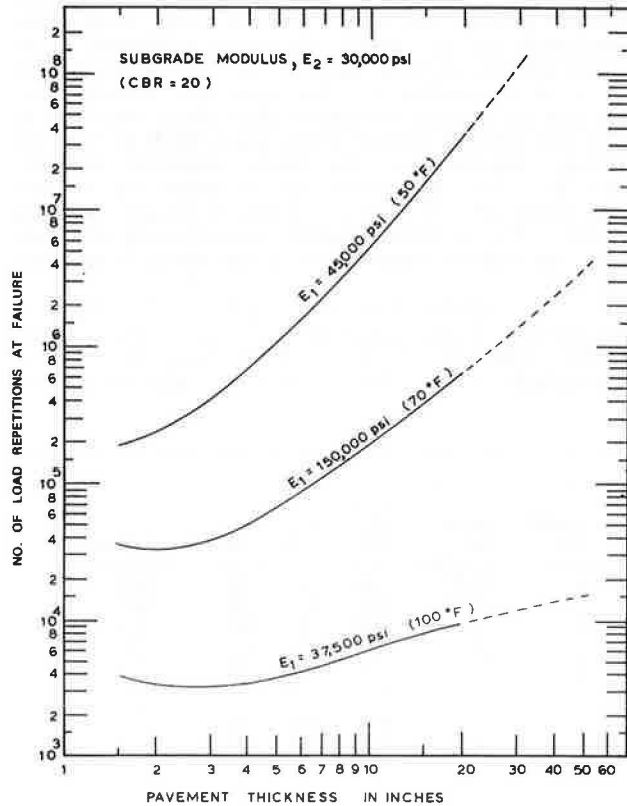


Figure 13. Design chart (18 000-lbf single-axle load) for mix of dune sand and asphalt.



PRELIMINARY THICKNESS DESIGN

A preliminary theoretical analysis by using the elastic-layered system was conducted to determine the thickness requirements for selected mixes of asphalt, sand, and sulfur under typical loading and subgrade conditions (3). The thickness design requirements were evaluated for a 9000-lbf wheel load (18-kip axle load); this load was applied by dual wheels (each 4500 lbf). The contact pressure was assumed to be 80 psi acting on a circular area of 4.23-in radius. The center-to-center spacing of the wheels in the dual axle was also assumed to be three times the radius of the loaded area, i.e., 12.69 in. The Chevron Computer Program (19) with a slight modification that permits the use of dual-wheel loading (17) was used to calculate the developed stresses and strains at various strategic locations in the pavement section. The computer program listing and description are given elsewhere (3,17).

Based on data reported by others (17,20), an average dynamic modulus for dune-sand subgrades is about 3×10^4 psi. The E_D -values used for the mixes of sand, asphalt, and sulfur in this analysis at 40, 77, and 140°F are 1.11×10^6 , 6.3×10^5 , and 2.4×10^5 psi, respectively.

Based on the above design criteria and the elastic properties of the dune sand and (A-S) mixes, a set of thickness design charts was developed (3). The outline of the development procedure for the charts is detailed elsewhere (3). Figure 12 presents a thickness design chart for the (A-S) mixes at these three effective temperatures with a subgrade modulus of 3×10^4 psi [California bearing ratio (CBR) of 20]. As shown, the design curves are steep and closely spaced, which indicates that a great increase in the expected number of load applications requires only a small increase in pavement thickness and that this type of pavement can withstand a great number of load repetitions with reasonable thickness requirements, even at high temperatures.

AlSalloum (17) presented a similar design chart for sand-asphalt mixes (without sulfur), which is reproduced here as Figure 13. If we compare Figures 12 and 13, there is an indication that significant reductions in pavement thickness required to withstand the same conditions are achieved with additions of sulfur.

It may be pointed out that the above analysis and chart are of a preliminary nature and were based on the assumed design criteria. Therefore, they should be used as guidelines for preliminary thickness design only until more detailed and inclusive design charts are developed.

CONCLUSIONS

A mix of dune sand and asphalt is weak, unstable, easily deformed under light loads, and nondurable. It is not acceptable for pavement construction in a desertlike environment. The introduction of a crushed gravel at different ratios improved the mix but not to the required level. The introduction of a dense-graded aggregate resulted in some improvement, but the quantity of costly high-quality mineral aggregate required to meet specifications made the mix uneconomical.

The introduction of 15 percent sulfur to a mix of dune sand and asphalt gives the following beneficial results:

1. Reduces the optimum asphalt content of the mix from 6.4 to 5.0 percent, a reduction of 22 percent;
2. Considerably increases the engineering quali-

ties of the mix even under severe environmental conditions (such improvements are evident in the Marshall criteria, tensile strength, compressive strength, modulus of rupture, and dynamic modulus of elasticity values);

3. Significantly reduces the pavement thickness required under similar loading and environmental conditions; and

4. Removes the need to import dense-graded aggregate to the site because the engineering qualities of the mix of dune sand, asphalt, and sulfur are equal to or better than those of conventional dense-graded aggregate-asphalt mixes. Not having to use dense-graded aggregate in the mix further reduces construction costs, especially if the aggregate has to be hauled a considerable distance. An additional benefit is that high-quality mineral aggregate may be saved for other purposes and/or for reducing the ecological damage caused by excavating borrow areas and hillsides.

ACKNOWLEDGMENT

The reported investigation was conducted at the Department of Civil Engineering, University of Arizona, while M.N. Fatani was a graduate research associate. Assistance rendered by R.A. Jimenez and A.H. Aboaziza is gratefully acknowledged.

REFERENCES

1. H.A. Sultan, M.N. Fatani, and A.H. Aboaziza. Dune Sand-Asphalt-Sulfur Mixes for Asphalt Concrete Pavements. Presented at Symposium on Geotechnical Problems in Saudi Arabia, College of Engineering, Univ. of Riyadh, Saudi Arabia, May 11-12, 1981, 29 pp.
2. A.A. Rula, W.E. Graber, and R.D. Miles. Forecasting Trafficability of Soils, Air Photo Approach. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MI, Tech. Memo 3-331, Rept. 6, 1963.
3. M.N. Fatani. Dune Sand Aggregate Mixes and Dune Sand-Sulfur Mixes for Asphaltic Concrete Pavements. Univ. of Arizona, Tucson, Ph.D. dissertation, 1980, 319 pp.
4. J.C. Rosner. Let's Design Fly Ash Concretes: Not Compare Them. Proc., Fourth International Ash Utilization Symposium, St. Louis, MO, March 1975, pp. 560-572.
5. Asphalt Institute. The Asphalt Handbook. Manual Series No. 4 (MS-4), 6th ed., College Park, MD, 1970.
6. Asphalt Institute. Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2 (MS-2), 4th ed., College Park, MD, 1974.
7. A.H. Aboaziza. Characterization of Sulfur-Asphalt-Dune Sand Paving Mixtures. Univ. of Arizona, Tucson, Ph.D. dissertation, 1981, 333 pp.
8. I. Deme. Processing of Sand-Asphalt-Sulfur Mixes. Proc., AAPT, Williamsburg, VA, Vol. 43, 1974, pp. 465-490.
9. D.Y. Lee. Modification of Asphalt and Asphalt Paving Mixtures by Sulfur Additives. Industrial and Engineering Chemistry, Product Research and Development, Vol. 14, No. 3, 1975, pp. 171-177.
10. R.A. Burgess and I. Deme. The Development of the Use of Sulphur in Asphalt Paving Mixes. Presented at Sulfur Use Symposium, American Chemical Society, National Meeting, Los Angeles, CA, 1974.
11. W.F. Chen. Double Punch Test for Tensile Strength of Concrete. Journal of the American Concrete Institute, Proceedings Vol. 57, 1970, pp. 993-995.
12. H.Y. Fang and W.F. Chen. New Method for Determination of Tensile Strength of Soils. HRB, Highway Research Record 345, 1971, pp. 62-68.
13. R.A. Jimenez. Testing for Debonding of Asphalt Pavements (Arizona)...Phase II. Arizona Department of Transportation, Phoenix, May 1975.
14. A.F. Bissada. Analysis of High Temperature Instability Failures of Heavily Trafficked Asphalt Pavement. Presented at Annual Meeting, AAPT, Louisville, KY, 1980.
15. R.A. Jimenez. Structural Design of Asphalt Pavements. Arizona Department of Transportation, Phoenix, Final Rept., Nov. 1975.
16. B.M. Gallaway and D. Saylak. Tasks I: Familiarization and Verification of Existing Technology. Texas A&M Univ. Res. Foundation, College Station, TX, Project RF 983, Vol. 1, 1974.
17. N.M. AlSalloum. Fatigue Characteristics of Asphalt Stabilized Dune Sand. Univ. of Arizona, Tucson, Ph.D. dissertation, 1973.
18. E.J. Yoder and M.W. Witczak. Principles of Pavement Design, 2nd ed. Wiley, New York, 1975.
19. H. Warren and W.L. Dickmann. Numerical Computation of Stresses and Strains in Multiple-Layered Asphalt Pavement System. California Research Corporation, Richmond, CA, 1963.
20. W. Heukelom and A.G. Klomp. Dynamic Testing as a Means of Controlling Pavements During and After Construction. First International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1962, pp. 667-679.

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

Asphalt Concrete Mixtures Made with Cement-Coated Aggregates

H.R. GUIRGUIS, O.E.K. DAOUD, AND S.K. HAMDANI

High-quality asphaltic mixtures for pavement construction in Kuwait are produced by using local aggregates with or without a treatment consisting of 1-2 percent of hydrated lime to improve adhesion and enhance resistance to stripping. In recent years, a third type of mix has been introduced in which the local aggregate is coated with about 4 percent of its weight of portland cement, allowed to hydrate, and then used conventionally in the asphalt plant. The merit of the cement-coated aggregate mix is investigated by comparing its laboratory performance with that of both the untreated mix and the hydrated lime-treated mix, all at the same gradation and under the same conditions of preparation and testing. The properties compared included stability, compactibility, resistance to water action, and dynamic response. Three levels of coating with cement (4.9, 6.3, and 8.1 percent) were considered. The findings indicated that the major advantage of using cement-coated aggregates in hot climates is the production of densely graded mixes of higher stability and lower potential for bleeding. The resistance of these mixes to water is as high as those treated with hydrated lime; the index of retained strength after immersion is almost 100 percent. In addition, cement-coated aggregate mixes showed lower susceptibility of stiffness to temperature at a low frequency of loading, with higher values of stiffness in the higher-temperature range.

Owing to the rapid rate of development in the State of Kuwait in recent years, a significant part of the country's road network was completed during the relatively short period of time since 1975. Unfortunately, the active program of road construction during this period allowed little opportunity to adequately monitor and collect feedback on actual pavement performance. The lack of such information made it impossible to identify accurately and at an early stage those forms of pavement distress that proved subsequently to be often encountered in the hot and dry climate of the region.

In recent years, such forms of distress as surface corrugations, excessive permanent deformation in the wheel paths, and fatting-up have significantly increased, and apparently there are no effective remedial measures available to the road authorities. This situation urgently called for a long-term solution to the problem. The initial diagnosis stressed the weakening effect of the very high temperatures in the asphalt pavement layers during the long summer season. These temperatures obviously caused large reductions in stability, which permitted plastic flow, and stiffness, which meant lower load spreadability. An obvious need has therefore arisen for the development of improved asphaltic mixtures with lower susceptibility to temperature.

Experimentation by using a coating of local aggregates with portland cement for the production of high-quality asphaltic mixtures started in early 1977. It was intended to improve the general mix performance by enhancing the frictional component of stability and the aggregate-binder affinity and adhesion. The aggregate was treated with about 4 percent of its weight of portland cement with sufficient water to satisfy the requirements of aggregate absorption and cement hydration. The treated aggregate was then allowed to cure for two days to form a coating of hydrated cement before it was used conventionally in the asphalt plant. The encouraging results of this technique led to the construction and monitoring of a 500-m trial section on a major road in Kuwait. The short-term performance of this section was judged by road authorities to be very satisfactory, and the technique was subsequently

introduced into the construction specification of the major road network (1).

This paper is concerned with the laboratory performance of asphaltic mixes made with cement-coated aggregates compared with the corresponding performance of two conventional mixes of the same gradation and type of parent aggregate: an untreated mix and one (of wide local application) treated with 1 percent of hydrated lime. The effects of cement-treating the aggregates and the degree of this treatment on the stability, compactibility, water resistance, and dynamic response of the resulting asphaltic mixtures were investigated.

ROLE OF CEMENT IN ASPHALTIC MIXTURES

The concept and practice of coating aggregates for use in asphaltic mixes is not new. In general, the process has been employed in an effort to ensure durable adhesion in the bitumen-aggregate system. Portland cement has been used primarily as a filler in warm-mixed bituminous mixtures to prevent stripping of the binder from previously dried aggregate; it has also been used to enhance the coating of wet aggregate with bitumen or tar (2).

Schmidt, Santucci, and Coyne (3) studied the effect of adding 1.3 percent and 3 percent of Type 1 portland cement in an attempt to improve the slow development of strength of emulsion-treated mixes. The cement was added to the aggregate at the time the asphalt emulsion was incorporated. It was concluded that mixes treated in this way cured faster, developed a high modulus of resilience (M_r) more rapidly, and were more resistant to water damage. Flexural fatigue (controlled-stress) experiments showed, however, that these treated mixes have less fatigue resistance than do similar mixes without cement. Terrel and Wang (4) had previously shown also that the rate of development of M_r in emulsion-treated mixes is greatly accelerated by the addition of cement.

Head (5) has reported the results of research on cement-modified asphalt cold mixes. He found that the addition of cement had a very significant effect on mix stability; addition of 1 percent produced an increase in stability of 250-300 percent over that of untreated samples. Specimens without cement immersed in water after stability tests disintegrated after 24 h, whereas cement-treated samples displayed no deterioration. The addition of 1 percent of cement also had the effect of doubling the flow values, but with 2 percent of cement these values were observed to decrease as the samples became more rigid. Results of the immersion-compression investigation indicated not only that moisture or prewetting of the aggregate is necessary to activate the cement but also, possibly, that the moisture available in the emulsion is not effective for hydrating the cement.

Schmidt and Graf (6) have shown that dramatic water resistance and with some aggregates a large increase in the dry M_r of the hot mixes were imparted by adding the cement and lime as a slurry to the aggregate 24 h before the hot mix was made.

Figure 1. Aggregate gradations used.

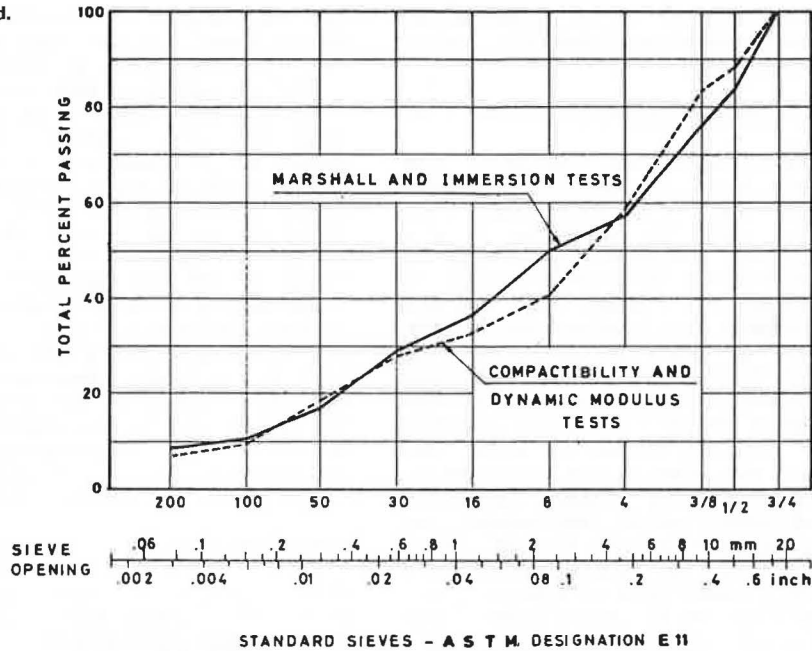


Table 1. Percentages of cement for coating aggregates at three levels.

Level of Coating	Cement (%)				Avg
	Nominal Maximum Size or Type ^a				
	19 mm	9.5 mm	Natural Sand	Crushed Sand	
Light	2.5	4.5	6	7	4.9
Medium	2.5	4.5	9	10	6.3
Heavy	2.5	4.5	14	13	8.1

^aThe ratios of water to cement were 0.65, 0.45, 0.35, and 0.35, respectively.

EXPERIMENTAL PROGRAM

Three different tests were carried out on compacted specimens of the untreated, hydrated lime-treated, and cement-coated aggregate mixes. These were the Marshall test (AASHTO T245), the immersion-compression test (AASHTO T165), and the dynamic-modulus test (ASTM D3497). Marshall tests were planned in two series to investigate the stability and the compaction of the different mixtures.

Previous work by Daoud (7,8) at the Road Research Center in Kuwait has shown that the optimum coating (in terms of the uniformity of distribution of the added cement among different-sized fractions of the treated sand) is usually achieved at 11-15 percent of cement, depending on a number of material and processing variables (including aggregate type and gradation, water/cement ratio, etc.). This is believed to be a range that is too high to prove economically feasible for many applications. It was therefore considered necessary to determine the effect on asphalt mix properties of using aggregates coated with lower percentages of cement. Three levels of cement treatment were selected and the resulting coatings were identified as light, medium, and heavy (Table 1).

In order to reduce sources of material variability, the aggregate gradation used was fixed as shown in Figure 1. This was done by sieving all the aggregates, untreated or cement-coated, into nine size fractions (by using sieve Nos. 1/2, 3/8, 4, 8, 16,

Table 2. Properties of materials used.

Property or Test	Aggregate			
	Asphalt Cement	Natural Sand	Crushed Sand	Coarse Aggregate
Penetration (25°C, 100 g, 5 s)	63			
Flash point (Cleveland Open Cup, °C)	260			
Solubility in trichloroethylene (%)	99.5			
Viscosity at 60°C (poises)	3300			
Bulk specific gravity ^a		2.64	2.67	2.52
Sodium sulfate soundness ^b (%)		9	6	-
Sand equivalent ^c		58	41	-
Los Angeles abrasion ^d		-	-	23

^aAASHTO T84/T85. ^bAASHTO T104. ^cAASHTO T176. ^dAASHTO 96.

30, 50, 100, and 200) and recombining these fractions on a weight basis to produce the test gradation.

Materials

Table 2 presents the results of tests on the materials used in this study. The asphalt cement is a 60/70 penetration grade produced from refining the local crude; it conforms to AASHTO M20.

The coarse aggregate is all of the crushed type, prepared from two nominal maximum sizes: 19 mm and 9.5 mm. The crushed aggregate is generally produced from hard and dense igneous rock excavated originally from watercourses that can be identified in the desert. The degree of crushing is such that more than 80 percent by weight of the aggregate particles had at least one fractured face.

The fine aggregate is a blend of 60 percent crushed and 40 percent natural sand. The natural sand is fine and evenly graded and consists of about 90 percent silica with some calcium carbonate.

The mineral filler used is a limestone dust that conforms to AASHTO M17. Both the filler and the hydrated lime have more than 75 percent of material passing sieve No. 200. Whenever the hydrated lime

was used in the asphaltic mixes, it was added to the aggregate as a slurry and mixed thoroughly before being heated for hot-mix production.

Preparation of Cement-Coated Aggregates

The process of cement-coating the aggregates was performed on a small scale in the laboratory by following the routine applied by local road contractors on a large scale and according to the Kuwaiti road construction documents (1). A 125-L concrete mixer was used for coating 50-kg batches of aggregates. The natural sand, crushed sand, and two sizes of coarse aggregate were treated with cement independently. (Table 1 shows the proportions of cement and water used in coating the four aggregate components.) The natural moisture content of the aggregate was determined in advance and was included in calculating the required amount of water for treatment. The cement was added to the aggregate in the mixer, and after 30 s of dry mixing, the proper amount of water was slowly added. Wet mixing was continued for 90 s, after which the treated aggregate was heaped on a concrete floor, covered with polyethylene sheet to prevent loss of moisture, and left to cure for at least 48 h. After curing, the aggregate was air-dried at room temperature and sieved into fractions.

Testing

Stability

Figure 2 shows the program adopted for determining

Figure 2. Program for testing effect of type and degree of treatment on stability.

BASIS OF COMPARISON	MIX TREATMENT				
	No Treatment	Hydrated Lime	C e m e n t ²		
			Light	Medium	Heavy ¹
Similar Aggregate Characteristics	X	X	X	X	X
Equal Compacted Voids	-	-	X	X	

1) this mix satisfies both criteria for comparison

2) see table 1

and comparing the Marshall stability of the different mixes. It was observed during the work that under the conditions of equal compaction effort and gradation, specimens made with heavily coated aggregates produced higher voids than did similar specimens made with lightly coated aggregates. This called for an additional comparison of mix stability on the basis of equal compacted voids. In order to obtain approximately equal voids in specimens of different treatments with the same compactive effort and gradation, an adjustment was made to the proportions derived from aggregates of several sizes used in preparing these specimens. The crushed fraction between sieves No. 4 and No. 8, for example, can be obtained either wholly from a crushed sand or wholly from 9.5-mm nominal maximum-sized aggregate or from the two materials in any ratio. Although they have the same size, the two fractions differ in particle angularity (due to the characteristics of the crushing process), which influenced the content of voids in the compacted specimens produced.

Compactibility

Compactibility was investigated by comparing the state of densification of otherwise identical Marshall specimens prepared for each mix at two different levels of compaction: 50 blows and 75 blows on each end. The specimens were prepared at the optimum asphalt content that corresponded to the type of mix and level of compaction used. The resulting bulk density, void content, voids in the mineral aggregate (VMA), and percentage of voids filled with bitumen were determined for each condition from three independent tests.

Water Resistance

The susceptibility of the mixes to the action of water was evaluated at two levels of cement treatment, light and heavy (Table 1), and two levels of filler content, 8 and 10 percent. Limestone dust was added to the aggregate to bring the fraction passing sieve No. 200 to the required level. Hydrated lime, when added, was regarded as part of the filler content.

The optimum asphalt content for each case was first determined by the Marshall design method by using a compactive effort of 75 blows on each end. This asphalt content was then used in preparing the cylindrical specimens for the immersion-compression test (AASHTO T165). The alternative procedure in which the immersed specimens are kept in water for 24 h at 60 ± 1°C was adopted.

Table 3. Results of Marshall stability tests.

Type and Degree of Treatment	Aggregate Proportions (%)					Properties of Compacted Mix ^a			
	Nominal Size (mm)		Sand		Limestone Dust	Optimum Asphalt (%)	Marshall Stability Ratio (%)	Bulk Density (g/cm ³)	Void Content (%)
	19	9.5	Crushed	Natural					
Untreated	28	20	30	14.5	7.5	4.00	100	2.37	3.1
Hydrated lime, 1 percent	28	20	30	14.5	7.5	4.25	109	2.36	3.1
Cement:									
Light	28	20	30	14.5	7.5	4.50	104	2.40	3.4
Medium	28	20	30	14.5	7.5	4.50	109	2.36	4.2
Heavy	28	20	30	14.5	7.5	4.50	124	2.35	4.9
Light	30	22.5	27	14	6.5	4.50	92	2.33	5.0
Medium	30	20	27.5	15	7.5	4.25	111	2.35	4.7
Heavy	28	20	30	14.5	7.5	4.50	124	2.35	4.9

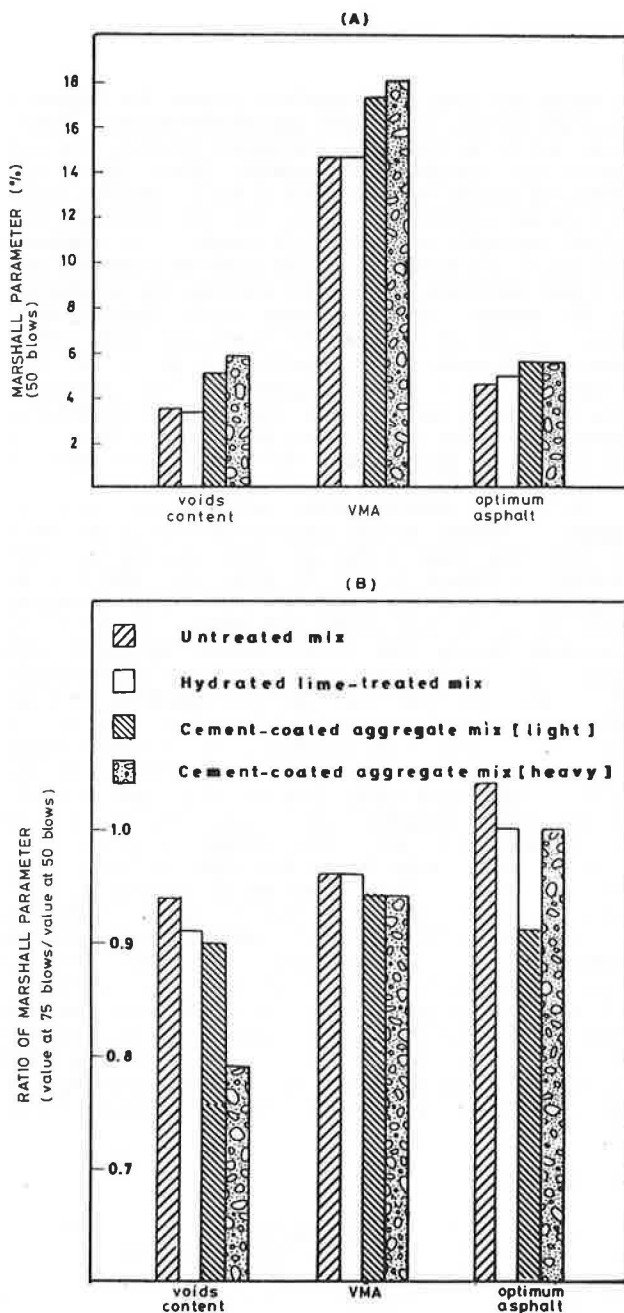
Note: 32 specimens were tested for each mix in rows 4, 5, 7, and 8; other mixes (rows 1, 2, 3, and 6) had 16 specimens each.

^aCompaction is according to AASHTO T245 with 100 blows per side.

Table 4. Results of compactibility tests.

Type and Degree of Treatment	Optimum Asphalt (%)	Void Content (%)	Bulk Specific Gravity (g/cm ³)	VMA Value	Voids Filled with Bitumen (%)
Untreated mix					
50 blows	4.6	3.4	2.338	14.6	73
75 blows	4.8	3.2	2.351	14.0	79
Mix with hydrated lime, 1 percent					
50 blows	4.9	3.3	2.343	14.6	78
75 blows	4.9	3.0	2.358	14.0	81
Mix with cement-coated aggregate					
Light coat					
50 blows	5.5	5.1	2.290	17.2	72.5
75 blows	5.0	4.6	2.306	16.2	70
Heavy coat					
50 blows	5.5	5.8	2.27	17.9	68
75 blows	5.5	4.6	2.30	16.9	73

Figure 3. Compaction characteristics of asphaltic mixes under two different treatments.



Dynamic Response

An electrohydraulic testing machine was used to determine the dynamic modulus of compacted specimens of two mix types--hydrated lime-treated mix and cement-coated aggregate mix with light treatment. The specimens, which were 101.6 mm diameter and 203.2 mm high, were prepared according to ASTM D3496, capped with sulfur mortar according to ASTM C617, and tested at three frequencies (1, 4, and 16 Hz) and three levels of temperature (5, 25, and 45°C) according to ASTM D3497.

RESULTS AND DISCUSSION

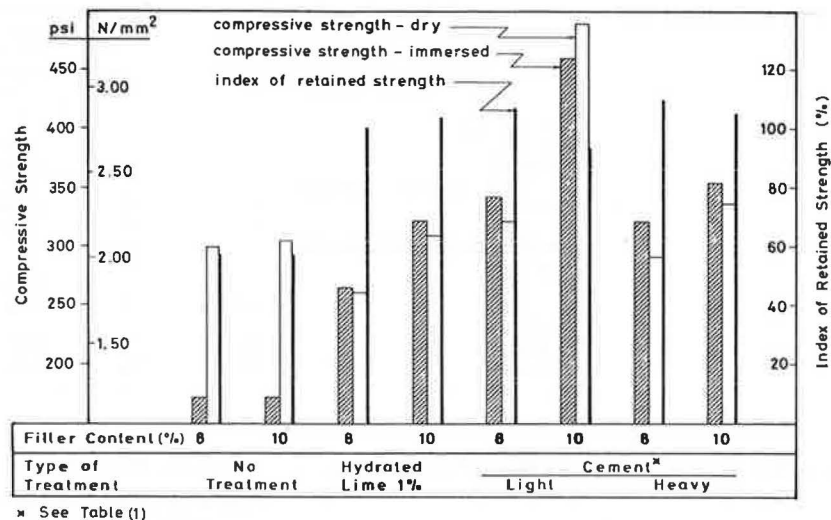
From the results of Marshall tests presented in Table 3, it can be seen that the untreated mix has the lowest stability in spite of its relatively high density (row 1). The addition of 1 percent by aggregate weight of hydrated lime (row 2) did not change the void content or density significantly, but it did increase stability by 9 percent. This increase in stability is believed to be due partly to the fineness of the hydrated lime compared with the limestone dust (specific surfaces of 750 and 260 m²/kg, respectively), which produced a binder of higher viscosity.

Rows 3, 4, and 5 of Table 3 show that both voids and stability of the cement-treated mixes increase with the degree of cement coating. Specimens made with increasing levels of cement coating are respectively 10, 35, and 58 percent higher in voids and 4, 9, and 24 percent higher in stability than specimens made with untreated aggregates. The higher void content obviously indicates a reduction in compactibility of these mixes and is expected consequently to result in lower stabilities. However, it seems that the gain in mix stability due to enhancement of aggregate texture (by precoating with cement) is more than the loss due to lower density and higher voids of the resulting less-compactible mix.

When the three cement-treated mixes are prepared and tested at approximately equal void content (rows 6, 7, 8), the effect of the degree of cement treatment becomes more evident. In this case, the stability values of the medium treatment and the heavy treatment compared with that of the light treatment are respectively 21 and 35 percent higher. One advantage of using cement-coated aggregate mixes in relation to a hot climate becomes apparent, namely, their higher stability values obtained at comparatively higher void content. The result is a reduction of the potential for bleeding usually developed when mixes with already low void content are post-compacted under heavy traffic.

The compaction characteristics of the mixes under two levels of Marshall compactive efforts are presented in Table 4 and Figure 3. It is seen in Fig-

Figure 4. Resistance of differently treated asphaltic mixes to water action.



ure 3A that by applying the same effort for all mixes at their respective optimum asphalt contents, those made with cement-coated aggregates resulted in about 25 percent higher VMA values compared with that of the untreated case. Since the same gradation is used, the higher VMA values are indicative of the difficulty with which these mixes are compacted. Under a kneading type of compaction, this increase in VMA would probably be smaller due to the relative freedom of movement of the particles during the process of densification. The higher VMA values and optimum asphalt content can be referred to the increase in roughness of particle surface texture (2,9) after having been treated with cement.

In the extremely hot climate of Kuwait, where pavement surface temperatures during summer can exceed 70°C, asphaltic mixes should preferably have void contents near the upper permissible limit as a safety factor against potential fatting-up or bleeding. Void contents in this range are especially suitable in such an environment due to the scarcity of rainfall. The cement coating of aggregates appears to allow densely graded mixes of higher stability values to be used without the risk of closing the voids to an undesirably low level.

Increasing the compactive effort to 75 blows has no significant effect on the optimum asphalt content of different mixes (Table 4). Figure 3B, however, shows that due to their higher initial VMA, mixes with cement-coated aggregates compressed slightly more during the additional 25 blows than did conventional mixes (4 percent versus 6-10 percent reduction in VMA values for conventional and cement-treated mixes, respectively). This extra consolidation due to a higher level of compaction may be regarded as a rough simulation of the postcompaction to which an asphalt layer would be subjected under traffic.

The effect of water immersion is shown in Figure 4. Although untreated specimens lost about half their compressive strength after immersion, specimens that had 1 percent hydrated lime and those treated with cement appeared completely resistant to water. The large drop in strength of untreated specimens occurs in spite of their fines content of 8 and 10 percent (mostly limestone dust), which ought to improve the mechanical component of adhesion and thus the resistance to stripping by increasing the viscosity of the original binder in the mix (2). However, unlike active fillers such as hydrated lime, limestone dust is inert and insoluble

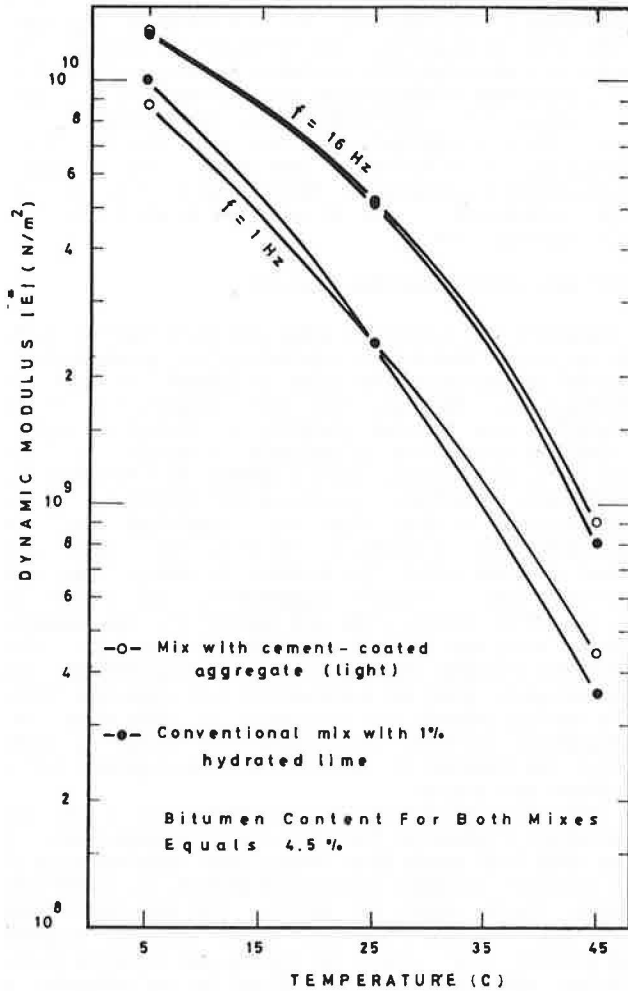
in water and does not therefore change the physicochemical mechanism at the aggregate-bitumen interface, which is known to influence greatly the adhesion and stripping of asphalt films (10). The change of filler content from 8 to 10 percent seems to have no significant effect on the index of retained strength in almost all cases. The compressive strength, however, in the case of both dry and immersed specimens is seen to increase by an average of 20 percent for specimens with hydrated-lime treatment and 13 percent for specimens with heavy-cement treatment. The maximum response to the change in filler content appears to be associated with the specimens that had light cement treatment. Increases of the compressive strength in this case are 52 and 34 percent for the dry and the immersed conditions, respectively.

The dynamic-modulus test results are shown in Figure 5. Values at the intermediate frequency of 4 Hz show the same trend but for clarity are not plotted in Figure 5. It is seen that the dynamic responses of the two mixes at the higher frequency level of 16 Hz are almost the same. At the lower frequency level, the cement-coated aggregate mix shows some improvement in temperature susceptibility. This is indicated by the slightly flatter stiffness-temperature relationship over that of the conventional mix. In addition, the cement-treated mix shows higher values of stiffness modulus at the higher temperature range (above 25°C), particularly at the lower frequency of 1 Hz. This could be interpreted as having better dynamic response as a pavement layer under heavy and slow traffic during the hot season. This increase in stiffness, however, is not of the order that permits a reduction in the design thickness of cement-treated asphalt layers (i.e., as a compensation for their improved load spreadability). Calculation of stress distribution, for instance, shows that the corresponding reduction in the vertical compressive stress on the subgrade due to the increase in stiffness at 45°C (Figure 5) is of the order of 7 percent only (assuming a two-layer pavement system with a modular ratio $E1/E2$ of 10).

CONCLUSIONS

1. The precoating of aggregates with 6.3 and 8.1 percent of portland cement (medium and heavy treatments) has the effect of increasing the Marshall stability of the resulting asphalt mixtures by 9 and

Figure 5. Effect of cement treatment on dynamic modulus of asphalt concrete.



24 percent, respectively. This was associated with an increase in the compacted voids.

2. When compared on the basis of approximately equal compacted voids, asphalt mixes with the medium and heavy cement treatments proved respectively 21 and 35 percent higher in stability than mixes with light cement treatment (4.9 percent cement).

3. The index of retained strength (immersion-

compression ratio) increased from less than 60 percent for untreated mixes to almost 100 percent for mixes made with cement-coated aggregates at any of the coating levels investigated.

4. The use of cement-coated aggregates improved temperature susceptibility of mix stiffness in the low-frequency range of 1-4 Hz.

ACKNOWLEDGMENT

This study was conducted at the Road Research Center under the sponsorship of the Ministry of Public Works (Department of Roads and Drainage) of Kuwait. We wish to thank A. Allam for doing the laboratory testing and F. Behn for reviewing the text.

REFERENCES

1. Specification for Road Work. Department of Roads and Drainage, Ministry of Public Works, State of Kuwait, 1980.
2. Bituminous Materials in Road Construction. U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1969.
3. R.J. Schmidt, L.E. Santucci, and L.D. Coyne. Performance Characteristics of Cement Modified Asphalt Emulsion Mixes. Proc., AAPT, Vol. 42, 1973, pp. 300-319.
4. R.L. Terrel and C.K. Wang. Early Curing Behavior of Cement Modified Asphalt Emulsion Mixtures. Proc., AAPT, Vol. 40, 1971, pp. 108-125.
5. R.W. Head. An Informal Report of Cold Mix Research Using Emulsified Asphalt as a Binder. Proc., AAPT, Vol. 43, 1974, p. 110.
6. R.J. Schmidt and P.E. Graf. The Effect of Water on the Resilient Modulus of Asphalt-Treated Mixes. Proc., AAPT, Vol. 41, 1972.
7. O.E.K. Daoud. Optimum Cement Ratios for Coating Crushed and Natural Sands. Road Research Center, Ministry of Public Works, Kuwait, Internal Rept. 80/4, 1980.
8. O.E.K. Daoud, H.R. Guirguis, and S.K. Hamdani. Factors Affecting the Coating of Aggregates with Portland Cement. Transportation Research Record (to be published).
9. J.M. Griffith and B.F. Kallas. Influence of Fine Aggregate on Asphaltic Concrete Paving Mixtures. Proc., HRB, 1958, pp. 219-255.
10. I. Ishai and J. Craus. Effect of the Filler on Aggregate-Bitumen Adhesion Properties in Bituminous Mixtures. Proc., AAPT, Vol. 42, 1973.

Publication of this paper sponsored by Committee on Characteristics of Non-bituminous Components of Bituminous Paving Mixtures.

Abridgment

Factors Affecting Unconfined Compressive Strength of Lime-Bituminous-Emulsion-Treated Clay

RAYMOND K. MOORE

A one-half 2⁷ fractional factorial experiment design was used to evaluate the effect of soil type, lime content, molding moisture content, modification curing time, bituminous emulsion type, bituminous emulsion content, and curing temperature on the unconfined compressive strength of compacted

specimens after a four-week postcompaction curing period. Analysis-of-variance techniques were used to determine the significant main effects, two-factor interactions, and three-factor interactions at alpha levels of 1 and 5 percent. In general, for the range of variables used in this research, the un-

Table 1. Factors and levels used.

Factor	Level	
	Low	High
Soil type	Houston Clay (HC)	Permian Red Clay (PRC)
Lime content ^a	3	8
Molding moisture content ^a	15	20
Modification curing time (days)	0	2
Bituminous emulsion type	CRS-2	SS-2
Bituminous emulsion content ^a	6	12
Curing temperature (°F)	75	110

^aPercent by air dry weight of soil.

Table 2. ANOVA for unconfined compressive strength.

Source of Variation	Degrees of Freedom	Mean Square	F-Value ^a	Significance Level (%)
A	1	36 130	49.06	1
G	1	32 572	44.23	1
C	1	21 301	28.92	1
AxD	1	7 933	10.77	1
DxExF	1	7 114	9.66	1
CxG	1	6 498	8.82	5
AxG	1	6 196	8.41	5
BxC	1	5 681	7.71	5
E	1	5 006	6.80	5
AxF	1	4 572	6.21	5
AxCxG	1	4 425	6.00	5
BxCxG	1	4 239	5.76	5
Residual	51	600	-	-
Total	63	-	-	-
Within treatments, treated alike	12	736	-	-

Note: A = soil type, B = lime content, C = molding moisture content, D = modification curing time, E = bituminous emulsion type, F = bituminous emulsion content, and G = curing temperature.

^aCritical F-values: $F(1, 12, 0.01) = 9.33$, $F(1, 12, 0.05) = 4.75$.

confined compressive strength of lime-bituminous-emulsion-stabilized soils was increased by using (a) 8 percent lime rather than 3 percent lime, (b) 20 percent molding moisture rather than 15 percent molding moisture, (c) curing temperature of 110°F rather than 75°F, (d) chloritic clay rather than a smectic clay, (e) two-day modification curing period after lime pretreatment of the smectic clay, and (f) SS-2 asphaltic emulsion instead of the CRS-2 emulsion. The correct emulsion content percentage appears to be a function of the individual soil type.

The research reported here was designed to investigate selected mixture design, construction, and environmental factors and their interactions that affect the bituminous emulsion stabilization of clay soils pretreated with hydrated high-calcium lime. This combination stabilization strategy appears to be complex, since the interaction of additives and various construction and environmental variables could make a mixture design process difficult to standardize and construction difficult to control.

EXPERIMENT DESIGN

The factors and levels used in the experimental investigation are given in Table 1. These factors represent a number of mixture design, construction, and environmental variables that have been shown to influence lime and bituminous stabilization of clay soil. It is therefore reasonable to use these variables in a study of a combination stabilization strategy by using lime and bituminous emulsion.

The experiment design used a one-half 2⁷ fractional factorial. All main effects were confounded with six-factor interactions, two-factor interactions were confounded with five-factor interactions,

and three-factor interactions were confounded with four-factor interactions. It was assumed that all six-factor, five-factor, and four-factor interactions were negligible. The estimate of experimental error was determined by randomly selecting 12 duplicate treatment combinations from throughout the factor space, and a replicate error was calculated by comparing the duplicate specimen with the factorial specimen. This technique has been used in other stabilization experiments that used statistical design techniques in the design and analysis of the experimental data (1).

SPECIMEN PREPARATION AND TESTING

Individual quantities of lime and soil were weighed in separate containers according to predetermined levels to minimize the time required for specimen preparation. The specimens were prepared by first measuring the desired quantity of distilled water, selecting the proper preweighed quantities of lime and soil, dry mixing, adding water, and blending by hand until a uniform appearance was attained. If no modification curing time was specified for the specimen, the bituminous emulsion was added and mixed by hand until the mixture of soil, lime, and emulsion had a uniform appearance. If a two-day modification curing time was specified, the mixture was covered and allowed to cure for two days. The moisture content was determined before curing, and any moisture loss by evaporation was replaced after the curing period had expired. At this time, the bituminous emulsion was added and mixed by hand until the mixture of soil, lime, and emulsion had a uniform appearance.

The soil mixtures were compacted in a Harvard Miniature Compaction Mold by using a scaled model of the ASTM D697 compaction hammer that weighed 0.53 lb to simulate standard compactive effort (25 blows per layer, three layers per mold). The specimens were extruded from the compaction mold, weighed, wrapped in plastic wrap, sealed to ensure no moisture loss, marked, and stored at the desired curing temperature until the proper test day. Following the four-week curing period, the specimens were removed one at a time from their respective temperature chambers, weighed, and tested to failure in unconfined compression by using a strain rate of 1 percent/min.

STATISTICAL DATA

The analysis-of-variance (ANOVA) results are given in Table 2 for main effects, two-factor interactions, and three-factor interactions found to be significant at alpha levels of 1 and 5 percent.

DISCUSSION OF RESULTS

The ANOVA data in conjunction with an analysis of treatment combination means indicated that the unconfined compressive strength of lime-bituminous-emulsion soils was increased by using the following:

1. Permian Red Clay (PRC) rather than Houston Clay (HC),
2. 8 percent lime rather than 3 percent lime,
3. 20 percent molding moisture rather than 15 percent molding moisture,
4. Two-day modification curing period after lime pretreatment of HC,
5. SS-2 bituminous emulsion rather than CRS-2 emulsion, and
6. Curing temperature of 110°F rather than 75°F.

Soil Type

PRC is a reddish-colored medium-plastic chloritic

(CL) soil common to Central Oklahoma with a mineralogical profile dominated by chlorite. Mica composes approximately 10 percent of the soil, and the remainder is illite and smectite.

HC is a yellowish-grey soil of the Alabama Black Belt; 30-45 percent of the total soil is smectite. Kaolinite constitutes about 25 percent of the clay fraction. The unified classification is CH.

Soil type was a significant main effect in two-factor interactions with modification curing time, curing temperature, and bituminous emulsion content and in a three-factor interaction with molding moisture content and curing temperature.

Lower unconfined compressive strengths were associated with HC. Clay soils that have a high surface area create mixability difficulties, even with lime pretreatment, which produces an uneven distribution of bituminous emulsion. Furthermore, the lime-treated HC appears to have a more fragile matrix of clay agglomerations, as proposed by Ford (2). The chloritic PRC would have better mixing characteristics with, perhaps, a stronger matrix of clay-particle agglomerations.

Lime Content

Although lime content does not appear significant as a main effect, it does affect the unconfined compressive strength through a two-factor interaction with molding moisture content and through a three-factor interaction involving molding water content and curing temperature. Higher unconfined compressive strengths were associated with the 8 percent lime content.

Eight percent lime is sufficient for free calcium to remain after the modification of both PRC and HC soils. The presence of bituminous emulsion does not appear to eliminate the formation of lime-soil pozzolanic compounds during the postcompaction period.

Molding Moisture Content

Molding moisture content was a significant main effect in a two-factor interaction with curing temperature and in three-factor interactions with soil type, lime content, and curing temperature. Higher unconfined compressive strengths were associated with the 20 percent moisture content.

The series of significant molding moisture content interactions is commonly associated with lime-soil treatment, since the development of lime-soil pozzolanic products is dependent on the presence of adequate water. Furthermore, the higher moisture content would also beneficially influence the densification of the soil system, since the 20 percent moisture content is approximately the optimum of the lime-treated PRC and HC. The distribution of the bituminous emulsion during the mixing process would also be facilitated at the higher moisture content with less risk that the SS-2 emulsion would break prematurely.

Modification Curing Time

Modification curing time was not a significant main effect. However, the factor was significant in a two-factor interaction with soil type and in a three-factor interaction with bituminous emulsion type and bituminous emulsion content.

The modification curing period of two days improved the mean unconfined compressive strength of HC, but it was detrimental to PRC. This interaction suggests that a modification curing period may be advisable for clay soils with a high percentage of smectite but that less-active clays should be stabilized with bituminous emulsion immediately after

modification lime and moisture have been added and thoroughly mixed. Furthermore, the interpretation of the three-factor interaction suggests that the effect of modification curing time on unconfined compressive strength will also be related to the emulsion percentage used.

Emulsion Type

Rapid-set emulsions have little or no ability to mix with an aggregate or with soil-aggregate mixtures. Typical applications of CRS-2 emulsions include surface treatment, penetration macadam, and single or multiple coarse-aggregate seal coats (3, ASTM D2397-71).

Slow-set emulsions are designed to mix with fine aggregate. Typical applications of SS-2 as a specific emulsion are not given in general reference data (3, ASTM D977-73). However, SS-2 emulsion only differs significantly from the more frequently referenced SS-1 by having a higher viscosity. Typical values for SS-1 are a minimum Saybolt Furol viscosity at 77°F of 20 s and a maximum of 100 s (ASTM D977-73) as compared with 40 s and 400 s of the SS-2 used in this study. Therefore, the typical applications of SS-2 would be those of SS-1 and would include cold plant mix, sand mix, slurry seal coat, tack coat, fog seal, dust layer, and mulch.

Certainly the CRS-2 emulsion would not be used or recommended for soil-bituminous stabilization, and it is not the intention of this study to evaluate its use. The emulsion type was helpful in better defining the complex interactions among soil, construction, and environmental factors investigated although it has no potential application in mixture design for lime-bituminous-treated clay soils. The SS-2 does have a potential application for field use, but the focus of the research does not have its evaluation in terms of mixture design as an objective.

The use of anionic emulsion (SS-2) with lime pretreatment produced a higher mean unconfined compressive strength than did the cationic emulsion (CRS-2). Since the CRS-2 is not manufactured for mixing with fine-grained soils, a tendency for poor mixtures would be expected. Visual inspection of the compacted specimens confirmed the poor mixability of CRS-2 emulsion with the fine-grained lime-modified soils. Another minor reason for the poor mixability may be that the clay platelets become saturated with calcium and have a net positive charge. Therefore, the flocculated and agglomerated groups of clay platelets would repel the cationic charged emulsion. Future research by using a CSS emulsion should be designed to investigate the influence of this surface-chemistry-oriented repulsion hypothesis for lime-modified soil.

Curing Temperature

Curing temperature had a significant effect on the unconfined compressive strength as a main effect as well as in two two-factor interactions and two three-factor interactions, as shown in Table 2. In all cases, the curing temperature of 110°F was associated with higher strengths than a 75°F curing temperature. The higher curing temperature would encourage the formation of pozzolanic compounds through the chemical interaction of the free lime and clay soil. The increased curing temperatures could also encourage a continuation of emulsion dispersion within the soil mass after compaction during the four-week curing period. The higher degree of dispersion could produce higher unconfined compressive strengths.

CONCLUSIONS

The experimental results are indicative of the complexity inherent in this combination stabilization strategy. An improved understanding of the effect of emulsion type on the engineering and physical properties of the lime-treated clay is needed. Potential physical interactions involving clay surface chemistry and emulsion chemistry require investigation, since they may influence the ability of the bituminous emulsion to be satisfactorily dispersed during mixing. Other test data are needed to define tensile properties, stress-deformation characteristics, and the response of the material to moisture penetration.

ACKNOWLEDGMENT

The research reported here was sponsored by the Research Grant-in-Aid Program, Auburn University, Auburn, Alabama. I am indebted to Gregory Rosser, who

assisted in the preparation and testing of the soil specimens. I also gratefully acknowledge the cooperation of the Hunt Oil Company, Tuscaloosa, Alabama, which provided the asphaltic emulsions used in the research effort.

REFERENCES

1. R.K. Moore, T.W. Kennedy, and W.R. Hudson. Factors Affecting the Tensile Strength of Cement-Treated Materials. HRB, Highway Research Record 315, 1970, pp. 64-80.
2. C.M. Ford. Reaction Products of Lime Treated Alabama Soils. Auburn University, AL, M.S. thesis, 1978.
3. A Basic Asphalt Emulsion Manual. Asphalt Institute, College Park, MD, Manual Series 19 (MS-19), March 1979.

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

Laboratory Test Method for Predicting Moisture-Induced Damage to Asphalt Concrete

ROBERT P. LOTTMAN

A laboratory test method is described for the prediction of moisture damage in dense-graded asphalt concrete mixtures. The method consists of obtaining diametral (or indirect) tensile-strength and modulus ratios of compacted specimens subjected to vacuum saturation with water and to freeze-plus-warm-water-soak accelerated moisture conditioning. Test results for dry specimens are used to form the ratio bases. The ratios are used to predict short-term and long-term field damage. Fatigue curves for two mixtures exposed to the dry, vacuum-saturation, and accelerated moisture conditioning of the test method are presented to show effects of moisture on fatigue life. A tentative relationship shows a correlation between tensile-strength ratios obtained by the test method and pavement fatigue-life ratios. An example of the practical use of the correlation is shown. Results are presented from a five-year field evaluation study conducted by seven highway agencies on eight pavement test sections to determine whether the test method's predictive ratios and stripping tendencies correlate with field results. Short-term ratios from laboratory vacuum saturation were reached at four years of pavement age or before. Long-term ratios from laboratory accelerated moisture conditioning ranged from 0 to 0.80; they were reached at five years for some pavements and probably will be reached in a few more years for the other pavements. This ratio is considered one of maximum moisture damage to minimum moisture damage. Visual stripping in the field cores appears similar to the predicted laboratory stripping. Agency-determined layer coefficients decreased due to the loss of moisture cohesion from the associated stripping observed in the field.

The destructive influence of moisture in dense-graded asphalt concrete has been recognized for decades. Laboratory tests have been developed to predict potential moisture damage. The purpose of the tests is to assess the redesign of asphalt mixtures (changes of aggregate type, asphalt, and compaction and addition of antistripping treatments) prior to paving in order to minimize the damage.

Immersion-compression and Marshall-type tests on asphalt concrete mixtures evaluate the effect of water on the asphalt concrete mixture in a compacted state in order to find the interaction of all the mixture constituents. The evaluation methods relate the wet strength to the dry strength either as a

ratio or as a percentage of retained strength. Highway agencies have developed specifications for the ratios or retained percentages; low values imply high moisture damage and the necessity to redesign or alter the asphalt concrete mixture being evaluated.

Moisture mechanisms that cause damage have been the objective of studies by many investigators. Simulation of the mechanisms to produce closer field-related moisture-damage conditioning in the laboratory should give more realistic predictive damage ratios. The following are some of the major moisture-damage mechanisms that cause stripping or mixture softening or both:

1. Pore pressure of water in the mixture voids due to wheel-loading repetitions; thermal expansion-contraction differences produced by ice formation, temperature cycling above freezing, freeze-thaw, and thermal shock; or a combination of these factors;
2. Asphalt removal by water in the mixture at moderate to higher temperatures;
3. Water-vapor interaction with the asphalt-filler mastic and larger aggregate interfaces; and
4. Water interaction with clay minerals in the aggregate fines.

Added to the importance of simulating the proper moisture-damage mechanism in laboratory tests is the selection of methods to measure damage. Loss of bond due to stripping seems to be measured more directly by tensile-type tests. Also, moisture-damaged asphalt concrete loses cohesion and the pavements crack and deteriorate, especially when severe stripping is observed. Cracking and some of the deterioration result from repeated tensile stress (or strain) in the field due to wheel loads. Thus,

measurement by tensile-mode laboratory tests appears to show the field-associated moisture-damage effect in a realistic manner. Likewise, the stress (or strain) value obtained from these tests has the advantage of being directly used in the current mechanistic approaches of pavement fatigue-life prediction. Here, the indirect-tension (diametral-tension) test is useful; modulus or strength values or both can be used to evaluate effects of moisture. In addition, there is an advantage to using the indirect-tension test under repeated loads as a fatigue test. The relationship of fatigue life to indirect tensile-strength ratios from laboratory moisture-damage simulation may also become practical for pavement life assessment and should provide a closer end-result examination of mixture design and treatment alternatives.

The objectives of this paper are as follows:

1. To describe the laboratory moisture-damage predictive test method that was developed under NCHRP Research Project 4-8(3) for dense-graded asphalt concrete (1),
2. To show the practical implications of how moisture-damage ratios might be related to the change of asphalt mixture fatigue life as a result of the effects of moisture conditioning applied in the test method, and
3. To determine the correlation of the predicted moisture-damage ratios and stripping (determined by the test method) with the damage ratios and stripping found in the asphalt concrete pavements selected for study.

MOISTURE-DAMAGE TEST METHOD

Background

The destructive mechanism of water pressure in the voids of dense-graded asphalt concrete was employed in the development of a laboratory test method for the Idaho Department of Highways in 1971 (2). The test method consisted of saturating compacted mixtures and subjecting them to thermal cycling. Moisture damage was measured by the indirect-tensile test, and the tensile-strength ratio was used as the measure of moisture sensitivity (damage). During the following years, a similar test method was developed under the National Cooperative Highway Research Program (NCHRP) Research Project 4-8(3) (1) with modifications that included the addition of the resilient-modulus ratio and the substitution of the accelerated moisture conditioning variant of a freeze-plus-warm-water soak in place of thermal cycling. In this project a number of cores were obtained and tested from 3- to 12-year-old moisture-damaged pavements in the United States. These results were compared with the damage resulting from several modifications of accelerated moisture conditioning by using laboratory-compact specimens with aggregate and asphalt types incorporated in the pavements. A close field match was observed with the freeze-plus-warm-water-soak conditioning procedure (3). It was observed that this accelerated conditioning was responsible for the majority of moisture-damage mechanisms.

The moisture conditioning and the testing procedure use routine laboratory equipment. The saturation and testing portion of the test method can also be used to monitor pavement damage by testing cores drilled from the asphalt concrete layer under investigation. The evaluation of the effectiveness of antistripping additives and treatments is also a potential application of the test method.

Test Method

Details of the test method are in the final report

of NCHRP Research Project 4-8(3)/1 (1). A summary of the test method is as follows:

1. Nine specimens 4 in in diameter by 2.5 in thick are made from mixtures of aggregate and asphalt materials to be used in the pavement and compacted to the field-estimated permeable voids (based on volume of water absorbed by vacuum saturation into the initially dry specimen).

2. After one or two days of room-temperature curing, the specimens are divided into three sets of three specimens each. One set is selected for the dry test, the second set for the vacuum-saturation test, and the third set for the accelerated-conditioning test (vacuum saturation followed by freeze-plus-warm-water soak). (Permeable voids are calculated during the vacuum-saturation procedure for the second and third sets.) Vacuum saturation consists of immersing the specimens in jars filled with distilled water, pulling a 26-in Hg vacuum for 30 min, and keeping the submerged specimens in the jars for an additional 30 min at atmospheric pressure.

3. The first (dry) and second (vacuum-saturation) sets are submerged in a water bath at the mechanical test temperature for 3 h. Dry specimens are maintained dry, e.g., placed in watertight jars in the bath. If the resilient modulus is desired, then it should be measured first at 55°F (or at room temperature) by using the Schmidt or Chevron procedure (4). By using the same specimens, their indirect tensile strength is measured at 55°F by using a vertical deformation rate of 0.65 in/min (3). Average values for each set are calculated. Visual stripping is also recorded for the two interior faces of each split specimen.

4. Before application of the accelerated conditioning, each wet, vacuum-saturated specimen of the third set is tightly wrapped in thin plastic wrap. Each wrapped specimen is placed in a heavy-duty plastic bag with about 3 mil of distilled water, sealed, and immediately placed in a 0-10°F freezer for 15 h. The wrapped, frozen specimens are then quickly submerged in a 140°F water bath for 3 min. The thawed wrappings are rapidly removed and the specimens are immediately replaced in the 140°F bath for 24 h. The specimens are then submerged in a cooler water bath (set at the desired mechanical test temperature) for 3-5 h prior to testing according to step 3 above.

5. Two resilient-modulus ratios and two indirect-tensile-strength ratios are calculated from the average test values. The ratio of vacuum-saturated specimens to dry specimens is considered to be a short-term ratio that simulates moisture in the asphalt concrete at the moisture peak or saturation in the field. The ratio of accelerated-conditioning specimens to dry specimens is considered to be an ultimate, long-term moisture-damage measurement, which occurs in the asphalt concrete (after the saturation effects) due to the forces of environment and traffic. This ratio is almost always less than the vacuum-saturation ratio, and severe stripping is almost always associated with very low ratios.

MOISTURE-DAMAGE AND FATIGUE-LIFE RATIOS

Field moisture damage, e.g., stripping, can destroy most of the cohesion of the asphalt concrete, and the pavement layer does not withstand bending stress to the degree exhibited initially after paving. Severe cases deteriorate the asphalt concrete to a virtually cohesionless base material. This will lead to early cracking and premature pavement distress. Therefore, low indirect-tensile-strength and resilient-modulus ratios should be related to low asphalt concrete fatigue life. Since the fatigue-

Figure 1. BK 142 mix: fatigue relationships at 70°F for moisture conditioning.

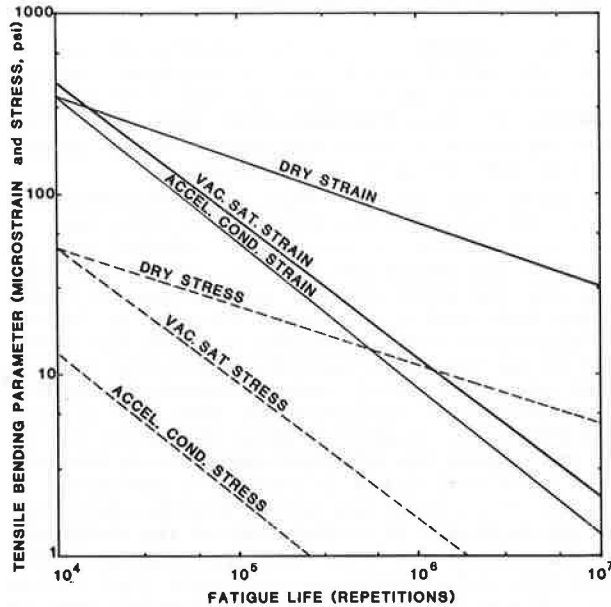
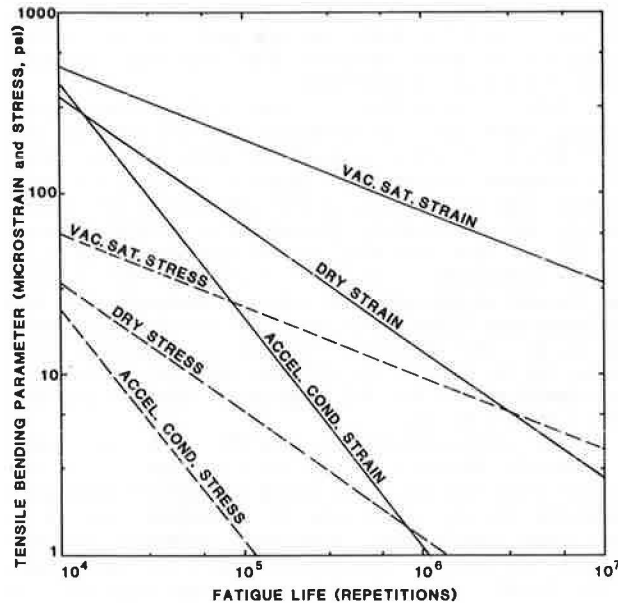


Figure 2. BK 117 mix: fatigue relationships at 70°F for moisture conditioning.



life consideration is an end result, its correlation to moisture-damage ratios should provide a practical basis for the selection of laboratory tensile strength or modulus, which denotes acceptable (or unacceptable) mixtures.

Research has shown that changes of indirect tensile strength and resilient modulus for asphalt concrete affect bending fatigue life and the relationship is somewhat orderly (5). Diametral or indirect tensile fatigue tests also have been performed on a number of asphalt concrete mixtures. The test data were transformed to construct bending fatigue curves (6). As a consequence of these practical developments, constant-stress diametral (indirect) tensile fatigue tests were performed on specimens of two different mixtures subjected to dry, vacuum-saturation, and accelerated moisture

conditioning to provide the rudiments of a laboratory correlation between the ratios of indirect tensile strength and fatigue life. These mixtures were made with aggregates that have exhibited differences in moisture sensitivity: Idaho BK 142 aggregate has a history of being associated with severe moisture damage, and Idaho BK 117 aggregate has a history of less moisture damage. A control AC-10 asphalt was used. The mixtures were compacted to 8-9 percent permeable voids in order to emphasize moisture damage or sensitivity differences.

Laboratory-developed fatigue curves for the two mixtures are shown in Figures 1 and 2. The constant-stress diametral (indirect) tensile fatigue data were transformed to uniaxial-type bending fatigue by using the stress difference multiplier of 4.0. Fatigue strains corresponding to transformed fatigue stresses were calculated by dividing the stresses by the appropriate modulus (dry, vacuum saturation, or accelerated conditioning) and then multiplying by the following term: $3 \text{ times Poisson's ratio} + 1$. Poisson's ratio was assumed equal to 0.33.

Indirect tensile strength, modulus, and maximum repeated stress for a fatigue life of 100 000 cycles are listed in Table 1 with their ratios. For these data, tensile-strength and modulus ratios are close to each other for a given mixture and type of moisture conditioning. However, the ratios for the two mixtures are different and this difference is also apparent in and related to the positioning of the fatigue curves. The fatigue curves are roughly proportional to the changes of tensile strength and modulus brought about by the moisture conditioning. This is observed by the tensile-stress ratio corresponding to a fatigue life of 100 000 cycles. Tensile-strength (or modulus) ratios less than 1.0 reflect steeper and lower fatigue curves—a decreased fatigue life for the moderate stress and strain range encountered in pavements.

Mechanistic-theory fatigue-life ratios were calculated for computed stresses and strains in the bottom of the asphalt concrete for two simulated pavements subjected to the 18-kip equivalent single-axle load (ESAL). Resilient-modulus values that represent each type of moisture conditioning for each mixture were used in the computation. The fatigue-life curves (Figures 1 and 2) were entered at the computed stress and strain values for the pavements, and the corresponding lives for each type of moisture conditioning were found. Fatigue-life ratios were then calculated based on the fatigue life in the dry condition. The stress and strain values and the resulting fatigue-life ratios are listed in Table 2.

It should be noted that the decrease of modulus produces a decrease of stress but an increase of strain in the asphalt concrete. Therefore, the decrease of modulus due to moisture damage will affect strain fatigue more adversely than stress fatigue. The resulting strain increase due to moisture damage results in a fatigue-life decrease that is somewhat greater than that apparent in Figures 1 and 2.

Tensile-strength ratios are plotted versus corresponding mechanistic-theory strain fatigue-life ratios for the two Idaho aggregate mixtures in Figure 3. The semi-log graph of Figure 3 shows great sensitivity between the two. The tensile-strength ratio calculated from the moisture-damage test is based on a simple arithmetic scale, and the fatigue-life ratio is calculated from a log-log relationship. Field performance could also behave with similar sensitivity. For instance, a tensile-strength ratio of 0.80 could mean that the fatigue-life ratio is no better than 0.15 to 0.40, which

Table 1. Tensile-strength, modulus, and stress fatigue-life ratios for two permeable asphalt concrete mixtures.

Mixture	Moisture Condition	Tensile Strength ^a		Resilient Modulus ^b		Maximum Repeated Tensile Stress at 100 000-Cycle Fatigue Life ^c	
		Psi	Ratio	10 ⁵ psi	Ratio	Psi	Ratio
BK 142 aggregate with AC-10 asphalt; voids, 9.0 percent	Dry	46	1	3.10	1	24	1
	Vacuum saturation	41	0.89	2.34	0.75	9	0.38
	Accelerated conditioning (Severe stripping observed)	14	0.30	0.69	0.22	2	0.08
BK 117 aggregate with AC-10 asphalt; voids, 8.6 percent	Dry	39	1	1.74	1	7	1
	Vacuum saturation	47	1.21	2.64	1.52	24	3.43
	Accelerated conditioning (Slight stripping observed)	34	0.87	1.25	0.72	1	0.14

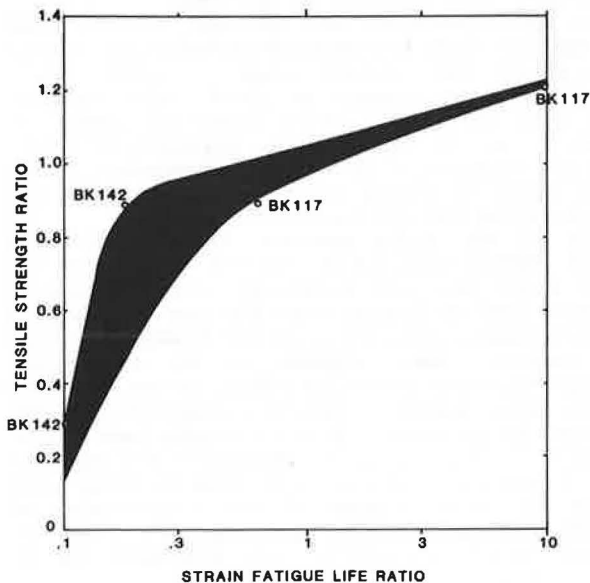
^aTest performed at alternate temperature of 70° F (0.10 in/min) to coincide with fatigue-test temperature.
^bTest performed at 70° F (0.10-s load-pulse-duration time) to coincide with fatigue-test temperature.
^cCalculated from fatigue lives for each moisture condition (see Figures 1 and 2) at 1 x 10⁵ repetitions.

Table 2. Mechanistic-theory tensile stress and strain fatigue-life ratios for two permeable asphalt concrete mixtures.

Mixture	Moisture Condition	Asphalt Concrete Pavement Bending Parameters		Fatigue-Life Ratios ^c	
		Tensile Stress ^a (psi)	Tensile Micro-strain ^b	Tensile Stress	Tensile Strain
BK 142 aggregate with AC-10 asphalt; voids, 9.0 percent	Dry	153	117	1	1
	Vacuum saturation	139	136	7.0	0.18
	Accelerated conditioning (Severe stripping observed)	58	238	4.0	0.07
BK 117 aggregate with AC-10 asphalt; voids, 8.6 percent	Dry	119	158	1	1
	Vacuum saturation	145	128	0.71	10.0
	Accelerated conditioning (Slight stripping observed)	100	185	2.1	0.63

^aThin pavement: 3.6-in asphalt concrete with 9.6-in untreated base.
^bThick pavement: 9.6-in asphalt concrete with 3.6-in untreated base.
^cCalculated from fatigue lives for each type of moisture conditioning (see Figures 1 and 2) by using asphalt concrete bending parameters.

Figure 3. Tensile-strength ratio versus mechanistic-theory strain fatigue-life ratio.



results in a much greater drop in field performance than that indicated by a tensile-strength ratio. A region of large fatigue-life change occurs at tensile-strength ratios between 0.95 and 1.00. Ratios above this range imply much better fatigue life and ratios below this range imply much worse fatigue life as compared with ratios for dry mixtures.

Although a curve similar to that in Figure 3 (or a family of such curves) may exist for dense-graded asphalt concrete, there are not enough data at present to form a reliable correlation. Further testing of other mixtures could produce a wider band of data on tensile-strength ratios and mechanistic fatigue life that would result in an obscure correlation. Tensile-bending strain ratios could be lower or higher than corresponding indirect-tensile-strength ratios, depending on the relative positioning of the dry versus accelerated-conditioning stress fatigue curves (e.g., stress ratio at 100 000 cycles) and of the difference in resilient modulus (e.g., resilient-modulus ratio). As an alternative to mechanistic theory, highway agencies may prefer to use the stress ratio at fatigue life of 100 000 cycles when developing a correlation similar to that of Figure 3. A reliable correlation substantiated by field performance could then be used to predict the reduction of the structural

coefficient of the asphalt concrete pavement layer due to loss of cohesion from moisture damage.

Specific fatigue-life analyses by using moisture-damage ratios, e.g., the tensile-strength ratio, and Figure 3 can be made with a knowledge of asphalt concrete design life, the rate of 18-kip ESALs and field time to reach the equivalencies of vacuum saturation and of accelerated conditioning. For example, suppose the anticipated design life for an asphalt concrete in a new pavement is 4 000 000 18-kip ESALs. At an average rate of 250 000 repetitions per year, this gives 16 years of life in a dry condition. Suppose estimates of field times are 4 years to reach vacuum-saturation equivalency and 6.5 years to reach accelerated-conditioning equivalency and that the laboratory-determined tensile-strength ratios are as follows: 0.95 for vacuum saturation and 0.70 for accelerated conditioning (moderate stripping observed). By using the middle of the band in Figure 3, the corresponding fatigue-life ratios are 0.60 for vacuum saturation and 0.20 for accelerated conditioning. The estimated (non-cracked) asphalt concrete fatigue life might be calculated in a simplified way, as follows:

- (a) Years 1-4, dry: Repetitions = $250\ 000 \times 4 = 1\ 000\ 000$.
- (b) Years 4-6.5, vacuum saturation: Equivalent repetitions = $250\ 000 \times (6.5 - 4) \times 0.60 = 375\ 000$.
- (c) Years 6.5-16, accelerated conditioning: Equivalent repetitions = $250\ 000 \times (16 - 6.5) \times 0.20 = 475\ 000$.

The sum of a + b + c equals 1 850 000 repetitions. At 250 000 repetitions per year, the estimated life is 7.4 years (instead of 16 years). The predicted life would be considered too low. The asphalt concrete mixture should be redesigned or antistripping treatments used in order to achieve higher tensile-strength ratios for longer asphalt concrete fatigue life.

A longer fatigue life will occur in the field if moisture conditions, climate, and traffic produce seasonal cyclic asphalt concrete drying and healing, even if they are only temporary. These conditions will increase the time to reach the ultimate damage predicted by accelerated conditioning. Effects of temporary recovery of moisture damage in the laboratory have been observed (7).

FIELD OCCURRENCE OF PREDICTED MOISTURE-DAMAGE RATIOS

Eight asphalt concrete pavements were evaluated periodically for approximately five years in NCHRP Research Project 4-8(3)/1 (1). These pavements were constructed in 1975 through 1977 by Arizona, Colorado, FHWA Region 10, Idaho, Georgia, Montana, and Virginia highway agencies. A variety of climatic regions and mixtures were represented. Aggregates were generally chosen that had a history of moisture damage when incorporated into asphalt concrete pavement.

Each highway agency selected a 1000-ft evaluation section of the pavements and performed the NCHRP 4-8(3) moisture-damage test method as previously described to obtain moisture-damage predictions. Laboratory specimens incorporated the aggregate and asphalt materials used in the lowest asphalt concrete layer of their respective pavements. In addition, each agency performed the test method on the lowest asphalt concrete layer cores drilled from the pavement sections immediately after paving. Then the predicted tensile-strength and modulus ratios were compared with ratios from the field cores obtained periodically throughout the study. The field ratios were calculated by using vacuum

saturation only because, over a period of time, the natural environmental forces will produce an accelerated-conditioning equivalency on their own and thus would be built in with the measurement of the field vacuum-saturation ratio. One half of each set of periodic cores was desiccated in the laboratory in order to obtain the dry base for the field ratios. If the field-core ratios and stripping became close to the predicted laboratory ratios and stripping, then there would be a good indication that the test method (and its ratios) reasonably predicted the occurrence of moisture damage in the pavements. The following is a summary of the comparison.

Figures 4 through 7 show tensile-strength ratio predictions, and the lines represent the lower running periodic field-core ratios for tensile strength. (Resilient-modulus ratios showed similar trends.) These lines are labeled WP (wheelpath) or BWP (between wheelpath), depending on which core location produced the lower field ratio. Arrows in the figures represent ratios of the strength of saturated final drilled cores to the strength of maximum-dry-strength cores that occurred over the pavement age to date. For this ratio calculation, the reduction of dry-core strength due to moisture damage is minimized and should more closely match the physical nonaging conditions of the laboratory test method used for predictions.

Short-term predictive ratios were determined by vacuum saturation and long-term predictive ratios were determined by accelerated conditioning. Each predictive ratio is identified by highway agency abbreviation and a C or an L in parentheses; C denotes initial core prediction and L denotes laboratory-fabricated specimen prediction.

In most cases, predictive ratios determined from initial cores were greater than the ratios predicted by laboratory-fabricated specimens. Current laboratory mixing and compaction methods seem to be at best a fair estimate of the compacted paved field mixture when moisture damage is considered. Therefore, it appears that moisture damage is overestimated by the use of laboratory-fabricated specimens. However, ratios of tested laboratory specimens are practical predictions that are useful to highway agencies.

The field ratios do not really show a moisture-damage bias for WP or BWP locations. For some pavement sections the BWP location contains more severe moisture damage than the WP location does. For others, this situation is reversed.

For the asphalt concrete pavement mixtures that had low long-term (accelerated-conditioning) predictive ratios that denote severe moisture damage (stripping), the decrease of field ratios occurred soon after an initial two to three-year period of pavement age. At this time the beginning of stripping was observed; later it was accompanied in the worst cases by severe stripping that caused some core disintegration in the field.

Six of the eight pavement sections developed ratios greater than 1.0 during the initial period, which meant that the saturated cores had greater strength (and stiffness) than the dry cores. This was not always predicted from the laboratory-specimen ratios. There appears to be an initial strengthening and stiffening effect in the field due to the early phases of moisture conditioning; some addition to the fatigue life may result (see Figures 2 and 3). However, field predictions for the initial period may be difficult to make by using laboratory specimens because of the complexities of interaction between early moisture conditioning, asphalt aging mechanisms, and aggregate surface reactions.

Figure 4. Periodic field ratios versus prediction ratios for Arizona and Colorado test sections.

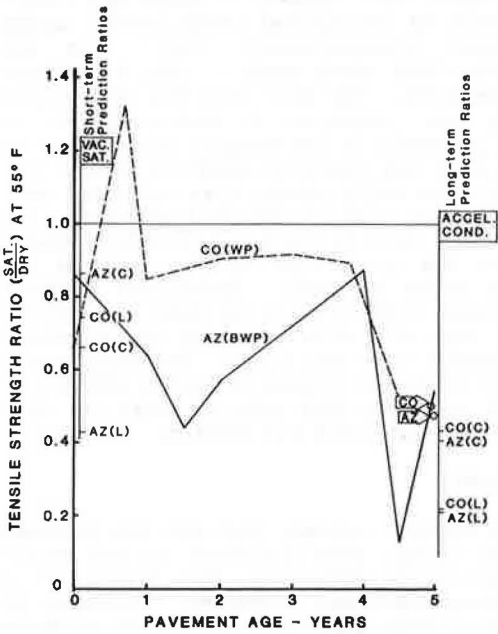


Figure 6. Periodic field ratios versus prediction ratios for Georgia-B and Idaho test sections.

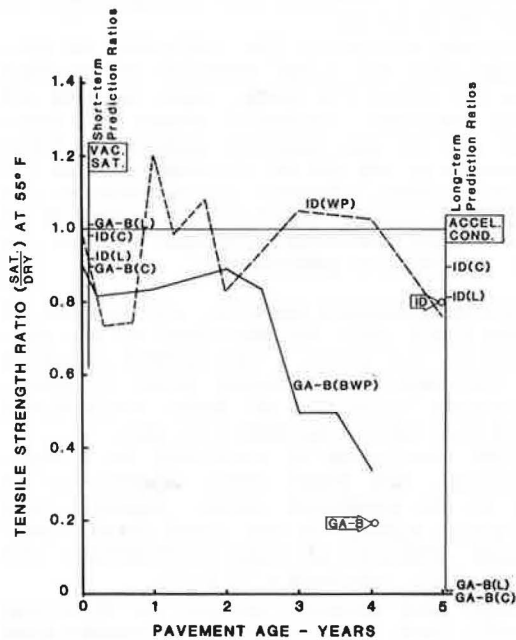


Figure 5. Periodic field ratios versus prediction ratios for FHWA 10 and Georgia-A test sections.

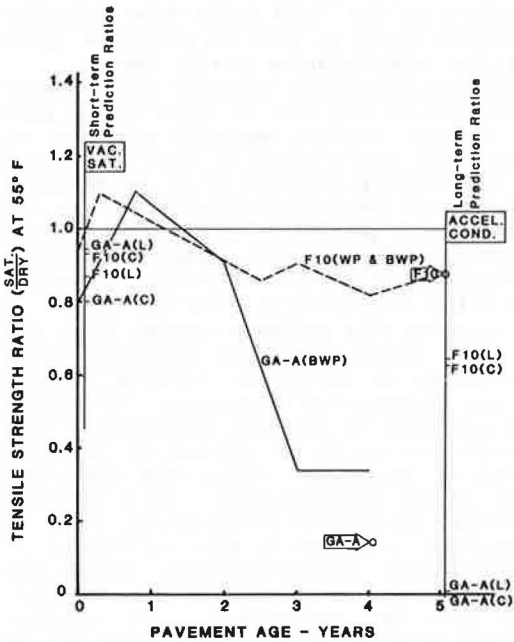
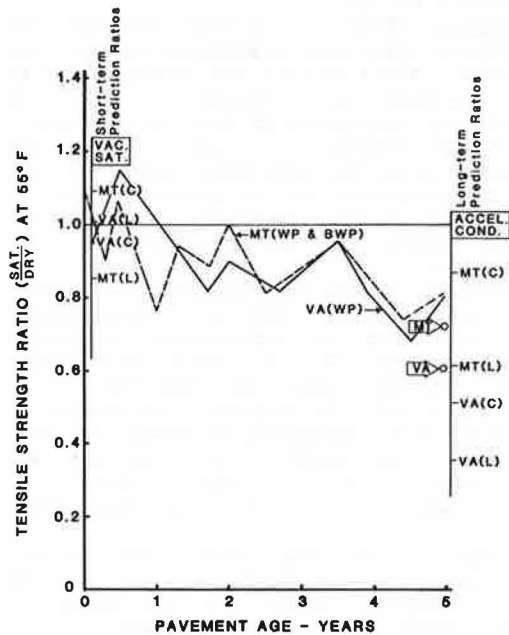


Figure 7. Periodic field ratios versus prediction ratios for Montana and Virginia test sections.



If there are fatigue-life and other benefits derived from the field-conditioning effect, they are overcome by stripping brought on gradually by the environmental moisture forces during the later stages. For those pavement sections for which stripping was predicted by the accelerated conditioning of laboratory specimens, the ratios decreased below 1.0. Stripping was first observed when the field ratios decreased to 0.80 and became more severe as the ratios decreased further. Minimum field ratios (denoted by arrows) provided a good correlation to the stripping severity observed at the end of the field measurement. They show that

the moisture-damage ranking of pavements is essentially the same as the predicted ranking by using the test method's long-term ratios. However, for most of the test sections, the field ratios remain greater than the predicted long-term ratios.

The field-conditioning effect and partial rehealing due to moisture and environmental changes appear to be more responsible than test variability for the zig-zag ratio patterns shown. (Coefficients of variation for tensile-strength and modulus tests of the field cores averaged 15 percent.) For those mixtures for which stripping was predicted, the rate of overall decrease of the field ratios appears

proportional to the heavy traffic volume on the pavements. A correlation of the moisture-damage rate to temperature extremes and precipitation is not apparent at this time.

The following summarizes the comparison of predicted damage with the field moisture damage that occurred at the end of the study. Also included are the highway agencies' estimated change of layer coefficient due to the moisture damage in their asphalt concrete at the end of the study and for the long-term estimation. Numbers in parentheses are the corresponding minimum field ratios at the end of the study and the predicted long-term ratios, respectively, expressed as percentages.

AZ: Although temporary rehealing was apparent at 5 years, the field ratio had decreased to the predicted ratio at 4.5 years. Some severe stripping and core disintegration occurred after 4 years. Percent cohesive retention of layer coefficient: end of study = 42 (48); long-term = 21 (22).

CO: After four years of remaining in a high-ratio condition, the field ratio appears to be decreasing to the predicted ratio. However, only slight stripping appears in the field cores. Percent cohesive retention of layer coefficient: end of study = 87 (50); long-term = 73 (22).

FL0: This FHWA pavement section is in Crater Lake National Park. It is used by recreational vehicles when it is open during the warmer visitor's season; 18-kip ESALS are negligible. The field ratio remains above the predicted ratio at five years with very slight stripping. Percent cohesive retention of layer coefficient: end of study = 100 (90); long-term = 43 (65).

GA-A and GA-B: These two pavement sections show severe stripping; some core disintegration occurred after two years. Field testing was terminated after four years (pavement was constructed in 1977) to coincide with the conclusion of the overall study. Field ratios have decreased to 0.20 or less. The predicted ratio is 0; the additional length of time to reach 0 is uncertain, but in effect it will be reached in a year or two. Percent cohesive retention of layer coefficient: end of study = 50 (19); long-term = 20 (0).

ID: Predictive ratios for this pavement section were high. The five-year field ratio also remains relatively high with no appreciable stripping. Percent cohesive retention of layer coefficient: end of study = 100 (80); long-term = 100 (82).

MT: The field ratio is apparently decreasing to the predictive ratio; stripping is considered moderate to light so far, slightly less than what was ultimately predicted. Percent cohesive retention of layer coefficient: end of study = 64 (72); long-term = 21 (60).

VA: Stripping became severe after three to four years. The field ratio is about 0.60 and is above the predictive ratio. Traffic is not heavy and there are definitely rehealing cycles present in the field, but the field ratio zigzag pattern appears to be downward, probably to reach the predictive ratio at about eight years of pavement age. Percent cohesive retention of layer coefficient: end of study = 62 (62); long-term = 62 (36).

The retention of the layer coefficient, based on the asphalt concrete as a cohesive material versus an untreated gravel, shows a general relationship to the ratios but leans more toward the values due to the stripping observed in the final pavement cores. Considering the possibility of fatigue-life decrease, the estimated decrease of layer coefficients might not be enough for about half of the test sections.

In summary, short-term ratios and stripping, predicted by vacuum saturation, can occur up to four years of pavement age. Long-term ratios and stripping, predicted by accelerated conditioning, appear to be maximum moisture-damage levels that are achievable to field times greater than five years for most pavements. The test method's accelerated conditioning was correlated to damage in 3- to 12-year-old pavements in the earlier study (2). It is expected that the predicted moisture damage will be reached in the field within this variable time. For those pavements with heavier traffic volume, the associated field times to reach the long-term damage should be on the lower side of the pavement age range (eight years or less). Associated pavement surface distress brought about by loss of cohesion and fatigue life will also occur in the moisture-damaged pavements in a few years. The time cannot be accurately predicted because of cyclic changes of moisture and rehealing that interact with the different pavement thicknesses and traffic.

RECOMMENDATIONS

Dense-graded asphalt concrete that has low moisture damage (high ratios) when subjected to the accelerated conditioning withstands the rigors of a reasonable moisture-damage mechanism. Evidence so far shows that these mixtures should have no worse damage or stripping in the field and provide long-term service in the presence of moisture. Conversely, if low ratios and severe stripping are predicted, then they will also occur in the field and these mixtures will not provide long-term field service. Ratios of the field cores from moisture-damaged pavements could be used to determine layer coefficients for rehabilitative overlay design.

The effects of particular climatic and environmental differences on the rate of field damage were uncertain. However, highway agencies may want to use an accelerated conditioning of cold-water-plus-warm-water soak for mild climates if it is warranted by their field and laboratory experience. In the meantime, the freeze portion is recommended for all locations because of the wide range of asphalt concrete materials variables encountered throughout the country and in situations of heavy traffic.

Highway agencies will probably develop their own acceptable ratio limits and visual stripping limits for specifications as acceptance criteria for mix design and for antistripping treatments. If fatigue life is as sensitive to moisture conditioning (and tensile-strength ratio) as it appears to be, then the selection of acceptable ratios and specifications should be done conservatively. It is possible that a developed correlative relationship, shown in a preliminary way in Figure 3, will be helpful in selecting acceptable tensile-strength ratios for dense-graded asphalt concrete. Data for two mixtures show that the mechanistic-theory tensile-strength ratios should not be much less than 1.0 in order to maintain the better dry-condition strain fatigue life in the pavement. (It could be a difficult assignment to improve these mixtures to that level.) The upgrading of asphalt concrete mixtures by the use of additives should be assessed by their ability to increase fatigue life when the additive-incorporated mixtures have been subjected to accelerated conditioning and referenced to the untreated control mixture's fatigue life. This seems to be a practical supplement to the use of tensile-strength ratios by themselves.

ACKNOWLEDGMENT

The major portion of the research reported here was

performed under NCHRP Projects 4-8(3) and 4-8(3)/1. NCHRP Project 4-8(3) was conducted at the University of Idaho; part of the work was performed by Battelle-Northwest and the University of Washington under subcontracts with the University of Idaho. NCHRP Project 4-8(3)/1 was coordinated by the University of Idaho; predictive moisture testing and subsequent periodic testing of field cores from pavement test sections were performed by seven highway agencies: Arizona, Colorado, FHWA Region 10 (Western District Federal Division), Georgia, Idaho, Montana, and Virginia. The continued cooperation and steadfastness of the materials and research personnel at these agencies has been gratefully appreciated throughout the five-year field evaluation study.

The opinions and findings expressed or implied in this paper are mine. They are not necessarily those of the Transportation Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or the individual states participating in the National Cooperative Highway Research Program.

REFERENCES

1. R.P. Lottman. Predicting Moisture-Induced Damage

to Asphaltic Concrete: Field Evaluation Phase. NCHRP, Project 4-8(3)/1, Final Rept., Jan. 1982.

2. R.P. Lottman. The Moisture Mechanism That Causes Asphalt Stripping in Asphaltic Pavement Mixtures. Univ. of Idaho, Moscow, Final Rept., Feb. 1971.
3. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete (Interim Rept.--Field Evaluation Phase). NCHRP, Feb. 1979.
4. R.J. Schmidt. A Practical Method for Measuring the Resilient Modulus of Asphalt-Treated Mixes. HRB, Highway Research Record 404, 1972, pp. 22-32.
5. G.W. Maupin, Jr. Test for Predicting Fatigue Life of Bituminous Concrete. TRB, Transportation Research Record 659, 1977, pp. 32-36.
6. T.W. Kennedy. Pavement Design Characteristics of In-Service Asphalt Mixtures. TRB, Transportation Research Record 659, 1977, pp. 24-32.
7. R.J. Schmidt and P.E. Graf. The Effect of Water on the Resilient Modulus of Asphalt-Treated Mixes. AAPT, Proc. Vol. 41, 1972, pp. 118-162.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Chemistry of Asphalt-Aggregate Interaction: Relationship with Pavement Moisture-Damage Prediction Test

J.C. PETERSEN, H. PLANCHER, E.K. ENSLEY, R.L. VENABLE, AND G. MIYAKE

Relationships were found between fundamental chemical and physical properties of the asphalt-aggregate bond and moisture-induced damage in asphalt pavement mixtures subjected to the Lottman conditioning procedure in National Cooperative Highway Research Program (NCHRP) Field Evaluation Project 4-8(3)/1. The relative tendency of different chemical functional types in asphalts to be strongly adsorbed on aggregate surfaces and their relative displacement from aggregate surfaces by water were determined. The affinity of the aggregates for pyridine-type nitrogen was also determined. For most asphalt-aggregate mixtures of the Lottman-NCHRP study, resistance to moisture-induced damage appeared to be controlled by a number of interrelated variables. These variables must be considered in concert to rationalize pavement moisture damage. The sensitivity of pavement mixtures to moisture-induced damage was explained by considering fundamental physicochemical properties of the asphalt-aggregate bond.

Premature pavement failure attributed to moisture-induced damage has long been recognized, but solutions to the problem have been far from satisfactory. This type of damage, generally believed related to rupture of the adhesive bond at the asphalt-aggregate interface, is a complex phenomenon involving physical and chemical properties of both the asphalt and the aggregate. Moisture damage is also strongly influenced by pavement-mixture morphology and external environmental factors.

A laboratory test method for predicting moisture-induced damage in asphaltic pavements was developed by Lottman (1) during work sponsored by the National Cooperative Highway Research Program (NCHRP). Phase 2 of this program involved a field evaluation to determine test-method predictability. The field evaluation commenced in 1974 under Lott-

man's direction with participation by seven state and federal agencies (2, 3) and was completed in 1981.

Our study reported in this paper was conducted by using materials identical to those used in the field-evaluation pavements. Objectives of our study were (a) to determine physicochemical properties of the asphalt-aggregate interaction as related to pavement moisture damage, (b) to correlate these findings with the predictive results of the Lottman test obtained by the participating agencies, and (c) to evaluate potential moisture-damage test methods developed in our own laboratory. The correlation of Lottman test-method results with actual field performance, which requires consideration of construction and environmental factors, is germane to the NCHRP study but is not considered in this paper.

In our study we have used methods developed in our laboratory for qualitatively and quantitatively determining a number of chemical functional group types in asphalts (4,5), their relative tendency to be adsorbed on mineral aggregate surfaces (6-8), and their relative displacement from aggregate surfaces by water (8). The methods are based on selective solvent desorption of asphalt components from aggregate surfaces followed by functional group characterization of the components by using differential infrared spectrometry and selective chemical reactions. These methods provided fundamental data on the chemistry of the asphalt-aggregate interaction by using microcalorimetry (9) and characterization

of aggregate surfaces by using a pyridine adsorption-desorption technique (10). Fundamental factors affecting the performance properties of the individual pavement mixtures used in the NCHRP field-evaluation project are discussed.

EXPERIMENTAL

Materials

Asphalts and Aggregates

Seven asphalt cements and aggregate lots, each from the same source used in actual pavement construction, were supplied by the following participating state and federal transportation agencies: Arizona, Colorado, FHWA Region 10 (Oregon), Georgia, Idaho, Montana, and Virginia. The Georgia asphalt was evaluated both with and without 0.25 weight percent antistripping additive. Aggregate types are identified as follows by Lottman (2): Arizona--Santa Cruz River gravels; Colorado--Morrison Creek stone coarse aggregate and Platte River (Littleton) fine aggregate; FHWA Region 10--Pole Creek stockpile and Kalamath County with 14 percent blended sand; Georgia--granite gneiss; Idaho--Salmon River gravels; Montana--bench gravels; and Virginia--granite coarse aggregate plus natural sand. Locations and details of pavement construction have been reported (2). For our study, aggregates were crushed, wet-screened without surfactants to 0.85-0.50 and 0.25-0.18 mm (20-35 and 60-80 mesh size), rinsed with distilled water, and dried at 150°C for 24 h prior to use.

Solvents

Reagent-grade solvents were used. Benzene and pyridine were dried by refluxing for 8 h over calcium hydride before distillation through a Vigreux column. Peroxide-free tetrahydrofuran (THF) (11,4) was kept dry by storing over 4-A molecular sieves.

Procedures

Asphalt-Coated Aggregates

The asphalts (63 g) and aggregates (1197 g) representing each field mixture were preheated at 150°C for 1 and 24 h, respectively, prior to mixing. The asphalt-aggregate mixtures were held at 150°C for 4.5 h in an oven, slowly cooled to ambient temperature by shutting off the oven, and then stored in the dark at ambient temperature for an additional 62 h before further treatment.

Isolation of Asphalt Fraction Not Strongly Adsorbed on Aggregate

The asphalt-coated aggregate was transferred as a benzene slurry into a separatory funnel containing a glass-wool plug between the main body of the funnel and the stopcock. The bulk of the asphalt was washed from the aggregate by slowly percolating cold benzene through the aggregate bed without agitation until the benzene was colorless. The benzene extract, concentrated by solvent removal in a rotary film evaporator, was filtered through a 0.9- to 1.4-micron fritted glass funnel before final solvent removal at 92°C in a rotary film evaporator at 0.3 kPa (2 Torr). The absence of benzene in the recovered asphalt fraction was established by the absence of an intense benzene infrared absorption band at 670 cm^{-1} in a carbon disulfide solution.

Isolation of Asphalt Fraction Strongly Adsorbed on Aggregate Surface and Displaced by Water

The benzene-washed aggregate particles from the

above separation were dried overnight at 30°C and 8-13 kPa (60-100 Torr). The dried aggregate particles were transferred into a glass vessel containing sufficient distilled water to cover the aggregate. Pressure within the vessel was slowly reduced to about 0.8 kPa (5 Torr) over a 1.5-h period and the reduced pressure maintained for an additional half hour after gas bubbles had ceased to appear in the water above the aggregate. After restoration to atmospheric pressure, the vessel was subjected to a 21-h freeze cycle at -10 to -12°C followed by a 24-h heat cycle at 60°C. These exposure conditions were similar to those used by Lottman (1-3) for pavement mixtures. The contents of the glass vessel were transferred to a separatory funnel, and the water was drained from the aggregate. Several bed volumes of benzene (1000 ml total) were slowly percolated through the wet aggregate and combined with the water initially drained from the aggregate. The water was distilled from the combined water-benzene mixture by using the water-benzene azeotrope. The water-displaced asphalt fraction was recovered from the benzene solution by concentration, filtration, and solvent removal as described above for the benzene extract.

Isolation of Asphalt Fraction Strongly Adsorbed on Aggregate Surface and Not Displaced by Water

The aggregate particles separated from the benzene-water mixture described above were washed several times with pyridine followed by an 8-h pyridine extraction in a modified Soxhlet extractor (12). The displaced asphalt components were recovered from the pyridine by concentration, filtration, and solvent removal as described above for the benzene extracts.

Functional Group Analyses

Ketones, carboxylic acids, anhydrides, and 2-quinolone types were quantitatively determined by selective chemical reactions and differential infrared spectrometry as previously described (4,5) except that the sample size was reduced from 0.125 to 0.0312 g for strongly adsorbed and water-displaced fractions. Sulfoxide concentrations were determined from the area of the 1030- cm^{-1} infrared absorption band (12). By using the general differential techniques previously described (4), phenols were determined from the differential spectra of sodium hydroxide-treated asphalt versus untreated asphalt by using the phenol-THF solvent hydrogen-bonding infrared band at 3300 cm^{-1} . Pyrroles, which also absorb at 3300 cm^{-1} in THF, were assigned the balance of the 3300- cm^{-1} band not attributed to phenolics. Apparent integrated absorption intensities (b) used for phenolic OH and pyrrolic NH, not previously reported, were 1.54×10^4 and 1.45×10^4 $\text{L mol}^{-1} \text{cm}^{-2}$, respectively. Total nitrogen determinations were made with an Antek Model 720 digital nitrogen detector.

Pyridine-Aggregate Interactions

Aggregates (0.25-0.18 mm, 60-80 mesh) were prepared for analysis by submerging them in pyridine followed by air drying and then by heating the aggregates for two days at 115°C in an air-purged oven. Treated aggregate (0.2500 g) was then placed in an Antek Model 772 microcomputer-controlled pyroreactor coupled to an Antek Model 720 digital nitrogen detector, and the temperature was increased to 900°C at 50°C/min with a 5-min isothermal period at 100°C and 7-min isothermal periods at 150 and 300°C.

Table 1. Selected aggregate properties.

Source	Type ^a	Aggregate Surface Properties						
		Area (m ² /g)	Elements by X-Ray Fluorescence ^b					
			Al	Si	S	K	Ca	Fe
Idaho (no lime)	Salmon River gravels	2.30	M	M	L	L	H	H
FHWA Region 10	Pole Creek stockpile with 14 percent blended sand	1.92	M	M	L	L	H	M
Montana	Bench gravels	0.47	M	M	H	H	M	M
Virginia	Coarse granite	2.61	M	M	M	H	M	M
Colorado	Coarse-Morrison crushed stone; fine-Platte River	1.95	M	M	H	H	M	M
Arizona	Santa Cruz river gravels	3.00	M	M	M	H	M	M
Georgia	Granite gneiss	0.10	M	M	M	H	M	M
Georgia + additive	Granite gneiss	0.10	M	M	M	H	M	M

^aLottman (2, 3).

^bRelative abundance: L = low, M = medium, H = high (abundance levels can be compared for each element among different aggregate systems but not for one element with another of a different type).

Table 2. Selected properties of asphalt-aggregate mixtures.

Source	Lottman's TSR ^a (conditioned/dry)	WST ^b (cycles to failure)	Bonding Energy ^c (mJ/g-min)
Idaho	0.8 (lime)	2 (no lime)	2.8 (no lime)
FHWA Region 10	0.6	5	2.9
Montana	0.6	5	2.1
Virginia	0.4	3-4	4.2
Colorado	0.2	3	0.50
Arizona	0.2	2	1.05
Georgia	0	2	1.7
Georgia + additive	0	2	-

^aTensile-strength ratio of laboratory-fabricated pavement specimens (2, 3).

^bWater-susceptibility test.

^cRate of energy release of the asphalt-aggregate interaction after 3 h in a microcalorimeter at 130°C.

Aggregate and Mix Properties

Aggregate surface areas of the 0.85- to 0.50-mm (20- to 35-mesh) fractions were determined by the BET (Brumauer, Emmett, and Teller) method by using krypton as the absorbate. The relative abundance of the aluminum, silicon, sulfur, potassium, calcium, and iron in the aggregates was determined by X-ray fluorescence on pulverized specimens.

Water-susceptibility test (WST) determinations were made on miniature briquets prepared from 0.83- to 0.42-mm (20- to 35-mesh) aggregates as previously reported (13). Briefly, briquets mounted on a stress pedestal and submerged in water were repeatedly subjected to freeze-thaw-warm-water-soak cycling until the briquets failed from crack propagation or fracture.

Asphalt-aggregate interaction energies (bonding energies) were determined by using a sensitive microcalorimeter (9). The interaction energies are reported as the rate of energy release after 3 h of contact between the asphalt and aggregate at 130°C.

RESULTS

Properties of Aggregates

Selected aggregate properties are shown in Table 1. Surface areas ranged from 3.00 m²/g for the Arizona aggregate to 0.10 m²/g for the Georgia aggregate. Relative concentrations of selected elements in the mineral surfaces were determined by X-ray fluorescence spectroscopy; however, the chemical structural composition of the mineral was not determined. The Idaho and FHWA Region 10 aggregates were notably different from the remaining aggregates, being lower in minerals containing sulfur and potassium and higher in calcium-containing minerals.

Idaho aggregate was highest in iron.

Properties of Asphalt-Aggregate Mixtures

Selected properties of asphalt-aggregate mixtures are shown in Table 2. Values for the Lottman tensile-strength ratios (TSRs) are those obtained by the participating agencies on laboratory-fabricated Marshall specimens before extended aging. TSR is the average tensile strength of a set of specimens after conditioning by vacuum-water saturation, freeze-thaw cycling, and a 24-h warm-water soak at 60°C divided by the average tensile strength of a set of specimens dried in a desiccator to constant weight (1-3). This conditioning procedure was used to predict long-term moisture-induced pavement damage. TSRs for Idaho, FHWA Region 10, and Montana were 0.6 or greater, predicting moisture-resistant mixtures; values for Virginia, Colorado, and Arizona were intermediate; Georgia specimens lost all strength during conditioning.

Results by using the WST developed in our laboratory (13) showed a fair correlation (Table 2) with Lottman's TSR. Our test, which uses a narrow aggregate size range to maximize sensitivity to asphalt bonding properties, subjects a miniature briquet mounted on a stress pedestal under water to repeated freeze-thaw-heat cycling until the briquet fails. A notable exception to the correlation was the Idaho sample (discussed later).

Table 2 also shows the energy of interaction, called bonding energy, of aggregate with asphalt 3 h after mixing as measured with a sensitive microcalorimeter (9). A relationship between the bonding energy and the TSR or cycles to failure in the WST was not apparent.

Analyses of Asphalts and Asphalt Fractions Not Strongly Adsorbed

The concentrations of selected chemical functionality in the original asphalts and in the fractions desorbed from asphalt-coated aggregates with benzene are shown in Table 3. Oxygen-containing functional groups formed by air oxidation, such as ketones (14), carboxylic acids (11), anhydrides (15), and sulfoxides (12,16) were usually found in higher concentrations in the fractions not strongly adsorbed on the aggregate surface than in the original asphalts. This result is attributed to air oxidation during the initial 4.5-h oven aging of the asphalt-aggregate mixtures at 150°C. In some cases, carboxylic acid concentrations are an exception because they are selectively adsorbed on aggregate surfaces (6-8), thus depleting their concentration in the asphalt fraction not strongly adsorbed. As

Table 3. Concentrations of chemical components in original asphalts and their amounts and concentrations in asphalt fraction not strongly adsorbed on aggregate surface.

Source	Percent of Original Asphalt	Concentration (moles/L)							
		Total Nitrogen	Pyrrlic NH	Phenolic OH	Ketones	Carboxylic Acids	Anhydrides	2-Quinolone Types	Sulfoxides
Original Asphalts									
Idaho (no lime)		0.55	0.13	0.06	Trace	0.021	Trace	0.016	0.022
FHWA Region 10		1.07	0.42	0.06	0.03	0.052 ^a	Trace	0.029	0.024
Montana		0.46	0.12	0.02	Trace	Trace	Trace	0.025	0.018
Virginia		0.21	0.04	0.02	0.21	0.014	Trace	0.014	0.010
Colorado		0.53	0.093	0.035	Trace	0.0035	0.001	0.023	0.022
Arizona		0.86	0.38	0.10	0.05	0.018	Trace	0.026	0.022
Georgia		0.31	0.05	0.02	0.04	Trace	Trace	0.017	0.036
Georgia + additive		0.35	0.05	0.02	0.024	0.0026	Trace	0.017	0.014
Fraction Not Strongly Adsorbed on Aggregate Surface									
Idaho (no lime)	98.7	0.55	0.14	0.07	0.19	0.005	0.002	0.021	0.28
FHWA Region 10	98.0	1.03	0.46	0.09	0.46	0.037	0.026	0.026	0.18
Montana	99.3	0.46	0.12	0.06	0.25	0.003	0.007	0.014	0.21
Virginia	98.1	0.20	0.05	0.02	0.35	0.014	0.006	0.014	0.18
Colorado	99.1	0.53	0.08	0.08	0.14	0.006	0.005	0.017	0.29
Arizona	98.4	0.82	0.44	0.08	0.46	0.018	0.018	0.024	0.20
Georgia	99.8	0.31	0.05	0.02	0.25	0.006	0.005	0.010	0.23
Georgia + additive	99.9	0.34	0.05	0.07	0.29	0.009	0.007	0.010	0.21

^a Fifty percent were acid salts.

Table 4. Amounts and concentrations of chemical components in asphalt fraction strongly adsorbed on aggregate surface and their concentration ratios.

Source	Percent of Total Asphalt	Total Nitrogen	Pyrrlic NH	Phenolic OH	Ketones	Carboxylic Acids	Anhydrides	2-Quinolone Types	Sulfoxides	Sum ^a
Concentration of Total Fraction Strongly Adsorbed on Aggregate Surface (moles/L)										
Idaho (no lime)	1.3	0.36	<0.01	<0.01	0.23	0.13	0.07	0.07	0.72	1.5
FHWA Region 10	2.0	0.33	0.07	0.84	0.69	0.45	0.27	0.10	0.31	2.9
Montana	0.72	0.22	0.03	0.20	0.53	0.14	0.12	0.11	0.56	1.5
Virginia	1.9	0.19	<0.01	<0.01	0.35	0.38	0.08	0.05	0.64	1.6
Colorado	0.86	0.10	0.06	0.27	0.26	0.12	0.07	0.10	0.90	1.7
Arizona	1.6	0.42	0.14	0.67	0.36	0.23	0.10	0.06	0.53	2.3
Georgia	0.16	2.39	0.02	0.08	0.29	0.41	0.16	0.10	0.38	3.7
Georgia + additive	0.13	2.78	0.34	0.48	1.8 ^b	0.25	0.20	0.09	0.23	5.7
Concentration Ratio ^c										
Idaho (no lime)		0.65	<0.06	<0.15	1.2	26	4	3	2.5	
FHWA Region 10		0.32	0.15	9.3	1.5	12	10	4	1.7	
Montana		0.48	0.25	3.3	1.0	47	17	8	2.7	
Virginia		0.95	<0.2	<0.5	1.0	27	13	4	3.6	
Colorado		0.18	0.75	3.4	1.9	20	14	6	3.1	
Arizona		0.51	0.32	8.4	0.78	13	6	3	2.7	
Georgia		7.7	0.40	4.0	1.2	68	32	10	1.6	
Georgia + additive		8.2	6.8	6.9	6.2 ^b	28	29	9	1.1	

^a Sum of functional groups less pyrrolic NH and 2-quinolone types that contribute to total nitrogen.

^b Because of the additive, amide and/or ester carbonyl may contribute to this analysis.

^c Concentration in strongly adsorbed fraction divided by concentration in fraction not strongly adsorbed.

indicated in Table 3, more than 98 percent of the asphalt was recovered from all aggregate mixtures by benzene extraction. Hydrated lime was used in the actual Idaho pavement mixture but was inadvertently omitted in our laboratory studies. Beneficial effects of lime in reducing moisture damage have been reported in both laboratory (13) and field studies (17).

Analyses of Strongly Adsorbed Asphalt Fractions, Their Relative Affinity for Aggregate Surfaces, and Their Relative Displacement by Water

Following extraction of the asphalts from the aggregates with benzene, the aggregates were subjected to a water-saturation freeze-thaw procedure to displace strongly adsorbed water-sensitive components from the aggregate surface; the water-sensitive components were subsequently recovered. Finally, the

strongly adsorbed components not displaced by water were recovered from the aggregate surface with refluxing pyridine. Percentages recovered and functional group analyses of the recovered fractions are reported in Tables 4, 5, and 6. Because the entire strongly adsorbed fraction was not isolated as a discrete fraction but as water-displaceable and not-water-displaceable subfractions, data in the upper half of Table 4 were calculated from data obtained on the two subfractions (Tables 5 and 6). The entire strongly adsorbed fraction ranged from 0.13 to 2.0 percent of the total asphalt (Table 4). Amounts of the strongly adsorbed fraction displaceable by water ranged from 1.9 to 9.7 percent (Table 6). The sum of the functional group components of the asphalt found in the strongly adsorbed fraction (Table 4) ranged from 1.5 moles/L for the Idaho and Montana systems to 5.7 moles/L for the Georgia-plus-additive system. Assuming an average molecular

Table 5. Amounts and concentrations of chemical components in strongly adsorbed asphalt fraction that was not displaceable with water.

Source	Percent of Original Asphalt	Concentration (moles/L)							
		Total Nitrogen	Pyrrholic NH	Phenolic OH	Ketones	Carboxylic Acids	Anhydrides	2-Quinolone Types	Sulfoxides
Idaho (no lime)	1.3	0.35	ND ^a	ND	0.23	0.13	0.07	0.07	0.69
FHWA Region 10	1.9	0.32	0.07	0.84	0.70	0.43	0.26	0.10	0.31
Montana	0.71	0.21	0.03	0.20	0.53	0.14	0.12	0.14	0.56
Virginia	1.8	0.19	ND	ND	0.35	0.39	0.08	0.05	0.63
Colorado	0.82	0.10	0.06	0.27	0.26	0.12	0.06	0.10	0.90
Arizona	1.6	0.40	0.14	0.67	0.36	0.24	0.09	0.06	0.50
Georgia	0.14	2.6	0.02	0.08	0.31	0.43	0.17	0.08	0.36
Georgia + additive	0.12	2.9	0.34	0.48	1.8 ^b	0.23	0.21	0.07	0.22

^aNot detected.^bIncludes possible nonketone carboxyl from antistripping additive.

Table 6. Amounts and concentrations of chemical components in strongly adsorbed asphalt fraction that was displaceable with water.

Source	Percent of Total Strongly Adsorbed Asphalt	Concentration (moles/L)							
		Total Nitrogen	Pyrrholic NH	Phenolic OH	Ketones	Carboxylic Acids	Anhydrides	2-Quinolone Types	Sulfoxides
Idaho (no lime)	2.5	0.63	ND	Trace	0.18	0.31	0.27	0.19	1.76
FHWA Region 10	3.0	0.49	ND	ND	0.41	1.12 ^a	0.74	Trace	0.46
Montana	1.9	0.54	ND	ND	0.14	0.40	0.34	0.43	2.63
Virginia	3.8	0.21	ND	ND	0.25	0.53	0.22	0.11	0.89
Colorado	3.6	0.10	ND	ND	0.31	0.25	0.41	0.24	0.92
Arizona	2.7	1.21	ND	0.6	0.24	0.60	0.45	0.45	1.60
Georgia	9.7	0.31	ND	ND	0.10	0.22	0.11	0.30	0.61
Georgia + additive	6.3	1.4	ND	Trace	Trace	0.49	0.16	0.32	0.20

Note: ND = not detected.

^aPlus some insoluble carboxylate salts.

the concentration of a given functional group in the entire strongly adsorbed fraction divided by the corresponding concentration in the fraction not strongly adsorbed. As found earlier (6-8), carboxylic acids and anhydrides have the greatest affinity for aggregate surfaces. Although it seems justified to compare concentration ratios of strongly adsorbed species within a single asphalt-aggregate system, caution should be exercised in comparing concentration ratios from one system to another because concentrations in the initial asphalt, which affect the numerical value of the ratio, vary greatly from one asphalt to another. weight of 700 and a density of 1 for the strongly adsorbed components, a concentration as low as 1.5 moles/L indicates an average of more than one functional group per molecule, virtually assuring poly-functionality for many strongly adsorbed molecular species.

The relative affinity of functional group types for aggregate surfaces is shown in the lower half of Table 4 as the concentration ratio. This ratio is

The water-displacement ratios (8) shown in Table 7 were calculated from data in Tables 5 and 6. These ratios indicate the relative tendencies for the different functional group types in the strongly adsorbed fraction to be displaced from the aggregate surfaces with water. This ratio is the functional group concentration in the water-displaced fraction divided by the corresponding concentration in the strongly adsorbed fraction not displaced with water. Carboxylic acids, anhydrides, and 2-quinolone types in most systems were the functional types most readily displaced with water. Nitrogen compounds (determined as total nitrogen) and sulfoxides also generally showed significant water displacement. It should be noted that concentration values are not a measure of the amounts of material actually displaced.

Pyridine-Aggregate Interactions

The relative concentration density of adsorption sites on aggregate surfaces that strongly adsorb pyridine (amount divided by surface area) and the thermal desorption characteristics of adsorbed pyridine are shown in Table 8. Pyridine is the lowest-molecular-weight member of a nitrogen type found in petroleum. The data reflect the relative amounts of pyridine nitrogen desorbed from pyridine-treated aggregates as the temperature is slowly raised in the oven of a nitrogen analyzer. No pyridine nitrogen remained on the aggregate above 600°C. Montana aggregate showed the highest pyridine adsorption density and Georgia, the lowest. Most of the pyridine nitrogen desorbed from the aggregates appeared in three desorption temperature ranges with maxima at about 140, 280, and 480°C. The relative amounts desorbed at the different temperature ranges varied with aggregate source.

DISCUSSION OF RESULTS

The adhesive bond between asphalt cements and aggregate surfaces and the effect of moisture on this bond are complex phenomena primarily because of the complex nature and variable composition of materials involved. Consequently, tests to predict the moisture sensitivity of asphalt-aggregate mixtures have been largely empirical. Empirical tests, however, do not provide the fundamental information necessary to optimize the material choices or determine corrective measures when loss of the asphalt-aggregate bond is a potential problem. In this study we have attempted to identify fundamental chemical and physical properties of the asphalt-aggregate bond and show how these properties are affected by moisture.

Table 7. Displacement of strongly adsorbed asphalt components by water treatment.

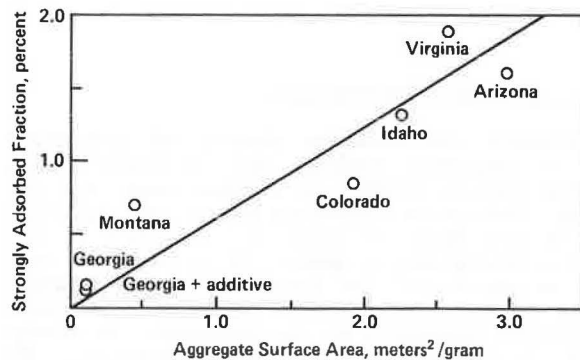
Source	Water-Displacement Ratio ^a							
	Total Nitrogen	Pyrolic NH	Phenolic OH	Ketones	Carboxylic Acids	Anhydrides	2-Quinolone Types	Sulfoxides
Idaho (no lime)	1.8	ND ^b	ND ^b	0.78	2.4	3.9	2.7	2.6
FHWA Region 10	1.5	Low ^c	Low	0.59	2.6	2.8	<0.1	1.5
Montana	2.8	Low	Low	0.26	2.9	2.8	3.1	4.7
Virginia	1.1	ND ^b	ND ^b	0.71	1.4	2.8	2.2	1.4
Colorado	1.0	Low	Low	1.2	2.1	6.8	2.4	1.0
Arizona	3.0	Low	Low	0.09	2.5	5.0	7.5	4.4
Georgia	0.12	Low	Low	0.32	0.5	0.6	3.8	1.7
Georgia + additive	0.48	Low	Low	<0.06	2.1	0.8	4.6	0.9

^aConcentration in fraction displaced by water divided by concentration in fraction not displaced by water.
^bNot detected in either fraction.
^cToo small to be measured.

Table 8. Relative surface density of aggregate adsorption sites and their affinity for pyridine as determined by pyridine desorption temperature profile.

Source	Aggregate Surface Area (m ² /g)	Pyridine Nitrogen Desorbed				
		<100°C	100-150°C	150-300°C	300-600°C	Total
Idaho (no lime)	2.30	1.4	14.3	17.2	8.8	41.7
FHWA Region 10	1.92	1.8	9.3	20.2	14.8	46.1
Montana	0.47	3.4	68.3	30.2	30.8	132.7
Virginia	2.61	0.7	8.5	7.0	1.1	17.3
Colorado	1.95	3.1	20.4	12.5	3.8	39.8
Arizona	3.00	1.1	13.9	13.2	5.2	33.4
Georgia	0.10	0	1.0	0.8	0	1.8

Figure 1. Comparison of aggregate surface area and amount of strongly adsorbed asphalt fraction.



Relationships Between Aggregate Surface Area and Adsorption of Asphalt Components

As shown in Table 4, up to 2 percent of the asphalt in the asphalt-aggregate mixtures was so strongly adsorbed on the aggregate surfaces that it was not extractable with benzene. A previous study (8) indicated that aggregates from benzene-extracted mixtures were coated with approximately a monomolecular layer of asphalt molecules, indicating generally a high surface density of active sites. These results are supported by those of the present study (Figure 1) in which a relationship was found between measured aggregate surface area and the amount of strongly adsorbed components. Because all crushed aggregates were screened to a similar particle-size distribution, variations in the measured aggregate surface areas must necessarily relate to variations in aggregate surface characteristics; i.e., higher aggregate surface area indicates more porous or rough surface texture.

The unusually low surface area of the Georgia aggregate is believed related to the high moisture sensitivity of the Georgia pavement mixture. Its low surface area was similar to that of a highly

crystalline calcite aggregate characterized previously (8). The low surface areas of both aggregates suggest a smooth, crystalline surface with low voids or surface roughness. This was verified by microscopic examination. From purely mechanical considerations, this should reduce adhesive bond strength. Both the Georgia and the calcite aggregate produced highly moisture-sensitive asphalt-aggregate mixtures.

An important comparison of aggregate adsorption characteristics is provided by the Montana and the Colorado aggregates. Although both had nearly the same amounts of strongly adsorbed asphalt components (Table 4), the Colorado aggregate had four times the surface area (Table 1). This result suggests that the Montana aggregate has either a higher surface density of adsorption sites than the Colorado aggregate or that Colorado aggregate adsorption sites, because of steric hindrance, are not accessible. In any event, the density of asphalt components is much higher on the Montana aggregate than on the Colorado aggregate. Pyridine adsorption and desorption data shown in Table 8 support the high density of surface adsorption sites for the Montana aggregate. Based on the above discussion, one is led to conclude that both chemical and physical properties of the aggregate surfaces play an important role in determining the adhesive properties of the asphalt-aggregate bond.

Relative Affinity of Aggregate Surfaces for Asphalt Components

The varying composition of the strongly adsorbed fraction among asphalt-aggregate systems is demonstrated by the data in Table 4. The functional group types determined are representative of the major polar types initially present in asphalts or formed during pavement preparation and subsequent aging. Although the determinations were made on components recovered from aggregates following a benzene extraction of the asphalt-aggregate mixtures, reasoning suggests that polar components were present in high concentration on the aggregate surfaces prior to the benzene-extraction step. Most asphalts have a large excess of adsorbable species

over that needed to saturate the aggregate adsorption sites. High temperatures during pavement-mixture preparation should allow the more polar species to equilibrate with the aggregate surface and displace the more weakly adsorbed or nonpolar components that may have been initially adsorbed. Furthermore, multilayer adsorption (9), which might interfere with the adsorption of polar components at the asphalt-aggregate interface, has not been observed in our laboratory in microcalorimetric studies of nonpolar asphalt fractions. The ability of polar asphalt components formed during oxidative aging to migrate to the aggregate surfaces at pavement mixing temperatures has been demonstrated by inverse gas-liquid chromatography studies (18). In any event, the distribution of strongly adsorbed polar components on aggregate surfaces following benzene extraction serves to characterize the adsorption properties of the aggregate with regard to asphalt components.

The high sum of polar group concentrations in the strongly adsorbed fraction (Table 4) indicates that molecules in this fraction, as previously explained, probably contain more than one functional group and/or that adsorbed molecules are of unusually low molecular weight. Polyfunctionality complicates the determination of adsorption characteristics of adsorbed species, making it difficult to determine the relative contribution to adsorption of different functional groups on the same molecule. This problem can often be partly resolved by knowledge of the chemical properties of the different functional types and general trends observed when comparing similar data on different systems. The high sum of functional group concentrations for the Georgia systems (3.7 and 5.7 moles/L) is attributed largely to the high nitrogen content.

The relative affinity of the aggregate surfaces for the different chemical functional types in each pavement mixture can be judged from the concentration ratios (lower half of Table 4). It becomes apparent, except for the Georgia-plus-additive system, that carboxylic acids are the functional types most strongly adsorbed by all aggregates. The lower value for Georgia-plus-additive than for Georgia alone probably results from competition by the antistripping agent for potential carboxylic acid adsorption sites. The high concentration ratios for carboxylic acids cannot be overemphasized. Carboxylic acids often occur in only trace amounts in asphalts (Table 3) but are usually significant contributors to the strongly adsorbed fraction (Table 4). Phenols, anhydrides, sulfoxides, and 2-quinolone types are also significantly concentrated on the aggregate surface. Notable exceptions are the phenols on the Idaho and Virginia aggregates. These exceptions may be more apparent than real because of possible inability of the solvent to displace phenols from these aggregates; phenols were present in the original asphalt. The relatively low concentration ratios for sulfoxides and ketones do not necessarily reflect correspondingly low contributions to the strongly adsorbed fraction but often result from their relatively high initial concentrations in the asphalts.

Low concentrations of pyrrolic NH in the strongly adsorbed fractions were expected because these types form weak hydrogen bonds (19) and would not be expected to associate strongly with most mineral adsorption sites. The high pyrrolic concentration in the Georgia-plus-additive system probably results from the antistripping additive. Other nitrogen types such as the pyridine type, which have been shown to interact strongly with aggregate surfaces (13, 20, and Table 8 of this paper), probably largely account for the strongly adsorbed nitrogen types.

The high concentration of nitrogen in many of the original asphalts and/or the low density of nitrogen adsorption sites on the aggregates may in part account for the low total nitrogen concentration ratios. The unusually high concentration attributed to ketones for the Georgia-plus-additive system was indicated from detailed infrared analyses to result largely from amides and/or esters from the antistripping agent that were concentrated on the aggregate surface. The high concentration of ketones in the FHWA Region 10 system is noteworthy since ketones have been shown to resist displacement from the aggregate surfaces by water (8).

X-ray fluorescence analysis (Table 1) showed that Idaho and FHWA Region 10 aggregates were lower in sulfur and potassium and higher in calcium than the other aggregates, suggesting that these aggregates produce more moisture-resistant mixtures for the following reasons. Sulfur-containing inorganic compounds (probably sulfates) and potassium ions often readily associate with water, which could disrupt the asphalt-aggregate bond. Also, aggregate treatment with a hydrated lime slurry (calcium hydroxide) is known to improve moisture resistance of many aggregates, presumably from the interaction of asphalt components with the calcium.

An inescapable conclusion resulting from analyses of the strongly adsorbed fractions is that asphalt-aggregate interaction chemistry is highly complex and variable among different asphalt-aggregate systems and that the interactions are a function of both asphalt and aggregate properties. This conclusion is supported by asphalt-aggregate interaction energies (bonding energies) as measured by microcalorimetry (9); these interactions are highly asphalt- and aggregate-dependent, as illustrated in Table 2. High interaction energies, proposed to result from multilayer adsorption of asphalt components on aggregate surfaces (21), should be related to bond-strength properties. A high interaction energy may not, however, be a necessary condition for resistance to moisture damage in asphalt-aggregate mixtures.

Water Displacement of Asphalt Components from Aggregate Surfaces

The ability of water to displace strongly adsorbed components from aggregate surfaces should significantly affect the sensitivity of asphalt-aggregate mixtures to moisture damage. Fundamental studies in our laboratory (8) have shown that the water desorption characteristics of the strongly adsorbed fraction are dependent both in quantity and quality on individual mixture characteristics. The systems of the present study were no exception. The Montana aggregate had the highest density of pyridine adsorption sites (Table 8) and the least amount of the strongly adsorbed fraction displaced by water (1.9 percent, Table 6). The Georgia aggregate, on the other hand, had the lowest density of pyridine adsorption sites and the highest amount of strongly adsorbed fraction (9.7 percent) displaced by water. Based on these facts alone, moisture-damage resistance should be high for the Montana system and low for the Georgia system. Colorado aggregate, with nearly the same amount of strongly adsorbed fraction as Montana aggregate, showed twice the quantity of water-displaced material; thus, based on this consideration, the Montana system would be rated superior to the Colorado system.

The relative tendency for the various chemical functional groups to be displaced by water, as indicated by the water-displacement ratio, is shown in Table 7. These ratios should be considered together with the relative amounts of strongly

adsorbed fraction displaced by water in assessing moisture-damage potential of pavement mixtures. In general, those functional types most highly concentrated in the strongly adsorbed fraction, with the exception of phenols, were most readily water-displaced. This result is consistent with previous work (8). Thus, high concentrations of these groups on aggregate surfaces may contribute to the moisture sensitivity of mixtures. Carboxylic acids, anhydrides, and 2-quinolones showed the highest displacement ratio, probably because these functional group types and the aggregate surfaces both have a high affinity for water via hydrogen bonding. Sulfoxides, major components of the strongly adsorbed phase, were generally selectively displaced by water, as was total nitrogen. Many nitrogen types may be present; those types more readily displaced by water are probably weakly adsorbed or sterically hindered. The low water-displacement ratio of pyrrolic nitrogen together with low original adsorption (Table 4) suggest that the pyrroles in the strongly adsorbed fraction may exist in polyfunctional molecules and are not adsorbed via the pyrrolic functional group alone. The high resistance of ketones and phenolics to water displacement together with their relatively high concentrations in the strongly adsorbed fraction suggest that they should contribute to increased moisture-damage resistance.

Interactions of Pyridine with Aggregate Surfaces--Model Studies

Pyridine treatment of a moisture-sensitive aggregate (13) and the addition to petroleum asphalt of pyridine-type weak bases from shale oil (20) significantly improved the resistance of corresponding asphalt-aggregate mixtures to moisture damage in the water-susceptibility test. Asphalts prepared from shale oil residues also produced water-resistant pavement mixtures (22). Because pyridine-type functionality may exist in petroleum asphalts and strongly adsorb on aggregate surfaces, the stability of pyridine-aggregate bonds was investigated. Preliminary studies of the pyridine aggregate bond have been reported (10).

Data in Table 8 show that even at temperatures above 300°C, significant amounts of pyridine (boiling point, 115°C) remained strongly adsorbed to the aggregates; strong chemisorption is indicated. A high aggregate surface density for adsorbed pyridine resistant to desorption at the higher temperature ranges suggests that resistance to moisture should be enhanced when the aggregate is used with asphalt that has a high availability of unhindered pyridine-type nitrogen. Montana aggregate showed the highest surface density of adsorbed pyridine at the higher temperatures. FHWA Region 10 aggregate also showed a relatively high adsorption density at high temperatures. Georgia aggregate had almost no affinity for pyridine and the small amount that was adsorbed was desorbed below 150°C.

The Colorado and the Arizona aggregates (Table 8) showed a greater affinity for pyridine than the Virginia aggregate. Colorado and Arizona asphalts also had higher total nitrogen contents than the Virginia asphalt (Table 3). However, the amount of strongly adsorbed nitrogen was lower for the Colorado system than for any other system (Table 4), and nitrogen compounds displaced by water were greatest for the Arizona system. These data indicate that the nitrogen compounds in the Colorado and the Arizona asphalts are less effective than those in the Virginia asphalt in forming strong, pyridine-type bonds with the aggregate and thus are less effective in reducing mixture sensitivity to mois-

ture damage. Differences in nitrogen types and/or steric hindrance of the nitrogen atom partly explain these results.

Rationalization of Fundamental Asphalt-Aggregate Interaction Data with Moisture-Induced Damage Observed in Laboratory Pavement Mixtures

It is apparent from the data presented that pavement sensitivity to moisture damage is probably a function of many interrelated variables. Thus, no one factor may be identifiable as the cause of moisture-induced failure except where a factor may dominate. Therefore, the various factors involved must be considered in concert to make a judgment of their net effect. In this section, significant data are summarized for the individual systems and related to moisture-induced damage in the laboratory-prepared pavement mixtures of the Lottman-NCHRP study. The systems are discussed in increasing order of sensitivity of moisture-induced damage.

Idaho

The high TSR (0.8) for the Idaho mixture is predictive of high moisture resistance. The high concentration of nitrogen compounds in the strongly adsorbed fraction should promote moisture resistance. The potential for stable, high-quality aggregate bonds with nitrogen compounds is suggested by the significant proportion of pyridine-aggregate bonds that are stable at high temperatures. The moderately low carboxylic acid, anhydride, and 2-quinolone-type concentrations in the strongly adsorbed fraction together with their moderate water-displacement ratios should promote moisture resistance, as might also the high calcium and low potassium contents of the aggregate. Although lime was used in the Idaho pavement, it was omitted in our studies; this omission may in part be responsible for the failure of our miniature briquet specimen in the water-susceptibility test after two cycles. In the absence of lime, adsorption data suggest a system with moisture resistance somewhere between that of the Montana and that of the Virginia systems.

FHWA Region 10

A number of factors that should promote pavement moisture-damage resistance are prominent with this system. These include the highest concentration of ketones and phenolics in the strongly adsorbed fraction of any system studied; these types are resistant to water displacement. Strongly adsorbed nitrogen is moderately high. The high concentration of nitrogen in the original asphalt should promote adsorption of nitrogen types most strongly adsorbed. An abundance of high-quality adsorption sites for nitrogen is shown by the resistance to displacement of more than 75 percent of the pyridine from pyridine-treated aggregate at temperatures above 150°C--more than for any other aggregate. The high carboxylic acid concentration in the strongly adsorbed fraction, which could promote moisture sensitivity, is apparently more than offset by the many positive factors cited above to produce a moisture-resistant pavement mixture, as indicated by the TSR of 0.6.

Montana

A number of prominent features of the Montana system (TSR = 0.6) also suggest moisture resistance. Probably the most dominant feature is the unusually high surface density of aggregate adsorption sites for both pyridine and the strongly adsorbed fraction. The aggregate also showed a high percentage

of pyridine adsorption sites with high thermal stability. The Montana system also had the lowest percentage of the strongly adsorbed phase displaceable by water. Finally, the concentration of potentially moisture-sensitive carboxylic acids and anhydrides was relatively low in the strongly adsorbed fraction.

Virginia

With a TSR of 0.4, the Virginia system should have intermediate resistance to moisture-induced damage. Our data support this prediction with a balanced array of factors suggesting both increased and decreased moisture resistance. Moisture resistance should be increased by the low acid and anhydride concentrations in the strongly adsorbed fraction and by their low water-displacement ratios. Offsetting these factors is the low amount of nitrogen in the strongly adsorbed fraction. Although the water-displacement ratio for nitrogen was low (increases moisture resistance), this factor was offset by a low pyridine surface density of adsorption sites. The amount of strongly adsorbed fraction and the aggregate surface area were high, which promoted water resistance. However, bond quality may be poor, as suggested by the relatively high displacement of the strongly adsorbed fraction by water.

Colorado

The low TSR of 0.2 for the Colorado mixture predicts a system with a significant potential for moisture-induced damage; a number of factors support this prediction. The ratio of aggregate surface area to the amount of strongly adsorbed fraction was high--four times greater than that for the Montana system--indicating reduced reactivity of the aggregate surface for asphalt components. In addition, the nitrogen content of the strongly adsorbed fraction was lower than that for any system, even though nitrogen content of the asphalt was moderately high, indicating a low interaction of nitrogen compounds with the aggregate. Because of the moderate interaction of the Colorado aggregate with pyridine nitrogen, these data suggest that the nitrogen compounds in the Colorado asphalt, for either chemical or physical reasons, are relatively unreactive with the aggregate. Thus, both asphalt and aggregate possess properties that could contribute to moisture sensitivity.

Arizona

Like the Colorado system, the TSR of 0.2 for the Arizona system indicates sensitivity to moisture damage. The potential benefits of high aggregate surface area and high nitrogen content of both the asphalt and the strongly adsorbed fraction appear to be more than offset by the ability of water to displace polar functional groups from the aggregate surface. The amount of nitrogen compounds in the strongly adsorbed fraction and their corresponding water-displacement ratio were higher for the Arizona system than for any other system. Further, concentrations and water-displacement ratios for carboxylic acids, anhydrides, 2-quinolones, and sulfides were also high. In addition, the Arizona system was the only system showing significant amounts of phenolics displaced by water. In summary, this system's moisture sensitivity seemed to result largely from the combined effect of water disruption of a wide variety of chemical functional groups forming the asphalt-aggregate bond.

Georgia and Georgia-Plus-Additive

The high moisture sensitivity of the Georgia sys-

tems, as predicted by a TSR of zero, was supported by our fundamental studies. The extremely low aggregate surface area suggests a smooth, crystalline surface with poor mechanical bonding properties. The strongly adsorbed fraction was several times more sensitive to water displacement than that of any other system. The aggregate surface showed almost no affinity for pyridine, which suggests a surface unable to form moisture-resistant bonds. Although the nitrogen-containing antistripping agent added to the one Georgia system was found concentrated at the aggregate surface and did displace some carboxylic acids from the strongly adsorbed fraction, the poor bonding properties of the Georgia aggregate may have rendered the additive relatively ineffective in preventing moisture damage.

SUMMARY

Relationships were found between fundamental chemical and physical properties of the asphalt-aggregate bond and moisture-induced damage in laboratory-prepared pavement mixtures that were subjected to accelerated conditioning by using the Lottman procedure. For most mixtures of the Lottman-NCHRP study, resistance or susceptibility to moisture-induced damage appeared to be controlled by a number of interrelated variables. These variables must be considered in concert to rationalize pavement moisture damage with fundamental bond properties. The moisture sensitivity of the pavement mixtures in the Lottman-NCHRP study, as predicted by the TSR ratios, was explained by considering fundamental properties of the asphalt-aggregate bond.

ACKNOWLEDGMENT

Partial financial support by the Federal Highway Administration is greatly appreciated. Asphalt and aggregate samples were supplied by the highway departments of the following states and agency: Arizona, Colorado, Georgia, Idaho, Montana, Virginia, and FHWA Region 10. Chee-kin Chow provided valuable laboratory assistance. Discussions with R.P. Lottman of the University of Idaho and his cooperation during this study are gratefully acknowledged.

REFERENCES

1. R.P. Lottman, R.P. Chen, K.S. Kumar, and L.W. Wolf. Laboratory Test System for Prediction of Asphalt Concrete Moisture Damage. TRB, Transportation Research Record 515, 1974, pp. 18-26.
2. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete--Progress Report on Field Evaluation Phase of NCHRP Project 4-8(3)/1. AASHTO 66th Annual Meeting, Las Vegas, NV, Nov. 16-19, 1980.
3. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete--Field Evaluation Phase. NCHRP, Project 4-8(3)/1, Summary Interim Rept., Feb. 1979.
4. J.C. Petersen. A Quantitative Method for the Determination of Compound Types in Asphalts Absorbing in the Carbonyl Region of the Infrared Spectra. Analytical Chemistry, Vol. 47, 1975, pp. 112-117.
5. J.C. Petersen and H. Plancher. Quantitative Determination of Carboxylic Acids and Their Salts and Anhydrides in Asphalts by Selective Chemical Reactions and Differential Infrared Spectrometry. Analytical Chemistry, Vol. 53, 1981, pp. 786-789.
6. J.C. Petersen, S.M. Dorrence, E.K. Ensley,

- F.A. Barbour, R.V. Barbour, and W.E. Haines. Paving Asphalts: Chemical Composition, Oxidative Weathering, and Asphalt-Aggregate Interactions: Part II. FHWA, Interim Rept. FHWA-RD-74-71, June 1974.
7. J.C. Petersen, E.K. Ensley, and F.A. Barbour. Molecular Interactions of Asphalt in the Asphalt-Aggregate Interface Region. TRB, Transportation Research Board Record 515, 1974, pp. 67-78.
 8. H. Plancher, S.M. Dorrence, and J.C. Petersen. Identification of Chemical Types in Asphalts Strongly Adsorbed at the Asphalt-Aggregate Interface and Their Relative Displacement by Water. Proc., AAPT, Vol. 46, 1977, pp. 151-175.
 9. E.K. Ensley. A Study of Asphalt-Aggregate Interactions and Asphalt Molecular Interactions by Microcalorimetric Methods: Postulated Interaction Mechanism. Journal of the Institute of Petroleum, Vol. 59, 1973, pp. 279-89.
 10. H. Plancher, C. Chow, S.A. Holmes, and J.C. Petersen. Moisture-Induced Damage in Bituminous Pavement--A Study of Nitrogen Compound Interactions with Aggregates. Proc., International Symposium, Progressi Nella Tecnologia Dei Bitumi, Stazione Sperimentale per i Combustibili, Viale A. Gasperi 3, 20097 San Donato Milanese (Milano), Italy, 1981, pp. 263-273.
 11. J.C. Petersen, R.V. Barbour, S.M. Dorrence, F.A. Barbour, and R.V. Helm. Molecular Interactions of Asphalt: Tentative Identification of 2-Quinolones in Asphalt and Their Interaction with Carboxylic Acids Present. Analytical Chemistry, Vol. 43, 1971, pp. 1491-1496.
 12. H. Plancher, E.L. Green, and J.C. Petersen. Reduction of Oxidative Hardening of Asphalts by Treatment with Hydrated Lime--A Mechanistic Study. Proc., AAPT, Vol. 45, 1976, pp. 1-24.
 13. H. Plancher, G. Miyake, R.L. Venable, and J.C. Petersen. A Simple Laboratory Test to Indicate the Susceptibility of Asphalt-Aggregate Mixtures to Moisture Damage During Repeated Freeze-Thaw Cycling. Canadian Technical Asphalt Association, Vol. 25, 1980, pp. 247-262.
 14. S.M. Dorrence, F.A. Barbour, and J.C. Petersen. Direct Evidence for Ketones in Oxidized Asphalts. Analytical Chemistry, Vol. 46, 1974, pp. 2242-2244.
 15. J.C. Petersen, F.A. Barbour, and S.M. Dorrence. Identification of Dicarboxylic Anhydrides in Oxidized Asphalts. Analytical Chemistry, Vol. 47, 1975, pp. 107-111.
 16. J.C. Petersen, S.M. Dorrence, M. Nazir, and H. Plancher. Oxidation of Sulfur Compounds in Petroleum Residues: Reactivity-Structural Relationships. Division of Petroleum Chemistry, American Chemical Society, Preprints, Vol. 26, No. 4, 1981, pp. 898-906.
 17. C.V. Chachas, W.J. Liddle, P.E. Peterson, and M.L. Wiley. Use of Hydrated Lime in Bituminous Mixtures to Decrease Hardening of Asphalt Cement. Materials and Test Division, Utah State Department of Highways, Salt Lake City, Final Rept., Dec. 1971. NTIS: PB 213 170.
 18. F.A. Barbour, R.V. Barbour, and J.C. Petersen. A Study of Asphalt-Aggregate Interactions Using Inverse Gas-Liquid Chromatography. Journal of Applied Chemistry and Biotechnology, Vol. 24, 1974, pp. 645-654.
 19. J.C. Petersen. An Infrared Study of Hydrogen Bonding in Asphalt. Fuel, Vol. 46, 1967, pp. 295-305.
 20. H. Plancher, G. Miyake, and J.C. Petersen. Shale Oil Products as Replacements for Petroleum Counterparts in Pavement Applications. Proc., 13th Oil Shale Symposium, Colorado School of Mines, April 16-18, 1980, pp. 261-268.
 21. E.K. Ensley. Multilayer Adsorption with Molecular Orientation of Asphalt on Mineral Aggregate and Other Substrates. Journal of Applied Chemistry and Biotechnology, Vol. 25, 1975, pp. 671-682.
 22. J.W. Button, J.A. Epps, and B.M. Gallaway. Laboratory Evaluation of Selected Shale Oil Asphalts in Paving Mixtures. U.S. Department of Energy, Rept. LERC-3695-1, Jan. 1978.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Silane Pretreatment of Mineral Aggregate to Prevent Stripping in Flexible Pavements

JOSEPH A. DIVITO AND GENE R. MORRIS

Debonding of asphalt from mineral aggregates (stripping) was termed a problem as far back as 1938, yet it continues to plague the paving industry today. Commercial antistrip additives are available, but their long-term effects are not well understood. A silane coupling agent was compared with a well-known commercially available liquid antistrip agent (amine) in the immersion-compression and double-punch debonding tests on two Arizona mineral aggregate sources. The silane was used as a mineral aggregate pretreatment, whereas the amine was added to the asphalt. The results of this research are encouraging and indicate that the silane generally performed as well as the liquid antistrip agent or better. Further testing is recommended along with construction of experimental projects to evaluate field performance.

An asphalt concrete pavement is inherently dependent on the cohesive and adhesive characteristics of the binder to hold it together. As a result, the bond

between the asphalt binder and the mineral aggregate is of special importance. It is critical that a good bond be developed during construction and maintained for the life of the pavement. Any degree of loss of the asphalt-aggregate bond will result in a corresponding loss of pavement performance in one manner or another. The strength of an asphalt concrete mixture is a result of the cohesive resistance of the binder, the adhesive bond between the binder and the aggregate, the aggregate interlock, and the frictional resistance between aggregate particles.

Several methods have been used to limit the possibility of stripping. Some of the more common methods are as follows:

1. Addition of dry lime or portland cement in small percentages to the mix or lime-slurry treatment of mineral aggregate;
2. Precoating aggregates with bitumen or diluents prior to asphalt concrete production;
3. Addition of selected natural mineral fillers;
4. Disallowance of known hydrophilic aggregates;
5. Washing, wasting, or blending of aggregates; and
6. Addition of chemical antistripping agents.

All these methods, for one reason or another, are not always acceptable or economical in every situation.

Under certain circumstances, an asphalt binder will separate from the aggregate, a complex phenomenon known as debonding (commonly referred to as stripping). Debonding is a function of the environmental conditions, traffic loading, binder and aggregate characteristics, mixture properties, and more. However, it is generally agreed that the mechanism of debonding is the intrusion of water between the adherends. Even though a proper bonding of the asphalt to the aggregate may have taken place during construction, debonding is still possible. Water intrusion is the mechanism that will facilitate debonding by replacing the asphalt coating on mineral aggregates. Since water in one form or another will always be present in a pavement, stripping is always a possibility. An extensive study of debonding is available in the literature (1-16). The intent of this research project was to evaluate and compare an organofunctional silane as a mineral aggregate pretreatment with an amine liquid antistripping agent to determine whether the silane should be considered a practical antistripping treatment. It has been reported that amidoamine and imidoazoline antistripping compounds may actually increase "emulsion" formation at the aggregate-asphalt interface (1). These additives actually enable a better coating to be placed on the aggregate particles initially but could lead to accelerated stripping later. Emulsion formation is one mechanism by which stripping can occur; however, it is not an accurate description of the stripping mechanism. Although it is true that

the term "emulsion" can be applied to any asphalt-water mixture, it may cause confusion when used to describe the stripping mechanism.

In 1976, the Arizona Department of Transportation (ADOT) undertook a limited in-house study to evaluate liquid antistripping additives as well as alternative methods of preventing debonding. It was apparent then, and still is now, that the long-term effects of these additives are not well understood. Many of our new pavements exhibited asphalt stripping 6-12 months after construction even though a commercial antistripping agent was used. A simple but significant test program was initiated. Three aggregate materials from different sources were tested with each of two different commercial antistripping compounds and Dow-Corning Z-6020 organofunctional silane in the immersion-compression test (AASHTO T165).

The commercial antistripping compounds tested were Edoco and Pavebond Special. Each was added to the asphalt in the amount of 1 percent by weight of the asphalt. The silane was applied to the mineral aggregates as a pretreatment and allowed an ambient cure of 24 h before mixing. Two different silane-in-water solutions were tested: 1.5 and 2 percent. These solutions were applied to the dry mineral aggregate at the rate of 3 percent by weight of the mineral aggregate.

The results were encouraging. The silane pretreatment solutions imparted a better retained strength in the immersion-compression test in every case but one. The complete results are shown in Table 1.

Based on these results, a second project was initiated and a consultant was hired to run further tests to confirm the effect that the silane solution had on the retained strength of an asphalt concrete mix. This time two different material sources were selected for testing. Furthermore, the double-punch test procedure (3) was to be used in addition to the immersion-compression test to see whether the results were test dependent. It was decided that Pavebond Special and Dow-Corning Z-6020 silane were to be tested. Pavebond Special was chosen because of its widespread use in Arizona.

THEORY

It should be noted that after considerable work by ADOT in testing silane as an antistripping agent, it was discovered that previous work involved the use of silane coupling agents in asphalt concrete mixes both as an antistripping additive and as an aggregate pretreatment and is documented in a patent held by Chevron. However, silanes are not now, nor have they ever been, marketed as antistripping agents.

The patent describes dramatic asphalt retention by aggregates even when treated with as little as one part per million. Furthermore, it was reported that a tenfold increase in retained asphalt (on the surface of the aggregate) was demonstrated after a water-immersion test of 180°F for eight days. This work verified our early work and encouraged further testing. Significant favorable results were achieved with the silane as a mineral aggregate pretreatment and as an asphalt additive. This report addresses only the use of silanes as a mineral aggregate pretreatment.

DESCRIPTION OF TESTING

In March 1980, a proposal was submitted to the Federal Highway Administration to compare Dow-Corning Z-6020 and Pavebond Special as antistripping agents. R.A. Jimenez of the University of Arizona was commissioned to prepare and evaluate 240 asphalt con-

Table 1. Immersion-compression test: preliminary data.

Compressive Strength (psi)		Retained Strength ^a (%)	Treatment
Dry	Wet		
Aggregate Source 1: Pit 8567 Zuni			
353	118	33	None
	238	67	1 percent Edoco Anti-Strip
	326	92	1 percent Pavebond Special
	227	78	1.5 percent silane
	296	84	2.0 percent silane
Aggregate Source 2: Pit 8500 Globe			
287	119	41	None
	163	57	1 percent Edoco Anti-Strip
	237	83	1 percent Pavebond Special
	325	113	1.5 percent silane
	325	113	2.0 percent silane
Aggregate Source 3: United Metro No. 11 Yuma			
375	68	18	None
	125	33	1 percent Edoco Anti-Strip
	190	51	1 percent Pavebond Special
	207	55	1.5 percent silane
	264	70	2.0 percent silane

^a(Wet compressive strength/dry compressive strength) x 100 percent.

crete specimens by two different test methods: the double-punch debonding test and the immersion-compression test.

Immersion-Compression Test

A measure of resistance to debonding was obtained with the immersion-compression test (AASHTO T165). The AASHTO procedure was followed except that enough material was mixed at one time to produce three specimens instead of one. This change was necessary to assure that all specimens of a set had received the same chemical treatment. Work in the laboratory has shown variability in density, and strength measurements have met the usual requirements when enough material is mixed at one time to produce these specimens. After mixing at approximately 140°C, enough mixture was taken to produce a specimen 101 mm by 101 mm after compaction at 121°C. Following compaction, the set of six specimens was placed in a 60°C oven for 24 h. After the 24 h of curing and then cooling, specimens were weighed in air and submerged in water for density calculations. The six specimens were divided into two groups of nearly equal average density. One group was submerged in hot distilled water at 60°C for 24 h and the second group was stored in a 25°C room. After the 24-h hot-water exposure, the three specimens were transferred to a 25°C water bath for 2 h prior to testing under unconfined-compression conditions. The other three dry specimens were then tested under similar conditions. The effect of the hot-water exposure is found by dividing the strength of the wet specimens by the strength of the dry specimens and expressing this ratio as the percentage of retained strength.

Double-Punch Debonding Test

The double-punch debonding test was developed by Jimenez of the University of Arizona. The procedure has been described and published (3). The sequence of steps for this method is similar to that of the immersion-compression procedure, with the following exceptions:

1. Mixing and compaction: Size of specimen is 101 mm by 63 mm and compaction is by a vibratory kneading compactor.
2. Wet exposure: Specimen is not cured for 24 h. After density determination, specimen is submerged in 50°C distilled water for a minimum of 45 min and is vacuum saturated during this time. Following saturation and submersion, the specimen is stressed by the application of a repeated pore-water pressure that varies from 34 to 207 kPa and cycles at 10 Hz (580 cpm) for 10 min at a temperature of 50°C.
3. Strength test: Strength of the wet or dry specimen is obtained by an indirect tensile test referred to as the double-punch test. Two steel punches 25 mm in diameter stress the specimen on the centers of the two flat surfaces. Load is applied at a displacement rate of 25 mm/min.

The silane pretreatment of the mineral aggregate varied as to solution concentration as well as aggregate surface moisture condition [oven-dried or approximate saturated surface dried (s.s.d.)]. The silane was applied in two different ways depending on this condition. In the oven-dried state, the aggregate was treated with 3 percent (by weight of the aggregate) of four different silane concentrations. In the approximate s.s.d. condition, the aggregate was treated with 1 percent (by weight of the aggregate) of the same four silane-in-water con-

centrations, namely, 0.25, 0.75, 1.00, and 1.50 percent. These two different aggregate surface moisture conditions were selected to determine whether the silane-aggregate reactivity is dependent on this characteristic as well as to simulate field conditions. Figure 1 depicts the variables and test methods. Two different aggregate sources were tested--Salt River and Agua Fria--both from the Phoenix area. Pavebond Special was added to the asphalt binder in the amount of 1 percent by weight of the asphalt, which is the same amount used in construction.

MATERIALS USED FOR TESTING

Mineral Aggregates

Two sources of aggregate were selected for use in the test program. Crushed Salt River and Agua Fria aggregate samples were obtained for testing from commercial stockpiles. Physical characteristics and mix design data for each source are included in Table 2. Both are stream deposits in the Phoenix valley area; both mix gradations approach the Fuller maximum-density curve. The sand equivalent values indicate the primary difference between the two sources: 32 for the Agua Fria as compared with 55 for the Salt River sources. The sand equivalent test is a very good measure of the portion of detrimental fine dust or claylike minerals in the mineral aggregate. It is logical to conclude that a low sand equivalent number will indicate a higher potential for asphalt-aggregate debonding and therefore proves to be a valuable test for aggregate evaluation. A good correlation between sand equivalent value and stripping has been established (3).

Asphalt

One asphalt was chosen for use throughout the test sequence. The asphalt used conforms to an aged residue grading classification of AR 2000, which was obtained from Sahuaro Petroleum, Phoenix. Edgington Asphalt in Long Beach, California, is the asphalt source.

Additives

Pavebond Special is the registered trade name of a product of the Carstab Corporation. It is marketed as an asphalt additive to prevent debonding.

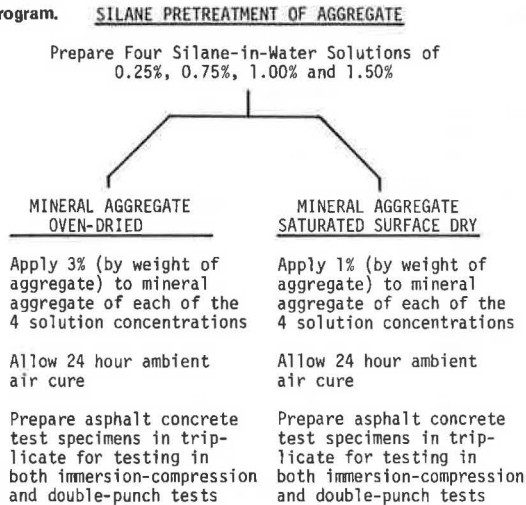
Silane coupling agents were first introduced to improve the water resistance of reinforced plastics. It was soon observed that they also imparted significant improvement to initial properties of laminates (2). Hydrophilic mineral surfaces were used in preparing composites with organic polymers; silanes were used to improve the bond. The similarity between the polymer-glass systems and the pavement materials was noted and it was felt that silanes may have the potential to increase the bond between asphalt and mineral aggregate surfaces.

Z-6020 Silane is a registered trade name of Dow-Corning. It is not marketed for the highway paving industry. It is primarily used as a coupling agent for the resin and plastic industry (2). It is a low-viscosity liquid aminoalkyl functional silane with the molecular formula $(\text{CH}_3\text{O})_3\text{SiCH}_2\text{NHCH}_2\text{CH}_2\text{NH}_2$. It is only one member of one subclass of the much larger group of organofunctional silane coupling agents. The chemical name for Z-6020 is N-(beta-aminoethyl)-gamma-aminopropyltrimethoxysilane.

SUMMARY AND DISCUSSION OF RESULTS

The complete test results are included in Tables 3

Figure 1. Test program.



PAVEBOND SPECIAL

Add 1% Pavebond Special to Asphalt and thoroughly mix.

Prepare asphalt concrete test specimens in triplicate for testing in both immersion-compression and double-punch tests.

NO TREATMENT

Prepare asphalt concrete test specimens in triplicate for testing in both immersion-compression and double-punch tests.

TEST MATRIX

TEST	DOUBLE PUNCH OR IMMERSION - COMPRESSION								
	TREATMENT*	SILANE 3% O.D.		SILANE 1% SSD		PAVEBOND SPECIAL		NONE	
CONDITION **	DRY	WET	DRY	WET	DRY	WET	DRY	WET	
CONCENTRATION	A	3	3	3	3	3	3	3	3
	B	3	3	3	3				
	C	3	3	3	3				
	D	3	3	3	3				

- * O.D. = OVEN DRY AGGR., SSD = SATURATED SURFACE DRY AGGR.
- ** WET OR DRY CONDITIONING OF SAMPLE
- PAVEBOND SPECIAL 1% BY WEIGHT OF ASPHALT
- 3 REPLICATES OF EACH TEST

Table 2. Mineral aggregate characteristics.

Physical Characteristic	Salt River (% passing)	Agua Fria (% passing)
Sieve Size		
1 in	100	100
3/4 in	94	94
1/2 in	82	80
3/8 in	71	66
No. 4	50	50
No. 8	40	44
No. 16	32	36
No. 30	22	25
No. 50	15	14
No. 100	8	7
No. 200	4	4
Sand equivalent	55	32
Centrifuge kerosene equivalent oil ratio (%)	4.9	4.5
Effective specific gravity	2.67	2.69
Specific gravity with 5 percent asphalt	2.47	2.49

and 4. The results are also plotted in Figures 2 through 5. The retained strengths of the laboratory-prepared asphalt concrete specimens were calculated two ways, as shown in the tabular results. In this manner one can compare results of a particular treatment (wet soaked) versus an untreated unsoaked specimen or that same particular treatment versus the specimen in the dry state. Density, voids, and the differences in compressive strength between wet and dry specimens are also reported.

The results indicate that silane pretreatment of mineral aggregate is effective in preventing debonding as indicated by the two different test procedures. Furthermore, it appears to be more effective on the Agua Fria mineral aggregate, which is a much "dirtier" source than the Salt River aggregate, as shown by the low sand equivalent value of 32. Densities of the silane mixtures are consistently higher with lower void contents. This may have contributed to the increased retained strengths of the silane-treated mixes, since it is well known

that mixes with low void content are more resistant to stripping than are mixes with high void content.

It is significant that the silane-treated specimens exhibited higher dry strengths than untreated specimens in all but three cases. Pavebond Special increased the dry strengths in every case; however, part of the increase may be a result of an increase in binder viscosity caused by the asphalt additive. It is known that amine additives facilitate better coating of aggregates, and this may also contribute to higher dry strengths.

Silane concentration, amount of silane solution applied, and application method all affect mix-strength retention as shown in Figures 2 through 5. For example, for the same given quantity of silane treatment (75 ppm), the retained strengths are substantially different in every case. This phenomenon indicates that aggregate surface condition (specifically, whether the surface is dry or approximately s.s.d.) has a definite impact on retained strength. Surface moisture may be the vehicle by which the silane is uniformly distributed over the aggregate surface area and, as such, could be an important factor influencing reactivity.

In summary, it appears that the silane pretreatment of mineral aggregates does improve resistance to debonding at least as well as Pavebond Special. Some retained-strength values are lower, but many more silane retained strengths are higher than those with Pavebond treatment. Further work must be conducted to optimize treatment methods and silane concentrations to accomplish the most economical application that yields the best results as well as to determine long-term results.

CONCLUSIONS AND RECOMMENDATIONS

1. In general, the silane pretreatment of two mineral aggregates improved the resistance to debonding of asphalt from mineral aggregate in the immersion-compression and double-punch tests.
2. The silane treatment appeared to have a more pronounced effect on the Agua Fria aggregate source. As a result, dirty aggregate sources (those

Table 3. Agua Fria aggregate: immersion-compression and double-punch test results.

Treatment	Density (pcf)	Voids (%)	Strength (psi)			Retained Strength	
			Dry	Wet	Difference ^a	A	B
Immersion-Compression Test							
Asphalt only	142.5	8.3	391	90	301	23	23
1 percent Pavebond Special	142.0	8.4	423	182	241	43	47
Silane concentration (%) applied to oven-dried aggregate							
0.25	143.0	8.0	439	158	281	36	40
0.75	143.5	7.5	511	276	235	54	71
1.00	142.5	8.1	464	269	195	58	69
1.50	144.0	7.5	510	352	158	69	90
Silane concentration (%) applied to s.s.d. aggregate							
0.25	144.5	7.0	478	220	258	46	56
0.75	144.5	6.8	514	288	226	56	74
1.00	144.5	6.8	454	268	186	59	69
1.50	144.5	6.2	477	329	148	69	84
Double-Punch Test							
Asphalt only	149.0	3.8	185	120	65	65	65
1 percent Pavebond Special	149.5	3.5	204	155	49	76	84
Silane concentration (%) applied to oven-dried aggregate							
0.25	150.5	3.0	204	145	59	71	78
0.75	152.5	1.9	230	214	16	93	116
1.00	152.0	2.2	238	238	0	100	129
1.50	151.5	2.4	230	230	0	100	124
Silane concentration (%) applied to s.s.d. aggregate							
0.25	151.0	2.8	158	136	22	86	74
0.75	152.0	2.1	193	176	17	91	95
1.00	150.5	3.1	151	139	12	92	75
1.50	152.0	2.2	169	159	10	94	86

Note: A = [wet strength (treated)/dry strength (treated)] x 100 percent; B = [wet strength (treated)/dry strength (untreated)] x 100 percent.
^aDry strength minus wet strength.

Table 4. Salt River aggregate: immersion-compression and double-punch test results.

Treatment	Density (pcf)	Voids (%)	Strength (psi)			Retained Strength	
			Dry	Wet	Difference ^a	A	B
Immersion-Compression Test							
Asphalt only	142.0	8.1	321	151	170	47	47
1 percent Pavebond Special	142.0	8.1	409	327	82	80	102
Silane concentration (%) applied to oven-dried aggregate							
0.25	142.0	8.1	325	312	13	96	97
0.75	142.5	7.8	379	292	87	77	91
1.00	142.5	7.8	428	321	107	75	100
1.50	143.0	7.2	337	283	54	84	88
Silane concentration (%) applied to s.s.d. aggregate							
0.25	144.5	6.4	498	478	20	96	149
0.75	143.0	7.5	419	335	84	80	104
1.00	145.0	5.9	537	537	0	100	167
1.50	143.5	7.1	447	384	63	86	120
Double-Punch Test							
Asphalt only	149.0	3.6	140	113	27	81	81
1 percent Pavebond Special	148.5	3.9	163	140	23	86	100
Silane concentration (%) applied to oven-dried aggregate							
0.25	151.0	2.3	165	160	5	97	114
0.75	151.5	2.0	181	181	0	100	129
1.00	151.0	2.1	157	143	14	91	102
1.50	151.5	1.7	167	167	0	100	119
Silane concentration (%) applied to s.s.d. aggregate							
0.25	151.0	2.2	172	136	36	79	97
0.75	151.5	1.9	207	143	64	69	102
1.00	151.5	2.0	200	200	0	100	143
1.50	150.5	2.4	181	179	2	99	128

Note: A = [wet strength (treated)/dry strength (treated)] x 100 percent; B = [wet strength (treated)/dry strength (untreated)] x 100 percent.

^aDry strength minus wet strength.

with low sand equivalent values) may benefit most from silane pretreatment. Furthermore, marginal aggregate sources may be allowed if testing with silane verifies that minimum retained-strength values can be obtained.

3. Silane concentration, application method, and aggregate surface moisture condition influence retained-strength values in both test methods. Therefore, a more detailed examination of these factors is the next logical step in future research endeavors.

4. Research should continue with silane chemicals as debonding preventives. In addition, silanes should be tested as additives in asphalts to compare with pretreatment of mineral aggregates as examined in this research effort. Preliminary tests by ADOT Research Center have shown the chemical to be effective down to 0.05 percent by weight of asphalt (the equivalent of approximately 25 ppm by weight of aggregate) as an antistripping agent.

5. It is recommended that experimental projects that use silane as an antistripping agent be con-

Figure 2. Agua Fria aggregate: immersion-compression test results.

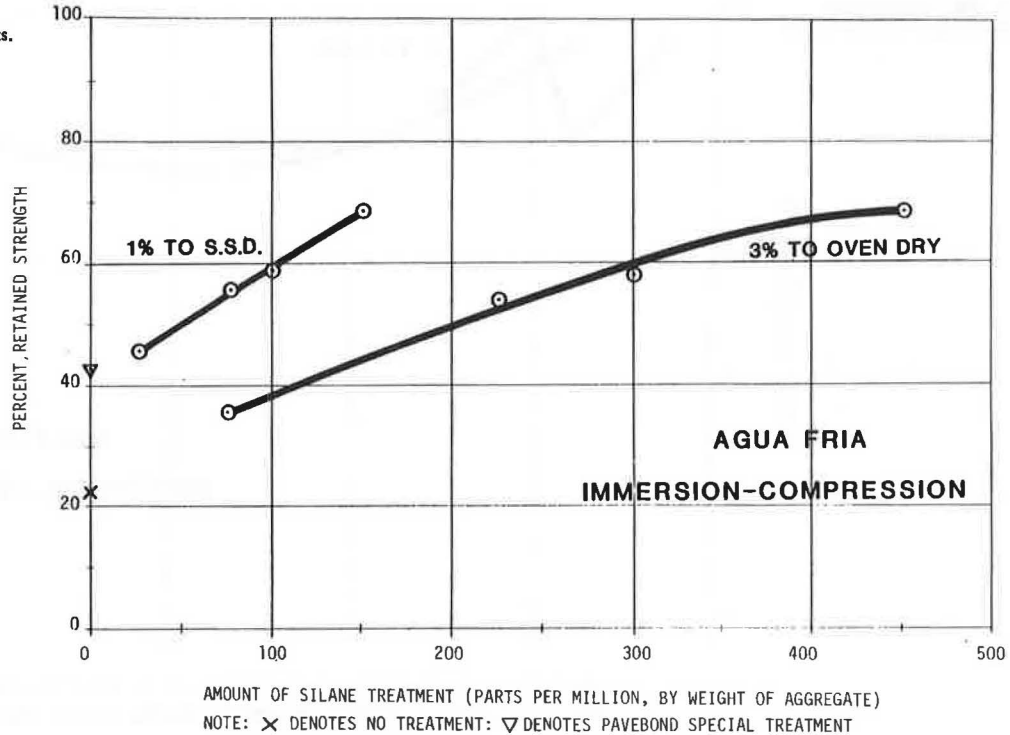
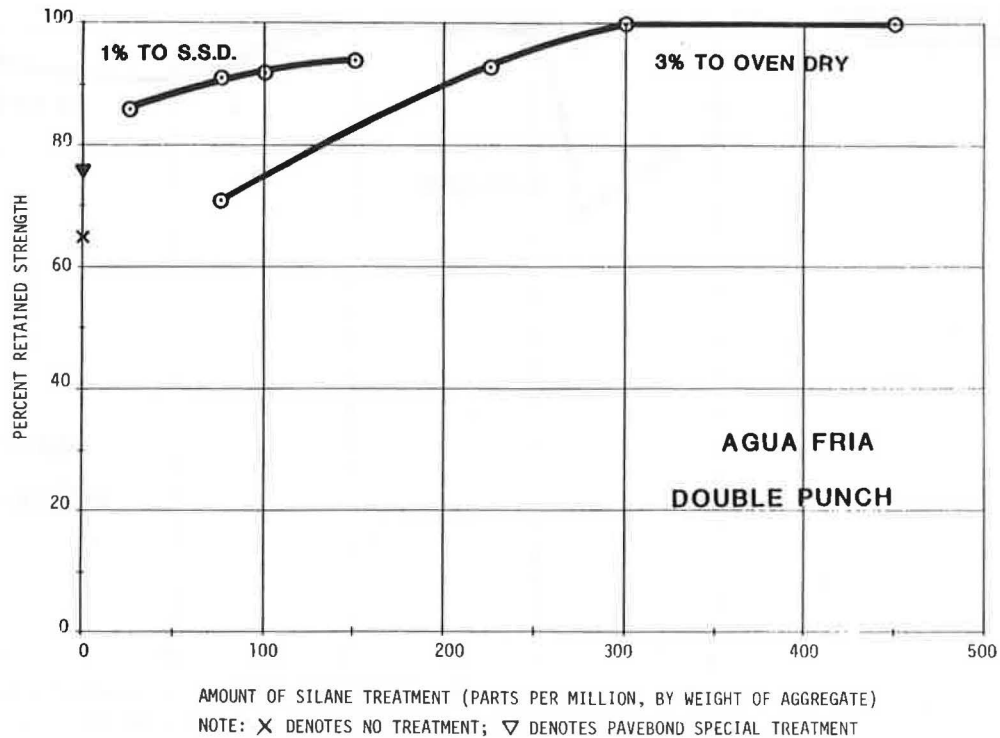


Figure 3. Agua Fria aggregate: double-punch test results.



structured to determine actual field effectiveness as well as long-term effects on asphalt concrete mixes.

6. Additional testing of aggregate pretreatment with silane solutions is warranted to compare the results of this research with various other aggregate types and characteristics, since this research was limited to testing of two local aggregate sources. In addition, other asphalt types and sources should be tested for the same reason.

ACKNOWLEDGMENT

John B. Hauskins, Jr., served as project consultant and did some very significant research into a vast array of chemicals early in the project. In addition, he was always willing to offer advice and direction. R.A. Jimenez of the University of Arizona served as consultant performing the testing. His assistance and guidance are always appreciated.

Figure 4. Salt River aggregate: immersion-compression test results.

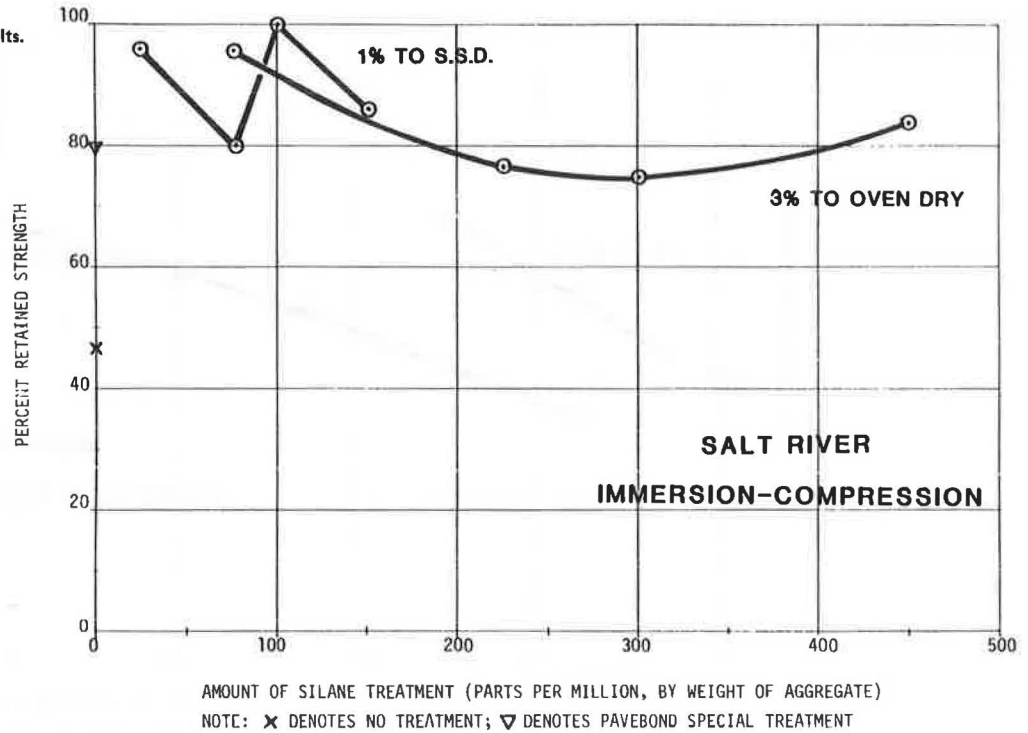
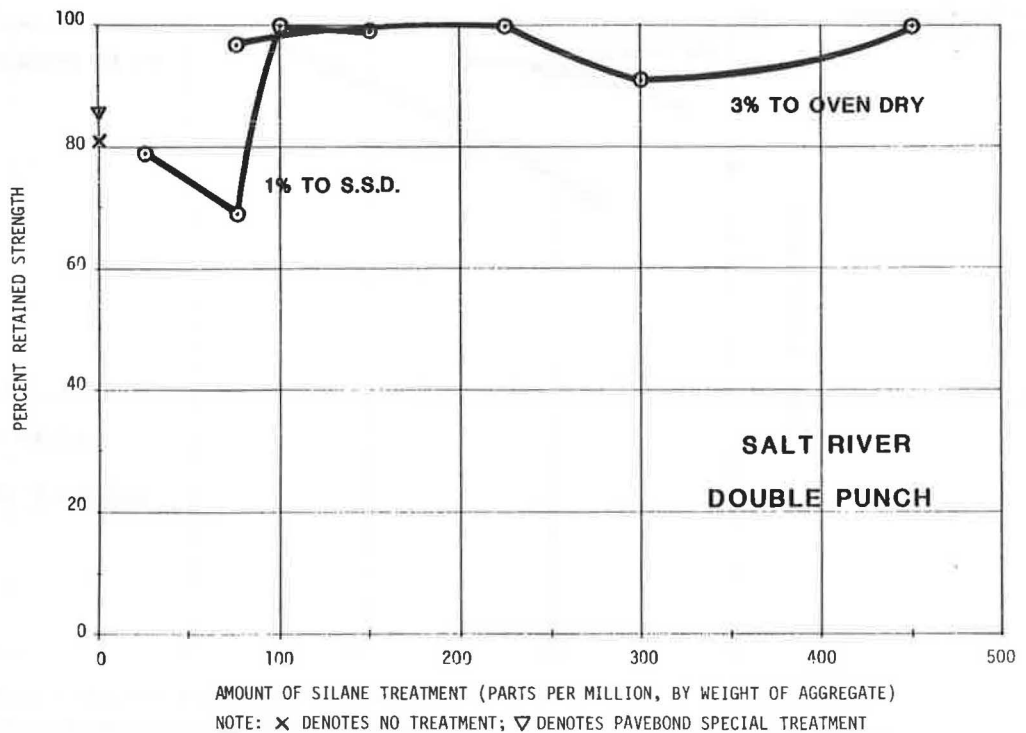


Figure 5. Salt River aggregate: double-punch test results.



This study was conducted under Federal Highway Administration project HPR-1-18(178). The opinions and findings expressed or implied are ours. The contents do not necessarily reflect the official views or policy of the U.S. Department of Transportation.

REFERENCES

1. H.J. Fromm. The Mechanism of Asphalt Stripping from Aggregate Surfaces. Proc., AAPT, Vol. 43, 1974.
2. E.P. Plueddemann. Mechanism of Adhesion of Coatings Through Reactive Silanes. Journal of Paint Technology, March 1970.
3. R.A. Jimenez. Testing for Debonding of Asphalt from Aggregates. TRB, Transportation Research Record 515, 1974, pp. 1-17.
4. J.M. Rice. Relationship of Aggregate Characteristics to the Effect of Water on Bituminous Paving Mixtures. Symposium on Effects of Water on Bituminous Paving Mixtures, American Society for Testing and Materials, Philadelphia, PA, ASTM Spec. Tech. Publ. 240, 1958.
5. K. Majidzadeh and F.N. Brovold. State of the Art: Effect of Water on Bituminous-Aggregate Mixtures. HRB, Special Rept. 98, 1968.
6. P. Hubbard. Adhesion of Asphalt to Aggregates in the Presence of Water. Proc., HRB, 1938, pp. 238-249.
7. Proc., International Symposium on Porous Asphalt, Amsterdam, Netherlands, 1976.
8. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete: Field Evaluation Phase. NCHRP, Project 4-8(3)/1, Interim Rept., Feb. 1978.
9. J.A.N. Scott. Adhesion and Disbonding Mechanisms of Asphalt Used in Highway Construction and Maintenance. Proc., AAPT, Vol. 47, 1978.
10. F.C. Sanderson. Methylchlorosilanes as Antistripping Agents. Proc., HRB, 1952, pp. 288-300.
11. L. Clplijauskas, M.R. Piggot, and R.T. Woodhams. Chemically Modified Asphalts for Improved Wet Strength Retention. American Chemical Society, I&EC Product Research and Development, June 1979.
12. M.C. Ford, P.G. Manke, and C.E. O'Bannon. Quantitative Evaluation of Stripping by the Surface Reaction Test. TRB, Transportation Research Record 515, 1974, pp. 40-54.
13. R.P. Lottman, R.P. Chen, K.S. Kumar, and L.W. Wolf. A Laboratory Test System for Prediction of Asphalt Concrete Moisture Damage. TRB, Transportation Research Record 515, 1974, pp. 18-26.
14. T.Y. Chu. Investigation of Roadway Drainage as Related to the Performance of Flexible Pavements. Univ. of South Carolina, Columbia, Aug. 1974.
15. J.G. Chehovits and D.A. Anderson. Upgrading of Marginal Aggregates for Improved Water Resistance of Asphalt Concrete. Pennsylvania State Univ., University Park, Jan. 1980.
16. J.C. Peterson and others. Paving Asphalts: Chemical Composition, Oxidative Weathering and Asphalt-Aggregate Interactions, Part II. FHWA, Rept. FHWA-RD-74-71, June 1974.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Evaluating Asphaltene Settling Test and Relating Results to Physical Properties of Paving Asphalts

FREDDY L. ROBERTS AND THOMAS W. KENNEDY

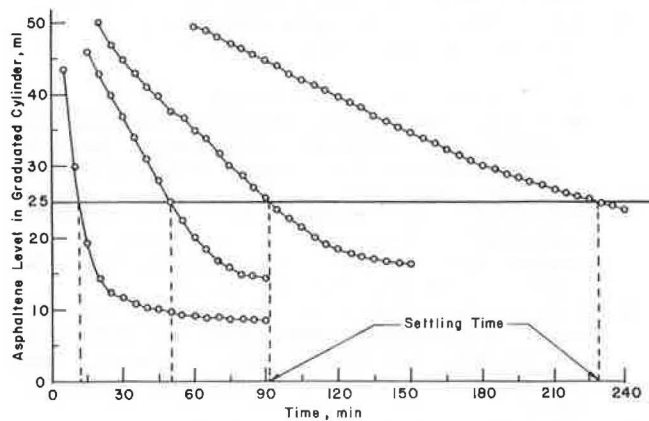
The asphaltene settling test was designed to measure the compatibility of the components of an asphalt cement by measuring the settling rate of the asphaltene for a mixture of asphalt in hexane. The results are reported as the length of time required for the asphaltenes to settle out of a solution of hexane. The main premise on which the settling test is based is that the asphaltenes with the longer settling times are better dispersed in the hexane-maltene phase and that longer settling times indicate that the mixtures have more compatible components. The purposes of this study were (a) to evaluate the settling test in terms of test repeatability and the effect of varying selected parameters of the procedure on test results, (b) to determine the settling time of a series of asphalt cement specimens in order to investigate potential relationships between settling time and asphalt characteristics, and (c) to evaluate the effectiveness of asphalt modifiers such as softening agents and antistripping agents. Based on the results, it is felt that additional work is required if the test is to have a practical value. Nevertheless, the findings indicate that (a) the test has a fair repeatability; the coefficients of variation of settling times for 262 asphalts ranged from 2.8 to 9.4 percent; (b) the settling time of asphalt is sensitive to the test temperature; (c) no well-defined relationship was found between settling time and specification-type asphalt characteristics (penetration, viscosity, specific gravity, flash point); (d) the major factor that affected test results was the asphalt producer; and (e) the addition of antistripping agents reduced the settling time with respect to the virgin asphalt, whereas

the effect of adding a softening agent was inconsistent, although there was a tendency for reduced settling times. A modified procedure to simplify the procedure, to reduce the time and cost of performing the test, and to improve its repeatability is presented.

The objective of the study summarized in this paper was to evaluate the asphaltene settling test, which has been suggested as a means to rapidly evaluate asphalt durability and the compatibility of asphalts and to determine how effective asphalt-softening agents, which have been proposed for use in recycled asphalt mixtures, are in redistributing the molecular agglomerates present in aged asphalts.

The test is based on previous work (1,2) describing relationships between asphaltenes and durability that led to the development of the settling test, which, according to Plancher and others (3), was developed by Hoiberg and Suhaka for the Asphalt Roofing Manufacturers Association. Later modifications by Plancher [comments in review of report by Kennedy and Lin (4)] adapted the test for paving-

Figure 1. Typical asphaltene settling curves.



grade asphalts (3). The test measures the relative degree of dispersion of asphaltenes of paving asphalts when the asphalts are dissolved in hexane. This dispersion is considered to be a measure of the compatibility of the asphalt components, which is important to asphalt chemistry (3). Test results are reported as a settling time or settling rate of the asphaltene in the solution of asphalt and hexane.

Based on the results reported by Plancher and Petersen at the Laramie Energy Technology Center (LETC) and by others, the test has begun to be used or to be considered for use by various agencies and groups. Plancher and others (3) reported several advantages to this test: (a) good repeatability and high sensitivity to changes in asphalt composition and (b) applicability to virgin, recovered or aged, modified, and blended asphalts. Modified asphalts are asphalts that have been mixed with modifiers, such as asphalt-softening or rejuvenating agents.

Even though a considerable amount of development work has been done with the test, it has not otherwise been used extensively or been applied to a large number of asphalts. The primary objective of this study is to evaluate the asphaltene settling test and determine the factors that affect it.

TEST DESCRIPTION AND EXPERIMENTAL PROGRAM

According to Plancher and others (3), asphalt has traditionally been considered to be composed of oils, asphaltenes, and resins, but it is more correctly a mixture of complex molecular structures ranging from paraffinic types to highly condensed ring structures, with varying degrees of aromaticity, substitution, and heteroatom content. It was also suggested that asphalt compatibility is related to the interaction of molecular components and that changes in the asphalt components could affect the compatibility of the asphalt mixture, which in turn could affect its performance characteristics. Some of the factors that could affect component compatibility are original composition, feedstock blending, aging, asphalt modifiers, and the adsorption characteristics of the mineral aggregates.

The asphaltene settling test was developed to measure the compatibility of the components of an asphalt cement by measuring the asphaltene settling rate for a mixture of asphalt and hexane. Typical settling-rate relationships are shown in Figure 1. Because many of the relationships related to particles settling from a solution were linear over a reasonable period, the time required for the asphaltenes to settle to the 25-ml level in a graduated cylinder was adopted as the settling time. The

slower the settling rate or the longer the settling time, the more compatible the asphalt components. This measure of compatibility is based on the premise that asphaltenes with longer settling times are better dispersed in the hexane-maltene phase than those with shorter settling times (3). Plancher also noted that the test results could be affected by the quantity, size, shape, and density of the asphaltene particles and by the viscosity and density of the hexane-maltene phase. Likewise, the effect of changes in test variables, such as quantity of asphalt, test temperature, etc., has not been systematically studied to any extent.

Aged asphalts generally have longer settling times than virgin asphalts. This increase in settling time is partly attributed to increased viscosity of the hexane-maltene solution, charged particles, changes in asphaltene geometry with increased aging levels (spherical to platelet), density, etc., and therefore it should not be construed that aged asphalts are more compatible or durable simply because they have longer settling times. According to Plancher, caution should be exercised when cross-comparing asphalt systems.

OBJECTIVES

This study was developed to evaluate the asphaltene settling test. This evaluation included determining the repeatability for (a) tests conducted simultaneously by the same operator or at the same time by two different operators, (b) tests conducted over a period of time by the same operator, and (c) tests conducted by different laboratories (asphalt cements have been sent to LETC, but results are not available at this time) and the effect of variations of the following parameters on test results: (a) amount of asphalt, (b) asphalt preparation temperature, and (c) test temperature. An additional objective was to use the test to investigate (a) the relationship between the asphaltene settling time and asphalt characteristics for various producers and (b) the effect of commonly used additives or modifiers, such as antistripping agents and asphalt-softening agents, on test results.

TEST METHOD

The asphaltene settling test proposed by LETC involves the following steps:

1. Dissolve 2.0000 g of asphalt cement in 50 ml of n-hexane by stirring the mixture for 16-24 h at 20°C (a stirring time of 20 h was established for use in this and in subsequent studies),
2. Transfer the digested mixture into a 50-ml graduated cylinder, and
3. Read the asphaltene level in the graduated cylinder at 5-min intervals and record.

The asphalt settling time is defined as the time in minutes required for the asphaltene meniscus to descend to the 25-ml level in the 50-ml graduated cylinder (3). Additional details of the test and required equipment are contained in a report by Kennedy and Lin (4).

MATERIALS

Materials included in the study were virgin asphalt cements, asphalts extracted from pavements, antistripping agents, and asphalt-softening agents. The antistripping and softening agents were included in order to determine the effect of these additives on the settling times.

Except for the extracted asphalts, the asphalt cements were identified by using an alphanumeric

Table 1. Summary of repeatability of asphaltene settling test: simultaneous tests.

Producer and Specimen No.	Operator	Asphaltene Settling Time (min)										Extremes (min)	Range (min)	Mean (min)	SD (min)	Coefficient of Variation (%)
		Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Test 8	Test 9	Test 10					
A9	1	30.0	28.2	30.0	29.6	28.7	29.3	28.1	29.0	27.6	28.2	27.6-30.0	2.4	28.9	0.84	3.0
	2	30.6	28.7	30.7	31.0	26.6	28.6		27.5		27.9	26.6-31.0	4.4	28.9	1.64	5.7
B15	1	33.5	34.3	30.4	30.5	29.2	33.9	34.8	34.2	36.4	35.6	29.2-36.4	7.2	33.3	2.41	7.2
	2	32.4	31.2	31.2	31.2	29.1	31.4	26.3	31.5	33.7	30.5	26.3-33.7	7.4	30.9	1.99	6.4
C20	1	32.7	31.1	28.8	28.0	26.6	27.8	23.6	30.0	31.2	30.9	23.6-32.7	9.1	29.1	2.69	9.2
	2	26.5	29.2	27.2	27.0	25.5	25.5	24.9	28.0	25.8	26.0	24.9-29.0	4.1	26.6	1.31	4.9
D4	1	59.0	62.6	53.8	53.7	57.6	51.0	57.6	60.0	65.0	59.0	51.0-65.0	14.1	57.9	4.24	7.3
	2	53.9	65.0	53.0	49.0	49.0	55.2		52.7	55.4	58.0	49.0-65.0	16.0	54.6	4.87	8.9
D5	1	60.0	62.6	57.6	59.5	62.2	62.6	54.0	62.6	62.5	62.2	54.0-62.6	8.6	60.6	2.88	4.8
	2	60.6	57.7	54.0	45.0	55.0	56.0		56.7	62.5	62.0	45.0-62.5	17.5	56.6	5.31	9.4
A249	1	24.1	21.7	22.5	22.5	22.2	21.1	22.0	21.1			21.1-24.1	3.0	22.2	0.96	4.3
H245	1	98.5	98.4	104.1	104.1	97.5	104.1	102.7	102.0	98.2		97.5-104.1	6.6	101.1	2.87	2.8

Note: Test temperature = 10°C.

Table 2. Summary of repeatability of asphaltene settling test: daily runs.

Producer and Specimen No.	Asphaltene Settling Time (min)										Extremes (min)	Range (min)	Mean (min)	SD (min)	Coefficient of Variation (%)
	Day 1	Day 2	Day 3	Day 4	Day 5	Day 6	Day 7	Day 8	Day 9	Day 10					
A9	28.8	28.7	31.5	29.4	29.4	28.6	29.1	29.4	29.9	30.4	28.6-31.5	2.9	29.5	0.89	3.0
A53	23.6	24.9	25.0	25.0	24.1	22.2	22.8	22.8	23.7	23.1	22.2-25.0	2.8	23.7	1.01	4.3
B15	30.0	30.0	32.6	34.8	33.3	31.3	30.9	31.5	34.0	30.7	30.0-34.8	4.8	31.9	1.68	5.3
C20	30.5	27.3	28.4	29.0	28.9	28.7	32.5	28.0	27.1	28.0	27.1-32.5	5.4	28.8	1.61	5.6
D4	59.7	56.3	58.4	61.8	69.0	57.3	62.0	60.0	58.9	62.3	56.3-69.0	12.7	60.6	3.58	6.9
D5	60.5	60.2	66.2	60.0	58.0	62.2	59.5	61.4	63.2	57.3	57.3-66.2	8.9	60.9	2.58	4.2
G178	73.5	78.0	75.0	79.6	69.7	72.5	71.4	72.7	77.2	77.5	69.7-79.6	9.9	74.7	3.25	4.4
P170	126.3	130.4	126.9	121.8	127.7	125.0	130.0	131.0	125.5	133.6	121.8-133.6	11.8	127.8	3.46	2.7

Note: Test temperature = 20°C.

code in which a letter represented the producer and a number represented a sample of asphalt from that producer. For antistripping agents, the producer identification letter was preceded by an A and for softening agents an R preceded the producer identification. The extracted asphalts were identified as EE. These asphalts were extracted in 1980 by using trichloroethylene.

Tests were performed on 262 virgin asphalts and 5 samples of an extracted asphalt that were obtained from the Texas State Department of Highways and Public Transportation (DHT). Virgin asphalts were secured from 16 different producers in Texas and Oklahoma and included grades that ranged from AC-3 to AC-20.

The two categories of additives or modifiers used were antistripping agents and asphalt-softening agents. Fourteen types of antistripping agents provided by seven producers and five types of asphalt-softening agents provided by five producers were included in the test program.

ANALYSIS AND DISCUSSION OF TEST RESULTS

The primary objectives were to determine the repeatability of the test, the effects and importance of modifying various test parameters on test results, the relationship between asphaltene settling time and asphalt cement characteristics, and the effect of commonly used modifiers on the settling times of virgin asphalt.

Repeatability Tests

Two types of repeatability tests were conducted: (a) repeatability of simultaneous tests by the same operator or different operators and (b) repeatability of tests over a period of time by the same operator. Approximately 10 settling tests were conducted simultaneously by one operator on seven asphalts and by a second operator on five asphalts. In addition, 10 replicated tests on eight asphalts were conducted over a period of 10 days by one

operator. The resulting settling times are summarized in Tables 1 and 2.

As can be seen, the range for the settling times obtained for the various test series varied from 2.4 to 14.0 min for operator 1 and from 4.1 to 17.5 min for operator 2. The corresponding values of the standard deviation were 0.84-4.24 for operator 1 and 1.31-5.31 for operator 2. Since the mean settling times varied significantly for the various test series, coefficients of variation were calculated. These coefficients varied from 2.8 to 9.2 percent for operator 1 and from 4.9 to 9.4 for operator 2. Thus, the repeatability was fair and appeared to be essentially the same for the two operators.

A comparison of the mean values, however, indicates that operator 2 generally obtained shorter settling times. This variation probably is significant statistically; however, it is not of practical significance since the range of values for any given test series was much larger than the difference between operators. This difference could be attributed to sampling techniques, temperature control, and rate of digestion, i.e., stirring rate. However, due to the consistency of the difference, it is felt that the more probable cause is the time required to transfer the solution to the graduated cylinder at the beginning of the timing process. It is therefore recommended that the timing of the test start 20 s after stirring ends.

In addition, consideration should be given to eliminating the 10-ml hexane wash since it is felt that the amount of residue left in the flask probably would not affect the test results but that the change in mixture concentration in the upper portion of the cylinder could quite easily affect them. The washing process definitely adds to the time required for transfer and in fact is responsible for the largest portion of the transfer time.

The settling times obtained from the series of tests conducted over a period of time are summarized in Table 2. These data do not indicate any consistent changes with time. The range and standard deviations varied from 2.8 to 12.7 and from 0.89 to

3.58, respectively. The coefficients of variation varied from 2.7 to 5.9 percent, which is consistent with the variation of values obtained by simultaneous multiple tests conducted by the same operator. These results suggest that well-trained operators can probably repeat the test over a period of time and also that they can repeat it at any given time.

Figure 2. Relationship between amount of asphalt in mixture and settling time.

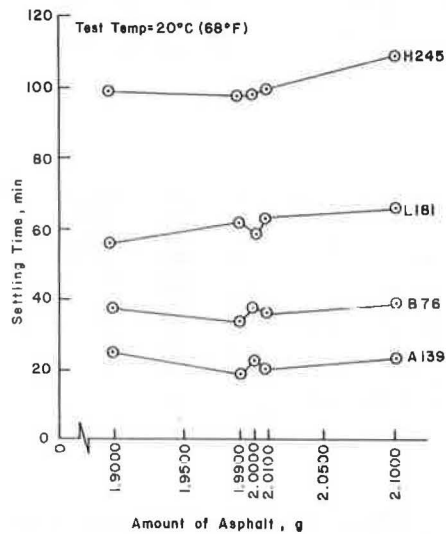


Figure 3. Effect on settling time produced by heating asphalt cement at time of weighing.

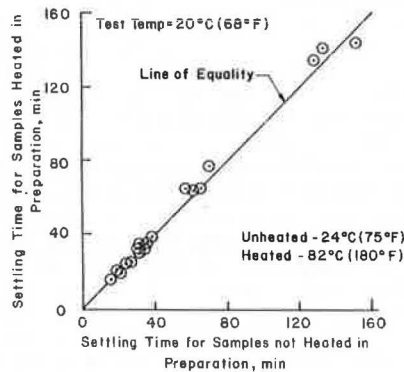
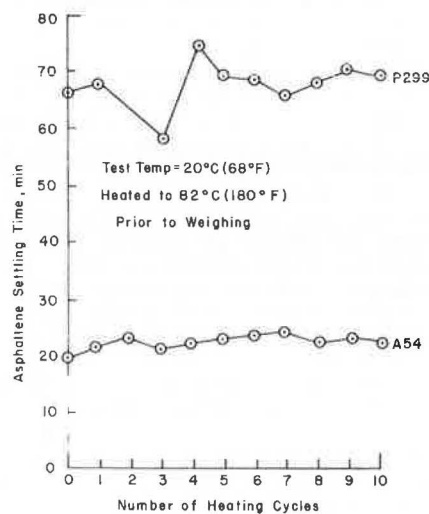


Figure 4. Relationship between settling time and number of heating cycles prior to weighing asphalt cement.



Samples of the asphalt were sent to LETC in order to develop between-laboratory comparisons; however, their results were not available at this time.

Test Parameters

Three test parameters, two associated with mixture preparation and the other with testing temperature, were evaluated to determine the effect of changes in the parameters on test results.

Mixture Preparation

The amount of asphalt and the effect of heating the asphalt during the weighing operation are important with respect to the time required for sample preparation. Thus, the effects of changes in these two parameters were evaluated.

Amount of Asphalt

A series of tests on asphalts that exhibited mean settling times ranging from 23 to 98 min was conducted to determine the effects and importance of variations in the amount of asphalt on test results. Mixtures containing 1.9000, 1.9900, 2.0000, 2.0100, and 2.1000 g of asphalt were prepared and tested. The resulting settling times are shown in Figure 2.

No consistent relationship was found between the amount of asphalt used in the mixture and the settling time. At or near 2.000 g there appears to be a significant hump in the relationship. This inconsistency is attributed to the short range over which these three tests were plotted, which exaggerates the variation. In fact, the magnitude of the effect is less than the range of values that could be expected with replicated tests (Tables 1 and 2). It therefore is apparent that the amount of asphalt within the limits evaluated is not particularly important. Thus, to facilitate sample preparation in production operations but at the same time to provide reasonable control, it is recommended that the asphalt be weighed to 2.00 g with an accuracy of ±0.01 g.

Heating the Asphalt

Heating the asphalt to 82°C to facilitate weighing the required amount of asphalt to be used in the partial solution of asphalt cement and hexane did not produce any consistent effect on the settling times nor did repeated heatings have an effect (Figures 3 and 4). Erratic results were observed in the repeated heating tests on asphalt cement P299 (Figure 4), which could have been the result of many testing or sampling variations. Since the accuracy of weighing has been reduced and since the asphalt properties could be adversely affected, it is recommended that the asphalt cement not be heated to facilitate weighing.

Test Temperature

Ten asphalts that have a wide range of settling times were selected and tested at temperatures of 24 and 20°C. The resulting settling times are shown in Figure 5. The asphaltene settling times at 20°C were substantially higher than those at 24°C. The effect is large enough to require control of the test temperature. Water baths or controlled-environment chambers are probably required and justified.

Relationship to Asphalt Characteristics

A limited analysis was conducted to investigate

potential relationships between settling time and physical specification properties supplied by the Texas DHT for 55 asphalt cements. Properties analyzed were viscosity, penetration, specific gravity, and flash point. Asphaltene settling times were determined for each virgin asphalt and the results compared with the physical properties of the asphalt cements contained in the report by Kennedy and Lin (4).

No relationships between asphaltene settling time and these physical properties of the asphalt cements were found to exist. This result was not unexpected

since many of the physical properties are controlled during production and are only indirectly related to the composition of the asphalt. Examples of the nature of the relationships involving viscosity and penetration are shown in Figures 6 and 7. According to Plancher, settling times, however, may be related to other rheological properties or characteristics.

The only factor found to affect settling time was the asphalt producer, which probably is related to composition. The means and ranges of settling times for the 16 producers located in Texas and Oklahoma are summarized in Figure 8. The variation of mean settling time between producers ranges from 16.2 to 235.0 min. Within the observation period of 132 h, the asphalt cement from producer E had no observed settling time and was considered to have an infinite settling time.

Large variations in settling time for some producers are evident; e.g., producers C, G, M, and N had coefficients of variation in excess of 30 percent. Such variation could be related to changes in crude, blending, etc.; however, there is no evidence available from this study to support or reject such a hypothesis.

Although no correlations with viscosity were detected previously, the settling times are shown in Figure 8 by viscosity grade for each producer. Within the limits of this study, no relationship could be found between settling time and the producer viscosity grade. For some producers the set-

Figure 5. Effect of test temperature on settling time.

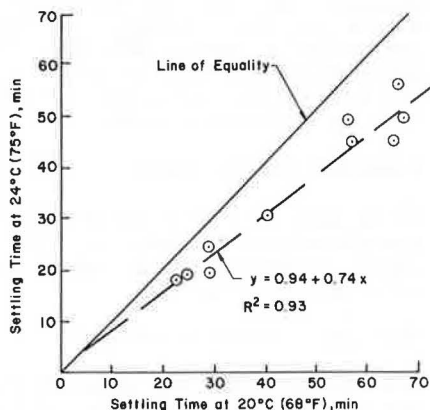


Figure 6. Relationship between viscosity and settling time.

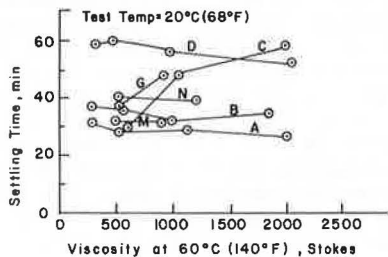


Figure 7. Relationship between penetration and settling time.

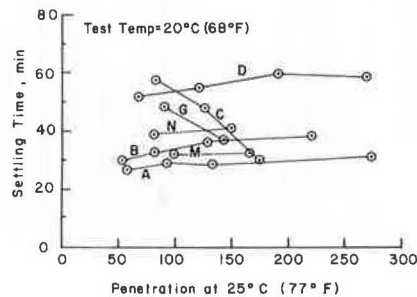


Figure 8. Means and ranges of settling times of various viscosity grades for various producers.

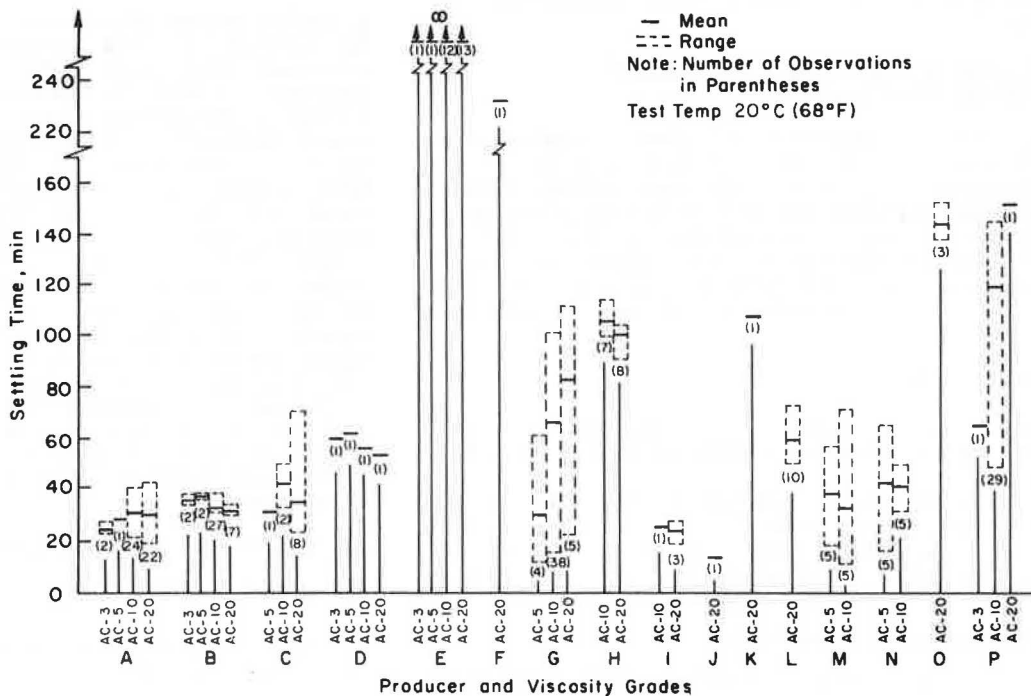


Figure 9. Relationship between settling time and volume of settled asphaltenes.

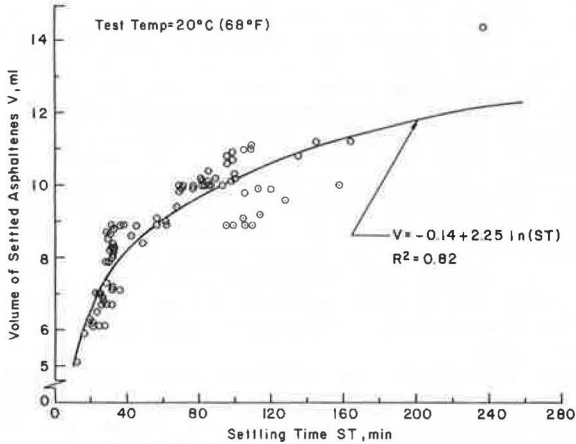
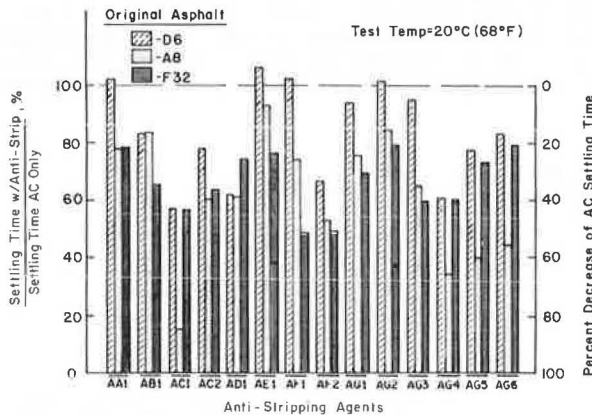


Figure 10. Effect of antistripping agents on settling time for various virgin asphalts.



tling times were essentially the same for all grades, whereas for others large differences occurred.

Relationship Between Settling Time and Volume of Asphaltenes

Since the bulk volume of asphaltenes should influence the settling times, a further analysis was conducted to establish the relationship between asphaltene settling time and the estimated volume of asphaltenes. For this analysis the volume of asphaltenes was defined as the cylinder reading in milliliters after 24 h settling. As shown in Figure 9, the nonlinear equation describing the relationship is

$$V = -0.14 + 2.25 \ln(ST) \tag{1}$$

where ST is the asphaltene settling time in minutes and V is the volume of settled asphaltene in milliliters.

The above relationship has a coefficient of determination R² of 0.82. A more meaningful relationship with less scatter might be obtained if other factors, such as density of the asphaltenes in the mixture, are considered.

Effects of Asphalt Modifiers

The two types of asphalt modifiers involved in these

tests were antistripping agents and asphalt-softening agents.

Antistripping Agents

Three asphalt cements were used to evaluate the effect of 14 different antistripping agents on settling time. Ten percent by weight of asphalt cement of each antistripping agent was added to the asphalt-hexane mixture. The resulting settling times are summarized in Figure 10.

Even though no well-defined relationships existed, the asphaltene settling times decreased when antistripping agents were added, indicating that the presence of the antistripping agents had an effect on test results. It should be noted that antistripping agents AC1, AG4, AG5, and AG6 apparently caused a residue to remain in the flask and on the stirring bar after the transfer of the mixture into the graduated cylinder.

Asphalt-Softening Agents

Tests were conducted to determine the effects of asphalt-softening agents on the settling times of virgin asphalts, artificially aged asphalts, and extracted asphalts.

Virgin Asphalt and Artificially Aged Asphalt

One asphalt cement was selected for artificial aging and subsequent determination of the settling times. The asphalt was artificially aged by placing about 5 g on a flat-bottomed glass pan 3.5 in in diameter in a 121°C oven for a specified period of time. Three different commercial asphalt-softening agents, identified as RB1, RC1, and RD1, were added to 2.0000 g of virgin or aged asphalts in the amounts of 1 and 15 percent and settling times were determined.

The resulting settling times are summarized in Figure 11. It was observed that no well-defined relationship existed between the settling time and the asphalt-softening agent (Figure 11) and that for one asphalt the asphaltene settling time increased with the aging time (Figure 12).

Extracted Asphalt

A limited amount of extracted asphalt was supplied by District 15, San Antonio. The asphalt was extracted from five cores obtained from five locations on IH10 in District 15. Four different commercial and asphalt-softening agents were selected as modifiers--RB1, RC1, RF1, and RGL--and 15 percent by weight was added to the extracted asphalt. Settling-time ratios for each softening agent are shown in Figure 13. It is assumed that when asphalt modifiers are added to recovered aged asphalt, longer settling times indicate a more compatible system of asphalt and modifiers (comments by Plancher, June 5, 1981). Thus, even though certain asphalt-modifier combinations decreased the settling times, the more compatible mixtures should still be those with the longer settling times.

The ratios of settling time for extracted asphalt with softening agent to initial settling time ranged from 66.6 to 116.0 percent. The more compatible systems are presumed to be those with longer settling times (3).

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The test is quite easy to conduct and quantitative test values can be obtained. Nevertheless, it is

tive as asphaltene dispersants for the extracted asphalts in the study. This agreed with the previously reported findings.

Recommendations

Test Procedure

It is recommended that 2.00 g with an accuracy of ± 0.01 g of asphalt cement be used in the preparation of the asphalt cement--hexane mixture. The asphalt cement should not be heated during mixture preparation. The testing temperature for the asphaltene settling test should be closely controlled, which probably requires a temperature chamber or water bath. Timing should start 20 s after stirring has been completed. The transfer of the asphalt-hexane mixture should be completed within 20 s. Consideration should be given to eliminating the 10-ml wash since it changes the mixture concentration in the upper part of the cylinder and is responsible for the major portion of the transfer time. It is also felt that the amount of residue left behind probably is not important.

Research

The significance and meaning of the test results need additional study if the test method is to have practical value. Supplemental tests may be required. Additional research should be conducted to evaluate the relationship between settling time and the performance of pavements. It would also be desirable to investigate the relationship between settling time and basic rheologic properties of asphalt.

ACKNOWLEDGMENT

This investigation was conducted at the Center for

Transportation Research, Bureau of Engineering Research, University of Texas at Austin. We would like to thank the sponsors, the Texas Department of Highways and Public Transportation and the Federal Highway Administration, U.S. Department of Transportation.

The contents of this paper reflect our views and we are responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration and the Texas Department of Highways and Public Transportation. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. E.W. Mertens. Predicting Weatherability of Coating Grade Asphalts from Asphaltene Characteristics. American Society for Testing and Materials, Philadelphia, PA, Bull. 250, 1960, pp. 40-44.
2. S.H. Greenfield and J.R. Wright. Four Methods for Predicting the Durability of Roofing Asphalts. Materials Research and Standards, Vol. 2, 1962, pp. 738-745.
3. H. Plancher, A.J. Hoiberg, S.C. Suhaka, and J.C. Petersen. A Settling Test to Evaluate the Relative Degree of Dispersion of Asphaltenes. Proc., AAPT, Vol. 48, 1979, pp. 351-374.
4. T.W. Kennedy and C. Lin. An Evaluation of the Asphaltene Settling Test. Bureau of Engineering Research, Center for Transportation Research, Univ. of Texas at Austin, Res. Rept. 253-2, July 1981.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Asphalt Temperature Susceptibility and Its Effect on Pavements

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

The objective of this study was to determine the influence of asphalt temperature susceptibility on pavement construction and performance. Review of published information and testing of 16 currently produced asphalt cements indicate that asphalts produced today have the same range of temperature susceptibility as those produced from 1960 to 1973. Data collected from specific refining sources have indicated that the physical properties of asphalt cements from selected refineries have changed with time, whereas asphalt from other refineries shows no statistically significant changes. This appears to be related to the crude from which the asphalt is produced. Results obtained from laboratory tests conducted on material from eight field projects indicate that asphalt cement properties do not correlate well with asphalt concrete tenderness problems observed in the field. However, mixture tests such as the resilient modulus and the indirect tensile test have potential for recognizing tender mixtures and defining mixture temperature susceptibility.

"Asphalt ain't as good as it used to be" (1). This statement is often repeated by field construction and maintenance personnel throughout the United States. Their general belief is that the oil compa-

nies are taking the "goodies" out of the asphalt and using them as feedstock for the petrochemical industry. Another widely held belief is that the oil embargo and this country's dependence on foreign crudes have forced the oil companies to use less-than-desirable crudes to manufacture asphalt. In addition, field personnel are convinced that the present asphalt specification tests, which are routinely performed, do not identify the important properties that control field construction and pavement performance.

As evidence of these statements, the field engineers cite a general increase in the occurrence of such problems as placement difficulties (tender mixes), excessive displacement under traffic (low stability), and thermal cracking, raveling, and stripping (water susceptibility) of asphalt concrete pavements. These problems result in shorter service

Figure 11. Relationship between amount of softening agent and settling time for virgin asphalts and asphalts artificially aged for 6, 24, or 48 h.

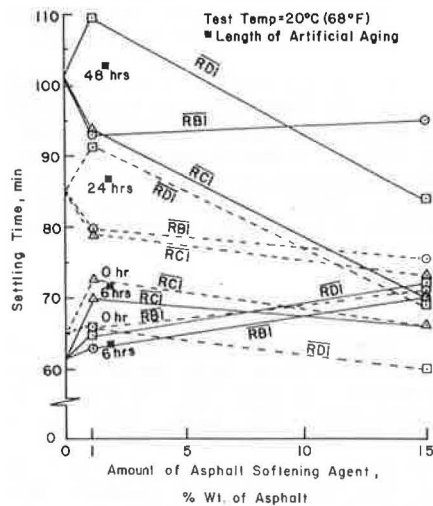


Figure 12. Relationship between asphalt aging time in oven and settling time.

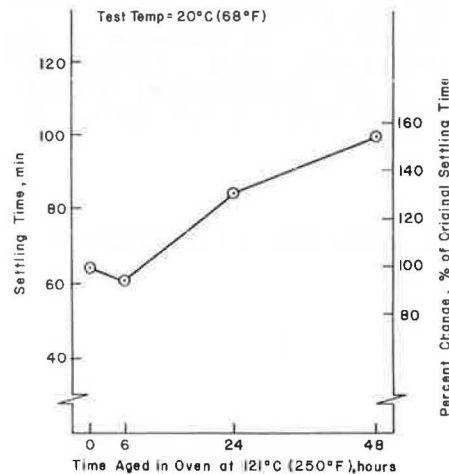
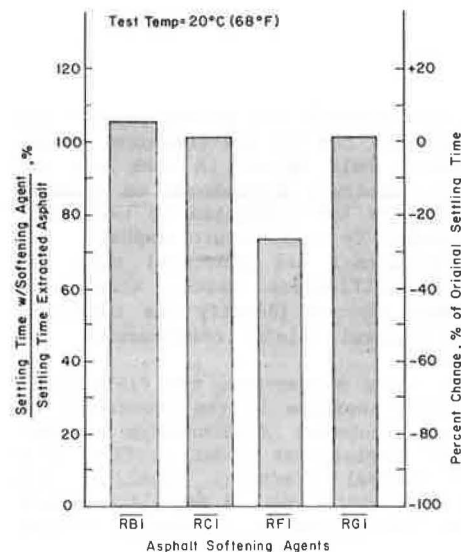


Figure 13. Effect of asphalt-softening agents on settling time for average of five samples of extracted asphalt.



felt that a great deal of additional work is required for the test to have practical value to practicing engineers. The complexity of the asphaltene, maltene, and hexane mixtures and the interaction of their components and their characteristics would appear to have a very definite effect on the test results and their meaning. Additional work on interpretation of test results will be required before the test can be used routinely.

Repeatability and Operators

The repeatability of the test values was fair and it was essentially the same for different operators and over a period of time:

1. The coefficients of variation of settling times ranged from 2.8 to 9.2 percent for operator 1 and from 4.9 to 9.4 for operator 2 in simultaneous tests.
2. The coefficient of variation of settling times varied from 2.7 to 5.9 for one operator conducting tests over a period of 10 days.

A definite operator effect was detected; one of the operators consistently obtained shorter settling times. This was attributed to the times required to transfer the mixture from the stirring flask to the graduated cylinder.

Test Parameters

The amount of asphalt (1.9000, 1.9900, 2.0000, 2.0100, and 2.1000 g) in the mixtures had no practical effect on the settling times in this study. It is felt that the precision should be 2.00 ± 0.1 g.

Heating the asphalt from room temperature to 82°C during mixture preparation did not affect the test results nor did repeated heating have an effect. Nevertheless, heating of the asphalt should be minimized.

Testing temperature produced a significant effect on the asphaltene settling time: the settling times were shorter at higher temperatures. Considerations probably should be given to conducting the test at room temperature, approximately 24-25°C.

Relationship to Asphalt Characteristics

The settling time varied significantly with producer and thus composition. In fact, for the asphalts tested in this study, the producer of asphalt was the only factor that affected settling time. No relationships were found between the settling time and specification-type asphalt characteristics such as viscosity, penetration, specific gravity, and flash point. Equation 1 was found to approximate the relationship between settling time and the volume of the settled asphaltene after 24 h settling. Other factors, such as the density of the asphaltenes in the mixture, probably should be considered in Equation 1.

Effects of Asphalt Modifiers

Settling times of virgin asphalt cements decreased when 10 percent antistripping agents was added. The settling time of asphalt cement increased with increased time of aging in an oven at 121°C. No well-defined relationship existed between the settling time and the addition of asphalt-softening agent for either the virgin or the artificially aged asphalt; however, the settling times tended to decrease when 15 percent of asphalt-softening agent was added. The asphalt-softening agents were not equally effec-

life, higher maintenance costs, higher life-cycle costs, and criticism by the driving public.

Certainly one must accept the opinions offered by those experienced field engineers; however, one must be cautious at the same time. For example, Hveem (2) indicated that tenderness problems were evident in California pavements in the 1940s. Field engineers complained, "Asphalt ain't as good as it used to be" as early as the 1930s (3-5), and asphalt cracking problems were evident early in the history of asphalt concrete use (6).

The overall objectives of the proposed research (1) are as follows:

1. To determine the range or extent of variability in temperature susceptibility of asphalt cements currently being used in road construction,

2. To evaluate the effects of the identified variability in relation to other factors and over the full range of service temperatures on pavement construction operations and short-term performance of pavements,

3. To identify the limits of variability in temperature susceptibility that can be accommodated through application of known asphalt technology by changes in asphaltic concrete construction procedures and mix design considerations, and

4. To determine procedures for accommodating or controlling that variability in temperature susceptibility of asphalt cements that cannot be accommodated by known asphalt technology.

These overall objectives are intended to be accomplished in two phases. Results of Phase 1 are contained in this report; they satisfy objective 1 as identified above. Phase 2 of this report will be concerned with objectives 2, 3, and 4 and will be initiated after the project panel staff of the National Cooperative Highway Research Program (NCHRP) review and approve the Phase 1 report.

The specific tasks of Phase 1 were to (a) gather and analyze data on the variability in the temperature susceptibility of asphalt cements over the full range of construction and service temperatures and (b) investigate in detail a minimum of four construction projects in which asphaltic concrete placement difficulties have been encountered and four projects in which placement difficulties were not encountered.

FINDINGS

For the purpose of this study, temperature susceptibility of an asphalt cement will be defined as the rate of change of viscosity (or other measure of asphalt consistency) with temperature. Asphalt temperature susceptibility is highly dependent on the temperature range considered and directly related to the type of equipment used to determine asphalt consistency.

Asphalt Cement Variability by Refinery Source

Published literature contains little information to indicate asphalt cement property variation by refinery source. In order to collect this information, a number of states were visited and asphalt cement physical-chemical property data were collected.

Detailed asphalt test data were obtained from 5 states representing 23 refineries in all regions of the United States. The temperature-susceptibility parameters penetration viscosity number (PVN) (77-275°F), (PVN') (77-140°F), and viscosity-temperature susceptibility (VTS) (140-275°F) were computed by using the equations shown below:

Penetration index PI_1 (1,2):

$$PI_1 = [30/(1 + 90PTS)] - 10 \quad (1)$$

where PTS = penetration temperature susceptibility = $(\log 800 - \log \text{penetration at } 77^\circ\text{F}) / [\text{ring-and-ball softening point } (^\circ\text{F}) - 77^\circ\text{F}] = (\log \Delta \text{ penetration}) / (\log \Delta T)$. (Large negative values of PI indicate greater temperature susceptibility; "typical" asphalts have values between +2 and -2.)

Penetration index PI_2 (2):

$$PI_2 = (20 - 500A)/(1 + 50A) \quad (2)$$

where A is $(\log \text{penetration at } T_1 - \log \text{penetration at } T_2) / (T_1 - T_2)$ and T is temperature ($^\circ\text{F}$), penetration at 100 g, 5 s. (Large negative values of PI indicate greater temperature susceptibility.)

Penetration ratio PR:

$$PR = [(\text{penetration at } 39.2^\circ\text{F}, 200 \text{ g}, 60 \text{ s}) + (\text{penetration at } 77^\circ\text{F}, 100 \text{ g}, 5 \text{ s})] \times 100 \quad (3)$$

(Lower PR indicates greater temperature susceptibility.)

PVN (temperature range, 77-275°F) (3):

$$PVN = [(4.258 - 0.7967 \log P - \log X) / (0.7951 - 0.1858 \log P)] (-1.5) \quad (4)$$

where P is penetration at 77°F in decimillimeters and X is viscosity at 275°F in centistokes. (Lower PVN indicates greater temperature susceptibility.)

PVN' (temperature range, 77-140°F) (3):

$$PVN' = [(6.489 - 1.590 \log P - \log X') / (1.050 - 0.2234 \log P)] (-1.5) \quad (5)$$

where P is penetration at 77°F in decimillimeters and X' is viscosity at 140°F in poises. (Lower PVN' indicates greater temperature susceptibility.)

VTS (4):

$$VTS = (\log \log 100 \eta_1 - \log \log 100 \eta_2) / (\log T_2 - \log T_1) \quad (6)$$

where

η_1 = viscosity at T_1 in poises,
 η_2 = viscosity at T_2 in poises, and
 T = temperature in degrees Kelvin.

(Greater VTS indicates greater temperature susceptibility.)

Temperature of equivalent stiffness (TES) (temperature at which stiffness of asphalt cement at 10 000-s loading time is equal to 20 000 psi):

1. Determine PI by using Equation 1,
2. By fixing stiffness of asphalt cement and time of loading and knowing PI, use Shell nomograph (5) to determine temperature difference, and
3. Compute TES by using the following calculation:

$$TES = \text{ring-and-ball softening point} - \text{temperature difference.}$$

(Higher TES indicates greater temperature susceptibility.)

These parameters together with penetration and viscosity were statistically analyzed by using the

Statistical Analysis System (SAS) computer program to determine annual means, standard deviations, and coefficients of variation for each refinery. Results of this analysis are given in a report by Button and others (1). Results indicate that, from a statistical standpoint, the temperature susceptibility of asphalts from a large number of refineries has changed with time. It is important to understand that this does not necessarily imply that the temperature susceptibility has changed to such a degree that construction and pavement performance will be affected.

Laboratory Testing of Selected Asphalts

Asphalt cements were collected and tested in the laboratory to define rate of change in consistency or hardness with temperature. Asphalts were obtained from refineries that produce asphalts having "best" and "worst" as well as "in between" reputations regarding asphalt-related construction difficulties. Asphalts were selected (in most cases) from refineries that use fairly constant sources of domestic crude. Test results provide an indicator of the range in temperature susceptibility of asphalts currently available to the user and define a general relationship between temperature susceptibility and pavement construction difficulties.

Sixteen asphalt cements of various grades were obtained from 10 different refineries. Table 1 gives a description of these asphalts.

Laboratory tests included the determination of viscosity and penetration over a range of temperatures and ring-and-ball softening points. These tests were performed on all the asphalts before and after the thin-film oven test (TFOT) and on selected

asphalts after the rolling thin-film oven test (RTFOT). Selected asphalts (before and after oven aging) were subjected to further viscosity testing at temperatures down to -50°F.

All tests performed on these asphalts used standardized procedures except the low-temperature viscosity tests. The Schwyer rheometer (7) was used to measure viscosity at 77 and 32°F. Viscosity at various temperatures ranging from 32 to -50°F were measured by using the Rheometrics mechanical spectrometer (8).

Test Results and Discussion

A summary of test results except those for the low-temperature rheology study is given for original asphalts in Table 2 and for oven-aged asphalts in Table 3. Observation of test results after TFOT and RTFOT reveals excellent correlation between the two tests. The report by Button and others (1) shows that the range of properties of these 16 asphalts is similar to the range of properties of asphalts produced up to 20 years ago.

Temperature Susceptibility

Seven different methods were used to compute temperature susceptibility. Results of these computations are presented in Tables 4-6. Asphalt H exhibits an unusually low temperature susceptibility, whereas asphalts B1, B2, B3, C, E, and J exhibit abnormally high temperature susceptibilities. Asphalts A1, B2, B3, C, and H show a significant increase in temperature susceptibility in the low-temperature (penetration) range. About half these asphalts show slight variations in temperature

Table 1. Asphalts used.

Asphalt Code	Grade	Refinery	Production Process	Crude Source	Crude Type	Market Area	User Comment
A1	AC-5	A, East Texas	Vacuum reduction	East Texas	Fairly heavy, sour	East Texas, Arkansas, Northern Oklahoma	Not normally a problem asphalt, from 1976 production
A2	AC-10						
A3	AC-20						
B1	AR 1000	B, California	Vacuum reduction	San Joaquin Valley, California	Napthenic, low in asphaltenes	Western United States	Not a problem asphalt; highly temperature susceptible; low volatility, low in asphaltenes; widely used on West Coast
B2	AR 2000						
B3	AR 4000						
C	AR 2000	C, New Mexico	-	Four Corners area of Arizona, New Mexico, Colorado, Utah	Light paraffinic, low in asphaltenes	Northern New Mexico, Southern Colorado, Southern Utah, Arizona	Causes construction difficulties, sets slowly, performs well once set up, very small production, excellent in certain blends
D	AC-20	C, Illinois	Vacuum flashed residue, air blown and blended	West Texas, Mexico, Mid-East	Mixed, sour, relatively light	East Central United States	Not a problem asphalt, relatively temperature susceptible
E	AR 2000	E, Oregon	-	-	-	-	Highly temperature susceptible, has produced tender paving mixtures
F1	AR 4000W	F, Wyoming	Vacuum reduction	Bighorn Basin, northwestern Wyoming	Fairly heavy	Colorado, Washington, Idaho, Wyoming	Not normally a problem asphalt, blamed for tenderness on one job
F2	AC-20	F, Wyoming	Vacuum reduction	Bighorn Basin, northwestern Wyoming	Fairly heavy	Colorado, Washington, Idaho, Wyoming	Not normally a problem asphalt, this supposedly solved above-mentioned isolated tender job (slightly harder grade)
H	AC-10	H, Texas Panhandle	Propane, deasphalting of flashed bottoms and blend back	Texas Panhandle area	Fairly light	Texas Panhandle, Oklahoma, New Mexico, Colorado, Kansas	Very low temperature susceptibility, good performance, minimum winter cracking
J	85-100 Penetration	J, Kansas	Vacuum reduction	West Texas, New Mexico, Wyoming	-	Midwestern United States	No problems before 1974, notable drop in viscosity for this penetration grade in about 1974, tenderness and slow setting since 1974
K	AR 2000	K, California	Steam vacuum, distillation	Santa Maria California coastal area	Very heavy	Central and Northern California	Low temperature susceptibility, high volatility, diesel-like oil is pumped into ground to soften crude to allow recovery
L1	AC-10	L, Arkansas	Vacuum reduction	Mostly Smackover, Arkansas, and other domestic	Heavy, cyclic, waxy, sour	Arkansas, Louisiana, East Texas, Mississippi	Fairly low temperature susceptibility, well known as exceptionally good asphalt
L2	AC-20						

Table 2. Properties of original asphalts.

Asphalt Code	Grade	Viscosity			Penetration (dmm)			Ring-and-Ball Softening Point (°F)
		77°F (poises x 10 ⁵)	140°F (poises)	275°F (poises)	77°F, 100 g, 5 s	39°F, 100 g, 5 s	39°F, 200 g, 60 s	
A1	AC-5	3.5	494	2.48	176	22	50	107
A2	AC-10	10.3	1218	3.6	106	14	34	116
A3	AC-20	35.0	2363	4.0	58	5	16	124
B1	AR 1000	3.2	556	1.3	131	8	38	105
B2	AR 2000	8.3	1037	1.8	85	5	22	112
B3	AR 4000	21.0	2142	2.4	50	1	8	120
C	AR 2000	6.5	554	1.89	111	5	26	110
D	AC-20	37.0	2140	3.99	58	6	19	126
E	AR 2000	24.0	736	1.83	60	4	14	124
F1	AR 4000	9.2	1571	3.55	80	8	23	116
F2	AC-20	12.5	1717	3.47	75	7	25	118
H	AC-10	8.5	1124	5.33	106	8	24	116
J	85-100	10.5	780	2.25	93	7	27	116
K	AR 2000	3.6	810	2.57	143	18	52	111
L1	AC-10	8.5	1036	3.2	115	11	30	108
L2	AC-20	14.0	1705	3.6	80	9	21	121

Table 3. Asphalt properties after TFOT and RTFOT.

Asphalt Code	Viscosity			Penetration (dmm)		Ring-and-Ball Softening Point (°F)	Loss (%)
	77°F (poises x 10 ⁵)	140°F (poises)	275°F (poises)	77°F, 100 g, 60 s	39°F, 200 g, 60 s		
After TFOT							
A1	5.5	1067	3.2	113	41	115	0.00
A2	24	2712	4.9	67	27	125	0.01
A3	82	6368	5.8	33	9	136	0.06
B1	7.9	900	1.6	85	21	111	0.74
B2	16	1338	2.2	58	12	117	0.46
B3	46	3210	3.0	33	6	128	0.44
C	13	1064	3.5	64	12	122	0.64
D	92	5236	5.3	37	14	135	0.07
E	44	1534	2.7	29	9	128	0.56
F1	35	4324	5.3	45	18	130	0.23
F2	36	4463	5.2	43	16	130	0.22
H	15	1823	7.4	73	-	124	0.00
J	24	1593	3.8	52	12	122	0.04
K	28	1946	4.9	65	23	128	1.55
L1	20	1875	3.9	70	19	124	0.01
L2	52	4614	5.8	48	11	130	0.03
After RTFOT							
A2	22	2497	4.8	65	27	126	0.14
B1	8.9	965	1.6	81	18	114	1.10
B2	19	1700	2.3	57	12	122	0.68
B3	68	3653	3.2	31	6	129	0.57
F1	27	3413	5.2	47	16	127	0.17

Table 4. Temperature susceptibility parameters before and after oven aging: penetration ratio, PTS, and penetration index.

Asphalt Code	Penetration Ratio			Penetration Temperature Susceptibility			Original Penetration Index ^a	Penetration Index ^b		
	Original	TFOT	RTFOT	Original	TFOT	RTFOT		Original	TFOT	RTFOT
A1	28	36	-	0.022	0.022	-	-0.48	+0.09	-0.04	-
A2	32	40	42	0.023	0.022	0.022	-0.30	-0.08	-0.06	-0.01
A3	28	27	-	0.024	0.024	-	-1.51	-0.57	-0.36	-
B1	29	25	22	0.028	0.029	0.027	-2.29	-1.49	-1.61	-1.23
B2	26	21	21	0.028	0.029	0.026	-2.37	-1.44	-1.58	-0.89
B3	16	18	19	0.028	0.027	0.027	-4.05	-1.48	-1.29	-1.29
C	23	19	-	0.026	0.024	-	-2.87	-1.02	-0.61	-
D	33	38	-	0.023	0.023	-	-1.03	-0.31	-0.23	-
E	23	31	-	0.024	0.028	-	-2.11	-0.49	-1.53	-
F1	29	40	34	0.027	0.024	0.025	-1.13	-0.93	-0.39	-0.67
F2	33	37	-	0.025	0.024	-	-1.31	-0.79	-0.49	-
H	23	-	-	0.023	0.022	-	-1.83	-0.08	+0.03	-
J	29	23	-	0.024	0.026	-	-1.84	-0.50	-1.11	-
K	36	35	-	0.022	0.021	-	-1.46	+0.07	+0.26	-
L1	26	44	-	0.027	0.023	-	-1.25	-1.30	-0.08	-
L2	26	46	-	0.023	0.023	-	-0.80	-0.15	-0.24	-

^aComputed by using Equation 2.

^bComputed by using Equation 1.

Table 5. Temperature susceptibility parameters before and after oven aging: PVN, PVN', and VTS.

Asphalt Code	PVN (77-275°F)			PVN'(77-140°F)			VTS (140-275°F)		
	Original	TFOT	RTFOT	Original	TFOT	RTFOT	Original	TFOT	RTFOT
A1	-0.30	-0.43	-	-0.62	-0.50	-	3.31	3.43	-
A2	-0.32	-0.37	-0.44	-0.46	-0.36	-0.49	3.39	3.46	3.45
A3	-0.81	-0.83	-	-0.71	-0.57	-	3.57	3.65	-
B1	-1.71	-1.77	-1.78	-0.90	-1.13	-1.13	3.98	3.99	3.99
B2	-1.60	-1.63	-1.58	-0.98	-1.28	-1.06	3.94	3.85	3.91
B3	-1.64	-1.67	-1.63	-1.02	-1.20	-1.16	3.97	3.93	3.93
C	-1.27	-0.89	-	-1.25	-1.37	-	3.61	3.35	-
D	-0.81	-0.85	-	-0.81	-0.59	-	3.53	3.65	-
E	-1.87	-1.90	-	-1.83	-2.03	-	3.77	3.73	-
F1	-0.65	-0.67	-0.66	-0.64	-0.50	-0.66	3.50	3.58	3.51
F2	-0.75	-0.73	-	-0.65	-0.53	-	3.56	3.61	-
H	-0.30	-0.32	-	-0.55	-0.63	-	3.04	2.98	-
J	-1.18	-0.98	-	-1.15	-1.26	-	3.61	3.45	-
K	-0.50	-0.40	-	-0.41	-0.74	-	3.50	3.33	-
L1	-0.40	-0.67	-	-0.50	-0.66	-	3.42	3.51	-
L2	-0.63	-0.48	-	-0.56	-0.34	-	3.53	3.53	-

Table 6. Temperature susceptibility parameters before and after oven aging: VTS and TES.

Asphalt Code	VTS (77-275°F)			VTS (77-140°F)			TES ^a °F		
	Original	TFOT	RTFOT	Original	TFOT	RTFOT	Original	TFOT	RTFOT
A1	3.65	3.59	-	4.27	3.88	-	-55	-44	-35
A2	3.64	3.62	3.61	4.09	3.90	3.92	-55	-33	-
A3	3.78	3.73	-	4.18	3.86	-	-27	-17	-36
B1	4.03	4.06	4.08	4.13	4.20	4.20	-28	-22	-22
B2	4.00	3.99	3.99	4.11	4.23	4.14	-22	-14	-17
B3	3.98	3.98	4.01	4.01	4.08	4.16	-18	-9	-
C	3.93	3.69	-	4.49	4.31	-	-33	-29	-
D	3.79	3.79	-	4.27	4.05	-	-29	-22	-
E	4.17	4.04	-	4.89	4.60	-	-28	-3	-20
F1	3.62	3.64	3.61	3.84	3.75	3.79	-27	-24	-
F2	3.69	3.65	-	3.93	3.74	-	-29	-22	-
H	3.40	3.33	-	4.06	3.97	-	-63	-37	-
J	3.90	3.75	-	4.45	4.29	-	-45	-20	-
K	3.64	3.64	-	3.88	4.22	-	-51	-27	-
L1	3.67	3.71	-	4.12	4.09	-	-47	-35	-
L2	3.69	3.66	-	3.99	3.88	-	-29	-26	-

^aTemperature at which stiffness of asphalt at 10 000-s loading time is equal to 20 000 psi, from nomograph for determining stiffness modulus of bitumens (15).

susceptibility after oven aging. Although oven aging will definitely affect the temperature susceptibility of most asphalts, there is no consistency in these effects. Temperature susceptibility of some asphalts increases with oven aging, whereas that of others may decrease and still others exhibit no appreciable change. TFOT and RTFOT induce almost identical changes in temperature susceptibility of individual asphalts (Tables 4-6).

Low-Temperature Asphalt Properties

Viscosity data from the capillary-tube viscometer, sliding-plate microviscometer, Schwyer rheometer, and the rheometrics spectrometer were plotted for selected asphalts on ASTM D2493 Viscosity-Temperature Chart for Asphalts. An example of these plots is shown in Figure 1.

Viscosities at 77°F measured by the sliding-plate viscometer are uniformly higher than those measured by the Schwyer rheometer. This is probably a result of the different shear rates employed and the configuration of the test specimens. Viscosities at 32°F measured by the Schwyer rheometer correlate fairly well with those of the Rheometrics device.

All asphalts appear to approach a viscosity of 10⁹ poises as an upper limit. The highly temperature-susceptible asphalts (C and E) approach this proximity at comparatively higher temperatures and

exhibit a notably sharper bend in the curve at approximately 32°F.

Field-Laboratory Test Program

The field-laboratory test program was undertaken to satisfy the second task as outlined by the NCHRP Project 1-20 panel. The intent of the program was to investigate in detail a minimum of four construction projects in which asphalt concrete placement difficulties were encountered and four projects in which placement difficulties were not encountered. The test program was aimed at locating and testing four tender or slow-setting asphalt concrete pavements. A review of the problems of tender mixtures will be presented prior to the results from the testing program.

Definitions and Causes of Tender Mixtures

Tender or slow-setting asphalt pavements have been defined by the Asphalt Institute as mixtures that have very low resistance to deformation by "punching loads" and/or scuffs under horizontally applied shearing loads after compaction has been completed (9). In general, tender mixes are difficult to roll, have difficulty in achieving a specified density, will occasionally rut, and will displace under high pressure and shove and scuff under traf-

Figure 1. Viscosity as function of temperature for asphalt F1 (after TFOT).

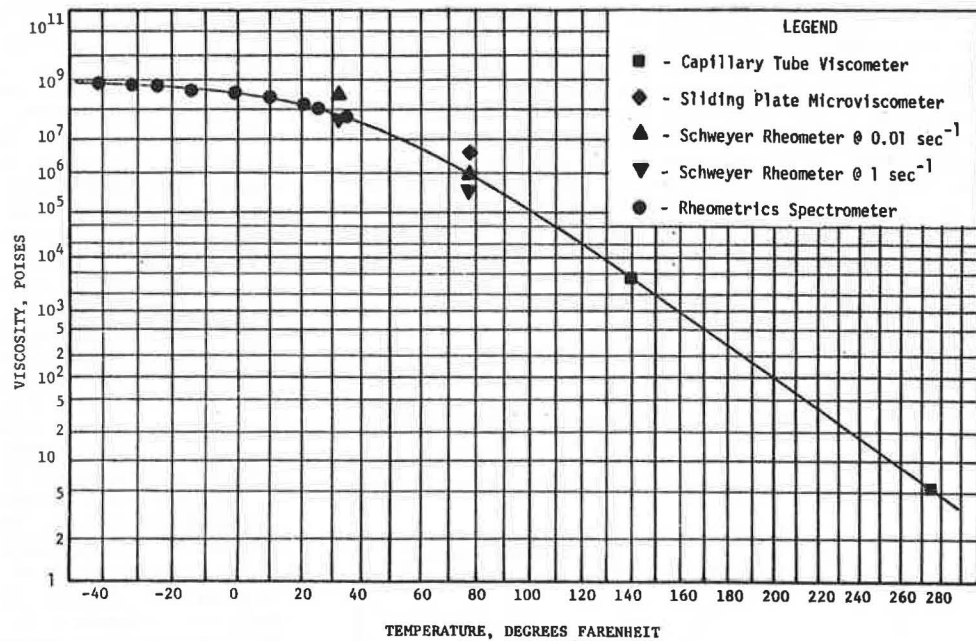


Table 7. Information on field cores tested.

Code	Construction Date	Pavement Location	Asphalt Description	Aggregate Description	Remarks of State Personnel
AG	1979	Ganado, Arizona; US-63	AR 2000, refinery C, temperature susceptible	Crushed rock, field sand	Very tender during and after construction, some rutting
AT	1979	Texas Canyon, AZ IH-10	AR 2000 + antistrip agent, producer same as AC (below), West Texas, temperature susceptible	River gravel with 40 percent crushed faces, river sand	Very tender during and after construction; additional No. 200 aggregate eliminated construction tenderness; may have contained excessive moisture; rutting, some flushing; used drum mixer and vibratory roller
AC	1975	Texas Canyon, AZ IH-10	AR 2000 + antistrip agent, producer same as AT (above), West Texas, temperature susceptible	River gravel with 40 percent crushed faces, river sand	Not tender, no rutting, generally performing well, used batch plant and pneumatic roller
I	1974	Western Iowa, IH-80	85-100 penetration, refinery J, temperature susceptible	75 percent crushed limestone, 25 percent field sand	Tender during and after construction; rutting soon after construction, asphalt concrete base rutting before surface course placed; weather hot during construction, batch plant
TA	1979	West of Amarillo, TX, IH-40	AC-10, same as asphalt H	Crushed limestone, field sand (somewhat dusty)	Slightly tender, blamed on fine aggregate; apparently water susceptible, blamed on dusty aggregate; no rutting; performance satisfactory
TS	1979	West of Sonora, TX, IH-10	AC-10, West Central Texas, temperature susceptible	Crushed limestone, field sand	Not tender during construction, used temperature-susceptible asphalt, no rutting
WW	1979	Clarkston, WA SR 129	AC-20 + antistrip agent, same as asphalt F2	Crushed gravel, river sand	Not tender during construction; asphalt contained 0.75 percent antistrip agent; performing well, no rutting
WL	1979	Clarkston, WA SR 129	AR 4000W + antistrip agent, same as asphalt F1	Crushed gravel, river sand	Somewhat tender during construction; asphalt contained 0.5 percent antistrip agent; performing well, no rutting

fic. However, some tender mixtures compact with little difficulty but remain soft and scuff under traffic.

It is evident that both the aggregate (grading, top size, particle shape, and particle surface characteristics) and the bituminous binder (viscosity, temperature susceptibility, chemical makeup) contribute to tenderness. However, it is a general belief that changes in the aggregate in a mixture are more effective in reducing or eliminating tenderness than changes in the consistency of the binder.

Test Program for Field Cores

The objective of this study is to attempt to establish a relationship between asphalt cement properties and construction problems. If relationships can be established, the engineer will be in a better position to accommodate the variability in asphalt properties and/or establish a rational specification for asphalt based on pavement performance.

On-going construction projects experiencing asphalt-related construction difficulties could not be located for testing according to the original plan; therefore, the test plan was modified to involve testing of mixtures from "completed" pavements that had experienced asphalt-related problems during construction or stability problems early in the pavement's life and similar constructed pavements that displayed acceptable performance during construction and early life.

Materials

Several cores 4 in in diameter were obtained from eight different pavements located in four different states. In each case, the asphalt concrete surface (the last overlay) was the layer of interest. One disadvantage was that the thickness of the layers was only about 1.5 in. Thus, core specimens sawed from the pavements were quite thin.

Table 7 gives a brief summary of the field specimens.

A subjective evaluation of the mixtures was made by the research team, based primarily on discussions with the highway department personnel supplying the mixtures and asphalts. The mixtures were rated according to tenderness on a scale from 0 to 5; zero represented no tenderness and 5 represented extreme tenderness. This was done to facilitate comparisons of laboratory-measured asphalt and mixture properties with difficulties experienced in the field. The values assigned to the mixtures are as follows: AG, 4; AT, 3; AC, 0; I, 4; TA, 1; TS, 0; WW, 0.5; WL, 1.5.

Testing

Pavement core specimens were subjected to a battery

of tests to determine the engineering properties of the mixtures, aggregates, and asphalt cements. Mixture tests included resilient modulus as a function of temperature, Hveem and Marshall stabilities, splitting tensile test, and water susceptibility.

After Hveem and Marshall stabilities and tensile properties of the core specimens had been determined, the failed specimens were heated to 250°F, mixed in a mechanical mixer, and "remolded" into cylindrical specimens 4 in in diameter and approximately 2 in in height. Molding was in accordance with test method TEX-206-F (Part II: Motorized Gyrotory-Shear Molding Press Operating Procedure) (10). As soon as the remolded specimens reached the appropriate temperature, the Hveem and Marshall stability tests and the indirect tension test were performed. According to Hveem, Zube, and Skog (11), tenderness of an asphalt mixture will recur on heating; that is, the slow-setting quality of an asphalt will reassert itself.

The resilient modulus of the core specimens was measured at several temperatures to define mixture temperature susceptibility. No consistent correlations between slope of the resilient modulus versus temperature curves and mixture tenderness were evident. However, it appears that mixtures with resilient modulus values (at 77°F) below 350 000 psi caused tenderness problems during construction.

Under controlled conditions, the resilient-modulus device has the potential for recognizing tender mixes and defining mixture temperature susceptibility. Figure 2 shows resilient-modulus test results from laboratory-compact specimens of subrounded gravel and crushed limestone of the same gradation with a single AC-10 asphalt (12). Figure 2 illustrates the dependence of resilient modulus on the aggregate characteristics. The round, smooth particles of the gravel aggregate produce less interparticle friction to aid in resisting shear stresses. The rough, angular particles of the crushed limestone produce considerable interparticle friction, which is manifested by the higher resilient moduli at higher temperatures. The overall temperature susceptibility of the gravel mixture is much greater than that of the limestone mixture.

Resilient modulus at 104°F as a function of time is plotted for mixtures WW and WL in Figure 3. The tender mixture (WL) exhibited a lower value of resilient modulus. Reasons for the decrease in resilient modulus during the first 8 h of specimen life have not been defined. Stress relaxation is a potential cause. Stresses imparted to the specimens

Figure 2. Resilient modulus of laboratory-compact specimens as function of temperature.

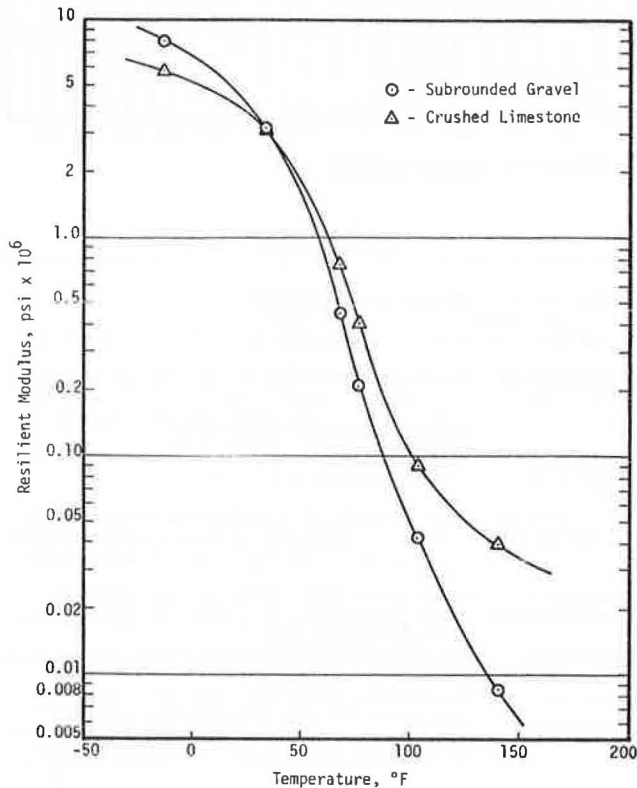


Figure 3. Resilient modulus of remolded Washington specimens versus time.

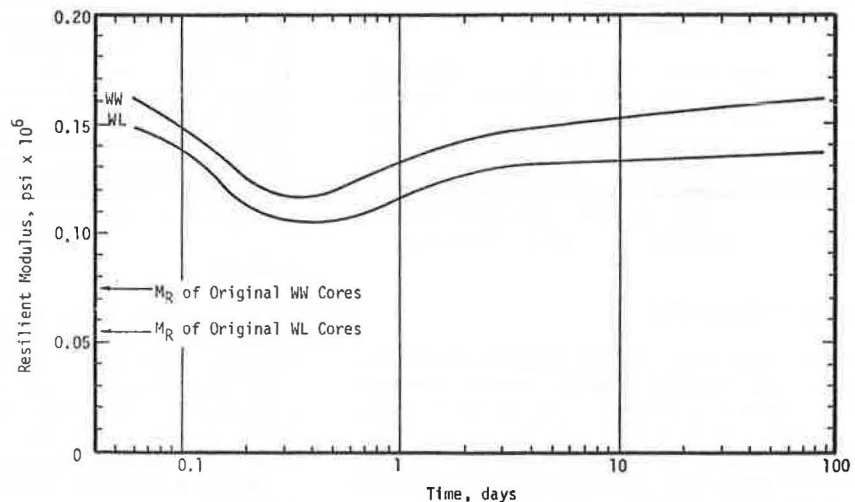


Figure 4. Relationship between mixture tenderness and resilient modulus.

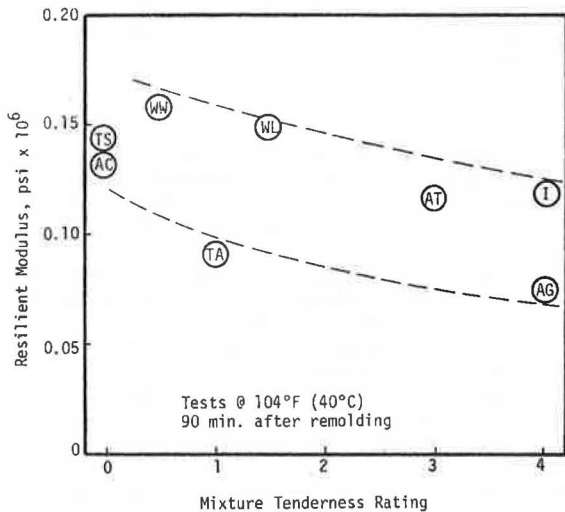
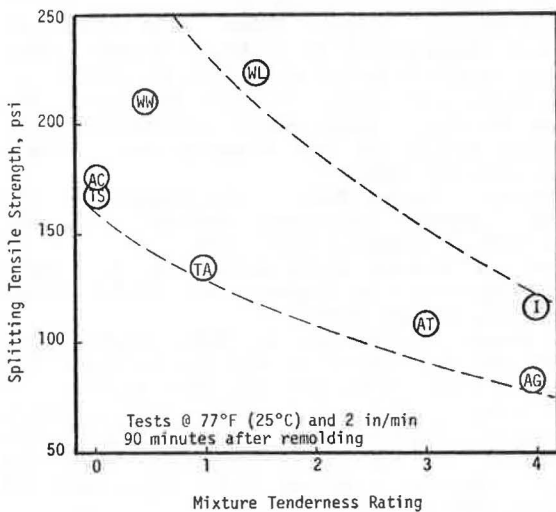


Figure 5. Relationship between mixture tenderness and indirect tensile strength.



by compaction and cooling are dissipating, which would diminish specimen stiffness. Stiffness of the specimens begins to increase almost immediately after the minima are encountered. It is postulated that early in the life of the compacted mixtures the rate of increase in resilient modulus depends on setting rate (structuring or thixotropic characteristics) of the asphalts and later on the age-hardening rate of the asphalt, both of which are related to the conditioning temperature. The resilient modulus at 104°F, 90 min after remolding, is compared with the subjective mixture tenderness ratings in Figure 4.

The splitting tensile test (indirect tension) performed at 77°F and at 2 in/min was employed to examine tensile properties of the specimens 90 min after remolding. Splitting tensile strength is compared with mixture tenderness rating in Figure 5, which shows increasing mixture tenderness with decreasing tensile properties. The influence of asphalt viscosity is shown by the higher tensile strengths of mixtures WW and WL. Tensile strength should exceed 125 psi if tenderness problems are to be avoided.

Considering the nonhomogeneity of these field

specimens and the uncertainty associated with the subjective rating of tenderness, drawing any conclusions from these experiments should be done with caution. Nevertheless, the relationships are mutually supportive and practical. Although preliminary in nature, these test results give strong evidence that resilient modulus and tensile properties, particularly of freshly compacted specimens, may be useful in predicting tender mixtures.

Summary of Field-Laboratory Test Program

Higher air-void contents of cores AG and AT when compared with those of core AC may be a result of tenderness during construction. However, it should be pointed out that the AC pavement was four years older. Prolonged traffic action densified pavement AC to some extent. Generally the original AC cores exhibited values of strength, stiffness, and stability significantly higher than corresponding values for AG and AT. Since all three mixtures contained asphalts with similarly high temperature susceptibilities, none of the differences in mixture properties or workability during construction can be attributed to variations in asphalt temperature susceptibility.

Mixture I exhibited tenderness during construction and instability during early pavement life. It contained an 85-100 penetration-grade asphalt from the producer of asphalt J. This is a highly temperature-susceptible asphalt, quite low in viscosity at 140°F, and fairly resistant to hardening. The aggregate was crushed stone of what appears to be a satisfactory gradation. The pavement was six years old when sampled and tested. It had a relatively low air-void content, which is likely due to its exposure to traffic and consequently resulted in comparatively high values of strength, stiffness, and stability.

Mixtures TA and TS contained crushed aggregates passing the same specifications. Mixture TA contained asphalt H, which has an exceptionally low temperature susceptibility and is highly resistant to oven hardening. The mixture was described as slightly tender during construction. Mixture TS contained a highly temperature-susceptible asphalt and was not tender during or after construction. The asphalt cement in mixture TS is usually furnished near the lower end of the 140°F viscosity specification and yet was harder than the asphalt recovered from mixture TA. Therefore, we may assume that the asphalt in mixture TS is hardened more during construction than the asphalt in mixture TA.

Mixtures WW and WL contained crushed gravel of virtually identical characteristics with similar quantities of AC-20 and AR 4000W asphalts, respectively, both from producer F. The asphalts are fairly temperature susceptible, and the only fundamental difference in the original asphalts is that the AC-20 is slightly harder. The slightly higher viscosity of the AC-20 was apparently just enough to eliminate the slight tenderness observed during construction with the AR 4000W. Although the original asphalts F1 and F2 have similar viscosities, the properties of the extracted and recovered asphalts reveal significantly greater hardening by asphalt F2, which was contained in mixture WW.

Relationships between mixture tenderness and asphalt temperature susceptibility are at best weak; nevertheless, they do appear to exist. However, a relationship between mixture tenderness and hardening resistance of the asphalt also appears to exist, which agrees with findings of other researchers (13). Sufficient data to illustrate this point were not generated in this experiment.

CONCLUSIONS

Conclusions obtained from data collected and analyzed during the Phase I study are given below:

1. Physical properties (including temperature susceptibility) of asphalt cements produced today have the same range of values as those produced in 1964 and immediately prior to the 1965-1973 preem-bargo period (14). Statistical computations performed by Pennsylvania State University (15) indicate that mean values of particular asphalt cement physical-chemical properties varied over the years on a national and regional scale. Data collected from specific refining sources in this study indicated that the physical properties of asphalt cements from selected refineries have changed with time, whereas asphalts from other refineries show no statistically significant change.

2. Although some asphalt cement properties have changed significantly from a statistical standpoint, it is uncertain whether these changes are significant from the standpoint of pavement construction and pavement performance. Limited analyses of information presented in the project indicate that the changes in asphalt cement properties will affect construction if adjustments in field compaction temperatures are not made.

3. Asphalt cements produced in the post-1976 period have a greater resistance to TFOT hardening and hence hot-mix hardening than do those asphalt cements produced prior to 1977. Asphalts tested in this study and obtained from 1979 to 1980 production have about the same range of basic properties as measured before and after TFOT as do those asphalt cements produced in 1977.

4. Temperature susceptibility of asphalt cements is affected very little by TFOT and RTFOT. On the average a slight increase in temperature susceptibility should be expected with oven aging. The effects of TFOT and RTFOT aging on asphalt consistency are nearly identical.

5. The correlation between asphalt cement properties and field tenderness was for the most part masked by variations in aggregate properties and/or construction techniques. However, highly temperature-susceptible asphalts have been related to tender pavements. These same asphalts exhibit undesirable low-temperature characteristics.

6. It is likely that the prediction of field tenderness will be based on mixture tests rather than asphalt cement tests. The resilient modulus and tensile strength of mixtures offer promise as methods for recognizing tenderness problems.

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration

and was conducted in the National Cooperative Highway Research Program, which is administered by the Transportation Research Board of the National Research Council.

REFERENCES

1. J.W. Button, J.A. Epps, D.N. Little, and B.M. Gallaway. Influence of Asphalt Temperature Susceptibility on Pavement Construction and Performance. NCHRP, Project 1-20, Interim Rept., Oct. 1980.
2. F.N. Hveem, E. Zube, and J. Skog. Proposed New Tests and Specifications for Paving Grade Asphalts. Proc., AAPT, Vol. 32, 1963.
3. Proc., Montana National Bituminous Conference, Sept. 1937, pp. 325-327.
4. R.H. Lewis and J.Y. Welborn. Physical and Chemical Properties of Petroleum Asphalts of the 50-60 and 85-100 Penetration Grades. Proc., AAPT, Vol. 12, 1940.
5. R.H. Lewis and J.Y. Welborn. Properties of Residues of 50-60 and 85-100 Penetration Asphalts from Oven Tests and Exposure. Proc., AAPT, Vol. 12, 1940.
6. F.N. Hveem. Asphalt Quality Controls. Presented at 33rd Annual WASHO Conference, Sun Valley, Idaho, Sept. 16-18, 1954.
7. H.E. Schweyer. Asphalt Cement Viscosities at Ambient Temperatures by a Rapid Method. HRB, Highway Research Record 404, 1972, pp. 86-96.
8. H.S. Pink, R.E. Merz, and D.S. Bosniack. Asphalt Rheology: Experimental Determination of Dynamic Moduli at Low Temperatures. Proc., AAPT, Vol. 49, 1980.
9. V. Marker. Tender Mixes: The Causes and the Cures. Asphalt Institute, College Park, MD, June 1977, Information Series 168.
10. Manual of Testing Procedures, Vol. 1. Texas State Department of Highways and Public Transportation, Austin, 1970.
11. F.N. Hveem, E. Zube, and J. Skog. Proposed New Tests and Specifications for Paving Grade Asphalts. Proc., AAPT, Vol. 32, 1963, p. 271.
12. J.W. Button, J.A. Epps, and B.M. Gallaway. Laboratory Evaluation of Selected Shale Oil Asphalts. Transportation Institute, Texas A&M Univ., College Station; U.S. Department of Energy, LERC-3695-1, 1978.
13. R.J. Schmidt, L.E. Santucci. Influence of Asphalt Type on Pavement Setting Rate. HRB, Bull. 333, 1962.
14. V.P. Puzinauskas. Properties of Asphalt Cements. Proc., AAPT, Vol. 48, 1979.
15. D.A. Anderson and E.L. Dukatz. Asphalt Properties and Composition: 1950-1980. Proc., AAPT, Vol. 49, 1980.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.