**US Army Corps of Engineers**
Waterways Experiment Station

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# Material Utilization in Military Pavement Systems (MUMPS) Program - FY 95

*By Larry N. Lynch, Marian P. Rollings, Reed B. Freeman,*
*J. Kent Newman, Randolph P. Ahlrich*

Prepared for Headquarters, U.S. Army Corps of Engineers

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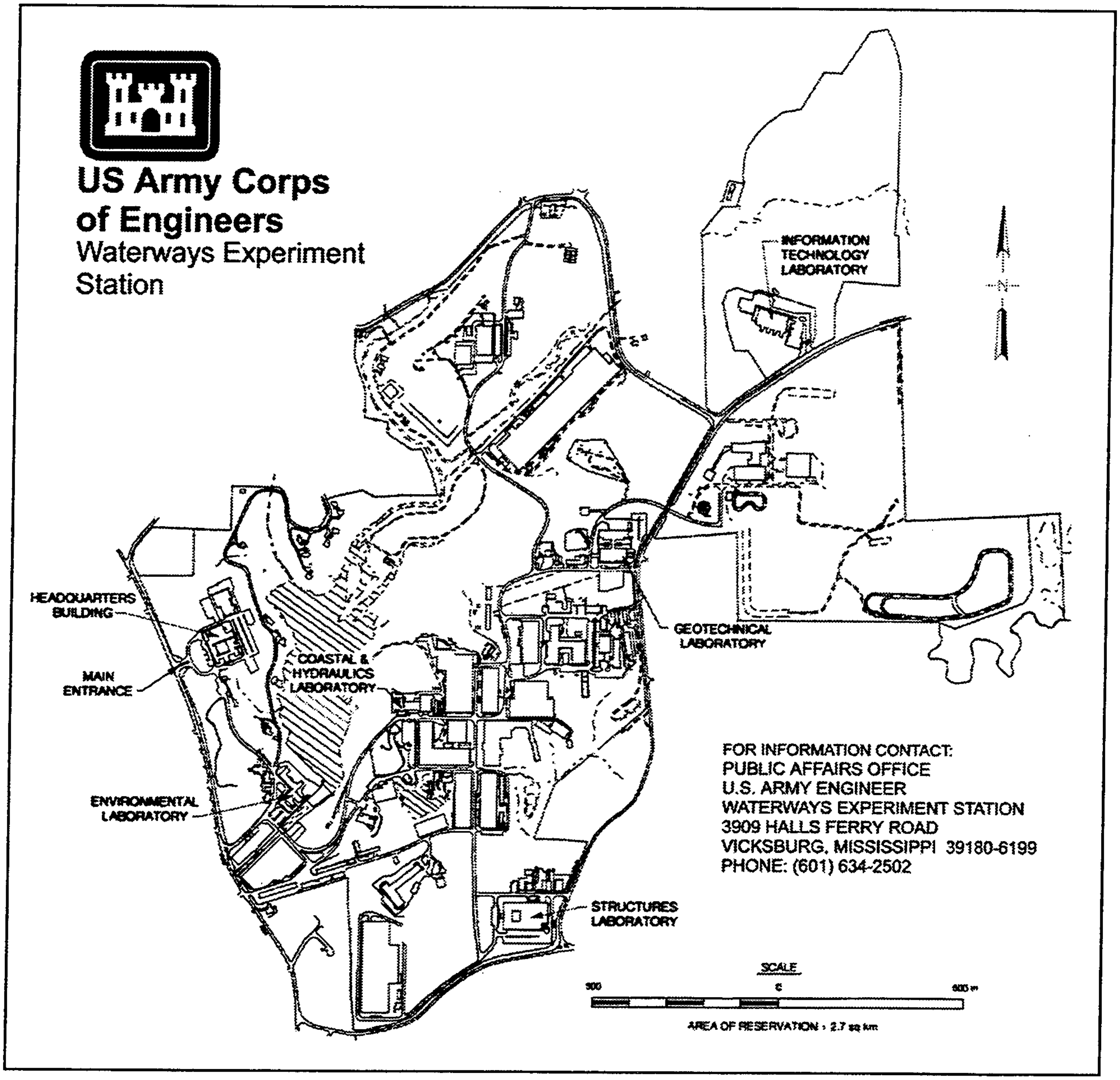

**US Army Corps of Engineers**
Waterways Experiment Station

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# Preface

This project was conducted by the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, during FY 95 and was sponsored by the Military Construction Appropriation Bill (Military Pavements Study), 1995, through Headquarters, U.S. Army Corps of Engineers (HQUSACE). The HQUSACE technical monitor was Mr. G. W. Hughes.

The project was conducted under the general supervision of Dr. W. F. Marcuson III, Director, Geotechnical Laboratory (GL), WES, and under the direct supervision of Dr. Raymond S. Rollings, Jr., Chief, Airfields and Pavements Division (APD), and Mr. T. W. Vollor, Chief, Materials Analysis Branch (MAB), APD. The WES Principal Investigator was Dr. Larry N. Lynch, MAB. This report was prepared by Drs. Lynch, Marian P. Rollings, Randolph C. Ahlrich, J. Kent Newman, and Reed B. Freeman. Laboratory testing support was provided by Messrs. J. K. Simmons, H. McKnight, R. Felix, R. T. Graham, and C. W. Dorman, MAB, Mr. H. T. Carr, Instrumentation Systems Division, and Ms. A. Williamson and Mr. D. Goodin, Applied Research Associates, Inc.

Dr. Robert W. Whalin was the Director of WES, and COL Bruce K. Howard, EN, was the Commander at the time this report was prepared.

# Executive Summary

Pavement research and development in the military and civilian sectors have never been more necessary nor more visible than at the present time. There are a number of reasons for the urgent need to accelerate the research and development efforts including, but not limited to: deterioration of our Nation's transportation infrastructure, increased congestion of our airports, proposed aircraft development of unprecedented size, depletion of paving materials and increased expense of existing pavement materials, increased dependence of our Nation's military on U.S. bases to project forces for regional conflicts. The Strategic Highway Research Program addressed many of the highway pavement industry needs, but these findings cannot be simply transferred to the military pavement industry because of the unique loading and traffic demands placed on military pavements.

The Department of Defense (DOD) approved a Tri-Service Civil Engineering Science and Technology Plan that included the subarea of "Airfields and Pavements." The impetus of this research was to sustain and extend the life of DOD pavements. The Department of the Army (DA) pavement system consists of approximately 565 million sq m (676 million sq yd) of pavements and annual maintenance expenditures of approximately $124 million. The DA pavement system includes approximately 74,230 km (46,000 miles) of roadways (comparable in size to the Interstate system), 4,023 km (2,500 miles) of rail (comparable in size to the Amtrak system), and approximately 150 airfields.

It has been estimated that increasing the service life of a pavement by 1 year will reduce maintenance costs by 20 percent and construction costs by 10 percent. Currently, it costs approximately $2.5M to reconstruct a 3,048-m (10,000-ft) runway using asphalt concrete. A 1-year extension in the service life would provide a $250K savings in construction costs alone, not including maintenance costs and the costs of relocating the mission during the reconstruction. If the service life was extended by 5 years, the construction cost savings would increase to $1.2M, approximately one-half of the initial reconstruction cost. If one runway at each of the 150 DA airfield locations were reconstructed, the savings would rapidly increase and could reach approximately $180M. If additional runways, taxiways, or parking apron pavements were included, the estimated savings would exceed $824M for the DA.

The major pavement distress area for DOD airfield pavements, as determined by a Tri-Service selection committee, was durability related distresses in asphalt pavements. Based on this major distress area and the original objectives described in HR 103-516 (which was the impetus for this research), the project objectives were:

*a.* Develop a methodology based on laboratory testing to quantify the improved resistance of asphalt pavements to durability distresses resulting from binder modification.

*b.* Provide current practice information on new or manufactured and waste or by-product materials used as modifiers and/or fillers in asphalt pavements.

The generalized approach to achieve the objectives included: obtain two different asphalt cements and four different modifiers for laboratory evaluation including unaged and aged binder testing and unaged and aged mixture testing, and determine the current industry practice involving asphalt binder modification through a market survey and literature review.

The addition of modifiers to asphalt pavements typically increases the initial costs of the pavement by 10 to 40 percent depending upon the modifier selected. Methodologies to predict or quantify modified hot mix asphalt (HMA) pavement performance are required to justify the increased initial cost through reduced life cycle cost. The development of a methodology is critical because of the reported variances in modified HMA performance. For example, in one specific application, the use of a modifier postponed all maintenance work on the specific pavement for over 10 years. However, a similar modifier used in similar application but a different climate exhibited no improvement in performance. Successful development of a laboratory-based methodology would allow user agencies to select the best material for a given application which would promote field performance closer to the first example of reduced maintenance costs rather than the second example of simply increased initial costs. Minimizing maintenance activities on pavements by a single year would provide a significant savings when considering the annual maintenance expenditure of $124M. Each additional year that maintenance activities could be postponed through the selection of the best technology or modifier would very quickly demonstrate even more significant savings.

The general conclusions from the Material Utilization in Military Pavement Systems (MUMPS) research program were:

*a.* Significant strides were made in this project toward developing the methodology required to develop a simplified process (as compared to the current process used in military construction) to evaluate materials and technologies, thus allowing the use of new and innovative materials and technologies in military pavement construction projects.

*b.* The recyclability of an HMA pavement containing a waste material or modifier must be determined before allowing widespread use of that particular material in HMA.

*c.* The use of a waste material or modifier in an HMA should be based on engineering, economic, and environmental factors, not legislated mandates. For example, the incorporation of a waste material or modifier into an HMA pavement should not adversely affect the performance of the pavement and preferably should enhance pavement performance.

*d.* Conventional binder and HMA tests have limited usefulness when evaluating modified binders and modified HMA due to their empiricism.

*e.* The highway industry is adopting the Strategic Highway Research Program (SHRP) binder testing criteria for use in HMA construction projects, but this criteria may not be directly applicable to airfield pavements because of their unique load requirements. For example, in the past, highway mix design methods and criteria have required modification before they could be used for airfield applications because of the differences in traffic loadings.

*f.* The predominant concern in relation to military airfield HMA pavements is durability related distresses.

The general recommendations based on the FY 95 MUMPS program include:

*a.* Complete the modification of the repeated-load deformation test and the development of dynamic testing in the linear viscoelastic region of the HMA. These two tests provide the data required for numerical analysis, thus allowing life-cycle costs of specific modifiers to be compared for a specific set of conditions.

*b.* Use the two finite element programs that were evaluated in the research effort to predict pavement performance. These programs will accept the data generated from the above mentioned tests.

*c.* Determine recyclability of modified HMA pavements.

*d.* Evaluate the SHRP adopted binder specification criteria to determine if any modifications are required before being applied to airfield pavement projects (this must include field test sections to verify the criteria).

*e.* Develop correlation between the accelerated laboratory HMA aging techniques used in the MUMPS project to the aging of HMA pavements in the field. The HMA aging techniques used in the laboratory were severe and delineated differences in the modified HMA mixture.

The results from the FY 95 MUMPS project indicate that it is possible to develop a procedure that can provide performance-related data on binder and HMA mixtures, thus allowing the use of new and/or waste materials in military pavement construction projects. To achieve the original objectives of the MUMPS project, a commitment must be made to a multiyear project which would include additional laboratory evaluations and more importantly, field evaluations. The development and verification of performance-related criteria must be made with actual field data, not assumed field conditions.

# 1 Introduction

Pavement research and development in the military and civilian sectors have never been more necessary nor more visible than at the present time. There are a number of reasons that point to the urgent need to accelerate the research and development efforts. Some of the reasons that delineate this need include: the deterioration of our Nation's transportation infrastructure, increased congestion of our airports, proposed aircraft development of unprecedented size, depletion of paving materials and increased expense of existing pavement materials, and the increased dependence of our Nation's military on U.S. bases to project forces for regional conflicts.

The Department of Defense (DOD) approved a Tri-Service Civil Engineering Science and Technology Plan that included the subarea of "Airfields and Pavements." The impetus of this research was to sustain and extend the life of DOD pavements. The Department of the Army (DA) pavement system consists of approximately 565 million sq m (676 million sq yd) of pavements and annual maintenance expenditures of approximately $124 million. The DA pavement system includes (see Table 1) approximately 74,230 km (46,000 miles) of roadways (comparable in size to the Interstate system), 4,023 km (2,500 miles) of rail (comparable in size to the Amtrak system), and approximately 150 airfields (Headquarters, Department of the Army (HQDOA) 1994). These figures do not include the Departments of the Air Force and Navy, but if one simply assumes that the Departments of the Air Force and Navy also have similar pavement systems, it becomes evident that a large investment is required to maintain the DOD pavement system.

**Table 1**
**Department of the Army Pavement System**

| Pavement System Item | Amount of Pavement System Item |
|---|---|
| Roadways | 74,230 km (46,000 miles)<br>*comparable to the size of the Interstate system* |
| Railways | 4,023 km (2,500 miles)<br>*comparable in size to the Amtrak system* |
| Airfields | 150 airfield locations |

There are three major facets required for rational pavement analysis. The three facets include laboratory analysis and material characterization, analytical modeling, and field validation. None of the three facets could be used independently to provide a design methodology, to predict the performance of a given pavement structure, and/or to determine the practicality of constructing the pavement structure. In the past, when conventional materials were used in the pavement construction process, each of the three facets had an extensive database that would allow pavement design, performance prediction, and construction practicality. However, the current trend of using additives, modifiers, and/or fillers in pavements as a means of enhancing specific attributes of the pavement or to dispose of waste materials has led to the necessity of improved material characterization and laboratory testing techniques, new analytical approaches, and rapid evaluation for verification.

## Project Background

In October 1994, the U.S. Army Engineer Waterways Engineer Experiment Station (USAEWES) was tasked by the Headquarters, U.S. Army Corps of Engineers (HQUSACE), to develop a scope of work in response to Bill Report HR 103-516. The project length and funding as described in HR 103-516 was 3 years and $2.5 million per year. The objectives of this original scope of work as outlined by HR 103-516 were:

*a.* Determine applications of new pavement materials in military roads, airfields, and other paved areas.

*b.* Determine the degree to which waste by-products and recycled materials (i.e., ground tires and plastics) can be used in military pavements.

*c.* Evaluate the requirements for performance-based specifications for military pavement construction, including asphalt, concrete, and unpaved surfaces.

*d.* Develop a simplified specification process to allow the integration of new materials and technologies into the construction of military pavements.

The objectives were to be achieved by evaluating the field performance of candidate materials through laboratory characterization and field test evaluations. Additional areas of interest included in the investigation were the environmental impact of using the new or waste materials in pavement systems, the effect of these materials on the performance of the pavement including the maintenance and life-cycle cost of the pavement, and potential constructability problems associated with their use. However, these original objectives were refocused due to reduced funding. The next few paragraphs provide a chronological accounting of the events that occurred during FY 95. Table 2 presents a chronological summary of the major items associated with the project that have been completed to date.

## Table 2
## Chronological Summary of FY 95 Project Events

| | |
|---|---|
| October 1994 | WES was tasked by HQUSACE, CEMP-ET to develop a scope of work based on the language of HR 103-516 (funding \$2.5M/year, project length 3 years). |
| Mid-October 1994 | Meeting held at CEMP-ET with WES and DAIM-FDR-X personnel to discuss scope of work. |
| Late-October 1994 | WES submitted a revised scope of work based on direction from mid-October meeting (funding \$200K, project length 3 months). |
| Mid-November 1994 | (1) Presented project objectives to industry and Government agencies at a meeting in Dalton, GA.<br>(2) Initiated a market survey. |
| Mid-December 1994 | DoD selection committee established and meeting held at WES - prioritized pavement distress areas and selected candidate materials. |
| January 1995 | A project review meeting was held at WES and a final report of the \$200K project was provided to HQUSACE, CEMP-ET. |
| February - Mid-June 1995 | Review of project objectives, approach, and funding requirement by DAIM-FDR-X and CEMP-ET. |
| Mid-June 1995 | A funding increment of \$750K was provided to WES. A new scope of work was developed based on the reduced funding and direction from DAIM-FDR-X and CEMP-ET. |
| July 1995 | Procurement of materials required for laboratory evaluation (aggregates, asphalt cements, etc.). |
| August - September 1995 | Completed laboratory test plan and other items necessary to initiate laboratory investigation. |
| Mid-October 1995 | Received necessary materials and initiated laboratory evaluation. |
| Mid-December 1995 | (1) Completed laboratory evaluation of two asphalt materials and four modifiers.<br>(2) Submitted draft report to CEMP-ET. |

In mid-October 1994, a meeting was held at CEMP-ET to discuss the original comprehensive scope of work. At this meeting, WES personnel were instructed by DAIM-FDR-X to initiate a small portion of the scope of work and to provide documentation for specific items that could be accomplished for an initial funding of \$200K. The portion of the scope of work that WES was specifically instructed to initiate was to conduct a market survey of potential products for use in military pavements and to attend a project meeting in Dalton, GA, which would include representatives from both the Government and private industry.

The revised scope of work was submitted to Corps of Engineers, Military Programs, Engineering Division, Technical Engineering Branch (CEMP-ET) in late October 1994. The objectives of the revised \$200K scope of work included:

*a.* Conduct a brief literature review concerning current practice in the use of new and waste materials.

*b.* Complete a market survey on potential products for use in military pavements.

*c.* Begin the development of a rigid and flexible pavement distress area database to determine the major pavement distress areas which occur in military pavement.

*d.* Establish a DOD selection committee to prioritize the pavement distress areas for investigation and the types of new and waste materials that should be evaluated in the laboratory portion of the investigation.

In mid-November 1994, a project meeting was conducted in Dalton, GA. The purpose of this meeting was to obtain input from private industry and Government agencies concerning the approach proposed for the investigation. This input was believed to be critical so that the findings of the investigation could be rapidly implemented into pavement construction projects.

Also in mid-November 1994, WES initiated a market survey which included reviewing product information previously submitted to WES by manufacturers, listing information of materials from the "Surfaced Area Material Utilization Catalog," and publishing an advertisement in the Commerce Business Daily (CBD). The CBD advertisement was used to ensure that all manufacturers were provided an opportunity to submit information concerning their products. Information on a total of 114 materials was collected through the market survey -- 56 asphalt related products, 48 concrete related products, and 10 soil related products.

The DoD selection committee was formed, and a meeting was held at WES in mid-December 1994. The members of the selection committee were:

| | |
|---|---|
| Dr. Larry Lynch | USAE Waterways Experiment Station |
| Mr. Terry Sherman | U.S. Army Corps of Engineer/Transportation Systems Mandatory Center of Expertise |
| Mr. Malcolm Martin | HQ, Training and Doctrine Command |
| Mr. Ali Achmar | U.S. Army Center for Public Works |
| Mr. James Greene | Air Force Civil Engineering Support Agency |
| Mr. Cliff Sander | HQ, Air Combat Command |
| Mr. Charles Schiavino | Northern Division, Naval Facilities Engineering Command |
| Mr. Kerry Nothnagel | Atlantic Division, Naval Facilities Engineering Command |

At this meeting, the pavement distress areas were prioritized for research, and the materials for laboratory evaluation were selected. The pavement distress area priorities established by the committee and the definition (as determined by the committee) of the pavement distress areas are as follows:

*a. Durability related distresses of asphalt pavements.* This category included asphalt mixture properties and distresses associated with both low and high temperatures. Additionally, distresses associated with temperature cycling and weathering caused by thermal, oxidative, and ultraviolet (UV) radiation processes as well as stripping were included in this category.

*b.* *Durability related distresses of concrete pavements.* This category included joint and corner spalling, freeze-thaw damage, and alkali-silica reaction degradation. The committee agreed that one way to address this problem would be not only to investigate modifiers and admixtures but to also examine the criteria used for the concrete mix design gradation. Specific methods such as those used in the Shiltstone method and current procedures used in Australia were mentioned as potential areas to investigate.

*c.* *Pavement joint sealant materials.* The largest portion of pavement distress in the concrete pavement database was related to joint sealant damage. The committee agreed that even though there was on-going research in the joint sealant area, the sealants should be included in this research for completeness. The focus of this research would be the development of new materials that do not require "hospital" clean joints.

*d.* *Load-related distresses on concrete pavements.* This category included corner breaks and slab cracking. One way the committee recommended that this could be addressed was to investigate the mix proportioning criteria in addition to investigating modifiers and admixtures.

*e.* *Load-related distresses on asphalt pavements.* The load-related distresses associated with airfield pavements were almost nonexistent according to the data collected for the airfield pavement evaluation database. This conclusion was reinforced by those present at the meeting. However, load-related distresses cannot be completely overlooked because some additives and modifiers used to improve the durability aspects of asphalt pavements can potentially decrease the ability of the mix to resist permanent deformation.

*f.* *Unsurfaced pavements.* The two basic areas often considered as potential problems with unsurfaced pavements are dust control and soil stabilization. The selection committee did not rate this area as a high priority. The committee recommended that if any work was conducted in this area, it should focus on the implementation of techniques and results from previous and existing research.

The selection committee prioritized 15 asphalt related products and 23 concrete related products for laboratory evaluation. The more promising products as determined from the laboratory evaluation would then be used in field evaluations to verify performance models developed through analytical means. Additionally, the selection committee decided that the major emphasis of this research should be airfield pavements. The committee agreed that the findings from the recently completed Strategic Highway Research Program (SHRP) should be evaluated for nonairfield applications.

On January 19, 1995, a project review meeting was conducted at WES in which the status of the alternate $200K plan of test was briefed to HQUSACE, CEMP-ET, representatives. A final report (Lynch and Hammitt 1995) of the $200K project was provided to HQUSACE, CEMP-ET, January 31, 1995. The submittal of the report completed the $200K project.

In June 1995, a funding increment of $750K was provided to WES to continue a portion of the Material Utilization in Military Pavement Systems (MUMPS) project. WES was instructed by the U.S. Army to limit the scope of work to asphalt pavement modification and the scheduled completion date for the project was December 1995. The remainder of this report documents the revised research project.

## Objectives

The major pavement distress area for DOD airfield pavements, as determined by the selection committee, was durability-related distresses in asphalt pavements. Based on this major distress area and the original objectives described in HR 103-516, the objectives of this research project were to:

*a.* Develop the basis of a methodology based on laboratory testing that can quantify the improved resistance of asphalt pavements to durability distresses resulting from binder modification.

*b.* Provide current practice information on new or manufactured and waste or by-product materials used as modifiers and/or fillers in asphalt pavements through a brief literature review.

*c.* Provide technical recommendations on requirements to advance from the conclusions of this research to the original objectives outlined in HR 103-516.

Without field verification, the development of criteria used in a performance-based specifications would not be defensible. Therefore, this research only addressed laboratory quantification to provide an initial basis for performance-based specifications.

## General Approach

The generalized approach to develop a laboratory-based methodology to quantify the improved resistance of asphalt pavements to durability distresses resulting from binder modification included the following. Two different asphalt cements were obtained and modified with four different modifiers. The modifiers selected were a styrene butadiene rubber (SBR), a styrene butadiene block co-polymer that reportedly reacts with the asphalt binder (this modifier will be referred to as a reacted SBS or RSBS), a modified ground tire or crumb rubber (MCR), and a recycled low density polyethylene (LDPE). Binder and mixture tests were then conducted on aged and unaged samples in an effort to quantify improved resistance to aging as a result of modification.

A brief literature review was conducted to determine the current industry practice involving asphalt binder modification. The information included a description of the materials used, areas of application, and potential advantages and disadvantages of modification. The current practice information was divided into two categories; new materials and waste or recycled materials.

# 2 Modifiers, Additives, and Fillers for HMA Pavements

## Waste Materials and By-products

### Introduction

The paving industry has received increasing pressure to incorporate waste materials and by-products into pavement structures. The increasing pressure has been a result of overwhelming environmental enthusiasm exhibited throughout the nation, the decreasing availability of landfill space (Flynn 1992), the large amount of space that the transportation corridors afford for the potential incorporation of waste and by-products, and the fact that once incorporated or encapsulated into a pavement structure, the potential exposure of the general public to the waste material would be minimal. Additionally, legislation has been enacted at both the state and national level requiring the incorporation of waste and/or by-products into pavement systems or as a minimum to investigate the feasibility of using waste materials and by-products in pavement systems (Kandhal 1993a). The most notable of the legislation passed concerning the incorporation of waste materials into pavements was the Federal Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991.

The ISTEA mandated that states receiving Federal highway funds must use recycled rubber in asphalt pavements. The amount of recycled rubber used by the states was determined as a percentage of the total tons of asphalt laid. In 1994, a percentage (5) of the total tons of asphalt laid by each state was to contain recycled rubber. The percentage of total tons required to include recycled rubber increased to 10 percent in 1995, 15 percent in 1996, and 20 percent in 1997. The percentages of recycled rubber required in asphalt pavements included in ISTEA were never implemented, and in 1995 the United States Senate passed legislation which established the National Highway System (NHS). Among other issues addressed, the NHS bill repealed the ISTEA mandate to include recycled rubber in asphalt pavements on federal highway projects (Betters Roads 1995). The NHS did not repeal state legislation concerning the use of waste materials in pavement structures. However, it did reduce some of the legislative pressure on the pavement industry and will allow sound engineering judgment coupled with economic factors to take a leading role in determining utilization of waste materials in pavements.

Tons of waste materials and by-products are generated every day from every segment of society. The two major categories of waste materials are municipal or domestic waste and industrial waste. Sometimes a third category is used for mineral wastes from mining and natural resource extraction. Table 3 lists typical types and quantities of various wastes generated in the United States that might be considered for use in hot mixed asphalt (HMA). For purposes of this report, further discussion of mineral wastes will be included under industrial wastes. The wastes currently being incorporated by state Departments of Transportation (DOT) into HMA pavements are listed in Table 4; this information was obtained through a survey of states (Ciesielski and Collins 1993).

**Table 3**
**Classification and Annual Quantities of Solid Waste Materials and By-Products with Potential Use in Hot Mix Asphalt-Concrete Pavements (after Ciesielski and Collins 1993)**

| Waste Category | Description of Waste | Annual Quantity Produced in the U.S. ($10^6$ metric tons) | Total Annual Quantity |
|---|---|---|---|
| Domestic | Household and commercial refuse | 34 | 34 |
| | Glass | 11 | |
| | Plastics | 13 | |
| | Incinerator ash | 7.8 | |
| | Scrap tires | 2.2 | |
| Industrial | Coal ash | 65 | 206 |
| | Blast furnace slag | 15 | |
| | Steel mill slag | 7 | |
| | Nonferrous slags | 9 | |
| | Reclaimed asphalt concrete pavement | 91 | |
| | Reclaimed concrete pavement | 3 | |
| | Foundry wastes | 9 | |
| | Roofing shingle waste | 7 | |
| Mineral | Waste rock | 925 | 1,395 |
| | Mill tailings | 470 | |
| Total | | | 1,635 |

When the estimated annual quantities of waste generated in the United States is considered, along with diminishing landfill space and difficulty in locating and permitting new landfills, it is obvious why emphasis is being placed both on society and industry to reduce waste, reuse materials, and/or recycle waste

## Table 4
## Waste Materials Used by State Departments of Transportation (after Ciesielski and Collins 1993)

| Waste Material | Number of States Using |
|---|---|
| Reclaimed Asphalt Pavement | 44 |
| Scrap Tires | 38 |
| Iron and Steel Slags | 13 |
| Reclaimed Concrete Pavement | 7 |
| Mining Wastes | 7 |
| Waste Glass | 6 |
| Coal Fly Ash | 5 |
| Coal Bottom Ash | 4 |
| Plastic Waste | 3 |
| Kiln Dusts | 2 |
| Incinerator Ash | 2 |
| Roofing Shingle Waste | 2 |
| Broken Concrete | 1 |
| Foundry Waste | 1 |
| Quarry Waste | 1 |
| Nonferrous Metal | 1 |

materials. Reduce, reuse, and recycle have been the three R's of the late 1980's and early 1990's. One of the often discussed means of recycling various waste materials which have not found other more productive uses has been incorporation into highway and airfield pavements. Although this use would appear to minimize human exposure and provide a permanent solution to waste disposal, these materials cannot be incorporated in engineered products such as asphalt and portland-cement concrete pavements without proper testing and evaluation to assess their impact on pavement performance. Waste materials should not be incorporated into pavements simply to dispose of the wastes; they should only be included when this inclusion will enhance some material characteristic or the performance of the pavement.

The following sections of this chapter provide a review of the current state of practice regarding use of the most promising and/or more common waste materials for inclusion in HMA. These waste materials may act as a chemical modifier to the asphalt cement, as an additive to impart particular properties or characteristics to the binder or HMA, or simply as a filler in the HMA.

## Municipal solid waste products

**Rubber tires.** Old, used tires constitute such a large volumetric waste and pose such significant problems at conventional landfills (e.g., hold fluids, cause air pockets, cannot be compacted, etc.) that they are normally excluded from municipal solid waste (MSW) landfills and are routinely removed from the incoming waste stream. Kandhal (1993a) reports that approximately 285 million tires are discarded annually in the United States alone; about 55 million of these are refurbished and used on vehicles, and about 42 million are used in some other fashion (Heitzman 1992). U.S. Environmental Protection Agency (USEPA) estimates that 2 to 3 million scrap tires are currently available for use (Ciesielski and Collins 1993). A number of Federal and state regulations have been enacted to address the waste tire problem. The ISTEA of 1991 required the use of waste tires in highway construction. Many of the issues concerning the use of waste tires in the ISTEA have been rescinded; however, regulations in about 35 states remain in effect. When tires are used in pavement construction, approximately 60 percent of each tire (the rubber portion) can be used; the other 40 percent (composed of 20 percent steel and 20 percent fibers and other materials) must be discarded (Estakhri, Button, and Fernendo 1992).

The main use for waste tires in highway construction is as ground (crumb) rubber in HMA concrete. Terminology relating to use of crumb rubber in HMA is identified in Table 5. Two procedures are available for incorporating crumb rubber into HMA. The crumb rubber may be mixed with the hot asphalt cement prior to the addition of aggregate (the *wet process* or *asphalt rubber*). Rubber particle sizes used in the dry process range from 12.5 mm (0.49 in.) to 75 $\mu$m (0.0029 in.). In the wet process, the rubber acts as an asphalt modifier when sufficient time and heat are provided to depolymerize the rubber. The wet process is the most common method of using rubber in HMA. Small-sized particles are typically used in this process to enhance the asphalt-rubber reaction.

**Table 5**
**Relationship of Crumb Rubber Modifier Terminology (after Heitzman 1992)**

| Material | Process | Technology | Product |
|---|---|---|---|
| Crumb Rubber | Wet | McDonald | Modified Binder<br>(asphalt-rubber binder) |
| | | Continuous blending | |
| | Dry | PlusRide | Rubber Aggregate<br>(rubber-modified HMA) |
| | | Generic | |
| | | Chunk rubber | |

Alternately, the rubber may be substituted for a portion of the aggregate, in which case it would be added to the aggregate before mixing with the asphalt (the *dry process* or *rubber-modified mixture*). In the dry process, some reaction may occur between the asphalt and the rubber, however the main purpose of including the rubber in this instance is "to provide solid elastomeric inclusions within the asphalt-aggregate matrix. These inclusions are thought to provide more rebound under traffic loading. When the dry process is used, a stable, long-term reaction with the rubber is important" (Stroupe-Gardiner, Newcomb, and Tanquist 1993). Table 6 gives information on the number of projects and quantities of rubber-modified asphalt used in the United States prior to the recent interest in rubber (Takallou and Hicks 1988).

**Table 6**
**Summary of Rubber-Modified Asphalt Projects in the United States**

| Year | No. of Projects | Tons of Mix |
|---|---|---|
| 1979 | 1 | 90 |
| 1980 | 1 | 1,700 |
| 1981 | 4 | 3,000 |
| 1982 | 8 | 5,867 |
| 1983 | 6 | 15,886 |
| 1984 | 7 | 18,883 |
| 1985 | 14 | 20,315 |
| 1986 | 11 | 38,370 |
| Totals | 52 | 104,111 |

The rubber from tires is ground into crumb rubber in one of two basic ways. It is ground at ambient (room) temperature or it may be ground at very low temperature (cryogenically ground). The crumb rubber produced by ambient grinding has a rougher and larger surface area which allows relatively rapid reaction with the asphalt cement. Cryogenically ground crumb rubber has clean, flat surfaces that retard asphalt-rubber reaction (Kandhal 1993a). It is much more expensive to produce, and some states prohibit its use because of its resultant surface texture (Amirkhanian 1993).

When rubber is added to asphalt, the rubber swells and softens in addition to reacting with the asphalt; the viscosity of the mix increases as the rubber and asphalt react. In fact, the high-temperature viscosity of an asphalt cement can be increased by a factor of 10 or more with the addition of 15 percent rubber (Heitzman 1992). The reaction between the asphalt and the rubber is enhanced when the asphalt contains a high percentage of light fractions (Stroupe-Gardiner, Newcomb, and Tanquist 1993). This can be achieved by selecting a low-viscosity asphalt or by adding an extender oil (Takallou and Sainton 1992). Either approach provides aromatics for absorption by and digestion of the rubber particles and reduced asphalt/oil viscosity to compensate for the increased viscosity when rubber is added. Table 7 gives some typical properties of asphalt rubber. A laboratory investigation conducted by WES and others (Hansen and Anderton 1993) recommended that softer asphalts be used for asphalt-rubber binders. This recommendation was made based on the fact that the performance of asphalt rubber produced with AC-5 asphalt cement was

**Table 7**
**Typical Properties of Asphalt-Rubber Binders (after OECD 1984)**

| **Properties** | **Range of Values** |
|---|---|
| Consistency | |
| Viscosity at 117 °C | 1 to 8 Pa.s |
| Viscosity at 60 °C | 700 to 6000 Pa.S |
| Ring and Ball Softening Point | 55 to 70 °C |
| Modulus* at 4 °C | |
| At loading times of | |
| 0.5 seconds | 700 to 17,000 kPa |
| 0.2 seconds | 1,700 to 27,600 kPa |

better than an unmodified AC-20. Binder tests used for this evaluation included softening point, resilience, penetration, and tensile creep. In the same laboratory study, an (AC-5) asphalt-rubber dense-graded mix was more resistant to permanent deformation at high temperatures and to thermal cracking at low temperatures than a dense-graded mix produced with either an unmodified AC-20 or AC-5 asphalt.

Asphalt-rubber may be used in HMA, in stress absorbing membranes (SAM), or in stress absorbing membrane interlayers (SAMI). Some of the earliest reported use of SAM and SAMI was in Arizona where they were used in both highway and airport applications (Vallerga et al. 1980). Recommendations on use of SAM and SAMI have been provided in Shuler, Pavlovich, and Epps (1985), Office of Economic Co-Operation and Development (OECD) (1984), and Estakhri, Button, and Fernendo (1992). Additionally, asphalt-rubber has been used as a crack and joint sealer (Estakhri, Button, and Fernendo 1992) and can be used in virtually any application requiring asphalt cement.

A number of laboratory investigations of asphalt rubber have been conducted (e.g., Khedaywi et al. 1993, Hansen and Anderton 1993, Stroupe-Gardiner, Newcomb, and Tanquist 1993, Hanson et al. 1994, Hui, Morrison, and Hesp 1994, Krutz and Stroupe-Gardiner 1992, Takallou and Hicks 1988, Takallou, Hicks, and Esch 1986, Jimenez and Meier 1985). These investigations have concentrated on binder properties as well as the behavior of HMA mixtures. The original properties of the asphalt cement and the type, size, texture, and properties of rubber used in each application will significantly affect the properties and behavior of the asphalt-rubber binder. Generally, asphalt rubber has lower penetration values, higher absolute and kinematic viscosities, higher softening point, lower temperature susceptibility, and lower water susceptibility. HMA mixtures using asphalt rubber normally have higher optimum binder content, lower Marshall stability, higher Marshall flow value, greater fatigue resistance, 25 percent lower resilient modulus, lower static creep resistance (especially at higher temperatures), and higher resistance to permanent deformation as indicated by dynamic creep testing.

Field performance of asphalt rubber has been evaluated in many applications and locations, and in most cases, the product has performed satisfactorily (e.g., Hanson et al. 1994, Fager 1994, Shuler and Estakhri 1993, Lundy, Hicks, and Zhou 1993, Aurilio, Lynch, and Northwood 1993, Morris 1993, Maupin 1992, Page, Ruth, and West 1992, Schnormeier 1986). Case studies from diverse geographic and environmental provinces have been monitored for various periods of time:

*a.* Mississippi, 2 years (Hanson et al. 1994).

*b.* Kansas, 2 years (Fager 1994).

*c.* Oregon, 10, 7, and 2 years (Lundy, Hicks, and Zhou 1993).

*d.* Florida, 1 year (Page, Ruth, and West 1992).

*e.* Virginia, 1 year (Maupin 1992).

*f.* Arizona, 15 years (Schnormeier 1986).

Some unexplained problems with longitudinal cracking have occurred in the Mississippi test sections, causing the Mississippi DOT to cease use of asphalt-rubber pavements until additional research is conducted (Robert Denson, Mississippi State Department of Transportation, private communication 1995).[1]

Although rubber-modified asphalt has received considerably less attention than asphalt rubber, it has been the subject of several research efforts and applications (Takallou and Sainton 1992, Takallou, Hicks, and Esch 1986). Successful projects with rubber-modified asphalt require a low percentage of voids in the total mix (Takallou and Hicks 1988). Two of the perceived advantages of rubber-modified asphalt are that reflective cracking is delayed (Heitzman 1992, Schnormeier 1986) and ice debonds more easily from this product because of the higher resiliency of the mix (Kandhal 1993a). Also, rubber-modified asphalt can use two to four times the quantity of rubber typically used in asphalt rubber (Kandhal 1993a). Disadvantages include the need for specialized mixing equipment, special mineral aggregate gradations, and special design criteria (Takallou and Sainton 1992). Takallou and Sainton (1992) reported that rubber-modified asphalt concrete has been recycled successfully in Canada.

From an environmental perspective, air quality monitoring during asphalt-concrete production in Canada (Aurilio, Lynch, and Northwood 1993) indicated that no increase in benzene emissions occurred with the addition of rubber in a rubber-modified pavement test section. Also the study reported that leachate testing showed waste asphalt binder and asphalt concrete modified

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[1] Private Communication, 1995, Robert Denson, Mississippi State Department of Transportation.

with rubber could be placed in Canadian landfills, while wastewater from air emission control equipment at the plant could not be discharged into stormwater sewers.

The cost of rubber-containing asphalt paving is greater than that of nonmodified asphalt paving. Aurilio, Lynch, and Northwood (1993) reported that rubber-modified HMA used in test sections cost 37 percent more than a convenional HMA when No. 4 mesh rubber was used, and it was 65 percent more expensive than a standard mix when No. 10 mesh rubber was used. A survey of states conducted by Amirkhanian (1993) indicated that 100 percent of states responding had found mixtures containing rubber cost about twice as much as conventional unmodified mixtures. Estakhri, Button and Fernendo (1992) reported that the normalized cost of typical in-place chip seals in Texas varied as follows according to the type of binder used: asphalt cement, 1.00; asphalt cement with latex, 1.02; emulsion, 1.02; emulsion with polymer, 1.19; and asphalt rubber, 2.43. Costs for asphalt overlays in Texas were about $1.00 more with a SAMI than without ($4.25/yd$^2$ vs $3.20/yd$^2$). In 1990, asphalt-rubber HMA cost (in-place) $52 per ton in Amarillo and $80 per ton in Tyler compared to an average of $30 to $35 per ton for conventional HMA in most Texas DOT districts (Estakhri, Button, and Fernendo 1992). Schnormeier (1986) reported that asphalt-rubber chip seals have virtually eliminated maintenance on a number of roadways in Arizona. Morris (1993) cited several asphalt-rubber applications and concluded that when life-cycle costs were considered (not simply initial costs) "asphalt-rubber can provide a first cost effectiveness in a majority of the cases. Asphalt-rubber will provide life cycle cost effectiveness virtually 100 percent of the time."

Reported benefits of using rubber in HMA include (OECD 1984, Amirkhanian 1993):

*a.* Thinner lifts required.

*b.* Increased pavement life.

*c.* Retarded reflective cracking.

*d.* Decreased traffic noise.

*e.* Reduced maintenance costs.

*f.* Decreased pollution and increased environmental quality.

Some of the more prevalent problems that tend to slow or prevent use of rubber in pavements are summarized in Table 8.

A number of technical and environmental questions must be addressed before asphalt rubber can be used routinely. An environmental aspect to be considered is the recyclability of rubber-containing asphalt concrete. To date, only a few in-service asphalt-rubber mixtures have been recycled (Kandhal

**Table 8**
**Barriers to Utilization of Rubber in Paving Applications (after Lundy, Hicks, and Zhou 1993)**

| Type of Barrier | Problem |
|---|---|
| Economic | Initial costs are double that of conventional materials |
| | Insufficient life-cycle cost data available |
| | Capital cost for equipment modification |
| Noneconomic | Insufficient and/or conflicting long-term field performance data |
| | Lack of uniformity in specifications for rubberized asphalt |
| | Patented processes limit competition |
| | Potential leaching from tire chips where roadbeds are exposed to saturated conditions |
| | Potential environmental concern with visible emissions emanating from asphalt plants in which crumb rubber is being used |

1993a, Aurilio, Lynch, and Northwood 1993, Takallou and Sainton 1992). Thus, one of the biggest concerns, the potential for air pollution upon mixing and heating of the recycled pavement, remains unanswered. Other concerns include the unknown fate of asphalt rubber during recycling and the lack of design procedures for recycled asphalt-rubber pavement mixtures. Also the long-term performance of recycled asphalt-rubber pavements has not been addressed. Further research and field experience on a national level is needed before long-term application of this technology is feasible (Kandhal 1993a, Heitzman 1992).

**Incinerator residue.** Approximately 26 million metric tons of municipal solid waste (MSW) are incinerated annually in the United States. There are approximately 190 thermal reduction facilities (MSW incinerators) located in 36 states and the District of Columbia (Styron, Gustin, and Viness 1993, Ciesielski and Collins 1993). The process of incineration reduces the quantity of waste to approximately 10 percent of its original volume and 25 to 30 percent of its original weight, thus leaving about 8 million metric tons of incinerator residue. This residue will typically be composed of 90 percent bottom ash and 10 percent fly ash. In most cases, the bottom ash and fly ash are combined prior to disposal.

The composition of incinerator residue is extremely heterogeneous. It contains large and small particles (the larger chunks of ferrous metal are removed using magnets) and consists of "agglomated fine ash and scrubber lime particles interspersed with and coating larger pieces of metals, glass, rock, slag, small quantities of uncombusted materials such as paper" (Collins 1979). The typical chemical composition of incinerator residue is given in Table 9. Similar compositions were reported by Collins (1979) in Federal Highway Administration studies.

For various disposal options, concerns exist about the potential for environmental pollution from incinerator ash or from leachate from the ash. Leachate

**Table 9**
**Typical Composition of Incinerator Residue - Combined Bottom Ash and Fly Ash (after Styron, Gustin, and Viness 1993)**

| Compound | Quantity in Ash, percent |
|---|---|
| Silica ($SiO_2$) | 40-50 |
| Alumina ($Al_2O_3$) | 5-15 |
| Ferric Oxide ($Fe_2O_3$) | 12-25 |
| Calcium Oxide (CaO) | 8-15 |
| Sodium Oxide ($Na_2$) | 3-6 |
| Magnesium Oxide (MgO) | 1-2 |
| Potassium Oxide ($K_2O$) | 0.75-1.5 |
| Titanium Dioxide ($TiO_2$) | 0.75-1.5 |
| Sulfur Trioxide ($SO_3$) | 0.50-1.5 |
| Phosphorus Pentoxide ($P_2O_5$) | 0.50-0.75 |
| Cupric Oxide (CuO) | 0.06-0.15 |
| Lead Oxide (PbO) | 0.04-0.22 |
| Zinc Oxide (ZnO) | 0.12-0.22 |
| Loss on Ignition @ 750°C | 1-3 |

studies of incinerator residue have indicated that leachate from fly ash samples alone generally exceed regulatory limits for cadmium and lead, but combined samples of bottom ash and fly ash do not typically exceed these limits (Schroeder 1994, Ciesielski and Collins 1993). The USEPA has indicated that only about 10 percent of all incinerator residue produced in the United States is used productively (Schroeder 1994), principally because of environmental concerns.

Various studies have been conducted during the past quarter century to evaluate the potential for use of incinerator residue as aggregate in pavement structures. The results have been varied. Incinerator residue has been used successfully in embankments, subbases, and bituminous base courses in pavement construction. Some of these installations have been in service with no reported problems for many years (Garrick and Chan 1993, Teague and Ledbetter 1979). Lauer (1979) reported that incinerator residue could be used as aggregate in asphalt and concrete pavements as well as for subbases and base courses. However, incinerator residue may be frost susceptible, may lead to environmental degradation, and has a tendency to swell. The latter characteristic can cause problems when the residue is used in portland-cement concrete (Permanent International Association of Road Congresses (PIARC) 1989, Lauer 1979). The potential for frost susceptibility should be evaluated when the incinerator ash is to be used in freezing areas. There have also been some stripping problems reported when traffic is placed directly on asphalt pavement layers containing incinerator residue (Pavlovich, Lentz, and Ormsby 1979), but some of this problem can be ameliorated by the addition of lime or lime slurry (Garrick and Chan 1993, Pavlovich, Lentz, and Ormsby 1979).

One study in Connecticut attempted to quantify the potential for harmful leachate production from an asphalt pavement which substituted incinerator residue for a portion of the traprock aggregate (Garrick and Chan 1993). Investigators subjected loose (uncompacted) samples of two asphalt mixtures (one with only traprock aggregate and one with 32 percent incinerator residue/ 68 percent traprock) to the USEPA extraction procedure toxicity (EP toxicity) test and the toxicity characteristic leaching procedure (TCLP) test. The leachate extracted from each sample was analyzed for various metals, according to the test procedures. Results of the leachate tests are shown in Table 10. As the center columns of the table show, for this particular incinerator residue, only lead was present in the leachate in appreciable levels, and it only exceeded the regulatory maximum contaminant level in the TCLP test (recognized to be the more severe of the two tests).

**Table 10**
**Leachate Test Results For HMAs Made With Traprock and Traprock/Incinerator Residue Aggregate (modified from Garrick and Chan 1993 and Styron, Gustin, and Viness 1993)**

| Element | Maximum Contaminant Level, ppm | Traprock Pavement Mixture[1] |  | Traprock/ Incinerator Residue Pavement Mixture[1] |  | TCLP Leachability of Raw Ash and TAP[2] |  |
|---|---|---|---|---|---|---|---|
|  |  | EP Toxicity ppm | TCLP ppm | Ep Toxicity ppm | TCLP ppm | Raw Ash mg/l | TAP mg/l |
| Arsenic | 5.00 | 0.0004 | 0.0009 | 0.0016 | 0.0024 | <0.002 | <0.05 |
| Barium | 100.00 | 0.0490 | 0.2470 | 0.1540 | 0.2040 | 2.93 | 0.55 |
| Cadmium | 1.00 | 0.0030 | 0.0030 | 0.0040 | 0.0190 | 1.07 | 0.0002 |
| Chromium | 5.00 | 0.0060 | 0.0110 | 0.0040 | 0.0260 | 0.024 | 0.063 |
| Lead | 5.00 | 0.0082 | 0.0258 | 2.3750 | 10.5000 | 5.6 | <0.0002 |
| Mercury | 0.2 | N/R[3] | N/R | N/R | N/R | 0.006 | 0.0003 |
| Selenium | 0.99 | 0.0027 | 0.0017 | 0.0023 | 0.0018 | <0.005 | <0.005 |
| Silver | 5.00 | 0.0000 | 0.0170 | 0.0000 | 0.0230 | <0.01 | <0.01 |
| Copper | 100.00 | N/R[3] | N/R[3] | N/R[3] | N/R[3] | 1.20 | 0.018 |
| Zinc | 500.00 | N/R[3] | N/R[3] | N/R[3] | N/R[3] | 120.25 | 0.01 |

[1] Data from Garrick and Chan 1993.
[2] Data from Styron, Gustin, and Viness 1993.
[3] N/R - data not reported for this parameter.

To use incinerator residue in asphalt pavements, the residue must generally be processed in some way (Kandhal 1993a, Garrick and Chan 1993, PIARC 1989, Collins 1979, Lauer 1979, Pavlovich, Lentz, and Ormsby 1979). When the bottom ash leaves the incinerator, it exits at a high temperature (around 815 °C (1,500 °F)), is graded to remove larger particles, and is quenched in a water bath. The fly ash may be kept in a separate waste stream or it may be added to the bottom ash in the quenching bath (Lauer 1979). The quenched ash must then be dried (sometimes simply by *aging* or stockpiling for about 2 months), sieved, and have larger metallic particles removed by trommeling.

The ash must then have its gradation checked against project specifications and adjusted, if necessary, by blending with natural aggregate (Pavlovich, Lentz, and Ormsby 1979). Several researchers have reported that the incinerator residue must be mixed with natural aggregate to produce a mixture containing no more than 40 to 55 percent incinerator residue (Garrick and Chan 1993, Kandhal 1993a). One project in Philadelphia was reported in which the incinerator residue was ground with a hammer mill, heated to 688 °C (1,270 °F) to remove combustible materials, and fused at a temperature of 1,093 °C (2,000 °F) into a column of solid material. The material processed using this patented process was then cooled and crushed to produce aggregate of the desired gradation (Garrick and Chan 1993, Kandhal 1993b).

Another patented process has been developed in which incinerator ash is combined with "binders and chemical fixation agents" to immobilize heavy metal contaminants (Styron, Gustin, and Viness 1993). The finished product, called Treated Ash Produce (TAP), meets the physical specifications of the Minnesota DOT for aggregates used in asphalt pavements, and the TAP leachate meets current drinking water standards when tested by USEPA TCLP leaching test (See Table 10).

Asphalt paving mixtures can be successfully designed using incinerator residue. However, the mixtures vary somewhat from more standard asphalt-aggregate mixes. Typically the asphalt content will be significantly higher. Garrick and Chan (1993) reported that 20 percent more asphalt was required in the HMA utilizing incinerator ash. This mix had an indirect tensile strength 20 percent lower than the control mix and stripping was more of a problem. Styron, Gustin, and Viness (1993) also reported high asphalt demands as a result of the absorptive nature of the aggregate. Stability, air voids, and voids in mineral aggregate may also pose mix design problems (Kandhal 1993a). The surface texture will usually be acceptable from a skid-resistance perspective.

Long-term field performance of several pavements constructed using incinerator residue has been documented. An intersection in Houston, TX, where incinerator ash was used as aggregate in the bituminous base course was monitored for 3 years and performed as well as the control section. In Washington, DC, a residential street with ash used in the base course was reported in good condition about 10 years after construction (Garrick and Chan 1993). Projects in Lynn, MA, and Harrisburg, PA, have also been reported to have performed satisfactorily for a number of years (Garrick and Chan 1993). Incinerator waste has found extensive use in Japan, Germany, France, The Netherlands, and Switzerland where it has been used as fill or aggregate in subgrades, subbases, and base courses.

Kandhal (1993a) made the following recommendations regarding use of incinerator residue in asphalt pavements:

*a.* Residues should be well burned out (loss on ignition of 10 percent or less).

*b.* HMA mixtures for base courses containing 50 percent natural aggregate and 50 percent incinerator residue hold the most promise.

*c.* Lime (2 percent) should be added to minimize stripping problems.

*d.* Because of the extremely variable nature of incinerator residue, it must be tested for compatibility of each source with each specific project.

Thus, the use of incinerator residue for aggregates in pavement applications seems to hold some feasibility. However, there remain a large number of unresolved technical and environmental issues to be solved before it becomes a viable technology.

**Glass.** Only about 20 percent of waste glass is currently being recycled; this is used mainly as cullet for glass manufacturing (Ciesielski and Collins 1993). Much of the 12 million tons of glass waste produced annually is discarded (Schroeder 1994). Several investigations have been conducted to assess the feasibility of substituting waste glass for a portion of the fine aggregate in asphalt pavements. Recent feasibility studies have been conducted in the states of Connecticut, Florida (West, Page, and Murphy 1993), and Virginia (Hughes 1990). Asphalt pavements which contain glass have often been referred to as "glasphalt."

Early studies were conducted in the 1970's and the results were summarized by Hughes (1990). Kandhal (1993a) summarized the recent Connecticut, Florida, and Virginia studies. The most significant findings were that waste glass could be used successfully in asphalt-concrete pavements; it has been used in at least 45 locations in the United States. It must be crushed to pass the 9.5-mm (3/8-in.) sieve (Kandhal 1993b, Hughes 1990), and lime should be added as an antistripping agent (West, Page, and Murphy 1993). Since glass cools more slowly than natural aggregate, the mix will likely retain heat longer after placement which may help compaction in cold weather but hinder it in hot weather (West, Page, and Murphy 1993, Hughes 1990). The structural performance was reported to be adequate and skid resistance was acceptable for low-speed applications, up to 48 km/h (30 mph) with light to moderate traffic (Malisch, Day, and Wixon 1973). However, use of studded tires will cause a degradation of glass aggregate and thus cause loss of skid resistance. The abrasiveness of glass-containing HMA concrete has been a concern (Malisch, Day and Wixon 1973) and was greater than HMA containing crushed limestone or natural gravel but less than those using traprock (Gupta 1972). Some limited problems with stripping have been reported and based on this potential problem, Hughes (1990) recommended that a maximum of 15 percent crushed glass be allowed in HMA. The cost of collecting and preparing the waste glass will often be greater than that of conventional aggregates (Hughes 1990, West, Page, and Murphy 1993).

The following conclusions of the Virginia DOT laboratory investigation relate to performance of the HMA containing crushed glass (Hughes 1990). The voids in mineral aggregate (VMA) and air voids, and therefore the optimum asphalt content, are reduced when glass was used in the mix. When a maximum of 15 percent crushed glass was used, neither the indirect tensile strength nor the resilient modulus values were adversely affected. The wet strength and retained tensile strength ratio (TSR) were unaffected by the amount of glass, but some separation was observed at the asphalt/glass interface.

Indications are that crushed glass can be substituted for up to 15 percent of the aggregate in HMA concrete. However, a significant amount of engineering effort will be required on a project-by-project basis to ensure a successful application. Several problem areas such as potential stripping, loss of skid resistance, glass particle breakage under studded tires, consistency of glass source (both quality and quantity), and increased asphalt-concrete production costs will likely be encountered and must be evaluated before considering use of glass in asphalt concrete (Kandhal 1993a, West, Page, and Murphy 1993, Malisch, Day, and Wixon 1973). The problems of skid resistance and surface raveling can be eliminated by using glass-containing HMA mixtures in the base course. It is strongly recommended that a test section be constructed prior to full-scale construction using glass aggregate (Kandhal 1993a, Hughes 1990).

**Plastics.** Plastics compose a significant portion of the MSW stream. Estimates indicate that plastics compose approximately 8 percent of the total weight of MSW and about 12 to 20 percent of the volume (Schroeder 1994). In 1992, only 2.5 percent of the 16.2 million tons of waste plastics generated in the United States were recycled (Little 1993). These statistics indicate that a large quantity of material is potentially available for recycling.

The most promising use for recycled plastics in pavement construction is as an additive to asphalt cement. The beneficial effects of polymers on asphalt cement are well known and documented. Polymer-modified asphalt exhibits increased resistance to low-temperature cracking and reduced deformation at elevated temperatures. To date, most uses of polymers in asphalt paving have incorporated virgin, not recycled, materials. However, there are two patented products that make use of recycled plastic as an additive to asphalt cement; these are Novophalt® and Polyphalt®. Details regarding types of virgin polymers, techniques of use, and effects on material properties and engineering performance when used in asphalt paving mixes are discussed in the "Manufactured Modifiers" section of this report. This section only addresses feasibility of using recycled polymers in asphalt paving materials.

One polymer used almost exclusively as an asphalt modifier is polyethylene. Most of the applications to date have used low-density polyethylene (LDPE), and recycled LDPE has proven to work equally as well as virgin LDPE. Little (1993) reported that research conducted in Austria in 1989 evaluated the effect of minor additions of other plastic materials to polyethylene-modified asphalt cement. In the Austrian study, various percentages of polypropylene (PP), polyethylene terephthalate (PET), polystyrene (PS), and polyvinyl chloride (PVC) were used. Results indicated that recycled LDPE "can be used to produce a premium binder" (Little 1993), while the use of high-density polyethylene (HDPE) caused "an excessive increase in ring and ball softening point and an unacceptable decrease in penetration" (Little 1993). Little (1993) speculated that aging of HDPE-modified asphalt binder would lead to embrittlement and premature cracking in pavements. The addition of other polymers to LDPE-modified asphalt did not adversely affect blending of the material nor the properties of the modified HMA. Figure 1 summarizes the changes in asphalt binder properties resulting from addition of various polymers. Based upon the findings of the Austrian research, it may be feasible to produce polymer-modified asphalt using a variety of recycled plastics from both pre- and postconsumer recycling efforts. However, removal of all paper and metal would be required to prevent rapid deterioration of the pavement, milling difficulties, and accelerated pavement aging (Little 1993).

Some problems with separation of asphalt and polymer have been reported during heated storage and/or transport of LDPE-modified asphalt cements. Researchers in Canada (Liang et al. 1993) reported on a proprietary product that combines asphalt, polyethylene, and a steric stabilizer to produce a permanent emulsion of polyethylene in asphalt. Laboratory testing indicated that the product will have strength, toughness, and resistance to deformation (Liang et al. 1993). Field trials of this product are being planned.

Another specialized product, an ethylene based copolymer (EBC) derived from reprocessed LDPE and developed in France, has been recently tested in the United States. When used in HMA, the product, marketed under the name Starflex, resulted in significant improvements in rutting resistance, fatigue cracking, and age hardening (Bayomy and Carraux 1993). The study found that the best results were obtained when the ethylene based copolymer was added to the aggregate before addition of the asphalt. The optimum treatment level for the EBC was about 1 percent of the aggregate dry weight.

The OECD (1984) reported that polyethylene can be used to completely replace asphalt in roadway construction. Conventional equipment can be used for mixing and placing these paving materials. However, there are at least two main drawbacks to using polyethylene as a pavement binder. Firstly, the cost was about three times that of asphalt concrete when virgin polyethylene was used and approximately two times that of conventional materials when recycled polyethylene was used. Secondly, polyethylene shrinks as it cools from as-placed temperatures to in situ temperatures. The shrinking leads to thermal contraction cracking, which is the most significant technical problem with thermoplastic resin concrete.

Recycled plastic has also been investigated for use as a fine aggregate substitute in portland-cement concrete. The study at Michigan State University is using recycled HDPE to replace 20 to 40 percent (by volume) of the fine aggregate in lightweight concrete mixes. Initial results indicated that compressive strengths were reduced, flexural strengths were unaffected, and impact resistance (flexural toughness) increased when HDPE was used in the concrete specimens (Schroeder 1994).

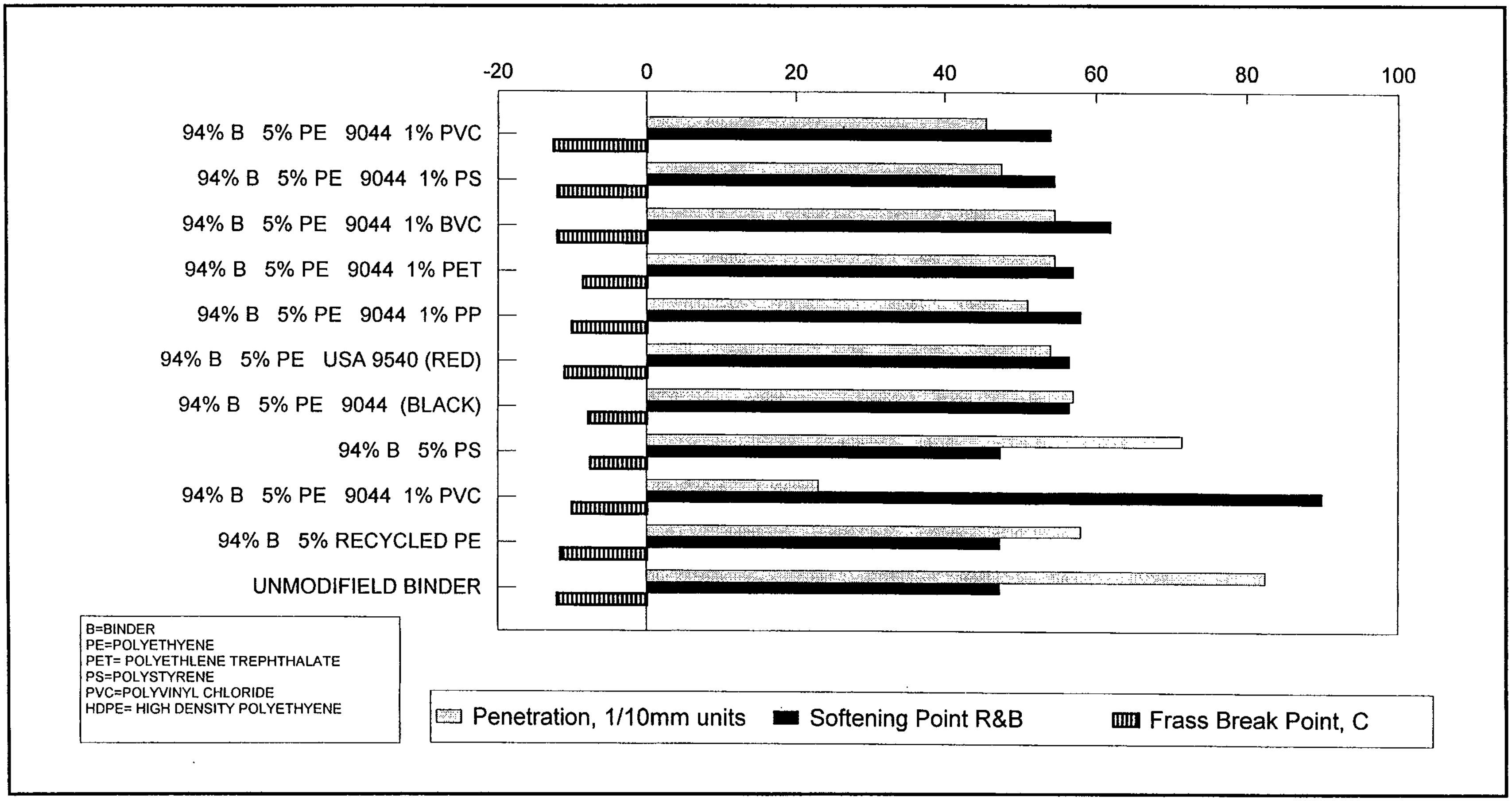

Figure 1. Changes in asphalt properties with addition of various types of recycled polyethylene (modified from Little 1993)

Other potential highway uses of recycled plastics include use in guardrails and supporting posts, noise barriers, snow poles, and sign, delineator, and fence posts. A number of industries and Government agencies are conducting research and engineering on such recycled plastic items to obtain the necessary engineering characteristics and behavior for various uses. Currently, the initial costs of these plastic items are generally higher than those of conventional materials.

**Roofing shingles.** Each year approximately 10 million tons of roofing shingle wastes are generated in the United States. This material includes scrap and waste from manufacturing plants and reroofing contractors (Schroeder 1994, Collins and Ciesielski 1994, Grzybowski 1993). The waste is generally composed of 30 to 40 percent asphalt, 50 to 60 percent inorganic mineral fillers (including 22 percent hard rock granules passing the No. 10 sieve), and 1 to 12 percent inorganic and organic fibers (Grzybowski 1993, Kandhal 1993a). Newcomb et al. (1993) reported a similar composition of asphalt roofing shingles: 35 percent asphalt, 45 percent sand, and 20 percent mineral filler. Some of the roofing waste from reroofing may contain materials such as roofing tacks and other "contaminants" (Collins and Ciesielski 1994) which may require removal prior to use.

The components of roofing shingles are generally similar to and compatible with HMA constituents. The asphalt cement used in shingles (and in other types of roofing) is a blown asphalt that is much harder than that normally used in asphalt-concrete pavements. Blown asphalt is produced by blowing air through a tank containing the hot asphalt; the combination of heat and oxygen causes oxidation of the asphalt, resulting in a harder final product. Blown asphalt typically has a penetration value of 15 rather than a value of 70 or greater as would normally be encountered in paving asphalts. The inorganic mineral fillers in shingles are often composed of limestone (Grzybowski 1993) which is often used in pavements. The reinforcing mat upon which the asphalt and mineral fillers are placed to form shingles are usually composed of fiberglass or cellulose fibers. Such fibers have been used in a number of pavements applications. Table 11 provides a comparison of the components of recycled asphalt roofing materials and commercially available products commonly used as constituents of HMA.

**Table 11**
**Comparable Constituents of Recycled Asphalt Roofing Waste and Asphalt Pavements (after Grzybowski 1993)**

| Material | Recycled Asphalt Roofing Component | Equivalent Asphalt Pavement Component |
|---|---|---|
| Asphalt | Air-blown asphalt | Trinidad Lake asphalt, Gilsonites, propane precipitated asphalts |
| Fillers | Limestone and other mineral fillers | Carbon black, limestone fines, hydrated lime, diatomaceous earths |
| Fibers | Fiberglass, cellulose | Minerals, polyester, polypropylene, cellulose (in stone mastic asphalt) |

Asphalt roofing wastes should be usable in asphalt paving mixtures, although the mixtures must be designed to accommodate their inclusion. Account must be made for asphalt content of the roofing waste, hardness of the roofing waste asphalt which will tend to stiffen the mix, and presence of roofing waste fillers and fibers which also tend to stiffen the mix. The latter two items will generally increase the optimum asphalt requirement of the mixture. One of the major problems with adding waste products to paving materials is maintaining consistency in the final product because the incoming waste product usually varies considerably. Consequently, to maintain desirable engineering characteristics of the mix, to avoid construction problems, and to minimize adverse effects of fluctuations in material properties of the roofing waste, only relatively small quantities of shingles and other roofing waste can be added to conventional HMA.

There is another potential approach to use of asphalt roofing waste in asphaltic concrete that shows more potential promise. Recent work at the WES (Ahlrich 1995b) with poorly graded aggregates has shown that highly stable asphalt-concrete mixes can be designed using gap graded aggregates. Additional unpublished work conducted by WES concerning stone mastic asphalt (SMA) mixes revealed that they are highly stable and relatively insensitive to the specific properties of the binder. Consequently, it would appear highly feasible to use a stone mastic-type mix design to incorporate shingles and roofing waste into asphalt pavement. Under the stone mastic design concept, a gap graded aggregate is used so that load is transmitted by point-to-point contact of the larger coarse aggregate particles. The void space between load bearing skeleton of coarse aggregates is filled with a mastic of sand, fillers, and asphalt to provide waterproofing and durability. Because of the high asphalt content used in the mastic, a filler such as cellulose fiber is usually added to prevent the asphalt cement from draining from the mix. Waste asphalt shingles could be used beneficially as part of the mastic phase of the mix -- the felt would provide filler in the same manner as the cellulose fiber, the stiff blown asphalt would simply help stiffen the mastic to avoid drainage and temperature susceptibility problems, and the mineral filler would become part of the mastic. The SMA mix is not sensitive to the specific mastic characteristics so variations in the characteristics of the shingle and roofing waste could be tolerated.

To use roofing waste in asphalt pavements, the waste shingles must be ground or shredded to a particle size of 12.5 mm (0.5 in.) or smaller prior to addition to the paving mix (Kandhal 1993a, Newcomb et al. 1993, Gryzbowski 1993). This will ensure meltdown and dispersion of the shingles throughout the mix. In the Minnesota laboratory study, the shingles were ground by hammer mills and then water cooled to prevent agglomeration of the material (Newcomb et al. 1993); this procedure caused a high water content in the roofing waste that had to be reduced prior to mixing with the HMAs. The preferred sequence for mixing components of HMA containing roofing waste is undecided. Grzybowski (1993) reported that the recycled roofing waste was added to the aggregate prior to addition of the asphalt in the dense graded HMA mix. Newcomb et al. (1993) report that the roofing waste was added to the mixture after the aggregates were coated with asphalt.

Recent studies by Newcomb et al. (1993) and Grzybowski (1993) indicate that roofing waste can be incorporated into both dense-graded and stone mastic asphalt mixes. The use of roofing waste was reported to enhance the performance of some dense-graded asphalt paving mixtures by increasing stabilities, reducing the tendency to rut, and improving high-temperature properties (Grzybowski 1993). Benefits for SMA included: increased binder content, reduced neat asphalt-cement content, and eliminated need for other modifiers. Both HMA and SMA with roofing wastes could be placed with conventional equipment (Grzybowski 1993).

**Miscellaneous.** Various other waste materials have periodically been considered for inclusion in pavements, their consideration many times driven by the desire to dispose of a waste material. Some of these materials proved to have detrimental effects on the performance of pavements under environmental and/or traffic loading. For instance, inclusion of paper in pavements would adversely affect the strength and durability of the paving materials; in fact, even very small amounts of paper on the surface of recycled plastics would lead to stripping and rapid deterioration in asphalt pavements (Little 1993). Other wastes that are available in large quantities, such as aluminum and ferrous containers, have too much intrinsic value to be used in pavements, and they would contribute nothing to the engineering characteristics or long-term performance of pavement structures.

Sewage sludge has not been used successfully in pavements. It has been used as a land treatment (fertilizer) for fields, rights-of-way, etc., in limited applications. Sewage sludge has been made into light-weight aggregate in California, Europe, and Japan.

Other potential waste additives have fallen out of favor for environmental reasons, particularly since pavement recycling has come into favor. The engineering community must not only consider the beneficial and detrimental effects of using different wastes in pavement, but also the potential effect on the recyclability of the pavement. For example, at one time asbestos was evaluated for use in HMA and was used at a number of locations. It provided fiber reinforcement to enhance strength but its environmental dangers were not recognized at that time. Given the current understanding of the hazard asbestos poses to human health, coupled with the doubled human exposure (initial construction and reconstruction) posed by recycling, asbestos will likely never be incorporated into another pavement. Indeed, where pavements with asbestos exist, they pose a particularly difficult rehabilitation problem. At present, the only consideration given to asbestos as it relates to paving is when naturally occurring asbestos is encountered. Certain soil and rock deposits contain naturally occurring asbestos (e.g., serpentinite in California and amphibole mineral deposits which contain actinolite asbestos and tremolite asbestos fibers) which can become airborne, particularly during blasting, grading, earthmoving and other construction activities (Dusek and Yetman 1993, Rude 1993, Huf and Stuart 1993).

# Industrial Waste Products

## Recycled pavements

Existing pavements with their aggregates and portland cement or asphalt binders represent a large potential source of recycled construction material. A typical asphalt concrete consists of approximately 5 percent asphalt binder and 95 percent aggregate by weight while portland-cement concrete pavements would typically consist of about 25 to 40 percent portland-cement paste and 60 to 75 percent aggregate by weight. These pavements can be recycled to produce new aggregates for use as fill, subbase, base, or as a constituent of new asphalt and portland-cement concrete.

Table 12 shows that there are already over 3.8 million miles of existing roadway in the United States. Recycling these pavements at the end of their life offers a powerful technique of salvaging some of our already hefty investment in our pavement infrastructure.

**Table 12**
**Road and Street Mileage in the United States Classified by Type of Surface (after Epps 1980)**

| Type of Surface | | Mileage | Percent of Total Mileage |
|---|---|---|---|
| Nonhard Surfaced | Unimproved | 283,976 | 7.3 |
| | Graded and drained | 397,986 | 10.2 |
| | Soil and rock | 1,192,052 | 30.7 |
| | Total | 1,874,052 | 48.2 |
| Hard Surfaced | Bituminous - low strength | 1,078,382 | 27.8 |
| | Bituminous - high strength | 811,553 | 20.9 |
| | Portland-cement concrete | 120,812 | 3.1 |
| | Total | 2,010,747 | 51.8 |
| Total mileage | | 3,884,761 | 100.0 |

The DOD was and continues to be a national leader in developing and using recycling technology for pavements. This leadership developed out of necessity because many military pavements are in remote areas (range roads, isolated facilities, overseas facilities in primitive environments, etc.) where conventional asphalt-concrete plants were not available and the paving jobs were too small to justify the expense of shipping in portable plants. Recycling proved to be a cost-effective technique for rehabilitating existing pavements under such conditions. Recycling is recognized as a standard pavement rehabilitation technique in much of DOD (e.g., HQDOA 1988) and is becoming more common in the civilian sector; however, this does not imply that the paving industry is exploiting the pavement recycling potential to the fullest degree possible. Challenging technical problems remain (e.g., safely recycling portland-cement concrete pavements that are undergoing alkali-silica reaction deterioration or recycling asphalt pavements that contain recycled rubber without adverse environmental emissions).

## Reclaimed asphalt-concrete pavements

Recycling technology is generally classified in one of three categories based on techniques used to recycle the pavement. The three general categories are:

a. *Surface techniques.* These techniques generally include surface heating or cold milling to correct pavement surface problems. The heater-planer heats the asphalt surface and then reshapes the pavement surface to the desired grade. The surface may also be scarified with the addition of new asphalt cement, recycling agents, or rejuvenators to improve the properties of the original aged asphalt binder. This technique is often used in conjunction with a follow-up overlay. The quality of the heater-planer process is highly dependent on the quality of the original asphalt concrete and quality of the construction effort. Production of smoke while heating the pavement is one drawback to this approach. A variety of commercial equipment is available to accomplish this type of work. Cold milling uses cutting teeth mounted in a helical spiral on a rotating drum to remove up to 101.6 mm (4 in.) of HMA concrete in a single pass. This equipment has undergone major improvements in the 1970's and 1980's with concurrent reductions in cost of operation. Today, the milling machine is the primary method of removing old pavement to reestablish grade or remove unsatisfactory material. The millings from this operation can then be used in either cold or hot mix recycling applications as discussed later. When milling is used alone as a surface preparation, the resulting surface has high skid resistance but tends to be objectionably rough to motorists and tends to have some raveling problems. Consequently, cold milling is most often used as a preparatory process before overlaying.

b. *Cold mix recycling.* In this process, the existing HMA pavement is removed by milling or by ripping and crushing. This reclaimed asphalt pavement (often called RAP) is mixed with a binder without heating, placed, and compacted. The binder is typically emulsified asphalt, but additives such as recycling agents or rejuvenators to soften the old oxidized asphalt, lime to combat stripping, or portland cement to stiffen the mix may also be used. The RAP may be mixed with the binder and additives, if any, either at a central plant or in situ with field mixers. The central plant provides the best quality control while field mixing offers the best economy.

c. *Hot mix recycling.* Hot mix recycling mixes RAP with virgin aggregates, asphalt cement, and possibly recycling agents or rejuvenators, if needed. All materials are mixed at elevated temperatures so the final product may be used essentially the same as any HMA concrete. The

> introduction of the drum asphalt plant has been particularly helpful in making production of hot mix recycled mixes economical. Up to 70 percent of the final asphalt mix can be RAP when produced in a drum plant while batch plants are limited to about 50 percent reclaimed asphalt pavement in the mix. Recent work with microwave heating of the reclaimed asphalt pavement has shown that up to 100 percent of the mix can be reclaimed asphalt pavement. Best current estimates are that somewhere between 20 and 50 percent of all milled asphalt pavements are recycled into hot mix asphalt concrete (Collins and Ciesielski 1994).

Table 13 compares the characteristics of different recycling technologies for asphalt pavements. These offer economical alternatives for pavement rehabilitation in many cases; however, they are not a panacea for every problem. Potential areas requiring further research include the use of polymer additives to cold mix recycled mixes to upgrade their structural characteristics, use of microwave heating to improve quality of recycled asphalt binder, and the recyclability of asphalt pavements containing rubber or other additives. Table 14 provides additional information on current references on specific recycling technology areas.

### Reclaimed concrete pavements

Recycling concrete consists of first breaking the pavement into pieces that are small enough to handle. This may be done using headache balls, guillotine blades, modified pile drivers, or similar such equipment. If reinforcing steel is present, it must be removed, and several techniques are available to accomplish this. The concrete is then crushed and screened to produce a new aggregate. This may be used as subbase or base course material, or it may be used as aggregate in new mixes of asphalt or portland-cement concrete. The fine (sand-sized) fraction of the recycled concrete tends to be highly angular and may lead to harsh, unworkable mixes when it is used in new portland-cement concrete. This can be controlled by using natural sands to supplement the recycled concrete fine aggregate.

Concrete pavements should be viewed as a resource and recycling of that resource as highly viable. Presently, approximately 321 km (200 miles) of concrete pavement are being recycled annually (Collins and Ciesielski 1994). Descriptions of specific concrete recycling projects can be found in Chase and Lane (1986), and McCarthy 1985. Many DOD applications would involve the use of relatively thick airfield pavements; technology for breaking and processing these thick pavements is less developed than for thinner highway pavements. Therefore, the projects must be large enough to mobilize the needed equipment or sufficiently near to crushing facilities to make the recycling process economically viable. The DOD has made relatively limited use of recycled concrete, but authority and guidance to use these materials exists in current manuals and guide specifications. Probably improved education of on-site engineers and effective demonstration projects will be needed to expand its use in DOD.

## Table 13
## Comparison of Asphalt Pavement Recycling Techniques (after Epps 1980)

| Recycling Technique | Advantages | Disadvantages |
|---|---|---|
| Surface | • Reduces frequency of reflective cracking<br>• Promotes bond between old pavement and thin overlay<br>• Provides a transition between new overlay and existing gutter, bridge, pavement, etc. that is resistant to raveling (eliminates feathering)<br>• Reduces localized roughness due to compaction<br>• Treats a variety of types of pavement distress (raveling, flushing, corrugations, rutting, oxidized pavement, faulting) at a reasonable initial cost<br>• Improve skid resistance | • Limited structural improvement<br>• Heater-scarification and heater-planning have limited effectiveness on rough pavement without multiple passes of equipment<br>• Limited repair of severely flushed or unstable pavements<br>• Some air quality problems<br>• Vegetation near roadway may be damaged<br>• Mixtures with maximum size aggregates greater than 25.4 mm cannot be treated with some equipment<br>• Limited disruption to traffic |
| Cold Recycling (In-Place) | • Significant structural improvements<br>• Treats all types and degrees of pavement distress<br>• Reflection cracking may be eliminated<br>• Frost susceptibility may be improved<br>• Improve ride quality<br>• Improve skid resistance<br>• Minimizes hauling | • Quality control not as good as central plant<br>• Traffic disruption<br>• Pulverization equipment in need of frequent repair<br>• PCC pavements cannot be recycled in place<br>• Curing is often required for strength gain |
| Hot Recycling (Central plant) | • Significant structural improvements<br>• Treats all types and degrees of pavement distress<br>• Reflection cracking can be eliminated<br>• Improve skid resistance<br>• Frost susceptibility may be improved<br>• Geometrics can be more easily altered<br>• Improved quality control if additional binder and/or aggregates must be used<br>• Improve ride quality | • Potential air quality problems at plant site<br>• Traffic disruption |

## Table 14
## Sources of Additional Information on Asphalt Pavement Recycling

| Topics | References |
|---|---|
| Surface Methods | HQDOA (1988), Emery and Terao (1992), Jimenez (1980), Roberts et al. (1991) |
| Cold Mix | HQDOA (1988), Emery (1993), Epps (1980), Kazmierowski et al. (1992), O'Leary and Williams (1992) Roberts et al. (1991), Rogge et al. (1992) |
| Hot Mix | HQDOA (1988), Emery (1993), Farrar et al. (1993), Hossain, Metcalf, and Scofield (1993), Roberts et al. (1991), Smith (1980) |
| Applications | Collins and Ciesielski (1994), PIARC (1989), Vicelja (1980) |
| Additives | DeKold and Amirkhanian (1992), Peterson et al. (1994) |
| Microwave | Shoenberger (1996) |

### Blast-furnace slag from iron production

This nonmetallic by-product of iron production consists primarily of silica, alumina, and oxides of silica and magnesium. Slow air cooling or more rapid quenching (typically with water) produces distinctly different final products. The more vesicular granulated slag ("popcorn slag") from rapid quenching is used as an aggregate for lightweight concrete block and similar applications, or it may be ground to produce a slag cement. The more crystalline air-cooled slags, which represent approximately 90 percent of all blast-furnace slags (Schroeder 1994, Collins and Ciesielski 1994), are widely used as aggregates for construction. About 13 to 15.5 million tons of blast-furnace slag are sold annually for construction in the U.S. (American Concrete Institute 1987a, Schroeder 1994, Collins and Ciesielski 1994), and its use is well accepted in both the DoD and civilian construction industry. Approximately 22 states have used blast-furnace slag as aggregate in both asphalt and concrete construction and as base and subbase materials. Similarly, over 7.3 million metric tons (8 million tons) are used for such purposes in Japan and over 10.8 millon metric tons (12 million tons) are used in the United Kingdom and France (PIARC 1989).

Suitable ground granulated blast-furnace slag (slag cement) is not as widely available as blast-furnace slag used as aggregate, but future supply is expected to increase as production facility and energy costs for conventional portland cement increase (American Concrete Institute 1987a). In past applications, ground granulated blast-furnace slag has generally been used as a cement substitute replacing 25 to 70 percent of the cement during the production of blended cements. It offers potential advantages in improved durability and reduced cost over conventional portland cement, but it generally has a reduced strength at early ages. The composition of portland cement, ground blast-furnace slag, and fly ash (discussed later in this section) are compared in Table 15 where it may be observed that ground blast-furnace slag composition is, more or less, intermediate between portland cement and fly ash. Ground granulated blast-furnace slags can be classified in one of three grades as stipulated in ASTM C 989 (ASTM 1994c), and requirements for blended cements

**Table 15**
**Typical Chemical Composition of Portland Cements, Blast-Furnace Slag, and Fly Ash (after Rollings and Rollings 1995)**

| | Chemical Composition | | |
|---|---|---|---|
| **Material** | **CaO** | **$SiO_2$** | **$Al_2O_3$** |
| Portland Cement[1] | 60-67 | 17-25 | 3-8 |
| Blast-Furnace Slags[2] | 29-42 | 32-40 | 7-17 |
| Fly Ash, Class C[3] | 12-29 | 44-64 | 20-30 |
| Fly Ash, Class F[3] | 1-7 | 45-64 | 20-30 |

[1] Neville 1981.
[2] American Concrete Institute 1987a.
[3] American Concrete Institute 1987b.

composed of portland cement and ground granulated blast furnace slag are specified in ASTM C 595 (ASTM 1994a). General guidance on the use of ground granulated blast-furnace slag in concrete mixtures is provided by the American Concrete Institute (1987a), but to date, there has been relatively little use of this product in DOD paving.

Ground granulated blast-furnace slag also has considerable potential as a soil or aggregate stabilizer. When combined with an activator such as lime or portland cement, it makes an effective cementitious stabilizing agent with greater working time than conventional portland cement stabilization. Stabilization with ground granulated blast-furnace slag is not common in the United States, but it has been proven useful overseas. Typical proportions have been 8 to 20 percent ground slag and 1 percent lime mixed with a soil or aggregate (Ray 1986). Improved technical documentation of properties, mixture proportioning methods, and field performance coupled with education are needed to widen use of this material in DoD stabilization work. As an additional benefit, stabilization has often been used to upgrade marginal or unsuitable materials for pavement use, so this application has the potential to use both an industry waste product and otherwise unsuitable materials for a beneficial application.

The WES has conducted preliminary research examining the potential for using high-alkali activators with ground granulated slag (and other glassy materials) as a new cementing medium. Results were generally promising, but more basic developmental work is needed in this area (Schilling 1992).

**Steel slag.** Slag from the production of steel contains magnesium oxides that hydrate slowly over time and expand as they hydrate. When these steel slag materials are used in construction as fill material, base material, or as concrete aggregate, they generally swell with time and have resulted in very dramatic failures (e.g., Crawford 1969, ENR 1983a, 1983b and 1986, Gnaedinger 1987, Waddell and Dobrowolski 1993). Because of this destructive expansive nature, steel slags currently have poor potential as a general construction aggregate or fill. (Note that these are not the same as *Blast-Furnace Slags from Iron Production* discussed earlier). The one exception appears to be their successful use as aggregate in HMA concrete (e.g., Schroeder 1994, Roberts et al. 1991, PIARC 1989). The bituminous coating of the slag in this application apparently waterproofs and protects the expansive oxides from hydrating. Specifications often limit the content of expansive material in the steel slag or require a period of outdoor aging. Schroeder (1994) reported that "bituminous mixtures containing steel slag exhibited high stability, high skid resistance, and longer heat retention resulting in easier compaction," although the high asphalt absorption of steel slag makes it uneconomical for use as an aggregate in asphalt concrete at this time.

**Coal by-products.**

*a. Fly ash.* Fly ash is a pozzolanic material[1] recovered from the burning of ground or powdered coal. Approximately 50 million tons are produced annually in the United States as a waste product from coal-fired electric power generation plants. Fly ashes are specified in ASTM C 618 (ASTM 1994b) as either class C -- normally the by-product of burning lignite or subbituminous coal, or class F -- normally the by-product of burning anthracite or bituminous coal. Class C fly ashes are often cementitious as well as pozzolanic while class F fly ashes are seldom cementitious when mixed with water alone. Typical compositions of these fly ashes were compared to that of portland cement and blast-furnace slag earlier in Table 15.

Fly ash is widely used in the United States concrete industry to improve workability of fresh concrete, to reduce temperature rise and shrinkage in concrete, to enhance durability, and to serve as a partial substitution for portland cement for economic savings. Guidance and experience on its use for such applications in the concrete industry are well documented (e.g., American Concrete Institute 1987a). It is also used as a fine mineral filler for asphalt concrete.

A promising application for fly ash that has received relatively little attention in the United States is its potential as a soil or aggregate stabilizer for pavement construction. Depending on the specific characteristics of the material to be stabilized, 8 to 30 percent fly ash and 2 to 8 percent lime (to serve as the activator for the pozzolanic reaction) are mixed with the soil or aggregate. When such mixtures are adequately cured, they will often achieve compressive strengths in the range of 3.4 MPa (500 psi) to 6.9 MPa (1,000 psi) with ultimate long-term strength gain above 20.7 MPa (3,000 psi) being possible (Terrel et al. 1979). This typically exceeds the strength achieved with lime or bituminous stabilization and equals that commonly seen with portland cement stabilization. Strength gain is slower than with portland cement, but in exchange the lime-fly ash stabilization has less shrinkage and cracking and allows greater time for field operations such as placement and compaction. Initial strength gain of the lime-fly ash mixture can be improved with additions of 0.5 to 1.5 percent cement.

Some guidance for use of fly ash as a soil or aggregate stabilizer is available (e.g., American Coal Ash Association 1991, HQDOA 1983), but the potential has not been widely recognized by DOD or private industry engineers. Consequently, lime-fly ash stabilization is seldom considered by the military or private industry pavement engineering community. Improved technical documentation of properties, mixture

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[1] Pozzolanic materials are siliceous or aluminous materials that are not cementitious when mixed with water alone, but when in finely powdered form, they will chemically react with calcium hydroxide (e.g., such as that found in portland cement or lime) to form cementitious compounds.

proportioning methods, and field performance coupled with education are needed to increase the usage of this material in stabilization work. As an additional benefit, stabilization has often been used to upgrade marginal or unsuitable materials for pavement use, so this application has the potential to use both an industry waste product and otherwise unsuitable materials for a beneficial application.

b. *Bottom ash.* Bottom ash has approximately the same composition as fly ash, but it is highly variable. Bottom ash is also much coarser than fly ash and is usually highly corrosive. It has been used as an additive in asphalt concrete where it increases resistance to stripping from moisture but this has been at the expense of decreased stability (Kandhal 1993b, Vassiladou et al. 1993). Wet bottom ash (also called boiler slag) has also been used to increase skid resistance of asphalt concrete in Texas. There is also active work examining potential application for bottom ashes in embankments or unbound base courses or as an aggregate in stabilized base courses (Collins and Ciesielski 1994).

c. *Gasifier slag.* Gasifier slag is produced from the modern, efficient burning of coal in coal gasification plants. These plants utilize a high-pressure and high-temperature burn to produce a high purity, medium-BTU gas for use by electrical power generation stations, while reducing sulfur dioxide emissions. The bottom ash from coal gasification plants is termed gasifier slag. This material is similar to bottom ash or boiler slag obtained from other coal burning processes. Gasifier slag is fairly fine and has a gradation similar to limestone screenings and field sands commonly used in asphalt pavements (Khatri et al. 1993). Results of an investigation by Khatri et al. (1993) indicated that gasifier slag could be economically substituted for limestone screenings or field sand in HMA and would produce a pavement with essentially equal performance. They expected no environmental problems or hazards from use of gasifier slag in HMA pavements.

d. *Coal tar.* Coal tar has been used in pavements as a substitute for asphalt for a number of years. However, it is much more temperature susceptible than conventional asphalts and is more brittle and subject to cracking (OECD 1984). Coal tar is not petroleum based, and therefore, it is not susceptible to damage from fuel spillage like conventional HMA. This has made it a leading candidate for use as a fuel resistant sealer for HMA concrete pavements or as a fuel resistant binder in a bituminous concrete. Coal tar's tendency to crack and other inferior characteristics to conventional asphalts has limited its usefulness. Additionally, the fumes produced during the production and laydown of coal tar pavements has raised considerable environmental concerns and has limited its use. Research has been conducted using polymer and rubber modifiers to improve its characteristics, but problems remain. Coal tars modified with PVC have been used successfully as a surface dressing in many countries; it has also been modified with 20 percent waste PVC and used in tar concrete in areas with severe traffic conditions (OECD 1984).

**Petroleum contaminated soil.** Petroleum contaminated soils occur wherever petroleum products come in direct contact with geotechnical materials such as clay, silt, sand, and gravel. Petroleum may escape from underground storage tanks (UST), be spilled in refueling areas, be illegally dumped, released in accidental spills, or leak from aboveground storage facilities. The petroleum products involved may range throughout the spectrum from light to heavy petroleum products, including gasoline, jet fuel, lubricating oil, and asphalt (National Cooperative Highway Research Program (NCHRP) 1993). Many motor pool and storage areas are contaminated with a variety of these products. Regulatory efforts are under way to remove sources of petroleum contamination and to provide safe, effective cleanup of contaminated soils.

With appropriate design, petroleum contaminated soil can be incorporated into asphalt paving layers without causing degradation in the quality of the pavement layer and without causing any short- or long-term environmental problems (Meegoda et al. 1993). Generally, the finer-grained contaminated materials such as clays and silts are used in stabilized subbase or base courses, while coarser gradations may be used in higher quality layers. For instance, the WES has participated in use of petroleum contaminated soils in pavements at several military bases. At one base in Alaska, old drums of asphalt had begun leaking into the underlying sandy silty gravels. This contaminated material was mixed in a pugmill with crushed RAP and an asphalt emulsion; it was then used as an intermediate pavement layer between two layers of cold mixed asphalt made from recycled asphalt pavement (Rollings and Vollor 1995). The roadway constructed of this material has continued to function well for 2 years. Meegoda et al. (1993) reported that laboratory investigations indicate petroleum contaminated soils can be used in HMA concrete pavements to produce mixtures that meet state specifications regarding Marshall stability and durability. No state highway agencies are presently conducting research into the use of contaminated soils in highway construction (Collins and Ciesielski 1994). Additional investigations are needed to determine which petroleum contaminants can be used and how different soils can be incorporated into pavement layers.

**Lignin.** Lignin is a high-volume inexpensive waste from paper production that can bond with silica surfaces and thereby act as a cementing agent. Consequently, the material has potential as a dust palliative and a limited soil stabilizer. A mixture of potassium dichromate and lignin has been particularly effective as the chromium ion enhances the lignin-soil bond. However, this mixture is expensive and it may pose environmental hazards. Consequently, its use has been limited, but it may prove effective in certain circumstances such as treatment of troublesome volcanic soils and soils containing chlorite (Ingles and Metcalf 1973).

Lignin has also been examined in the United States, the former Soviet Union, and France (OECD 1984) as a potential replacement for asphalt cement. Generally, this use has not proven promising, although a partial substitution of 30 percent of the asphalt binder with a lignin binder appears to be feasible (Kandhal 1993b). A recent summary by the National Center for

Asphalt Technology (NCAT) (Kandhal 1993a) indicated that lignin-asphalt binders are stiffer than conventional HMA mixtures and require a somewhat higher binder content. This would not be inconsistent with the OECD (1984) conclusion that lignins behave as an inert mineral filler, not as a bitumen extender. Laboratory experiments by Terrel et al. (1979) indicated that lignin-asphalt mixtures can be designed to provide strengths comparable to those of conventional HMA materials.

**Cellulose waste.** Cellulose is an amorphous carbohydrate polymer which is the main constituent of all plant tissues and fibers. Not surprisingly, cellulose wastes are derived from several diverse sources. Large quantities are generated from agriculture (e.g., crop residue and forest harvesting), specific manufacturing processes (e.g., food processing and wood and paper industries), and urban refuse such as municipal solid waste and manufacturing plant trash (Kandhal 1993b).

Attempts have been made to use cellulosic wastes to improve the properties of hydrocarbon binders (PIARC 1989). However, it is not directly applicable for use in asphalt concrete. Several processes have been used to try to convert cellulose waste to a usable binder or binder modifier. Kandhal (1993a) reports that a pyrolytic process (using heat to cause a chemical change) was used to convert cellulose waste to a binder. However, the material could not be substituted for asphalt cement because its rheologic properties did not meet standard performance criteria, and it could not be used as an asphalt extender or modifier because of incompatibility. Another pyrolytic process involving hydrogenation produced an oil that could be used as an asphalt-cement extender. Although compatible with asphalt, this oil did not significantly change the durability of an asphalt-surface course. No further work has been done to evaluate the long-term compatibility or performance of the cellulose-derived oil (Kandhal 1993b).

**Carpet fibers.** Approximately 2 million tons of carpet waste are produced annually (Collins and Ciesliески 1994 and Schroeder 1994). Polypropylene fibers from this waste have potential to function as reinforcement in portland-cement concrete, asphalt concrete, and soil. This source of pavement reinforcement has only had relatively cursory study to date.

Wang and Zureick (1993) and Groom et al. (1993) found that waste carpet fiber had potential to increase concrete toughness (important for loadings such as earthquake, blast, or projectile penetration) and to combat plastic shrinkage cracking in fresh concrete. Its use in conventional dense graded asphalt-concrete mixes found little difference between commercially produced fibers and waste fibers, but neither fiber performance in these particular tests was impressive (Schroeder 1994). Two other promising applications for carpet fibers include use in stone mastic asphalt where the fiber could provide the necessary thickening in the asphalt mastic phase and as an additive to reinforce sandy soils; both of these applications require investigation.

**Dredged material.** The sediments dredged from rivers and harbors to maintain navigation channels has been used as fill for construction sites, highways, dikes, and embankments (Rollings 1994). Because of the nature of river deposits, fairly clean deposits of sands and gravels may be found in the fast moving upper reaches of rivers. Through hydraulic segregation, as the velocity of flow decreases upon exit from an hydraulic dredge pipe, clean sand deposits are often encountered in dredged material disposal sites, even in coastal areas.

Normally at a high moisture content upon removal from the waterbody, dredged material must usually be dried prior to use. The fine-grained materials will usually require months or years to dry and then will find only limited use in pavement construction as subgrade or subbase courses. Fine-grained dredged material has been used to make light-weight aggregate in Russia, although presently the product is not readily available for commercial use. Coarse-grained dredged material, including both sands and gravels, typically mounds in the disposal site upon deposit, drains rapidly, and is soon available for use. It usually can be removed by conventional earthmoving equipment. Dredged sands and gravels have been sold or given away for use in road construction (HQDOA 1989).

**Miscellaneous slags, tailings, and other industrial by-products.** Industrial processes produce a wide variety of waste by-products such as foundry wastes, kiln dust, mine tailings, nonferrous metal slags, and phosphogypsum. Generally, these are only economically available in specific localities where particular industries operate. Consequently, while these may be locally important, their value to DOD facilities as a whole may be limited.

For industrial waste products to be used effectively in pavements, several problems must be overcome, the two most common being inconsistent composition and the presence of sulfates. The composition of the wastes may be variable which poses problems in maintaining quality. For example, lime and cement kiln dust is recovered though the air pollution system at the respective production plant. Although the recovered dust commonly has a high lime content, the content will vary, thus making it difficult to achieve consistent results if it is used for lime stabilization work.

Many waste products have high sulfate contents which can cause problems in pavement applications. For example, at the Port of New Orleans a local waste product from freon production (calcium hemihydrate) was stabilized with 6 to 7 percent portland cement for use as a high-strength base course in a marine terminal pavement. Since good quality aggregates are expensive in the New Orleans area because they must be shipped in from distant sources, this appeared to be a promising innovation. However, several failures occurred because of excessive swelling. This swelling developed when sufficient moisture accumulated under portions of the pavement to allow development of a highly expansive compound (ettringite - a calcium sulfoaluminate crystal) from the original cement stabilized material. Sulfate may also be present as a contaminant in the original material rather than as a byproduct of an industrial process. For example, waste material from processing coal has been stabilized with portland cement and used as a pavement base course. Pyrite present in the parent rock, oxidized causing the formation of sulfates which reacted with the cement hydration products and once again cause swelling in a manner analogous to the New Orleans case (Thomas, Kettle, and Morton 1989). Because there is often limited or no experience with a specific industrial waste or by-product in the pavement environment, its behavior can often be quite surprising. For example, a chromium slag was used as fill beneath a marine terminal pavement in Baltimore. Massive swelling later occurred in this fill, and although the Port Authority funded several investigations, a satisfactory explanation of the chemical and thermodynamic conditions necessary for this swelling to occur were not identified.

Each industrial waste or by-product must be individually analyzed for potential problems prior to use. Conventional aggregate specifications and testing may not identify unsuitable materials, and a substantial technical effort may be needed to adequately address engineering performance and environmental issues for use of this class of product. Because these products tend to be available only in limited areas where specific industries are in operation, there may be only a limited number of DOD facilities where any specific industrial waste or by-product would be useful. As noted in the opening of this entry, some of the more widely available industrial wastes and by-products are described in more detail in separate entries. Some general information on less widely available industrial wastes and by-products is presented in Table 16.

## Summary

As society produces greater quantities of waste products, especially nondecaying wastes, the need to find productive uses for these waste materials increases. A number of wastes seem to have potential for constructive uses in asphalt-concrete pavements; others are not so promising. Table 17 provides a summary of types and quantities of wastes currently known to be used in pavement applications along with a brief assessment of the success or lack thereof for each waste.

Large quantities of various waste materials can potentially be used by the pavement construction industry if they are determined to be compatible with paving materials and if they enhance, or at least do not degrade, long-term performance of the pavements into which they are incorporated. Some of the most promising waste materials that may be used in asphalt pavements are: reclaimed asphalt and portland-cement concrete pavements, rubber tires, incinerator residue, waste glass, waste plastics, roofing shingle waste, iron and steel slags, and coal by-products.

Much of the work to date on use of waste materials in asphalt pavements has been conducted in the laboratory, and few long-term field applications have been monitored to assess performance or actual life-cycle costs. The next

**Table 16**
**Miscellaneous Industrial Wastes and By-Products**

| Industrial Waste | Description | Past Pavement Applications | References | Comments |
|---|---|---|---|---|
| Foundry waste | Furnace dust, arc furnace dust, sand reclaimer residue | Fine aggregate in asphalt concrete | Collins and Ciesielski 1994, Javed 1994, Ciesielski and Collins 1993, Godwin 1983 | Variable materials, properties, and quality; trace metals may preclude use |
| Kiln dust | High CaO content | Stabilizing agent | PIARC 1989 | Quality is variable |
| Silica fume | By-product of silicon and ferrosilicon alloy manufacture; very fine pozzolanic particles | Partial replacement for portland cement in concrete | Collins and Ciesielski 1994 | Available as powder or as aqueous dispersion or slurry |
| Bag house fines | Fine particles collected in filter fabric baghouse dust collectors attached to asphalt production plants | Mineral filler in asphalt concrete | Lee and Fishman 1993, Lin 1990, Nelson and Wood 1990, PIARC 1989, Anderson and Tarris 1982 | Properties and quantities of fines vary from plant to plant |
| Mine tailings | Remnants from extraction of economically valuable minerals | Fill, subbase, stabilized base, and asphalt aggregate | Collins and Ciesielski 1994, Ciesielski and Collins 1993, PIARC 1989, Thomas, Kettle, and Morton 1989, Nanni 1988, Khedaywi 1988, OECD 1984, Godwin 1983, Sultan 1979, Rose 1979. | Highly variable materials and properties |
| Nonferrous metal slags | By-product from production of nickel, chromium, etc. | Subbase, base course, insulating layers, antiskid surfacing | PIARC 1989 | Variable properties requiring detailed study |
| Sulfate waste (flue gas desulfurization (FGD) sludge) | By-product from wet scrubbing of flue gases from coal-burning power plants | Stabilized base after dewatering and mixing with lime-fly ash, cement fly ash, or portland cement | Collins and Ciesielski 1994 | Must be dewatered before use |
| Phosphogypsum | By-product from production of phosphoric acid; primarily dehydrated calcium sulfate with environmentally sensitive contaminants | Stabilized bases, subbases, and subgrades, partial substitute for portland cement | PIARC 1989, Nanni and Chang 1989 | Reported susceptibility to water and frost, bound materials may swell when wet |

essential step is to assess the viability of their use in pavement construction projects. However, before any waste is placed in a pavement, its potential effect on the surrounding environment and on the recyclability of the pavement structure must be considered.

## Table 17
## Summary of Known Uses of Wastes in Pavement Applications (after Schroeder 1994)

| Waste Material | Annual Rates | | Current and Past Pavement Uses | | | | |
|---|---|---|---|---|---|---|---|
| | Produced | Recycled/ Reused | Asphalt Pavement | Concrete Pavement | Base Course | Embankment | Other |
| Blast furnace slag | ---[1] | 14.1 | Accepted use as an aggregate in base and surface (friction) course, research indicates good performance | Accepted use as a cement additive in granular form, research is ongoing | Accepted use; good, hard, durable aggregate | Limited but accepted use | Research in roller compacted concrete; accepted as ice control abrasive |
| Carpet fiber wastes | 2 | --- | Experimental stages in HMA and SMA; no field data | Experimental stages; no field data | No reported use | No reported use | No reported use |
| Coal combustion by-products | | | | | | | |
| Coal fly ash | 45 | 11 | Past use as a mineral filler; research ongoing | Accepted use; research ongoing | Used in soil stabilization | Used in flowable fills and embankments | Used in all types of PCC |
| Coal bottom ash or bottom slag | 16 | 5 | Combined ash as a fine aggregate; performance data limited | No reported use | Use unknown | Used as a subbase material and embankment material | Lightweight concrete, abrasive |
| Flue gas desulfurization waste | 18 | --- | Use unknown | Used as a set retarder | Used with cement in soil stabilization | Used as an embankment material | No reported use |
| Glass | 12.0 | 2.4 | Accepted use; long-term performance research underway | Past research indicated performance problems | Used in dense and open-graded bases | Some research projects under way | Limited use as a paint bead; pipe backfill |

*(Continued)*

[1] Not reported or unknown.

**Table 17 (Concluded)**

| Waste Material | Annual Rates | | Current and Past Pavement Uses | | | | |
|---|---|---|---|---|---|---|---|
| | Produced | Recycled/ Reused | Asphalt Pavement | Concrete Pavement | Base Course | Embankment | Other |
| Mill tailings | 432 | <1% | Accepted use; research indicates good performance | Limited but accepted use | No reported use | Accepted use | Accepted use |
| Municipal waste combustion ash | 7.3 | <10% | Past research indicated good performance; environmental questions | No known use | Used in cement-stabilized bases | Used in soil stabilization | No reported use |
| Plastic | 14.7 | 0.3 | Used as binder additive | Experimental stage | No reported use | No reported use | Used as fence or delineator posts & guardrail blockouts |
| Reclaimed concrete pavement | 3 | --- | Limited use; long-term performance research underway | Limited use; research underway | Accepted use | Accepted use | Used as rip-rap |
| Reclaimed asphalt pavement | 91 | 73 | Variety of accepted uses | Experimental stages | Accepted use | Accepted use | Used as shoulder material |
| Roofing shingle waste | | | Limited use; research under way | No reported use | No reported use | No reported use | Used as a pothole patching material |
| Industry produced | 0.4 | <1% | | | | | |
| Reroofing waste | 7.7 | <1% | | | | | |
| Scrap tires | 2.3 | 0.4 | Accepted use; extensive research being conducted | Experimental stages | Used as an insulator | Used with some success - research continuing | Being marketed for use as noise or retaining wall, molded posts, many minor uses |
| Steel slag | 7.5 | 6.9 | Past research indicates good performance | Extensive research; poor performance | Limited use | Accepted use | Ice control |
| Waste rock | 954 | <1% | Accepted use; research indicates good performance | Limited but accepted use | No reported use | No reported use | No reported use |

## Manufactured Modifiers

### Introduction

Some naturally occurring materials and industrial by-products and/or waste materials continue to be marketed as asphalt modifiers; however, the trend in today's market is toward high-tech, carefully engineered modifiers (Anderton 1990). Current emphasis for the use of modifiers is directed at correcting the following pavement performance problems: rutting, thermal cracking, placement difficulties, and water susceptibility (Haas et al. 1983, Terrel and Walter 1986, Epps 1986). Asphalt modifiers have also been used to improve the properties of mixtures containing marginal quality aggregates. The use of modifiers for this purpose can be very significant in terms of cost savings and/or the ability to build in remote locations using locally available materials (Jones 1990).

Several factors have led to the escalating interest in asphalt modification. First of all, there is a growing perception that asphalt cements have changed in recent years (Epps 1986, Terrel and Epps 1988). These changes are believed to have been caused by new refining technologies, which permit the extraction of more light-end petroleum products from crude oil. If the extracted material changes, so would the chemistry of the residue from this process (i.e., asphalt cement). Some additional factors that have led to the growing interest in asphalt modification include (Anderton 1990): increased traffic demands, an economic-driven trend toward thinner pavements and deferred maintenance, and an environmental-driven trend toward the use of industrial by-products and waste materials.

Today, both asphalt cements and asphalt emulsions are being modified. The modified binders are used for fog seals, slurry seals, chip seals, cold-mixed patching mixtures, dense-graded hot mixtures, and open-graded hot mixtures (Epps 1986). A popular classification scheme for asphalt modifiers is shown in Table 18. Roberts et al. (1991) warn that the dynamic nature of the paving industry requires that this list be updated frequently.

Because of the limited scope of this research project, only the polymeric modifiers will be discussed in any depth in this section. Brief descriptions of the other types modifiers are provided below.

*a.* Fillers are typically used to impose the following influences on HMA concrete (Roberts et al. 1991): to fill voids, to meet specifications for aggregate gradation, or to improve bond between asphalt and aggregate. These influences typically result in decreased optimum asphalt-cement content, increased density, and increased stability (Brown, McRae, and Crawley 1989). There are numerous types of materials that can be used as a filler material. Some of the more common types of filler materials include crushed fines, lime, portland cement, fly ash, and carbon black. Carbon black is a unique filler because it is comprised of very small particles. It can be dispersed within asphalt cement, thus serving as a

**Table 18**
**Classification of Asphalt Modifiers (after Terrel and Epps 1988, Terrel and Walter 1986)**

| Type (common percentages) | Examples |
|---|---|
| 1. Filler (see types) | Carbon black (10-15 percent)[1]<br>Sulfur (30-40 percent)[1]<br>Mineral filler (1-15 percent)[2]:<br>Crusher fines<br>Lime<br>Portland cement<br>Fly ash |
| 2. Extender (30-40 percent)[1] | Sulfur<br>Lignin |
| 3. Polymers (3-10 percent)[1] | Plastic:<br>Polyethylene<br>Polypropylene<br>Ethyl-vinyl-acetate (EVA)<br>Polyvinyl chloride (PVC)<br>Rubber:<br>Natural rubber<br>Styrene butadiene rubber (SBR)<br>Styrene-butadiene-styrene (SBS)<br>Combinations of plastic and rubber |
| 4. Fiber (0.1-0.5 percent)[2] | Natural rock wool<br>Polypropylene<br>Polyester<br>Fiberglass |
| 5. Oxidant (0.1-0.2 percent)[1,3] | Manganese salts |
| 6. Antioxidant (1-5 percent)[1] | Lead compounds<br>Carbon<br>Calcium salts |
| 7. Hydrocarbon (see examples) | Recycling and rejuvenated oils (3-10 percent)[1]<br>Natural asphalt cements (25-40 percent)[1]<br>Hard asphalt cements (10-20 percent)[1] |
| 8. Antistrip (see examples) | Amines (0.5-1 percent)[1]<br>Lime (1-2 percent)[2] |

[1] Percent by mass of total binder.
[2] Percent by mass of aggregates.
[3] Percentage of active ingredient only.

"microfiller." In addition to the typical filler effects listed above, carbon black can increase the high-temperature stiffness of asphalt cement (Yao and Monismith 1986).

*b.* Extenders are used to reduce the amount of asphalt cement used in a mixture (Roberts et al. 1991). The use of extenders is driven by economics, so the material and blending costs associated with the use of an extender must be less than the cost of asphalt cement.

*c.* Fibers may be used to increase the stiffness and tensile strength of asphalt concrete. These improvements are useful for combating both permanent deformations (rutting) when the HMA is subjected to heavy loads and crack formation/propagation when the HMA is placed over a cracked substrate (Roberts et al. 1991). Improved performance related to tensile strength and crack propagation is better documented than improved resistance to rutting (Freeman et al. 1989, Terrel and Walter 1986).

*d.* Oxidants are used to stiffen asphalt cement and, thus, stiffen the HMA with which it is associated. The increased stiffness helps to prevent rutting; however, caution must be exercised against increasing the susceptibility of the HMA to low-temperature cracking (Roberts et al. 1991). Manganese-based compounds have been employed as oxidizers for asphalt cement (Kennedy and Epps 1985). Hot mix asphalt concrete containing manganese has been reported to be soft during construction, stiff after a curing period, and resistant to property changes with age (Terrel and Walter 1986).

*e.* Antioxidants are used to minimize the oxidation-related aging that causes asphalt cement to become brittle. Maintaining ductility helps to prevent shrinkage cracking; however, caution must be exercised against increasing the susceptibility of HMA to rutting (Roberts et al. 1991). The chemical compatibility of the antioxidant with the asphalt cement must be considered. An antioxidant that is incompatible will cause the colloidal nature of the asphalt cement to become unstable.

*f.* Some hydrocarbons, in addition to polymeric modifiers, can be blended with asphalt cements to improve selected properties. Hard asphalt cements can be used to improve the hot-weather performance of relatively soft asphalt cements. Soft asphalt cements and/or rejuvenating oils can be used to improve the cold-weather performance of relatively hard asphalt cements (Roberts et al. 1991).

*g.* Antistripping agents are used to improve the resistance of HMA to moisture-related damage. The mechanism for improvement is typically described as an improved bond between the aggregate and asphalt cement. The effectiveness of antistripping agents is highly dependent on the asphalt/aggregate combination. Liquid antistripping agents may soften the physical properties of the base asphalt cement (Anderson, Dukatz, and Petersen 1982).

### Polymeric modifiers

**General.** Advantages of polymer modification are numerous. Depending on the polymer type, the polymer percentage, and the characteristics of the base asphalt cement, improved binder properties may include: reduced dependency of mechanical properties on temperature, increased flexibility at low temperatures, increased stiffness and toughness at high temperatures, improved adhesion, and improved tensile strength. These improved binder properties may impose some of the following enhancements for HMA concrete: reduced rutting, longer fatigue life, increased resistance to moisture-related stripping, and increased resistance to low-temperature cracking (Ali et al. 1994; Brule, Brion, and Tanguy 1988; Epps 1986; King and King 1986).

Polymer modifiers can be engineered to impart selected improvements for asphalt binders. It is therefore useful to separate them into two general categories: plastomers or plastics and elastomers or rubbers. Plastomers are reported to form a tough, rigid, three-dimensional network throughout the asphalt cement. The resulting binder is stiff at small strains, but may fracture with excessive strain. Common examples of plastomers include: EVA and LDPE (King and King 1986).

Polymers in the elastomer category resist permanent deformation by stretching and recovering their shape quickly when the force is removed. When used in asphalt cement, these polymers add little stiffness or strength to the asphalt until they are stretched. The tensile strength of the elastomer increases as it is stretched, so the resulting binder has improved resistance to fracture. Common examples of elastomers include: SBS and SBR (King and King 1986).

**Compatibility.** Although recent advances in macromolecular chemistry have permitted the development of a large number of polymeric products, relatively few polymers are compatible with asphalt cement. Compatibility problems are easiest to visualize with the help of a simple physical model for asphalt cement. Asphalt cement is a colloidal system consisting of asphaltenes that are in suspension in oil. Their suspension is facilitated by the existence of "resins" that have chemical compositions intermediate between the asphaltenes and the oil. These resins promote suspension by surrounding the asphaltenes and serving as protective envelopes. The introduction of any polymer into asphalt cement may disrupt this colloidal system. Poorly compatible polymers could potentially promote excessive flocculation of asphaltenes and oil bleeding. Asphalt modification with these incompatible polymers would result in binders with poor engineering properties (Brule, Brion, and Tanguy 1988).

A compatible polymer may be either soluble or insoluble in the asphalt cement, but it will remain well dispersed in the asphalt. If the polymer is soluble, all polymer-polymer interactions are replaced by polymer-solvent interactions. In these cases, a small concentration of the modifying agent can impose an increase in flow resistance. If a polymer is compatible with an asphalt cement, but is not soluble, it may remain as discrete globules. When dispersed and heated, these globules become swollen with the oily fraction of the binder, leaving the continuous phase enriched with asphaltenes and resins. The net result is an increase in viscosity similar to the previous case. Systems that include globules of polymer in suspension must be considered diphase materials. Handling of diphase materials requires special care to avoid problems associated with phase separation (Brule, Brion, and Tanguy 1988).

**Implementation.** Depending on the polymer type, one of several techniques will be required to incorporate the modifier in the asphalt cement.

Some modifiers are marketed in conjunction with various grades of a particular asphalt cement. In these cases, the manufacturer will typically have preblended the polymer with the base asphalt cement. Blending in these cases is often performed in large (e.g., 190 $m^3$ (6,700 $ft^3$)) tanks, which are equipped with both agitation and recirculation mechanisms (Button 1988).

Some polymer modifiers are marketed alone, without an accompanying asphalt cement. These modifiers, which are available in either powder, pellet, or latex form, may be blended with asphalt cement at the site of the asphalt mixing plant (Ali et al. 1994). Truck-mounted and stationary tank-mounted blending devices are available to facilitate this operation. When pellets are used, a blender that grinds the pellets as they are fed into the asphalt cement is advisable. When a latex is used, it may be added to either the asphalt cement or the HMA concrete. When a latex is added to the HMA concrete in a drum mix plant, it should be added downstream from the asphalt inlet. This procedure can minimize degradation of the polymer if prolonged exposure to heat is a concern (Button 1988).

Requirements for the blending temperature and blending duration are supplied by the manufacturers of the polymers. These requirements are highly variable. Once blending has been completed, homogeneity must be maintained with circulation. Tank trucks and stationary tanks are equipped with mechanisms for recirculation.

### Laboratory and field experiences

Polymers that are commonly used for asphalt modification will be presented in the following text. The polymers are listed according to abbreviated chemical descriptions.

**Ethylene-Vinyl-Acetate (EVA).** Anderton (1990) modified several AC-20 asphalt cements with 4 percent EVA by total mass of binder. The effect of modification on binder viscosity and stiffness was demonstrated by capillary tube viscosity tests and needle penetration tests, respectively. Tube viscosities at 135 °C (275 °F) were increased two- to three-fold. Needle penetration values decreased at both 4 °C (39 °F) and 25 °C (77 °F). Additionally, the modification with EVA increased the tensile strength and the resilient modulus (stiffness) of mixtures at both 25 °C (77 °F) and 40 °C (104 °F), without much effect at -18 °C (0 °F). Marshall stability was also increased (Anderton 1990). The EVA modification increased creep stiffness at 40 °C (104 °F) by 100 percent (Ahlrich and Anderton 1994).

Maccarone and Jameson (1988) reported the use of EVA-modified HMA concrete for an intersection with heavy traffic in Melbourne, Australia. In order to attain similar workabilities for unmodified and modified mixtures, the field compaction temperatures were 160 °C (320 °F) and 175 °C (347 °F), respectively. Initial skid resistance and the rate of decreasing skid resistance over time were approximately the same for unmodified and modified mixtures.

After 4 years of service, both the unmodified and modified mixtures had suffered little permanent deformation. In order to predict rutting under continued service, a wheel-track device was used to impose truck-type loads on sections constructed of similar materials. The EVA-modified mixture exhibited a significantly lower rate of rutting for these severe laboratory tests.

Pradhan and Armijo (1994) reported on the performance of an HMA concrete containing AC-10 asphalt cement, both with and without an EVA modifier. The materials were placed as overlays on an interstate highway in Montana. Conventional techniques were used for placement and compaction of the modified mixture and no problems were reported. After construction, in-place voids were approximately the same for both unmodified and modified mixtures. After 2 years of service, the EVA-modified section had rutted 50 percent less than the unmodified section. Both sections, however, had suffered transverse cracking on the order of 30 cracks per kilometer (50 cracks per mile). Most of the cracking damage occurred during the second winter when air temperatures got as low as -34 °C (-29 °F).

Rogge, Terrel, and George (1992) reported the use of EVA modifier in roadway pavements in Oregon. After 2 years of service of the Farewell Bend project, both the unmodified and modified pavements were performing well. Tests on laboratory-produced mixtures were used to predict long-term performance. Results from diametral fatigue tests at 25 °C (77 °F) and 0 °C (32 °F) showed that EVA-modified mixture should be more resistant to fatigue cracking. A comparison of force-ductility results before and after aging by the rolling thin film procedure provided a hint that the EVA-modified mixture may resist oxidative aging-related cracking better than the unmodified control mixture.

In another field project, Rogge, Terrel, and George (1992) reported that an EVA-modified section suffered three times as many transverse cracks as compared with a neighboring unmodified section. After extensive laboratory testing, they proposed that the brittleness was caused by overheating. The EVA-modified mixture was subjected to prolonged exposure at approximately 170 °C (338 °F). More recent projects in Oregon have included tighter monitoring/control of mixing temperatures.

**Low-density polyethylene (LDPE).** Anderton (1990) modified several AC-20 asphalt cements with 5 percent LDPE by total mass of binder. The effects of modification by LDPE were similar to those resulting from EVA modification. Capillary tube viscosities were increased two- to four-fold at 135 °C (275 °F). Needle penetration values decreased at both 4 °C (39 °F) and 25 °C (77 °F). HMA modification with LDPE increased tensile strength and resilient modulus (stiffness) at both 25 °C (77 °F) and 40 °C (104 °F), without much effect at -18 °C (0 °F). Marshall stability increased and creep stiffness at 40 °C (104 °F) increased by 150 percent. Modification with LDPE also helped to minimize the rate of creep for mixtures that contained rounded aggregates (Ahlrich and Anderton 1994).

The first documented use of LDPE-modified HMA in the United States was in 1986. The asphalt was placed at New York City's John F. Kennedy International Airport (Godfrey 1986). At this time, the use of LDPE-modified asphalt was already popular in Europe. Low-density polyethylene-modified asphalt concrete had even served successfully as a surface over cobblestones, which had historically been difficult to surface with modern materials. Overlays using unmodified HMA had failed in as little as 6 months due to their inability to adhere to the large, smooth stone surfaces (Godfrey 1986).

Anderton and Lewandowski (1994) reported on the performance of a LDPE-modified HMA overlay, which was placed on a runway at Houston Hobby Airport. As compared with neighboring unmodified HMA pavements, the LDPE-modified overlay exhibited improved resistance to fatigue cracking and reduced rutting. The high stability of the modified asphalt permitted grooving for drainage/skid resistance. Unlike many previous experiences with grooving unmodified HMA pavements, the grooves in the modified overlay did not close during summer months and did not ravel during winter months.

Little (1992) studied the effect of modification of AC-20 asphalt cement with LDPE. He measured shear strength and creep characteristics for mixtures containing both crushed limestone and rounded river gravel. Shear strength, as measured by conventional triaxial tests on cylindrical specimens, was increased by the use of 4.3 percent and 6.0 percent LDPE by mass of binder for each mixture. These effects were most significant for mixtures containing river gravel and for mixtures that were produced with binder contents 0.4 percent higher than optimum. Static-load creep was decreased by the use of LDPE, particularly under conditions of high load. When 6.0 percent LDPE was used to modify mixtures containing river gravel, the magnitude of creep was unaffected by the richness of the mix (optimum binder versus 0.4 percent higher than optimum).

**Styrene butadiene rubber (SBR).** Button (1988) reported the use of a latex SBR-modified HMA for a highway pavement in Texas. Production and placement operations were executed successfully, however, normal procedures had to be modified slightly. While unmodified mixtures are normally mixed at 140 °C (284 °F), the SBR-modified mixture had to be mixed at 160 °C (320 °F) in order to ensure adequate coating of aggregates. The SBR-modified mixture was reported to have been much more sticky than the unmodified mixture. With each dump truck delivery of the SBR-modified mixture, approximately 400 kg (882 lb) of material would adhere to the walls of the truck bed. This material had to be knocked out of the truck bed manually. The paving machine operator also reported that the SBR-modified mixture had a much higher drag than unmodified mixtures. Increased drag would only become a problem if it led to "tearing" of the pavement surface. Krater, Wolfe, and Epps (1988) reported a related construction problem. Their SBR-modified mixtures tended to stick to rubber tires during compaction. The rubber-tired rollers had to keep some distance from the paver to permit time for the mixture to cool.

Anderton and Lewandowski (1994) reported the performance of an SBR-modified, dense-graded asphalt concrete, which was placed on a runway at the Capitol City Airport in Lansing, MI. The SBR modifier was used for its anticipated ability to improve resistance to low-temperature cracking. After 5 years of service, the wearing course was performing well. A similar material was placed on two runways at the Detroit City Airport in Michigan. This material was also performing well. However, an attempt to cut transverse grooves for skid resistance was unsuccessful. The grooves in this material seemed to promote raveling.

The placement of an overlay in Texas included modification of asphalt concrete with an SBR latex. Use of the modifier increased the cost of the overlay by 10 percent. This cost was expected to be repaid if the overlay survived 6 months longer than the typical 5-year life span for neighboring unmodified overlays. Early performance indicated that the modifier would extend the life span of the overlay substantially (Godfrey 1986).

Anderton and Lewandowski (1994) also reported the performance of an SBR-modified porous friction course (PFC), which was placed on a runway at Oakland Pontiac Airport. The modified PFC was placed as a 25.4-mm-(1-in.-) thick surfacing over a dense-graded asphalt-concrete leveling course. The surfacing was applied to improve skid resistance during inclement weather. The surfacing had proved to be durable. Local authorities attributed the durability, at least partially, to the SBR modifier. The modifier had increased the viscosity and stiffness of the binder, thus permitting a thicker film of binder on the aggregates. Thick films of binder in a PFC resist oxidative aging better than thin films of binder.

Heather (1992) reported the use of SBR in porous friction courses for highway applications in the United Kingdom. Porous friction courses are popular in the United Kingdom for various reasons including reduced spray and reduced noise. However, the void contents of approximately 20 percent have historically caused durability problems, particularly those related to oxidative aging. These problems typically prevented unmodified mixtures from lasting more than 5 years. By modifying the asphalt cement with SBR, their design binder content increased from an average of 4.8 percent to an average of 5.2 percent by mass of total mix. This resulted in an increase in film thickness of approximately 10 percent, with no reported problems of "drain-down."

SBR, in latex form, has been used to modify asphalt for chip seal applications in Texas (Godfrey 1986). The modifier reportedly improved the binder's ability to retain aggregate, even under the abrasive action of a snowplow blade. After 15 months of service, no damage and no bleeding had been reported.

**Styrene-butadiene-styrene (SBS).** Anderton (1990) modified several AC-20 asphalt cements with 12 percent SBS by total mass of binder. The effects of SBS modification were slightly different than modification by LDPE and EVA. Capillary tube viscosities were increased two- to four-fold at 135 °C (275 °F). Generally, needle penetration values decreased at both 4 °C

(39 °F) and 25 °C (77 °F). However, needle penetration was increased for an AC-20 that exhibited high stiffness when unmodified. Modification with SBS increased the tensile strength and resilient modulus (stiffness) at both 25 °C (77 °F) and 40 °C (104 °F), with a slight decrease in resilient modulus at -18 °C (0 °F). Marshall stability increased and creep stiffness at 40 °C (104 °F), increased by 100 percent (Ahlrich and Anderton 1994).

Fleckenstein, Mahboub, and Allen (1992) reported the use of an SBS-modified asphalt concrete for a highway pavement in Kentucky. As part of preliminary laboratory testing, resilient modulus tests were performed at room temperature. Relative to the unmodified AC-20 asphalt concrete, the polymer-modified mixture demonstrated 35 percent lower permanent strain during specimen conditioning and 20 percent higher stiffness during the resilient phase of the test. After 2 years of service as a pavement wearing surface, the polymer-modified mixture had rutted 30 percent less than an unmodified mixture with similar traffic conditions. In a related study for the Kentucky Department of Highways, the Root-Tunnicliff test was performed to measure the ability of SBS modification to minimize moisture damage (stripping). Based on this limited amount of testing, SBS modification did not appear to be effective in this respect.

Although Fleckenstein, Mahboub, and Allen (1992) reported the polymer-modified mixture to be less susceptible to rutting, they also reported its cost to be approximately 30 percent higher than the cost of a similar unmodified mixture. The modified mixture cost about $52 per metric ton, while the unmodified mixture cost about $40 per metric ton.

Button (1988) reported the use of an SBS-modified asphalt concrete for a highway pavement in Texas. Placement and compaction were routine, with the exception of temperatures. While the unmodified HMA was mixed at 150 °C (302 °F), the modified HMA concrete required a mixing temperature of 170 °C (338 °F). In addition, the paving crew expressed concern over the color of the modified HMA. The SBS-modified mixture was brown in color, which led the crew to think that the mixture had been "scorched." The brown color; however, was typical for mixtures containing this particular SBS modifier.

Australian experience (Edser 1990) has included the use of SBS for both sprayed seals and hot mix asphalt. As compared with unmodified spray seal applications, polymer modification improved the retention of aggregates and minimized the propagation of cracks from underlying layers. As compared with unmodified HMA, polymer modification reduced rutting tendencies at intersections.

**Reacted styrene-butadiene-styrene (RSBS).** At least one manufacturer of a styrene butadiene block copolymer modifier proposes that their product chemically reacts with asphalt cement. This type of product is classified separately from SBS due to the potential influence of this reaction on the engineering properties of the modified binder. These products are preblended with asphalt cement by the manufacturer.

Ali et al. (1994) modified AC-5, AC-10, and AC-20 asphalt cements with RSBS. Modification increased binder viscosity and stiffness and approximately doubled the capillary tube viscosity at 135 °C (275 °F) for each asphalt cement. Modification also increased mixture stiffness. Modification decreased needle penetration and this decrease was most significant for the AC-5. Polymer modification of AC-5 and AC-10 mixtures also increased resilient modulus at test temperatures ranging from -18 °C (0 °F) to 60 °C (140 °F). Modification of the AC-20 mixture increased resilient modulus at high temperatures (60 °C (140 °F)), but did not significantly affect resilient modulus at low temperatures (-18 °C (0 °F)). Modification of AC-5, AC-10, and AC-20 mixtures decreased permanent deformation under static creep.

Button (1988) reported the use of an RSBS-modified asphalt concrete overlay for a Texas highway application. While unmodified mixtures could be mixed at temperatures less than 140 °C (284 °F), the modified mixture required production temperatures of about 150 °C (302 °F) to ensure uniform coating of aggregates. With increased mixing and placing temperatures, a binder containing 1 percent styrene-ethylene-butadiene-styrene (SEBS) had no detrimental effect on the in-place density of the HMA. However, relative to the unmodified mixture, the modified mixture was more susceptible to sticking to pneumatic tires. For quality control purposes, centrifuge extraction with trichloroethylene was used without problems.

Zhou, Nodes, and Nichols (1994) reported the use of RSBS to modify an AC-20 asphalt cement for use in an asphalt-concrete overlay on an Oregon highway. The modified binder mixed easily with aggregate, but it also tended to cling to machinery. Maintenance operations for plant and paving equipment became more extensive with the use of modified asphalt. The modified binder also caused pumping problems of the modified binder at the plant due to its viscosity, so the pumping temperature had to be increased to 180 °C (356 °F). Finally, the modified mixture could not remain in stagnant storage for extended periods of time. Due to its high temperature and its slightly higher optimum binder content, as compared with unmodified asphalt, it was susceptible to binder "drain-down." The modified mixture cost about $30 per metric ton, which was approximately 15 percent more than the unmodified mixture.

Based on needle penetration tests on binder recovered from field cores, the modified binder was hardened (aged) about the same amount as the unmodified binder. Field cores were obtained from both the modified and unmodified pavements for resilient modulus and fatigue tests. Although no significant difference between the mixtures was observed for resilient modulus, the polymer-modified mixture exhibited improved resistance to fatigue cracking. Condition surveys on the pavements over a period of 3 years revealed that the RSBS-modified mixture was slightly more resistant to rutting than the unmodified mixture. Both mixtures; however, had suffered transverse cracking with average spacings on the order of 35 m (115 ft) (Zhou, Nodes, and Nichols 1994).

**Others.** Krater, Wolfe, and Epps (1988) reported the use of 5 percent polyolefin, composed of ethylene and acrylic acid. Unmodified and modified HMA concrete mixtures were placed in states ranging geographically from Texas to Maine. While only 20 min was required to blend the plastic polyolefin pellets with asphalt cement in the laboratory, 8 hr was required for blending in distributor trucks. Due to the practical problems with this time requirement, the point of introduction for modifier pellets was changed to the pugmill. Mixing then went smoothly. Laydown and compaction also went well. A breakdown roller operator even commented on how well the sections with modified asphalt concrete compacted.

Field cores revealed that modification with this polyolefin increased resilient modulus at 40 °C (104 °F) by as much as 50 percent. This attribute was expected to contribute to rutting resistance. Resilient modulus tests on cores at -10 °C (14 °F) showed no significant effect by modification. Tensile strength tests at -10 °C (14 °F) on laboratory-compacted samples showed increased strength for the modified mixtures. These attributes indicated that the modification could potentially reduce susceptibility to low-temperature cracking. Lottman testing in the laboratory revealed that modification with polyolefin would not contribute significantly to resistance to moisture (stripping) damage (Krater, Wolfe, and Epps 1988).

Button (1988) reported the use of a SEBS modifier in an HMA concrete overlay for a roadway. The modifier was preblended with AC-5 at a level of 3 percent. For circumstances that could not be controlled, the modified binder was stored hot for 7 months prior to use. Force-ductility tests performed before and after storage showed little change in the binder's physical properties. Design asphalt-cement content, mixing temperature, and compaction temperature were all the same for both unmodified and modified mixtures. After 2-1/2 years of service, both pavements were performing satisfactorily.

### Concerns/cautions

**Engineering.** The modifier market is dynamic. New manufacturers appear frequently and experienced manufacturers often change their modifier formulations. This active environment is not conducive to the development of field data concerning the use of modifiers. Consequently, modifiers are often chosen based on their performance in field-related laboratory tests. These practices should be used with caution. In addition, small pilot projects or test sections should be constructed and monitored whenever possible.

The effects of asphalt-cement modifiers are dependent on the chemical and physical properties of the asphalt cement with which they are combined. Decisions related to the use of modifiers should be made with the source of the base asphalt cement in mind. Specifications should be based on the asphalt-modifier blend, rather than the modifier itself (Epps 1986).

Polymer modification can contribute to the variability of materials in real-life paving operations. Heather (1992) reported on the variability of HMA concrete that included a preblended modified binder. In this sense, preblended means the manufacturer of the polymer blended it with the asphalt cement in a laboratory-type environment. The specified proportion of modifier was 7 percent by mass of binder. Field cores revealed that actual proportions ranged from 3 to 10 percent. Heather attributed this variability to phase separation of the polymer from the asphalt cement during stages of storage and handling that included heat but did not include adequate stirring.

Typical HMA pavement design methods that are used today do not permit the engineer to take advantage of the performance improvements offered by modifiers. The material tests do not represent the loading and environmental conditions expected in service (Epps 1986). The new binder and mixture tests that have been developed under the National Research Council's SHRP will potentially improve this situation.

During the transition toward the use of SHRP performance-related tests, conventional standardized tests will have to be used with caution. As an example of potential problems, Shuler, Hanson, and McKeen (1988) found that the determination of mixing and compaction temperatures with conventional kinematic viscosity tests can be misleading for certain polymer modified asphalt binders. The kinematic viscosity results have indicated significantly higher mixing and compaction temperatures than were necessary during full-scale tests (Shuler, Hanson, and McKeen 1992). When the capillary viscometry data have been used for mixing and compaction temperatures, some polymer-modified mixtures have become brittle as a result of overheating during construction (Rogge, Terrel, and George 1992).

Conventional mix design procedures should also be used with caution during the transition toward SHRP procedures. Although Marshall and Hveem methods may be useful for estimating the optimum binder content for modified mixtures, additional research is needed to investigate the applicability of current stability, unit weight, and air void criteria (Jones 1990).

Problems with modifier blending, storage, and delivery have been observed on several projects (Epps 1986). Practical problems and solutions need to be documented and disseminated. During placement, both plastomer-modified mixtures and elastomer-modified mixtures have been observed to have a tendency to "tear" behind paving screeds (Krater, Wolfe, and Epps 1988). In order to alleviate some of these problems, increased temperatures in mixing, placing, and compaction may be required. Of course, increased temperatures equate to increased costs.

**Economic.** Most modifiers will significantly increase the cost of in-place asphalt concrete. These increases, which can be as high as 40 percent, result from the following (Roberts et al. 1991): the cost of the modifier itself, the cost of blending the modifier with asphalt cement, and costs associated with modifying equipment. In order for polymer modification of asphalt concrete to be economically advantageous, the increased initial costs must be offset by decreased maintenance requirements, increased performance life, or both. Implementation of performance-related specification testing will help predict/quantify cost-savings in terms of maintenance and extended performance.

**Environmental.** Modifiers that are known to increase the severity of pollution either during construction or during service should not be used. These effects are not common research topics, so they are not well-documented.

Modifiers that hinder the recyclability of HMAs should also be avoided. This decision could be justified by considering either economic or environmental concerns. The recyclability of modified asphalt concrete has not been studied extensively, so it is not well-documented.

## Summary

When blending polymers with asphalt cement, manufacturer's recommendations must be followed closely. If agitation is inadequate, the polymer may not disperse properly. Chopping mechanisms seem to facilitate blending operations for polymers that are sold in pellet form. Once blending has been completed, the binder should not be left hot and without circulation. Most polymer-modified binders are susceptible to phase separation to some degree.

Many polymers are available that improve the resistance of HMA concrete to permanent deformation at high service temperatures. Typically, these polymers do not, however, improve low-temperature performance. The most attractive application is thus to select a base asphalt whose grade is compatible with the low-temperature end of the environment and to modify with the percentage of polymer required to achieve acceptable resistance to deformation at high temperatures. Polymers are typically not effective for minimizing moisture-related damage (stripping) or preventing the propagation of cracks from underlying cracked substrates. The effects of polymers on aging have not been studied extensively.

In general, polymer modification can improve the rut-resistance of mixtures that contain marginal (e.g., rounded) aggregates. Polymers can also help minimize problems associated with deviations in binder content, relative to the optimum binder content, as defined by the mixture design. For chip seal applications, modification with polymers appears to help prevent the loss of aggregates.

Listing advantages and disadvantages for each type of polymeric modifier is difficult because the effectiveness of any particular modifier is highly dependent on its compatibility with the particular asphalt cement with which it is blended. However, in very general terms, plastomers and elastomers can be compared. Plastomers include EVA and LDPE. Elastomers include SBS and SBR. Binders that contain plastomers are more susceptible to phase separation. Mixtures that contain elastomers are more susceptible to both tearing under the paver screed and sticking to the rubber tires on pneumatic rollers. Both plastomers and elastomers can improve the resistance of mixtures to permanent deformation. Elastomers have performed slightly better in terms of improving low-temperature performance.

Problems associated with polymer modification include the following: Many of the current binder tests are not directly applicable to polymer-modified binders; many of the current mixture tests and mixture design procedures are not directly applicable to polymer-modified mixtures; most of these current, standardized tests are based on empiricism, rather than fundamental principles; criteria that have been developed from experience cannot be extrapolated to new materials that exhibit significantly different rheological behavior.

# 3 Asphalt Pavement Durability Issues

## Asphalt Pavement Failure Modes

There are two basic types of pavement failure modes generally associated with asphalt pavements—load-related failures and durability-related failures. However, identification of the particular failure mode is not always simple. Some of the observed pavement distresses may be similar for both types of generalized failures (i.e., cracking in the pavement could be caused by load- or durability-related failures). However, for the purpose of this report, load-related failures will be considered to be permanent deformation or rutting and structural failures denoted by alligator cracking. Durability-related failures (which are the focus of this study) will be considered to be thermal cracking induced by temperature extremes or temperature cycling, fatigue cracking, and moisture damage or stripping of the asphalt cement from the mixture. Consequently, a durable asphalt cement would be one that possesses physical properties that will produce the desired initial product performance and will resist changes in physical properties during long-term, in-service environmental aging (Petersen 1984).

Each of the durability-type issues listed above, with the exception of the moisture damage, denote cracking as the predominant result of the failure. There are differences, however, in the location of crack initiation. Generally, cracks associated with fatigue cracking initiate at the bottom of the HMA pavement and cracks associated with oxidation or aging initiate at the HMA pavement surface.

There are several factors associated with HMA that can result in durability related cracking of the in-place pavement (Malan, Strauss, and Hugo 1989):

*a.* Using an inappropriate asphalt grading (AC-20, AC-10, etc.) or type for the specific project and or pavement application.

*b.* HMA mixing temperature used at the plant (either too high or too low).

*c.* Temperature of the HMA during the paving operation (generally too high).

*d.* Filler content and/or type used in the HMA.

*e.* Compaction techniques and equipment used during paving.

*f.* Climatological factors such as oxidation, seasonal temperature changes, and UV radiation.

From the above partial list of factors that may lead to pavement cracking, most of the items can be or should be addressed through HMA design and construction quality control considerations. However, items (a) and (f) are related to the type and properties of the asphalt cement or binder. Item (a), concerning the use of an inappropriate asphalt cement, could also be addressed through the mix design process, but if only "poor" quality or inappropriately graded asphalt cements are locally available, then modification of the asphalt cement may be necessary.

## Asphalt Aging

One of the more predominant causes of durability distress is the aging process that occurs in the asphalt cement and HMA pavement. Aging will generally make the asphalt cement or binder become more stiff. The stiffening can occur through the loss of volatiles, steric hardening (a molecular structuring that causes the asphalt molecules to be restricted in movement), or a chemical reaction between the asphalt cement and atmospheric oxygen (oxidation). Oxidation is perhaps the most prevalent cause of asphalt aging, but, regardless of which mechanism or combination of mechanisms cause the aging, the increase in stiffness generally improves the load-carrying capability of the pavement making it more resistant to permanent deformation. However, the increased stiffness of the binder can also make the pavement more susceptible to cracking, raveling, and moisture damage, thereby reducing the wear resistance of the pavement. It was these latter types of pavement distress (cracking, raveling, and stripping) that were delineated in the Army and Air Force pavement evaluation database at WES as the predominant type of airfield pavement failure.

The fact that airfield pavements usually exhibit durability-related distresses as opposed to load-related distresses was not surprising. Generally, airfield pavements are constructed using lower asphalt-cement contents, as compared to highway pavements, to enhance their load-carrying capabilities. The lower asphalt contents imply that the binder coating around the aggregate in the mixture is thinner than those associated with highway mixtures and consequently, the airfield pavement would potentially be more susceptible to aging. From this brief discussion, one might be led to believe that the answer to the durability issue would be to simply increase the asphalt content of the HMA mixture. Increasing the asphalt content in the HMA pavement would improve the resistance of the pavement to durability type failures, but it would increase the susceptibility of the pavement to load related distresses. The loads associated with airfield pavements are much greater than those associated with highway and interstate pavements; therefore, using a typical highway mixture for an airfield pavement would not be acceptable. Figure 2 provides an illustration of the difference between airfield loadings and highway loadings. The figure illustrates the fact that the load exerted on the pavement by one cargo aircraft is similar to the load exerted by approximately 400 automobiles.

It becomes clear that one of the most important factors in developing an HMA design is determining the optimum asphalt-cement content. Sufficient asphalt cement must be present in the HMA to resist aging (i.e., improve durability) without adversely affecting the permanent deformation resistance (i.e., stability) of the HMA. The total amount of asphalt cement used in an airfield HMA typically ranges from 4 to 6 percent of the total weight of the mixture. As discussed above, the stability and durability of the HMA are related to the amount of asphalt cement or binder in the HMA and slight changes in the asphalt content can greatly affect the final properties and the life expectancy of the HMA. Increasing the asphalt content improves durability but it decreases stability; therefore, other solutions to improve durability are desired.

Just as important as the amount of asphalt cement used in a HMA is the type of asphalt cement used. There are different grades and qualities of asphalt cement derived from various crude sources. Additionally, the manufacturing processes used to produce the asphalt cement also vary. Each one of these asphalt cements will exhibit individualized performance as related to weathering and durability. Adding to the complexity of the selection of an asphalt-cement is that the exhibited differences in performance are also dependent upon the environment to which the asphalt cement is exposed. For example, if one particular asphalt cement is susceptible to cold temperature cracking but is used in an area that does not experience cold temperatures, then the asphalt cement may perform satisfactorily.

Usually there are only a few sources of asphalt cement available to the local pavement engineer or contractor. Shipping costs will typically prohibit the transport of a "good" asphalt cement for a project even if the local asphalt cement is of a poorer quality. Therefore, the pavement industry has a need for methods of economically improving and quantifying the improvement of asphalt cements that are available in a particular region. One potential method of improving the asphalt-cement material would be by the addition of a modifier to the binder.

The discussion of HMA durability (as defined in this research) quickly begins to focus on the asphalt cement. The focus results from the fact that the approximately 60 percent of fatigue related distress failures can be attributed to the properties of the asphalt cement. Additionally, approximately 85 percent of thermal related distress failures are related to the properties of the asphalt cement (Jones 1995). Table 19 provides the estimated effects of asphalt cement on various pavement distresses. The information in Table 19 implies that the majority of fatigue and thermal related distresses in HMAs can be addressed by modification or improvement of the asphalt-cement properties. This information also implies that laboratory testing during this research project can focus on binder testing with less emphasis on mixture testing. Mixture

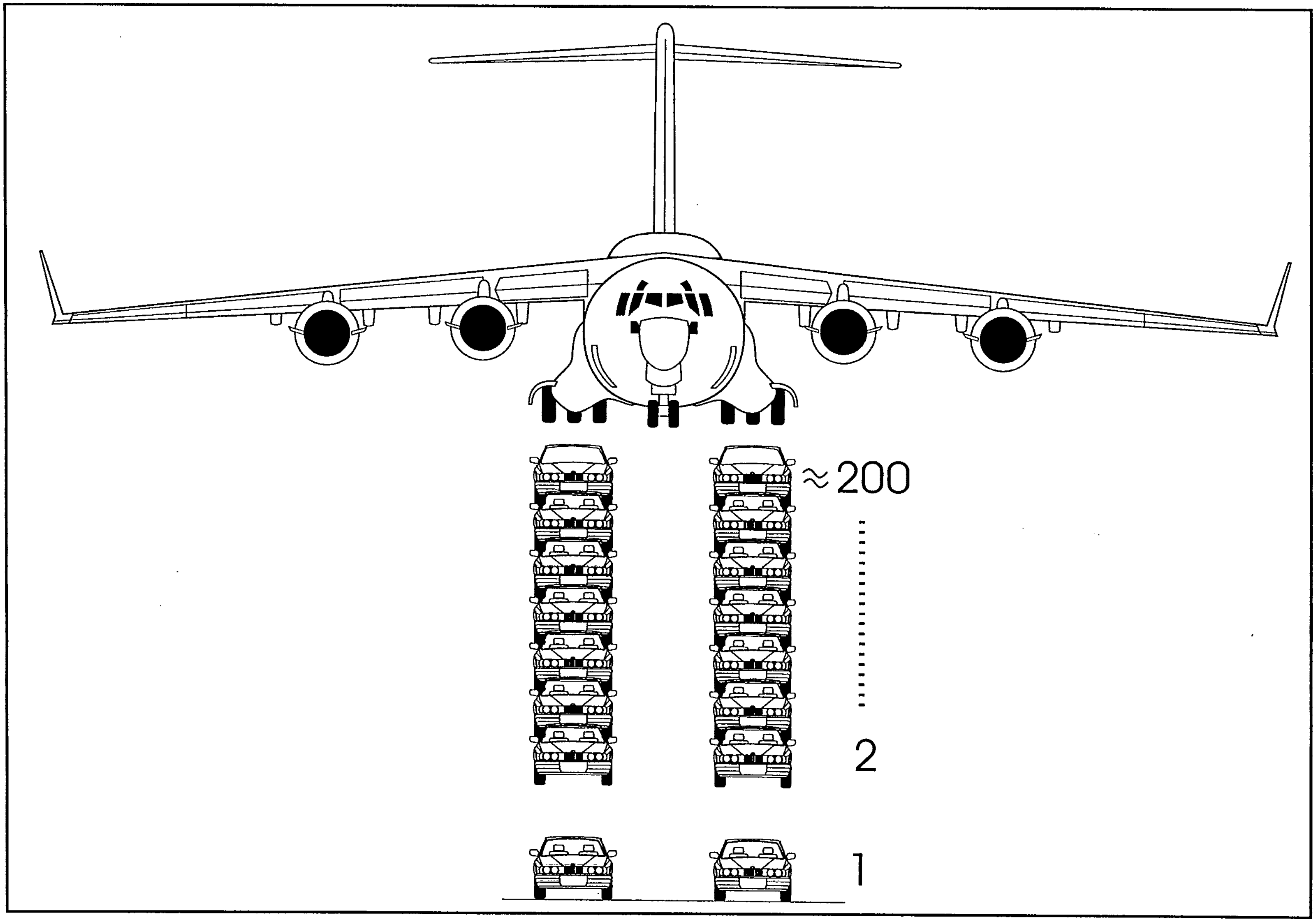

Figure 2. Comparison of airfield loading to highway loading

**Table 19**
**Percentage of Pavement Distresses Related to Asphalt-Cement Properties**

| Type of Pavement Distress | Estimated Effect of Asphalt Cement on Pavement Distress | Estimated Priority | |
|---|---|---|---|
| | | DoD | Highway |
| Rutting | 40 percent | 3 | 1 |
| Fatigue | 60 percent | 2 | 2 |
| Thermal | 85 percent | 1 | 3 |

testing cannot be completely eliminated because the aggregate does have some effect on the thermal and fatigue properties of the HMA.

## Chemical Analysis of Asphalt Cements

The aging process that occurs in most materials will alter the chemical constituents of that material. Asphalt cements generally follow that trend. As a result, several researchers have conducted chemical investigations in an attempt to identify asphalt cements that are more susceptible to the aging process, specific constituents in the asphalt cement that result in increased aging, and methods to retard or mitigate the aging process in those asphalt cements. Additionally, researchers determined that asphalt cements which meet the same specification requirements would often produce HMA pavements that varied considerably in both performance and serviceability (Petersen 1984). This implied that the chemical composition of the asphalt cements varied and that the tests used in the asphalt-cement specifications were not delineating some important material properties.

There are two major factors that are functions of the chemical composition of the asphalt cement that will affect its durability (Petersen 1984):

*a.* The compatibility of the interacting components of the asphalt cement; the elemental/molecular or asphalt fractions that make up a specific asphalt.

*b.* The sensitivity of the asphalt cement to oxidative aging.

A significant amount of research has been conducted in attempting to determine the chemical composition of asphalt cements. Petersen (1984) provided a comprehensive review of much of that research. The available literature suggests that the chemical model required to adequately describe an asphalt cement would be complex.

A brief discussion of the chemical composition of asphalt cements will be presented to provide an indication of the importance of the compatibility of interacting components within the asphalt cement. Elemental analysis of asphalt cements typically reveals the presence of carbon, hydrogen, nitrogen, sulfur, oxygen, vanadium, and nickel in varying percentages and quantities (Plancher, Green, and Petersen 1974). The elemental analysis indicates the type of elements present, but it does not provide any indication concerning the molecular types or structures present in the asphalt cement. Nuclear magnetic resonance (NMR) has been used to provide more specific information concerning how the molecules are structured in the asphalt cement (Petersen 1984). The NMR analysis has indicated that approximately 25 to 35 percent of the carbon present in the asphalt cement is the form of aromatic carbon. The aromatic carbon is generally incorporated into a condensed aromatic ring system containing from one to possibly 10 rings per aromatic moiety. The ring systems may be further associated with saturated naphthenic ring systems and both the aromatic and naphthenic ring systems may have normal and/or branched hydrocarbon side chains attached to them. The carbon associated with naphthenic ring systems typically range from 15 to 30 percent and nonaromatic and nonnaphthenic carbon contents typically range from 35 to 60 percent. Examples outside of these ranges have been found and the possible combinations associated with these molecular structures is astronomical. Additionally, the molecular structures will vary with the crude source and with the process used to produce the asphalt cement. The overall molecular structures of asphalt cements are further complicated by the presence of the heteroatoms such as sulfur, nitrogen, and oxygen that may be incorporated within the ring or nonring systems. Additional complexity occurs when a modifier is added to the asphalt cement. Since the composition of the asphalt cement is so varied, one cannot assume that the addition of a modifier to an asphalt cement from one source will exhibit the same performance when added to an asphalt from a second source.

As a result of the molecular variability in asphalt cement and the differing chemical structures and reactivities (i.e., polarity, etc.) that these molecules can exhibit, chemists have typically not seriously attempted to separate and identify all of those combinations. The researchers have, however, separated the molecules based on chemical functionality (i.e., sulfonates, ketones, etc.). This type of separation has been useful in providing additional characterization of asphalt cements and has assisted in determining how the various chemical functionality groups affect the physical properties of the asphalt cement and how aging alters those functionality groups (Petersen 1984).

The tests typically used to evaluate the chemical properties of asphalt cements are somewhat difficult and time consuming. Additionally, most of these tests have only been conducted on neat or unmodified asphalt cements. Some correlations have been made between specific chemical fractions or changes in chemical fractions and performance but in general, the pavement industry has preferred physical property or material characterization tests over chemical type tests. With this philosophy in mind, the MUMPS program followed a similar research approach to the SHRP in that it was believed to be more desirable to select a material characterization test or set of tests to delineate performance related properties. This philosophy also parallels the current trend of developing performance-related specifications whereby the chemical constituents of the material are left unspecified as long as the desired performance can be obtained.

## Strategic Highway Research Program (SHRP) Binder Review

One of the largest pavement research projects ever conducted was the SHRP. The SHRP was a $150 million project conducted from 1987 to 1993, which focused on four general highway related areas: asphalt pavements, concrete pavements, highway operations, and pavement engineering. One of the reasons the SHRP was initiated was to address growing concerns over the deterioration of the nation's aging infrastructure. In the asphalt pavement area, the SHRP had two major objectives:

*a.* Establishment of empirical relationships between laboratory measured binder properties and laboratory measurements of asphalt mixture performance.

*b.* Establishment of correlations between laboratory measured properties and field performance.

The objectives of the asphalt pavement portion of the SHRP are very similar to the MUMPS objectives. Therefore, it is important to review the findings from the SHRP to determine their applicability to the MUMPS research effort. The major difference between the MUMPS program and the SHRP research is the loading conditions. The SHRP program was directed toward highway pavements and the MUMPS research is directed toward airfield pavements. Because the loading conditions associated with airfield pavements are much more severe in terms of load magnitude and tire pressure, the HMA mixture and pavement designs, and the pavement performance are very different. Therefore, it is expected that some SHRP test procedures (i.e., binder testing procedures) may be applicable to airfield pavements, but the SHRP criteria for those tests, and the HMA mixture designs may not be applicable. The loading severity difference was demonstrated in Figure 2. Additionally, Table 20 illustrates some of the DOD pavement requirements that differ from the highway community.

**Table 20**
**Unique DOD Pavement Requirements**

| Requirement | Selected Examples |
|---|---|
| Unique, heavy and/or abrasive or harsh loads | Cargo aircraft up to 453,600 kg (1-million-lb load)<br>Fighter aircraft (tire pressures in excess of 2.4 MPa (350 psi))<br>Tracked vehicles (heavy loads combined with abrasive action) |
| Low Volume | Unpaved roads and airfields |
| Worldwide requirements | Pavement design criteria capable of producing a satisfactory pavement using local materials at any installation throughout the world. |

The SHRP binder research delineated three tests for binder characterization: dynamic shear rheometry (DSR), bending beam rheometry (BBR), and the direct tension test (DTT). The DSR and BBR tests will be discussed in more detail in Chapter 4; therefore, only a brief discussion of how the results correlate to different failure modes will be included in this section.

The DSR measures rheological properties, specifically the complex dynamic shear modulus or stiffness (G*), and the phase angle ($\delta$). The parameters from the DSR that relate to pavement performance are G*sin$\delta$ and G*/sin$\delta$. The SHRP research indicated that there was a relationship between G*sin$\delta$ and fatigue life. The SHRP research also indicated that G*/sin$\delta$ related to permanent deformation but the correlation was not strong (Leahy, Harrigan, and Von Quintus 1994).

The BBR test was used to measure creep stiffness (S) of the binder at low temperatures. From the measurement of creep stiffness versus logarithm of the loading time, an estimation of the slope of the creep stiffness curve (m) can be made. Both S and m were determined to correlate well to low-temperature cracking of the binder, thus allowing the temperature at which the material would fracture and the ultimate strain at failure to be predicted. Additionally, m was found to relate to fatigue cracking (Leahy, Harrigan, and Von Quintus 1994).

The DTT was used to measure low-temperature failure properties of the binder. The tensile strain at failure determined during the DTT reportedly provides an indication of the binder performance in cold environments. Some problems have been reported with the equipment used to conduct the DTT and there were some potential testing procedural problems. Therefore, this test was not included in the MUMPS research.

Leahy, Harrigan, and Von Quintus (1994) determined that HMA performance is not completely binder controlled even for low-temperature-type failures and that modifier performance is dependent on the original binder being modified. These conclusions were not new but they do reiterate several items important to the MUMPS project. First, HMA pavement performance cannot be predicted solely from binder tests; mixture tests will also be required. Second, they wanted to develop an empirical method to allow direct correlations between observed distresses and binder and mixture properties. The SHRP research also concluded that too many variables were involved to successfully develop an empirical approach within their project constraints. Therefore, a more complex mechanistic approach was developed in which material relationships were used in mechanistic models which would allow the prediction of pavement performance. The objective of the MUMPS research would be to develop a empirically validated mechanistic approach.

One final conclusion of the SHRP research validation process was that there were no specific material properties associated with aging and water sensitivity. The specifications developed by SHRP do include laboratory aging tests but the researchers stressed that "these effects should be evaluated in the asphalt-aggregate mix to be confident of their effects on pavement performance" (Leahy, Harrigan, and Von Quintus 1994).

## Summary

This brief review of HMA durability issues indicates that binder oxidation is the predominant mode of concern when considering aging. Additionally, the review indicates that aging is one of the areas that is most difficult to model because of the complex nature of the asphalt cement and the numerous mechanisms that are involved in the aging process. The fact that no specific physical or chemical properties have been identified as controlling factors for aging and moisture sensitivity further complicated the MUMPS research effort.

The review of the SHRP research indicates that rheological tests provide correlations to low-temperature cracking and fatigue life of asphalt binders, but the correlations were not necessarily valid for modified asphalt binders. Additional research has indicated that the modified asphalt binder performance is highly dependent on the original binder. All of these factors grouped together indicate that:

*a.* The research outlined in the original pavement study of HR 103-516 is greatly needed.

*b.* The quantification of modified binder performance will be dependent on the binder used for modification.

*c.* Both binder and mixture testing will be required for any developed performance-based specifications.

*d.* Field test sections will be required to validate performance models developed for HMA durability.

# 4 Plan of Test

The test plan for the FY 95 portion of the project was divided into two categories: asphalt binder testing and asphalt mixture testing. Additional HMA mixture tests and a modeling effort to quantify improved pavement performance resulting from binder modification was planned for completion in the FY 96 phase of the project.

Two asphalt cements were used: an asphalt cement obtained from an Arkansas refinery which is currently used as the WES laboratory stock asphalt-cement (AC-20a) and an asphalt cement obtained from a West Texas (AC-20b) refinery. The AC-20b material has a history of being sensitive to oxidation. Four modifiers were added to the asphalt cements: a styrene butadiene rubber (SBR), a styrene butadiene block copolymer which reportedly reacts when combined with the asphalt cement to form an RSBS, an LDPE, and a modified ground tire or crumb rubber (MCR). The crumb rubber product was classified as an MCR because the manufacturer indicated that a proprietary block copolymer had been blended in the asphalt cement with the crumb rubber. All of the modified binders were blended by the manufacturer of the specific modifier. The SBR, RSBS, and MCR were blended at a loading rate of 5 percent by total mass of the binder. The LDPE was blended at a loading rate of 5.5 percent by total mass of the binder. A total of 10 binder types, including 2 unmodified binders and 8 modified binders, were used to prepare the HMA mixtures. Mixtures produced with these binders were tested in unaged and aged conditions resulting in a total of 20 mixture types. Table 21 provides a listing of the HMA mixtures tested.

Only one loading rate (i.e., quantity of modifier added to the binder) was selected for the initial phase of the research. The loading rate selected was typically the highest loading rate (the maximum amount of modifier generally used to modify a binder) that would be used for the SBR, RSBS, and LDPE. Additionally, improvements in the low-temperature property improvements are not generally realized until a networking of the polymer occurs in the asphalt cement. This networking typically only occurs at high polymer loadings (approximately 5 percent). Selecting only one loading rate allowed multiple materials to be evaluated for comparative purposes.

**Table 21**
**Hot Mix Asphalt Description**

| Mixture Number | Type of Binder | Conditioning |
|---|---|---|
| 1 | AC-20a | Unaged |
| 2 | AC-20b | Unaged |
| 3 | AC-20a | Aged |
| 4 | AC-20b | Aged |
| 5 | AC-20a + 5% RSBS | Unaged |
| 6 | AC-20a + 5% RSBS | Aged |
| 7 | AC-20b + 5% RSBS | Unaged |
| 8 | AC-20b + 5% RSBS | Aged |
| 9 | AC-20a + 5% SBR | Unaged |
| 10 | AC-20a + 5% SBR | Aged |
| 11 | AC-20b + 5% SBR | Unaged |
| 12 | AC-20b + 5% SBR | Aged |
| 13 | AC-20a + 5.5% LDPE | Unaged |
| 14 | AC-20a + 5.5% LDPE | Aged |
| 15 | AC-20b + 5.5% LDPE | Unaged |
| 16 | AC-20b + 5.5% LDPE | Aged |
| 17 | AC-20a + 5% MCR | Unaged |
| 18 | AC-20a + 5% MCR | Aged |
| 19 | AC-20b + 5% MCR | Unaged |
| 20 | AC-20b + 5% MCR | Aged |

## Binder and Modified Binder Testing

Many of the binder conditioning and test methods employed in this study were developed under the SHRP. The SHRP test methods and practices are now being proposed for adoption by American Association of State Highway Transportation Officials (AASHTO). More detailed descriptions of the test methods, etc., can be found in the March 1995 edition of the AASHTO Provisional Standards (1995a) and Strategic Highway Research Program (SHRP) A-379 (1994a).

### Rolling thin-film oven test (AASHTO T240(1995a))

The Rolling Thin-Film Oven Test (RTFOT) is designed to simulate the aging of the asphalt binder that occurs during the heating, storage, mixing, and transport of the binder and mix prior to compaction. Duplicate asphalt binder samples weighing 35 grams are placed in special glass containers that fit a rack in a specially designed oven. The samples are carefully weighed on an analytical balance before starting the test. The oven is maintained at 163 °C

(325 °F) for 85 minutes, while the samples slowly turn in the oven. This results in a constant film thickness on the walls of the glass container. A small air pump is used to direct a stream of heated air into each bottle as it passes the nozzle with each revolution of the rack. At the completion of the test, the weight loss from the two asphalt-cement samples is determined. This gives a measure of the volatiles lost during the heating. The remaining material is collected for testing and further aging using the pressure aging vessel.

For the SHRP specifications, if the weight loss is greater than 1 percent of the initial weight, the asphalt exhibits a large volatile loss and is considered unsuitable as a paving asphalt.

### Pressure aging vessel (AASHTO PP1-93(1995b))

The method employed for accelerated aging of the asphalt binder to simulate field aging is accomplished using the Pressure Aging Vessel (PAV). The PAV practice was developed under SHRP and simulates between 5 and 10 years of pavement aging depending on the environment into which the binder will be placed. The SHRP procedures were used in this study for PAV aging.

The PAV device consists of a metal cylinder with a removable lid that can withstand routine compressed air pressures of 20.7 atm (300 psi) at temperatures ranging from 90 to 110 °C (194 to 230 °F). A rack designed to hold up to 10 thin-film oven pans fits inside the metal cylinder. Fifty grams of asphalt binder are placed on each pan to yield a film thickness of 6.5 mm. The device is sealed and heated to the aging temperature before charging the vessel to the operating pressure of 20.7 atm (300 psi). The pressure is maintained for 20 hr before it is released slowly to eliminate bubbling as air diffuses out of the aged binder.

### Rotational viscometry (AASHTO TP48-94 (1995c))

Rotational viscometry has been adopted for the analysis of modified binders. Capillary viscometry of polymer-modified asphalts may have problems with shear-thinning and crumb-rubber modified asphalts may clog the tube. The rotational technique obviates these problems. The sample is placed in a container heated to 135 °C (275 °F) and a spindle attached to a torque measuring device (Brookfield viscometer) is lowered into the asphalt. At a given number of revolutions per minute (rpm), the amount of torque required to maintain the specified rpm setting is measured. The torque is proportional to the viscosity of the sample in the absence of wall effects and thermal gradients.

For the SHRP test, the spindle is rotated at 20 rpm and the torque required to maintain that rpm is measured and converted into viscosity. The sample is tested after 20 min at 135 °C (275 °F). A maximum viscosity of 3 Pa/sec is allowed. A viscosity below this value ensures that the material can be pumped at the asphalt plant. If the material does not meet this value, the specification can be waived in lieu of assurances from the binder manufacturer that the material will meet the pumping requirements at the plant.

### Dynamic shear rheometry

DSR measures the complex modulus characteristics of the asphalt binder over a range of applied frequencies and temperatures. The sample is placed between two parallel plates and a sinusoidal oscillatory shear strain is applied at a range of frequencies generally from approximately $10^{-2}$ to 10 Hz at a specific temperature. The torque response of the sample is measured and the phase angle delta ($\delta$) between the applied shear and the response is determined. The magnitude of the complex modulus, $|G^*|$, the sample is determined from the torque amplitude, $\tau$, and the shear strain amplitude, $\gamma$, applied to the sample as measured at the edge of the plate, according to the equation:

$$G^* = \frac{\tau}{\gamma} = \sqrt{(G')^2 + (G'')^2} \tag{1}$$

The elastic, G′, and viscous, G″, components of the complex modulus can then be determined using $\delta$. G′ is the elastic portion of the modulus (also termed the storage modulus) in which the phase angle is zero and G″ is the viscous portion (also termed the loss modulus) in which the phase angle is $\pi/2$ radians or 90°.

$$G' = |G^*| \cos\delta \tag{2}$$

$$G'' = |G^*| \sin\delta \tag{3}$$

The ratio of the viscous to the elastic modulus is also useful.

$$\tan\delta = \frac{G''}{G'} \tag{4}$$

The data output from the test is the magnitude of the complex modulus and phase angle over a range of applied frequencies for a specific temperature. If this type of data are collected over a range of temperatures, a family of curves can be obtained that defines the material response for a narrow frequency range for each temperature. These curves can be combined using the time-temperature superposition principle (Ferry 1970) to generate the so-called "mastercurve." In short, time and temperature are equivalent in the stress-strain history of the sample; thus, a low-frequency load applied to a visco-elastic material at a low temperature is equivalent to a higher-frequency load applied at a higher temperature. Thus, the individual curves may be shifted on the frequency axis relative to a reference curve to generate a single mastercurve that defines the material response over a wider range of frequencies at a specific temperature or more simply, several relatively short experiments at different temperatures can be extrapolated to provide material properties at very long times which would normally take years of testing to develop or at extremely short times at which testing equipment may not be capable of accurately measuring. Mastercurves are useful for comparison of material properties of different materials at a common temperature. The changes in the modulus characteristics for a modified asphalt are generally apparent from a cursory inspection of the curve.

The mastercurves in this report were generated by measuring the DSR response at frequencies of 0.02, 0.04, 0.06, 0.08, 0.1, 0.2, 0.4, 0.6, 0.8, 1.0, 2, 4, 6, 8, and 10 Hz at specific temperatures of -30, -20, -10, 0, 10, 20, 30, 40, 50, 70, and 80 °C (-22, -4, 14, 32, 50, 68, 86, 104, 122, 158, 176 °F). These data were recorded on a Reologica Stresstech controlled stress rheometer equipped with a liquid nitrogen temperature control unit. At temperatures above 30 °C (86 °F) , 25-mm plates were used and below 30 °C (86 °F), 8-mm plates were used. The family of curves was shifted using the IRIS© software to generate the mastercurve at -30 °C (-22 °F) for a reference temperature. The software allows the mastercurve to be displayed at any temperature in the measured range and for the work described in this report the curves are displayed at 30 °C (86 °F). For the SHRP tests, the DSR response at a single frequency of 1.591 Hz (10 rad/sec) was measured at the appropriate temperatures.

The SHRP technology made extensive use of this technique for specification purposes. The specification of $G^*/\sin\delta$ is referred to as the "SHRP rutting parameter" and is a measure of the creep tendency of the binder. The minimum value for $G^*/\sin\delta = 1.0$ kPa for the original or tank asphalt. A $G^*/\sin\delta$ of 2.2 kPa after RTFOT conditioning ensures that enough elastic stiffness is present in the binder to resist permanent deformation immediately after laydown. The maximum value of $G^*\cdot\sin\delta = 5$ MPa after PAV aging provides for enough viscous flow in the sample to resist fracture of the binder due to repeated loadings of the sample.

### Bending beam rheometry (AASHTO TP1-93 (1995b))

The bending beam rheometer (BBR) test is designed to measure stiffness and creep rate of aged asphalt binders at low pavement service temperatures. The test measures the tendency of an asphalt binder toward thermal cracking. An asphalt beam approximately 125 mm long and 12.5 mm wide with a depth of 6.25 mm is subjected to a three-point bending test. The beam is supported at two points 100 mm apart and the sample loaded at the midpoint by a 100-gram static applied load. The applied load, sample deflection, and time of loading are monitored at a specified temperature throughout the test duration of 240 sec. At the completion of the test, the applied load and deflection are used to determine the stiffness, S, of the sample with time. A polynomial curve fit is employed to yield a mathematical representation of the log-log plot for determination of the slope of the curve, m, at specific times of 8, 15, 30, 60, 120, and 240 seconds. The BBR tests in this work were performed on a Cannon bending beam rheometer.

### Fluorescence microscopy

Fluorescence microscopy of the modified asphalts was conducted on an Olympus BX60 fluorescent microscope. Samples were viewed under magnifications ranging from 10 to 50 times. Under an ultraviolet light source, the asphalt absorbs much of the light, appearing as a dark gray or black surface. However, some polymer modifiers that are immiscible or incompatible with the asphalt exhibit phase separated domains that appear as bright areas where the light is reflected. Crumb rubber particles appear as black, nonabsorbing materials due to the large amount of carbon black in the rubber. The analysis of the images is somewhat subjective; however, much information on the morphology of the polymer in the asphalt matrix can be obtained. All observations were made at an ambient temperature of 23 °C (73 °F).

## HMA and Modified HMA Testing

### Volumetric and Marshall design properties

All mixture types were designed using the 75-blow Marshall mix design criteria. The 75-blow Marshall mix design was selected because this is the current design used for DoD airfields. The HMA mixtures were produced using a single source of crushed limestone aggregate and the 10 binders mentioned previously. The crushed limestone aggregate had a maximum size of 19 mm (0.75 in.) and was blended with 8 percent natural sand so that the HMA aggregate gradation was in the middle of the gradation band of the Corps of Engineers Specification for airfield pavements (Department of the Army and Air Force 1991). Figure 3 displays this aggregate gradation curve. The optimum asphalt-cement content was selected to provide 4 percent air voids (voids total mix) using Marshall mix design procedures.

The HMA properties measured during the mix design process included the void parameters, Marshall stability, and flow. The Marshall stability and flow test was conducted in accordance with ASTM D 1559 using a Marshall testing machine equipped with an automatic plotting device for graphing stability curves (ASTM 1994d). The Marshall stability of an HMA provides an indication of the mix strength defined as the resistance to deformation or plastic flow under a load (Headquarters, Departments of the Army and Air Force 1991) or as a measurement of the mass viscosity of an asphalt-aggregate mixture affected by the type, shape, and texture of the aggregate and the stiffness of the asphalt binder (Roberts et al. 1991). The flow value provides an indication of mix plasticity and is a measurement of deformation at failure or the maximum load of the stability test.

Once the optimum asphalt-cement content had been selected, the remaining samples were compacted using the Corps of Engineers gyratory test machine (GTM) which is described later in this report.

### Laboratory aging

One of the major objectives of this research was to quantify improvements in durability of HMA resulting from binder modification. To achieve that objective, it was necessary to age the mixes and determine differences between the aged and unaged materials.

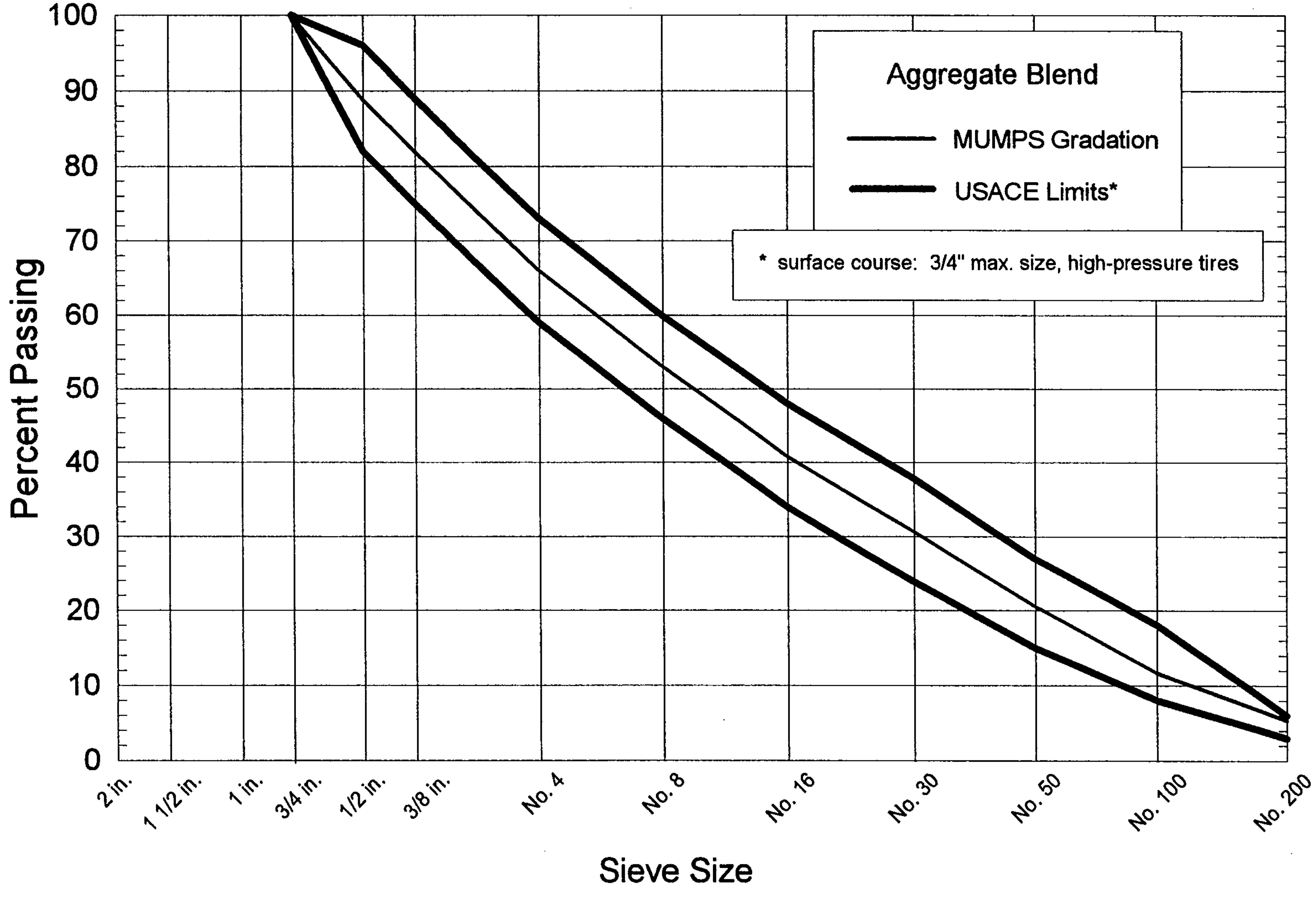

Figure 3. Aggregate gradation used for HMA mixtures

In this study, the conditions used to age the HMA consisted of placing the loose uncompacted mixtures in a forced draft oven set at 149 °C (300 °F) for 22 ± 2 hr. A similar long-term aging procedure had been considered in the SHRP research. However, the SHRP researchers were unable to compact the test specimens to the desired unit weight (Bell et al. 1994). The GTM was capable of compacting the specimens to similar unit weights as the unaged samples; therefore, the accelerated aging technique could be used in this study.

The aging of the uncompacted material using this technique provided for favorable laboratory productivity and allowed the collection of GTM data for comparisons between the unaged material.

### Gyratory testing machine

The GTM (Figure 4) developed by the U.S. Army Corps of Engineers (McRae 1965, McRae and Foster 1959) can be set to compact samples to a target unit weight. This practice can typically provide compacted samples with void contents within a close range of the desired value ( ± 0.3 percent).

Figure 4. Corps of Engineers gyratory machine

Compaction of HMA using the gyratory method applies normal forces to both the top and bottom faces of the material confined in a cylindrically shaped mold. Normal forces at the designated pressures are supplemented with a kneading action or gyratory motion to compact the HMA material into a denser configuration with aggregate particle orientation more consistent with in-place pavements (Ahlrich 1995a).

The gyratory compaction method (ASTM D 3387 (ASTM 1994f)) involves placing HMA into a 101.6-mm- (4-in.-) diam mold, positioning the mold in the GTM, and applying a normal stress to compact the HMA. The stress level used for compaction is selected to represent anticipated traffic contact pressure (ASTM 1994). The compaction pressure used for gyratory compaction in the MUMPS project was 1.4 MPa (200 psi) normal stress with a compaction angle of (1 deg) for the specified number of revolutions of the roller assembly. Thirty revolutions of the roller assembly at 1.4 MPa (200 psi) and 1 deg is equivalent to the standard 75-blow Marshall hand hammer compactive effort on unmodified HMAs (Brown and Basset 1990, Winford 1991, and Headquarters, Departments of the Army and Air Force 1991). The GTM compaction process produces stress-strain properties that are similar to those in field compacted samples (Ahlrich 1995a). A schematic of the gyratory roller assembly used in the compaction process is shown in Figure 5.

The gyratory compaction method using the GTM produces a gyratory graph or gyrograph that can be used to evaluate the HMA behavior, i.e., the relative stability or plastic behavior, during the compaction process. Figure 6 provides examples of typical gyrographs. When an unstable HMA is being compacted in the GTM, the gyrograph spreads or widens (the right-hand graph in Figure 6); when a stable mix is compacted in the GTM, the gyrograph will narrow slightly during the initial revolutions and then remain a constant width during the remaining revolutions or compactive effort (the left-hand graph of Figure 6).

The gyrograph also produces two indices that describe the relative stability of an HMA; the Gyratory Stability Index (GSI) and the Gyratory Elasto-Plastic Index (GEPI). The GSI is the ratio of the maximum width of the gyrograph at the end of compaction to the minimum width of the gyrograph (Figure 6). Numerically, the GSI is represented by the following formula:

$$\text{Gyratory Stability Index (GSI)} = \frac{\theta_{max}}{\theta_{min}} \qquad (5)$$

where

$\theta_{max}$ = the maximum width from the gyrograph

$\theta_{min}$ = the minimum width from the gyrograph

A GSI value greater than 1.0 indicates that the HMA is an unstable plastic mixture (i.e., exhibits a potential for rutting).

The GEPI is the ratio of the minimum width of the gyrograph to the initial width or machine setting (Figure 6). The GEPI is a measure of the shear strain experienced by the mixture during compaction and is an index of the angle of internal friction of the aggregate. Numerically, the GEPI is represented by the following equation:

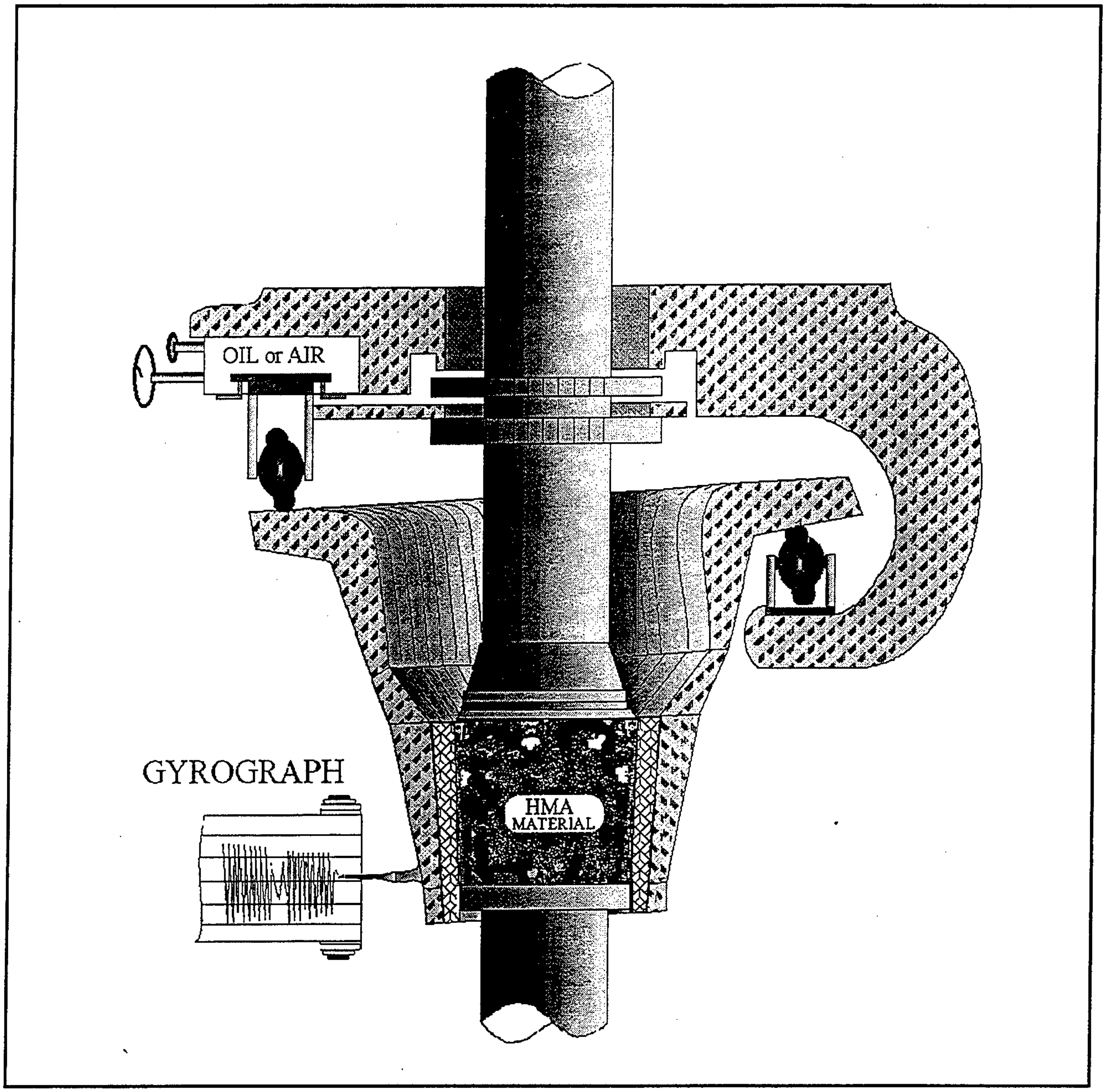

Figure 5. Corps of Engineers gyratory compaction assembly

$$\text{Gyratory Elasto-Plastic Index (GEPI)} = \frac{\theta_{min}}{\theta_0} \qquad (6)$$

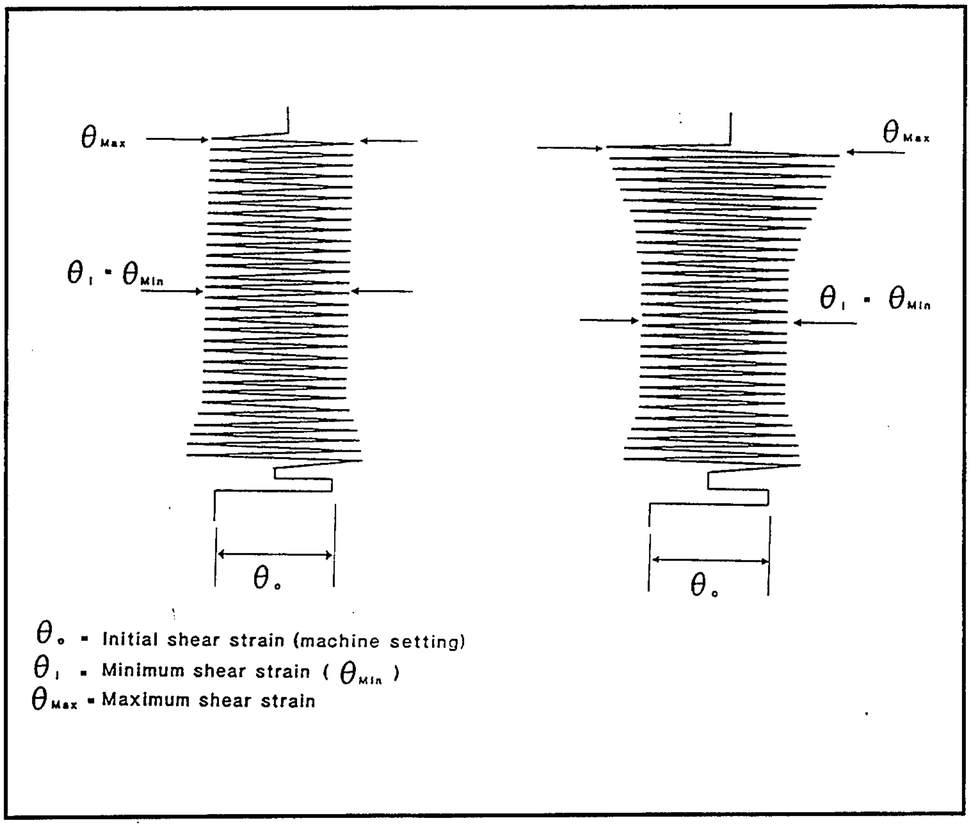

Figure 6. Typical gyrographs produced during HMA compaction

where

$\theta_{min}$ = the minimum width from the gyrograph

$\theta_0$ = the initial width from the machine setting

A GTM setting of 1 deg is used when the 75-blow Marshall mix design criteria is being used; therefore the GEPI is equivalent to $\theta_{min}$. A GEPI value of approximately 1.0 indicates satisfactory internal friction of the aggregate. GEPI values greater than 1.0 indicate that the internal friction of the aggregate is low as would be expected with rounded aggregate materials and thus less resistant to permanent deformation.

One final piece of information that was obtained from the GTM is the gyratory shear strength value. The gyratory shear strength value is related to the shear strength of the compacted specimen and is determined from the static roller pressure readings. The gyratory shear strength value can assist in the delineation of good HMA and poor HMA, with higher values being more favorable than lower values (Von Quintos, Scherocman, Hughes, and Kennedy 1991, Ruth, Shen, and Wang 1992). However, research conducted at WES indicates that the gyratory strength value should not be used without additional information such as the GSI and GEPI (Ahlrich 1995a).

### Indirect tensile testing

The indirect tensile test was developed to estimate the tensile strength of a material by placing a cylindrical sample of material horizontally between two loading strips. The specimen is loaded across its diameter until failure occurs. Figure 7 illustrates the indirect tensile test loading configuration. This loading configuration subjects the centerplane of the specimen between the two loading strips to a nearly uniform tensile stress resulting in failure of the material. This test procedure has been used to test and evaluate soils, concrete, and HMA (Al-Hussani and Townsend 1973).

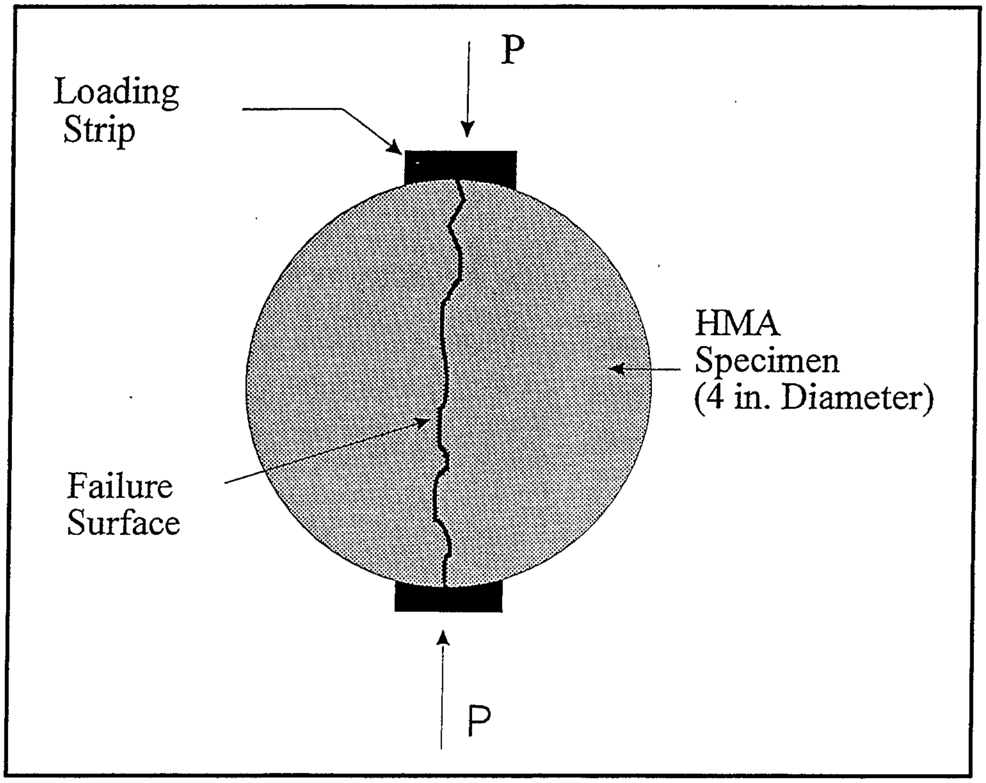

Figure 7. Schematic of indirect tensile testing

The indirect tensile test is commonly used to help determine the permanent deformation resistance of HMA. However, it has been determined that for thick HMA (i.e., those greater than 127 mm (5 in.)), that higher stiffness values at lower temperatures are indicative of greater fatigue resistance (Roberts et al. 1991). Therefore, the indirect tensile test data can provide an indication of improved fatigue resistance as a result of binder modification.

ASTM Method D 4123 provides guidance on indirect tensile testing of HMAs (ASTM 1994g). The indirect testing for the MUMPS project was conducted at two test temperatures; 25 °C (77 °F) and 40 °C (104 °F). The specimens were conditioned at the test temperature for a minimum of 2 hr before testing to ensure uniform specimen temperature. The vertical load was applied to produce a constant deformation rate of 50.8 mm (2 in.) per minute until failure. The ultimate load was recorded at failure and was used to estimate the tensile strength in accordance with ASTM D 4123 (ASTM 1994g). The equation used to estimate the tensile strength was:

$$TS = \frac{2P}{\pi t D} \tag{7}$$

where

TS = tensile strength, psi

P = ultimate load required to fail the specimen, lb

t = thickness of the specimen, in.

D = diameter of specimen, in.

The energy required to fracture specimens during the indirect tension test is related to the area under the load-deformation curve. Since the load-deformation curve is typically symmetrical and triangular, the area under the curve can be estimated by multiplying the peak load by the deformation at peak load. For the purpose of HMA mixture comparison, the approximate area under the load-deformation curve was designated the indirect tension toughness (ITT):

$$ITT = Pd \tag{8}$$

where

ITT = indirect tension toughness (lb-in.)

P = ultimate load (lb)

d = vertical deformation at ultimate load (in.)

The indirect testing was conducted on a minimum of three specimens for each of the aged and unaged mixtures.

### Confined repeated load-deformation testing

The additional aged and unaged samples were tested using a confined repeated load deformation or triaxial cyclic creep test. This test is traditionally used to evaluate the rutting potential of HMA and to determine the effectiveness of asphalt modification as a means of improving the rutting characteristics of the mix. A minimum of three samples of each mixture were tested to obtain a representative characterization of the mixture.

The test equipment used to conduct the confined repeated load-deformation test was developed by WES and was based on recent work conducted by the NCAT at Auburn University. The NCAT research indicated that the confined repeated load-deformation test provided a better laboratory indication of rutting than static creep tests (Mallick, Ahlrich, and Brown 1994, Gabrielson 1992, and Brown and Foo 1994).

The confined repeated load-deformation tests were performed on individual Marshall specimens that were 63.5 mm (2.5 in.) thick and 101.5 mm (4 in.) in diameter. The specimens were placed in the triaxial chamber with smooth, dense-graded paper on each platen. A rubber membrane was placed around the specimens to allow the application of a uniform confining pressure. The triaxial chamber was placed into an environmental chamber which was maintained at a temperature of 60 °C (140 °F). The 60 °C (140 °F) temperature was selected to be representative of the maximum temperature to which most pavements would be exposed. The specimen/triaxial chamber was conditioned in the environmental chamber for a minimum of 2.5 hr prior to testing to allow the specimen to equilibrate at the test temperature. Once the specimen had equilibrated at the test temperature, the triaxial chamber was pressurized with a confining pressure of 276 KPa (40 psi) for 5 min. Each specimen was then preconditioned using a 10-KPa (1.5-psi) preload followed by 30 cycles of a 69-KPa (10-psi) cyclic stress. The cyclic or repeated load was applied with a 0.1-sec load application and a 0.9-sec rest period.

Once the specimens had been preconditioned, a 1.4-MPa (200-psi) deviator stress cyclic load was applied to the specimen for 60 min. The deviator stress was then released for 15 min to allow the specimen to rebound. The test conditions on the specimens are illustrated in Figure 8. The deformations and loads were recorded at various times during the both the creep (cyclic loading) and rebound phases of the test. These measurements were used to calculate creep modulus (stiffness) and permanent strain values.

The results of the confined repeated load-deformation test can be used in several ways to evaluate HMA. The amount of strain after the creep and rebound phases of the test provides an indication of the HMA potential for permanent deformation. Smaller axial strains and lower creep values indicate that the HMA is more stable than a mix with larger values. An indication of the HMA stiffness can be determined from the creep modulus values. High creep modulus values generally indicate an HMA that has a minimum potential for permanent deformation. The creep modulus value or stiffness is calculated by:

$$CM = \frac{\sigma_D}{\frac{\Delta H}{H}} \tag{9}$$

where

CM = creep modulus value, psi

$\sigma_D$ = deviator stress (200 psi)

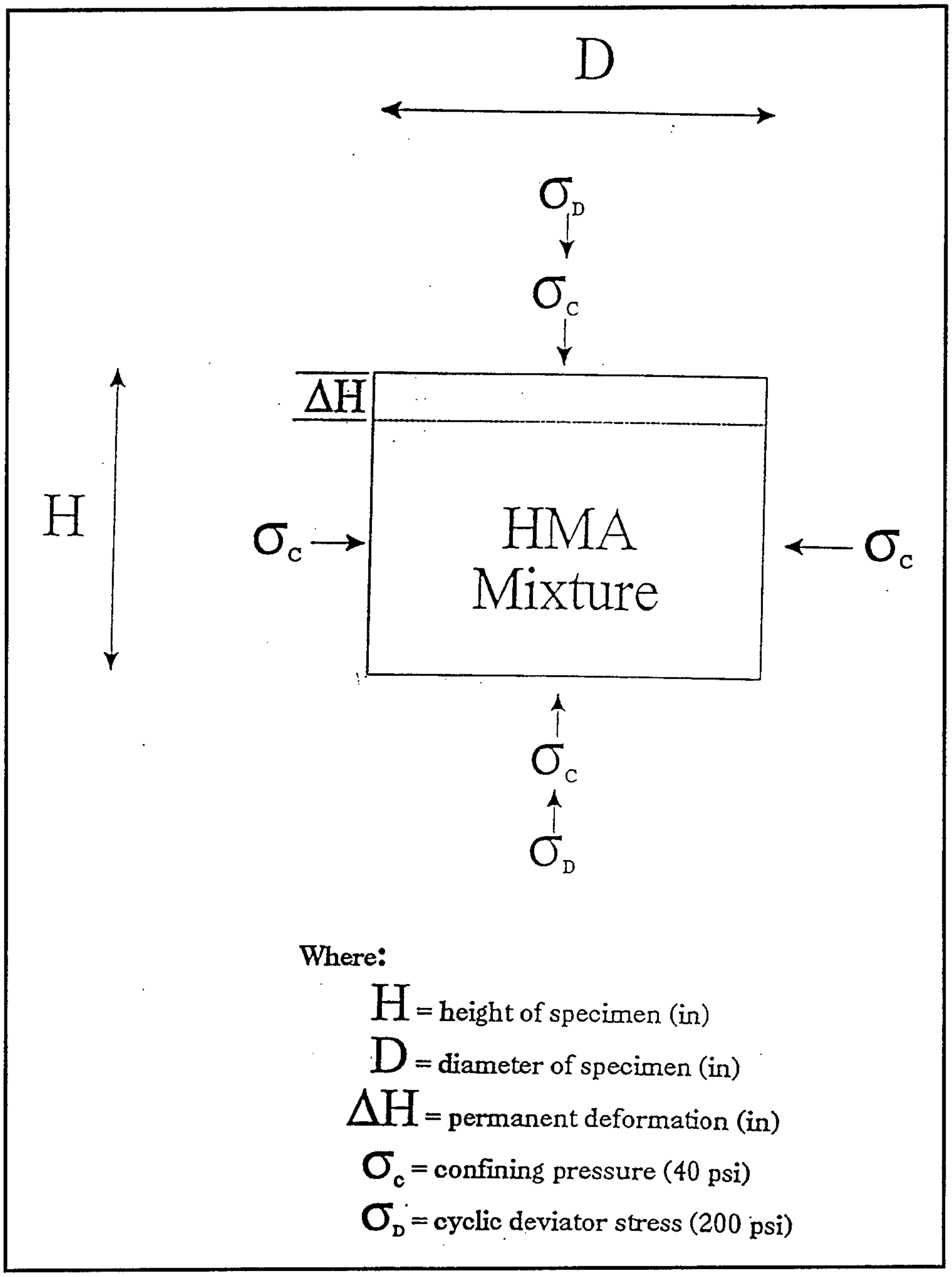

Figure 8. Schematic of confined repeated load-deformation test

H = initial height of the specimen, inches

$\Delta H$ = axial deformation, inches

One final test result from the permanent creep test used to evaluate an HMA is the slope of the log cumulative strain versus log time curve. A typical curve is shown in Figure 9. HMA that exhibit higher slope values have a greater

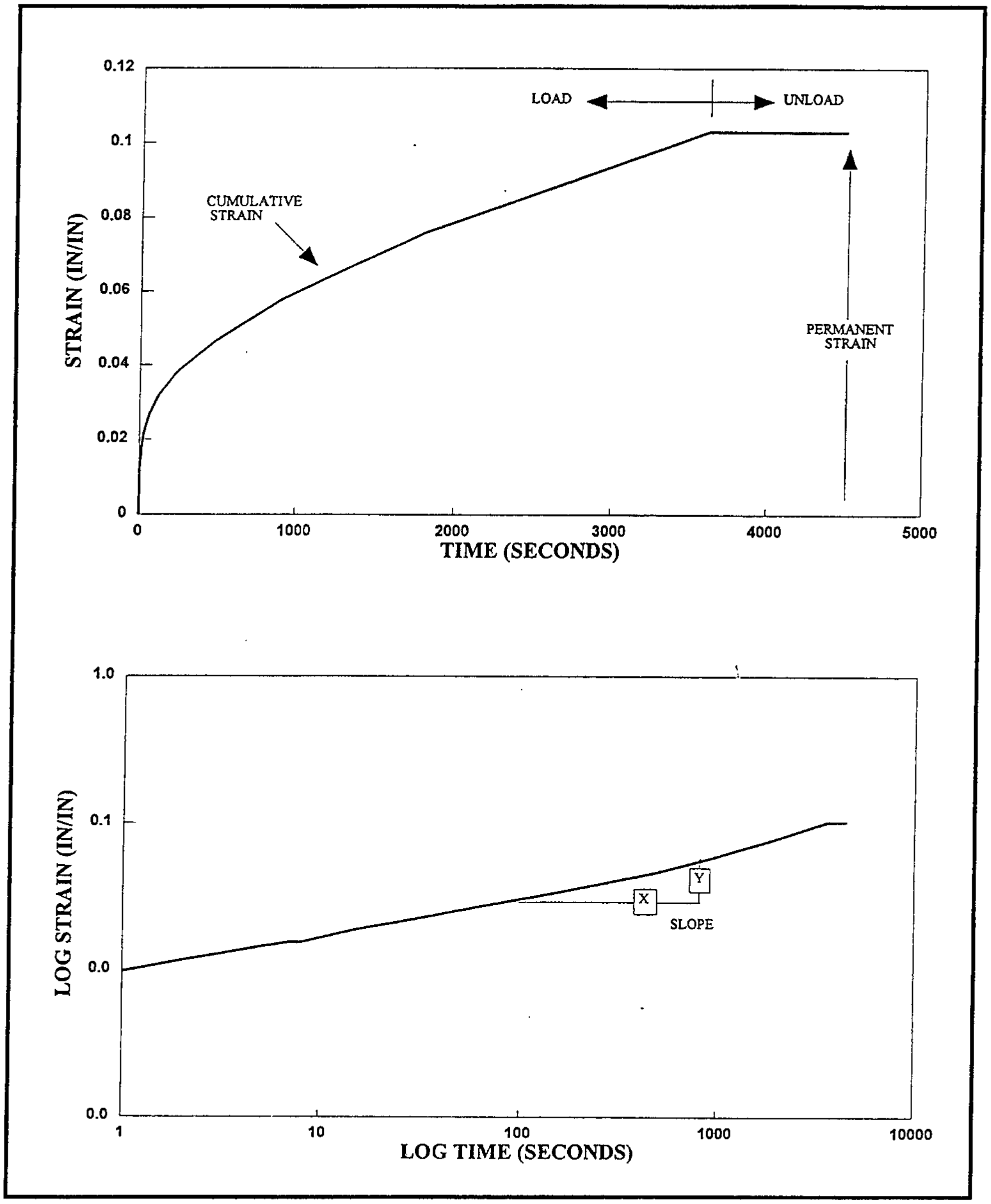

Figure 9. Typical curves obtained from confined repeated load test

potential for rutting than HMA that have lower slope values. The slope of the creep curve is defined as:

$$M = \frac{\log E_{T2} - \log E_{T1}}{\log T_2 - \log T_1} \tag{10}$$

where

M = slope of log - log creep curve

$E_{T2}$ = cumulative strain at 3,600 sec

$E_{T1}$ = cumulative strain at 900 sec

$T_1$ = 900 sec

$T_2$ = 3,600 sec

As previously discussed, permanent deformation is not the major concern in airfield pavements. However, it has been shown that for thick HMA (i.e., those greater than 127 mm (5 in.) thick), higher stiffness values at lower temperatures are indicative of a longer fatigue life. For thinner pavements (i.e., those less than 127 mm (5 in.) thick), higher stiffness at low temperatures are indicative of pavements that are more susceptible to fatigue cracking (Roberts et al. 1991). It has not been established if these relationships are valid for modified HMA, but these data provide for a more complete characterization of the mix. Therefore, changes in stiffness between unaged and aged specimens can provide an indication of the relative susceptibilities of polymer modified HMA to durability problems.

# 5 Data Presentation, Analysis, and Discussion

The laboratory test plan was developed to determine the influence of binder modification on HMA performance, specifically durability-related performance or changes in performance after aging. The laboratory study was conducted in four phases: binder characterization, modified binder characterization, HMA characterization, and modified HMA characterization.

## Binder and Modified Binder Characterization

### Results and discussion

Testing modified binders provides some unique challenges for those who are familiar with only plain asphalt cements. The polymer-modified binders need higher temperatures and longer heating durations in order to properly prepare test samples. Once they are heated, the polymer-modified binders must be stirred in order to prevent separation of the polymer from the base asphalt cement. The LDPE polymer in this study was particularly prone to phase separation.

No attempt was made to classify any of the modifiers as superior. Only the binder properties were compared. Any ranking of the properties of these materials is valid for that particular asphalt only. The data presented here will clearly demonstrate that different modifiers affect different asphalts in dissimilar fashion. This is to be expected since the chemistry of the asphalts differ due to crude source, although rheological properties may be similar.

**Conventional tests.** Solubilities for most of the binders in this study were above 99 percent. The two exceptions both included LDPE: one with AC-20a base asphalt cement and one with AC-20b asphalt cement. Since the average solubility for these LDPE-modified binders was approximately 95 percent, it was evident that the LDPE was not soluble in trichloroethylene. Another solvent would have to be selected in order to be able to perform the necessary quality control tests on this binder.

Polymer modification did not seem to impose any fire-related safety problems for the binders. The binder flash points were not significantly affected by the addition of polymers. All flash points were within the 290 °C (554 °F) to 350 °C (662 °F) range. The flash point becomes a more critical aspect when considering modified asphalt binders because the temperatures used to produce the modified HMA are generally higher than those used to produce conventional HMA.

Polymer modification did not have significant effects on specific gravity. This would be expected with the polymer only accounting for 5 to 5.5 percent of the mass of the binder. Specific gravities for all binders ranged from 1.019 to 1.043.

The capillary tube viscometer could not be used to accurately measure the viscosity of polymer-modified binders at elevated temperatures. Many polymer-modified binders fall outside of the range of viscosities for which the tubes were designed. Secondly, globules or crumbs of the polymer tend to stick to the inside surfaces of the glass viscometer tube, thus preventing smooth flow. Finally, some polymers are very difficult to remove from viscosity tubes. As an alternative to the tube viscometer, a rotational viscometer was used to ascertain the pumpability of the polymer-modified binders. These results are presented with the SHRP binder test results.

Polymer-modification had variable effects on binder ductility, as measured at 25 °C (77 °F) and shown in Figures 10 and 11. The LDPE polymer decreased ductility most significantly, particularly after thin-film oven aging. When SBR was used as the polymer modifier, the binder remained ductile both before and after thin-film oven aging.

Penetration tests were performed at both 25 °C (77 °F) and 4 °C (39 °F). This was the only conventional binder test conducted at low temperatures. At room temperature, both polymer-modification and thin-film oven aging decreased penetration, as shown in Figures 12 and 13. The LDPE decreased penetration most significantly, while SBR and MCR had the smallest effects. At 4 °C (39 °F), the effects of both polymers and aging were less significant and less consistent, as shown in Figures 14 and 15. The low-temperature properties of the polymer-modified binders appeared to be controlled by the base asphalt which supports a well-publicized premise. The penetrations of the binders containing AC-20a were generally higher than the penetrations of the binders containing AC-20b. Relative to the AC-20a binder, the AC-20b binder appeared to be more susceptible to brittleness at low temperatures.

The effects of the modification on softening points were not significant, as shown in Figures 16 and 17. Relative to both unaged and aged plain asphalt cements, the modification generally increased softening point slightly. More importantly, a difference in aging tendencies between the two base asphalts was evident. Aging consistently increased the softening point for binders containing AC-20b. The effects of aging were less obvious for the binders containing AC-20a.

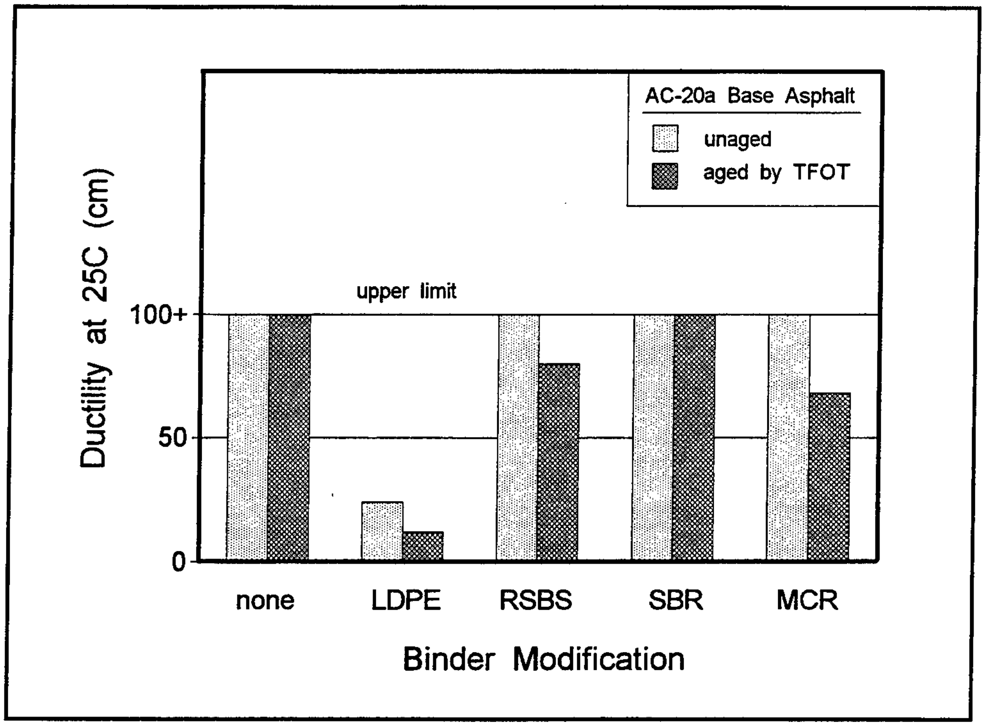

Figure 10. Changes in ductility resulting from modification of AC-20a

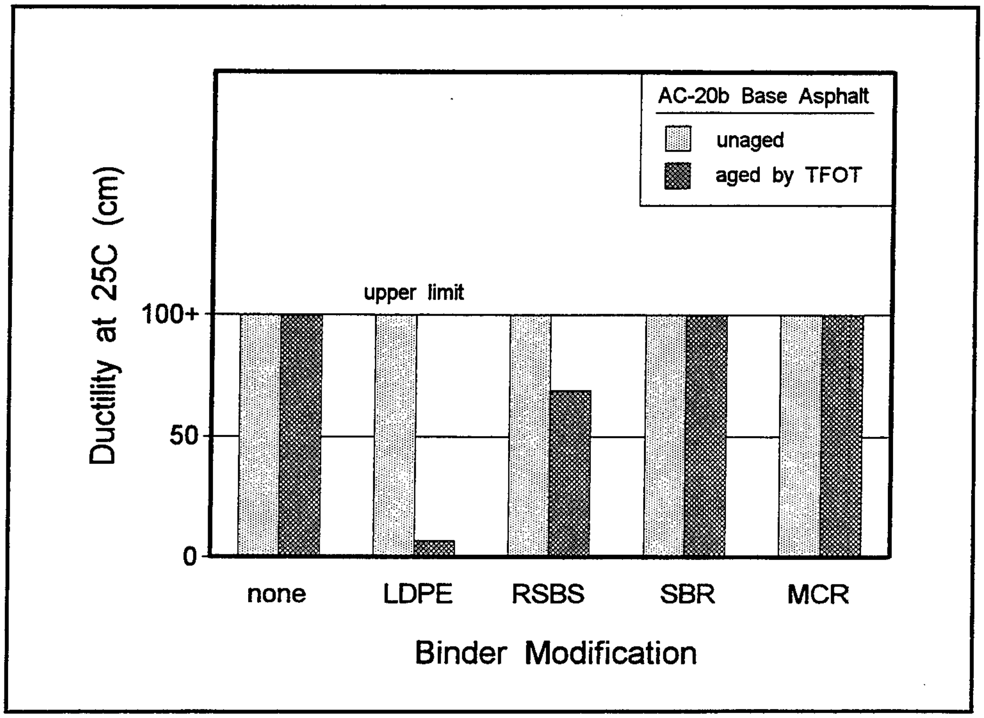

Figure 11. Changes in ductility resulting from modification of AC-20b

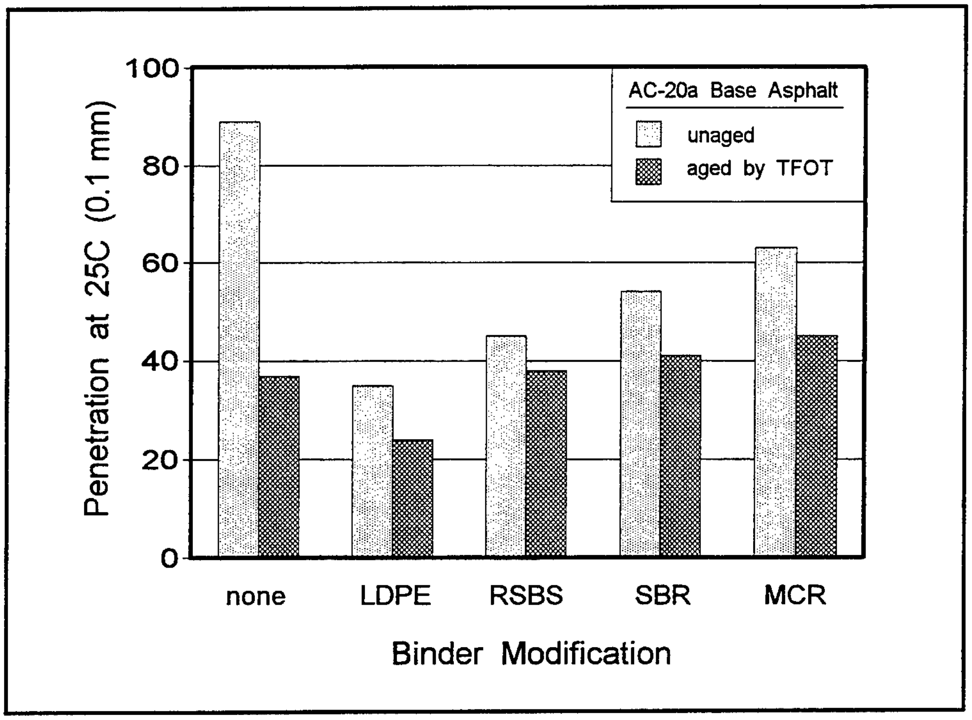

Figure 12. Changes in penetration at 25 °C resulting from modification of AC-20a

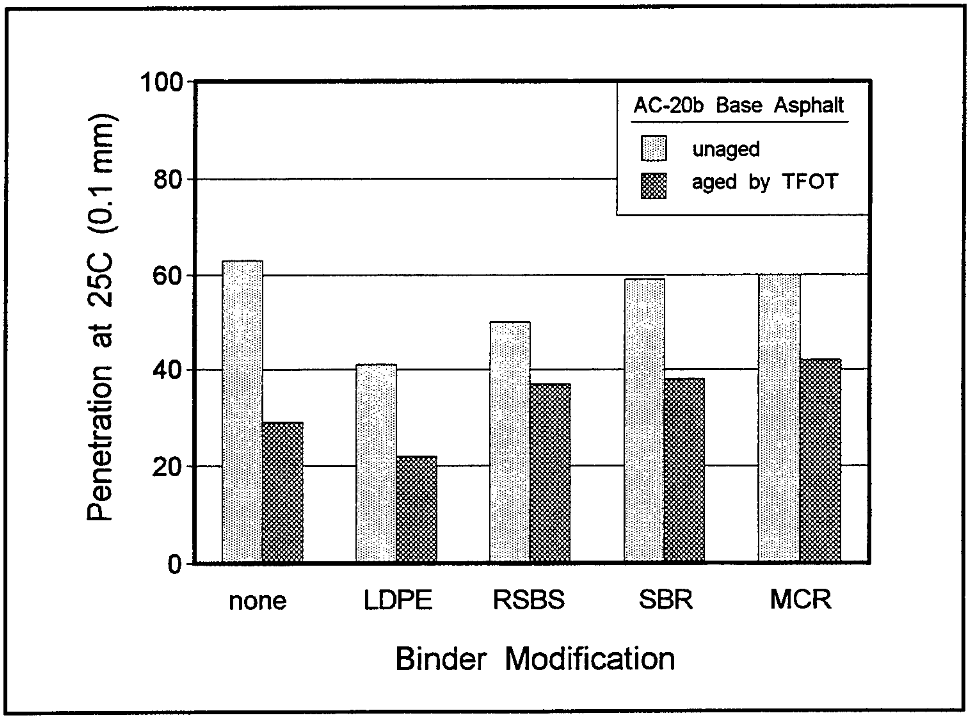

Figure 13. Changes in penetration at 25 °C resulting from modification of AC-20b

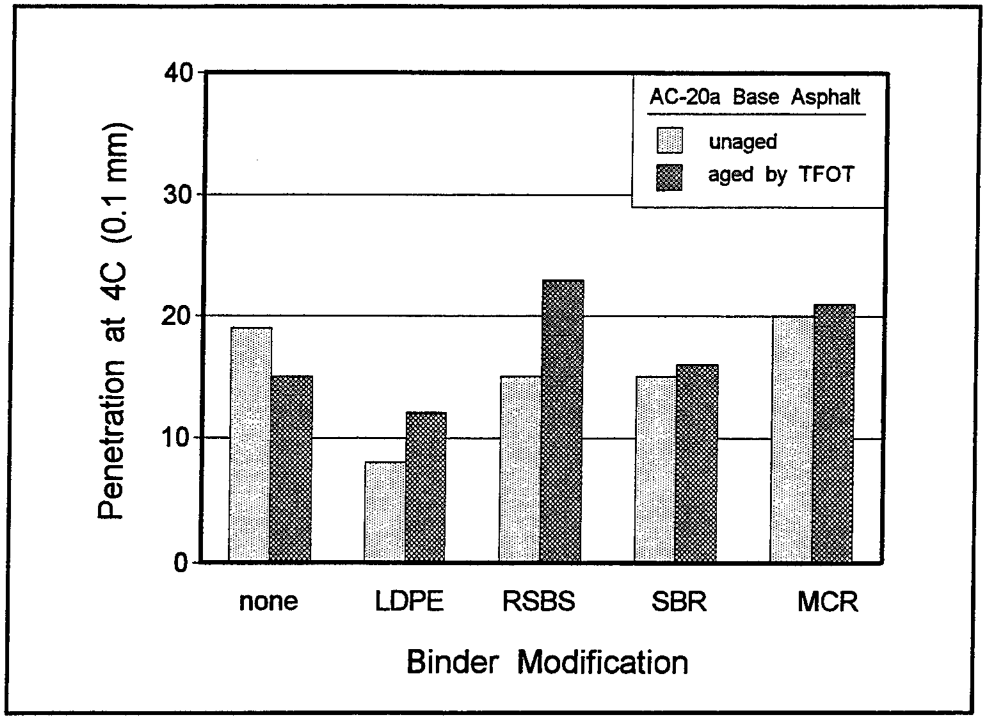

Figure 14. Changes in penetration at 4 °C resulting from modification of AC-20a

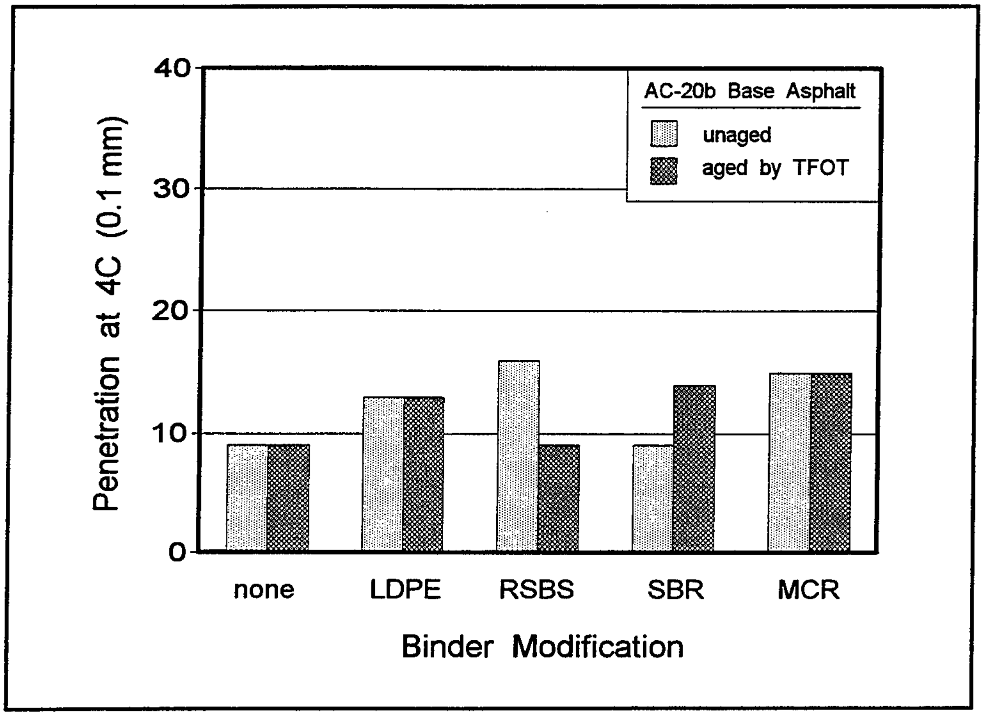

Figure 15. Changes in penetration at 4 °C resulting from modification of AC-20b

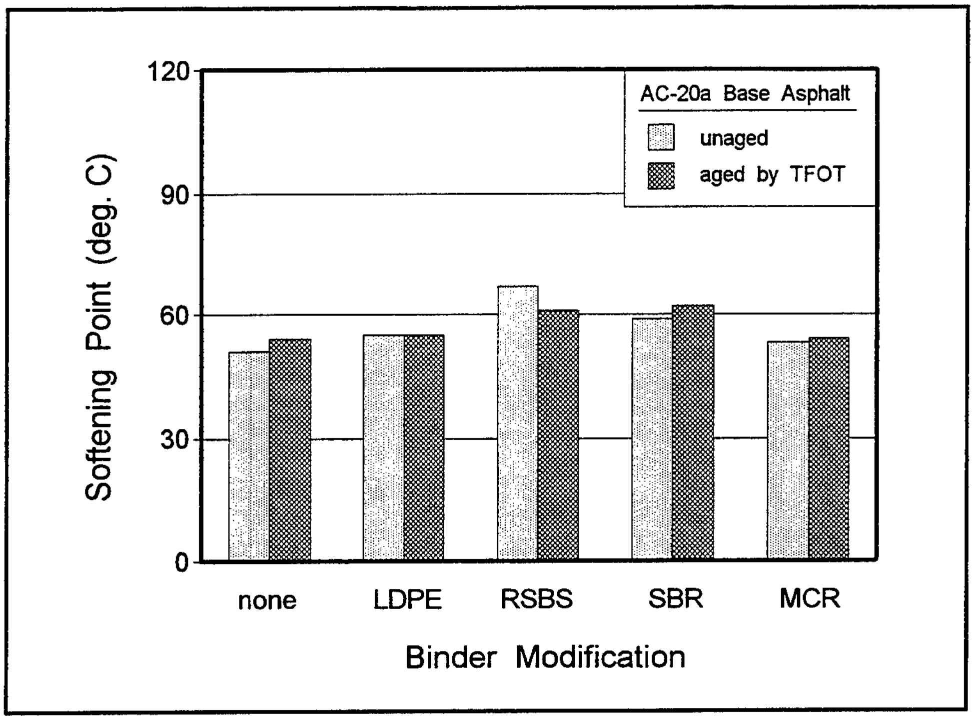

Figure 16. Changes in the softening point resulting from modification of AC-20a

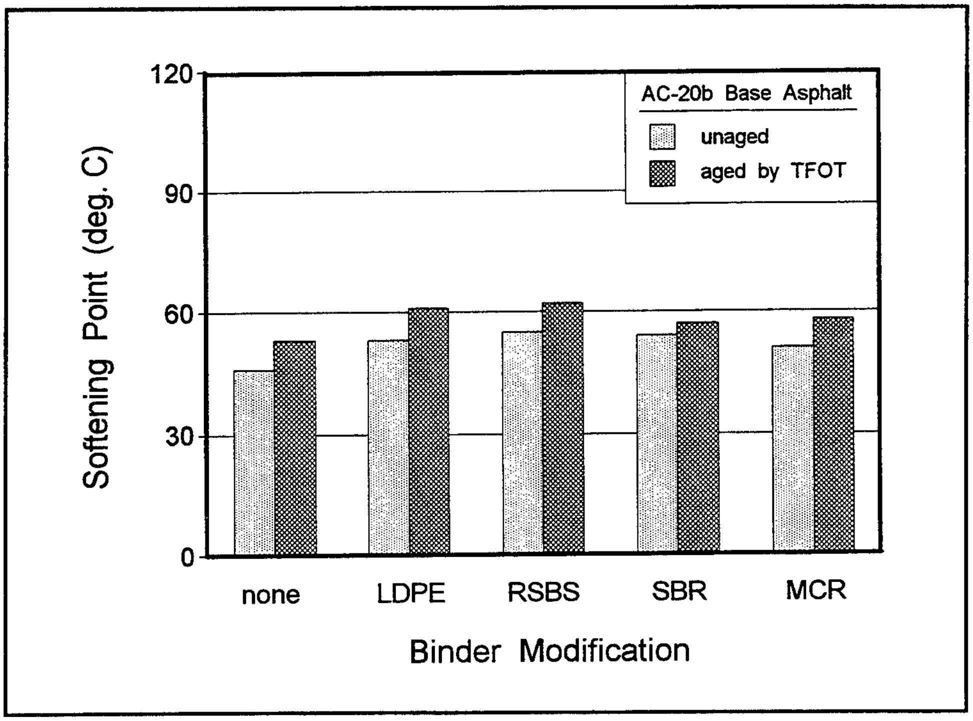

Figure 17. Changes in the softening point resulting from modification of AC-20b

**SHRP performance grading.** Table 22 lists the SHRP performance grades (PG) for the binders used in this study. During the SHRP project, these grades were determined by comparison of laboratory binder and mixture data to data collected from samples derived from highway test sections. Thus, these specifications are related to field performance. From a durability standpoint, the upper temperature limit reflects the asphalt binders ability to resist rutting tendencies while the low temperature limit reflects both the fatigue and thermal cracking potential of the binder. An increase in the upper temperature and decrease in the lower limit would result in an overall improvement in binder durability, as defined by the SHRP research. All of the modifiers used in this study improved the high-temperature properties as evidenced by the increase in the PG at the upper temperature limit, with the LDPE modifier having the largest effect. Addition of any of these modifiers to the asphalt studied here will, therefore, improve the initial durability of an asphalt pavement by reducing the rutting tendency of the pavement, as defined by the SHRP criteria.

**Table 22**
**SHRP Performance Grade of Binders and Modified Binders**

| Type of Binder | SHRP PG Grade |
|---|---|
| AC-20a | 58-22 |
| AC-20b | 58-16 |
| AC-20a + 5% SBR (A-SBR) | 64-28 |
| AC-20b + 5% SBR (B-SBR) | 64-22 |
| AC-20a + 5% RSBS (A-RSBS) | 70-28 |
| AC-20b+ 5% RSBS (B-RSBS) | 64-28 |
| AC-20a + 5.5% LDPE (A-LDPE) | 82-22 |
| AC-20b + 5.5% LDPE (B-LDPE) | 82-16 |
| AC-20a + MCR (A-MCR) | 64-28 |
| AC-20b + MCR (B-MCR) | 64-28 |

The RSBS, SBR, and MCR modifiers also improved the low-temperature grades of the base asphalts, but the LDPE modifier had no effect on the SHRP performance grading. Thus, the choice of modifier may have a significant effect on the properties of the base asphalt at low and high pavement service temperatures.

The limiting temperatures for the SHRP tests are presented in Table 23. Both of the asphalts are traditional AC-20 materials and display an upper PG of 58 °C (136 °F). After aging, asphalt AC-20b exhibited a value for $G^*/\sin\delta > 5.0$ MPa at 25 °C (77 °F) , limiting the low-temperature grade to -16 °C (3 °F), although the material passed the BBR tests at -12 °C (10 °F). Addition of 5 percent SBR extended the PG of the base asphalt at the upper- and lower-temperature limits. The upper limit of the tank asphalts were raised by two grades for AC-20a and one grade for AC-20b. However, A-SBR exhibited $G^*/\sin\delta < 2.2$ kPa at 70 °C for the RTFOT-conditioned material, limiting

**Table 23**
**Defining Temperatures (in Celsius) for the SHRP Binder Tests**

| Sample | Original[1] | RTFOT[2] | PAV[3] | BBR, S[4] | BBR, m[5] | PG |
|---|---|---|---|---|---|---|
| AC-20a | 63.9 | 65.1 | 20.0 | -15.9 | -16.9 | 58-22 |
| AC-20b | 63.5 | 62.2 | 26.3 | -13.8 | -15.8 | 58-16 |
| A-SBR | 75.8 | 69.9 | 14.8 | -19.9 | -18.5 | 64-28 |
| B-SBR | 70.2 | 66.9 | 19.5 | -15.8 | -17.1 | 64-22 |
| A-RSBS | 76.6 | 70.7 | 17.6 | -18.3 | -22.1 | 70-28 |
| B-RSBS | 71.7 | 69.7 | 18.1 | -15.2 | -18.2 | 64-28 |
| A-LDPE | 82.7 | 83.6 | 25.5 | -13.5 | -15.0 | 82-22 |
| B-LDPE | 85.6 | 82.7 | 28.1 | -14.2 | -11.6 | 82-16 |
| A-MCR | 70.3 | 68.4 | 15.1 | -18.8 | -20.7 | 64-28 |
| B-MCR | 71.0 | 68.3 | 19.7 | -18.0 | -18.9 | 64-28 |

[1] Maximum specification temperature at which $G^*/\sin\delta > 1.0$ kPa.
[2] Maximum specification temperature at which $G^*/\sin\delta > 2.2$ kPa.
[3] Maximum specification temperature at which $G^*\sin\delta < 5.0$ MPa.
[4] Minimum specification temperature at which $S < 300$ MPa.
[5] Minimum specification temperature at which $m > 0.3$.

the upper PG to 64 °C. The PAV-aged SBR binders showed reduced limiting temperatures for $G^*\sin\delta > 5.0$ MPa. For B-SBR, this limit was reduced from 25 °C to 16 °C, allowing the material to reach a lower PG of -22 °C, even though the BBR properties were not reduced sufficiently compared to the original asphalt to move into a lower PG bracket.

For the RSBS binders, the PG = s of both asphalts is extended at the high- and low-temperature limits. The original A-RSBS exhibits $G^*/\sin\delta > 1$ kPa at 76 °C, but the value for the RTFOT residue is less than 2.2 kPa at 76 °C, limiting the upper grade to 70 °C. Similarly, the tank B-RSBS demonstrates $G^*/\sin\delta > 1$ kPa at 70 °C, but the RTFOT material fails, limiting the PG to 64 °C. Both RSBS binders reduced the temperature at which $G^*\sin\delta >$ 5.0 MPa for the PAV residue. The B-RSBS binder displayed a BBR stiffness value, $S$ , greater than 300 MPa at -18 °C with a slope value, $m$, less than the 0.3 requirement. The SHRP protocols require the material to be tested in direct tension under these circumstances; however, because a direct tension device was not immediately available, the material was given the benefit and graded as passing at that temperature.

The LDPE modifier has a profound effect on the upper-temperature PG for both asphalts. These materials displayed an upper PG at the limit of the SHRP specification of 82 °C for $G^*/\sin\delta > 1$ kPa for the tank material and $G^*/\sin\delta$ $> 2.2$ kPa for the RTFOT residue. However, the temperature for passing the $G^*\sin\delta$ criteria was increased from 22 °C for AC-20a to 28 °C for A-LDPE and 25 °C for AC-20b to 31 °C for B-LDPE. The LDPE modifier also has little effect on the low-temperature properties for either binder, resulting in no changes in the PG compared to the unmodified asphalts. For B-LDPE, the value of *m* > 0.3 was not achieved at -6 °C and combined with the temperature limit of 31 °C for passing G*sin δ < 5.0 MPa, this material is limited to -16 °C for a low-temperature grade.

The MCR modified binders exhibited increases in both the upper and lower grades compared to the original binders. For A-MCR, the tank binder passes the 1.0 kPa limit at 70 °C, but the RTFOT material is less than 2.2 kPa at that temperature, limiting the upper PG to 64 °C. The B-MCR binder passed the criteria for the upper PG at 64 °C. The temperature limits for the PAV-aged material were reduced over those of the unmodified asphalts. Both *S* and *m* were improved for A-MCR to yield a lower PG of -28 °C. The *S* value was less than 300 MPa but the *m* value was less than 0.3 at -18 °C for B-MCR, limiting the lower PG to -22 °C.

**Brookfield viscosity.** The results of the rotational viscosity testing using the Brookfield device are presented below in Table 24. All of the binders passed the SHRP requirement of less than 3.0 Pa sec except for B-SBR.

**Table 24**
**Rotational Viscosities at 135 °C and 20 RPM**

| Type of Binder | Viscosity, Pa sec at 135 °C |
|---|---|
| AC-20a | 0.425 |
| AC-20b | 0.312 |
| A-SBR | 2.2 |
| B-SBR | 3.27 |
| A-RSBS | 1.4 |
| B-RSBS | 1.64 |
| A-LDPE | 2.38 |
| B-LDPE | 0.83 |
| A-MCR | 1.68 |
| B-MCR | 1.63 |

**Rolling thin-film oven test.** The weight loss from each of the binders during RTFO conditioning is presented in Table 25. All of the binders demonstrated less than 1 percent weight loss.

**Bending beam rheometry.** Figures 18 and 19 display the relative BBR stiffness and slope, respectively, for the PAV-aged binders. All values are relative to the base PAV-aged asphalt at that temperature. These results show that, in general, the modifiers were effective at reducing the cold temperature stiffness of the base asphalt, except for A-LDPE. The relative reduction in stiffness was most notable at -18 and -24 °C with the largest reductions observed in SBR, RSBS, and MCR modified binders. In Figure 19, the relative changes in the BBR slope after aging of the materials are favorable for the

## Table 25
## Rolling Thin-Film Oven Test Results

| Type of Binder | RTFO Weight Loss, percent |
|---|---|
| AC-20a | 0.40 |
| AC-20b | 0.42 |
| A-SBR | 0.60 |
| B-SBR | 0.58 |
| A-RSBS | 0.40 |
| B-RSBS | 0.28 |
| A-LDPE | 0.43 |
| B-LDPE | 0.40 |
| A-MCR | 0.49 |
| B-MCR | 0.45 |

RSBS and MCR modified binder systems. The results vary for SBR binders, but in general, the presence of SBR resulted in slight increases in the creep rate for aged AC-20b asphalt and slight reductions for AC-20a. However, the differences compared to the base asphalt are slight. The LDPE modified binders exhibited reductions in the slope values compared to the unmodified asphalts at all temperatures. Both the RSBS and MCR modified binders displayed higher slope values at all temperatures compared to the unmodified asphalt.

None of the BBR slopes of the modified asphalts showed an increase. Therefore, improvements in low-temperature properties (Figure 19) are not expected resulting from modification. Both asphalts with RSBS and MCR had slope values consistently higher than the original asphalt at all temperatures. At -24, -18, and -12 °C, the highest slope was exhibited by A-RSBS. At -18 °C, only the modified AC-20a with SBR, RSBS, and MCR had slope values higher than 0.3. At -12 °C, all of the binders except B-LDPE passed the SHRP requirement of 0.3. At -6 °C, the highest slope was found in A-MCR. All of the binders passed the SHRP 0.3 limit at -6 °C.

**Dynamic shear rheometry.** Figures 20 through 24 display the mastercurves of the storage and loss moduli, G′ and G″, respectively, and tan δ of the original, polymer-modified, and PAV-aged asphalt binders. These mastercurves curves are also presented in Appendix B where the modified binders are compared to the original and modified aged binders on an individual basis. In some cases, referring to the individual curves in the appendix may be helpful in delineating specific features of the curves. Examination of the mastercurves provides a convenient method for comparison of the complex modulus components of the modified binders. The upper plot is that for G′ and the lower plot for G″ and tan δ. In Figure 20, the response of the unmodified tank asphalts is typical for low molecular weight amorphous solids. No plateau is observed in the modulus and the curve for tan δ is smooth and increasing with decreasing frequency. It is apparent from the data that the rheological properties of these two tank asphalts are quite similar. The chemical properties of the two

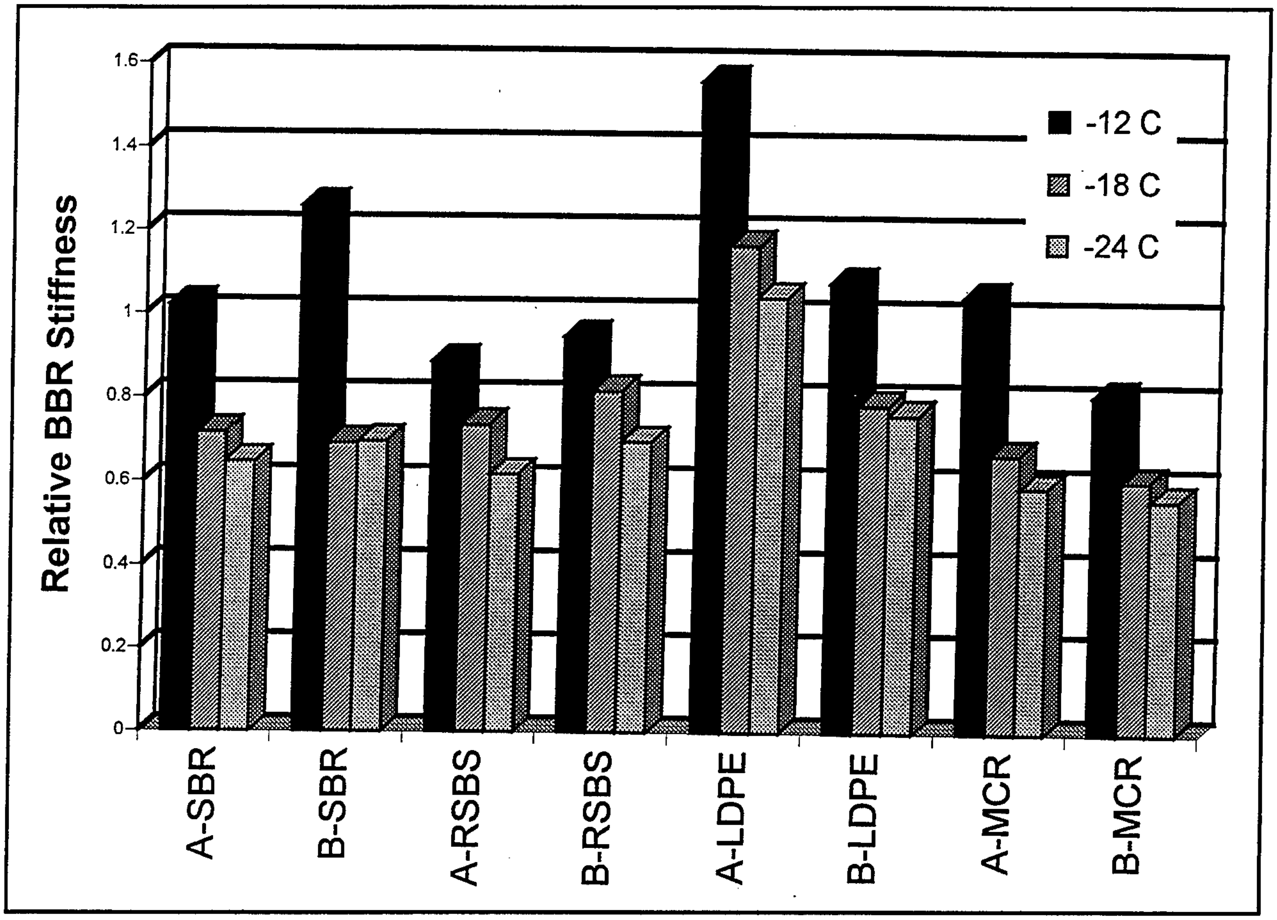

Figure 18. Stiffness results from BBR analysis

asphalts, although both from domestic sources, are measurably different with asphalt AC-20a having a slightly higher asphaltene and a lower aromatic content than asphalt AC-20b. After aging, the rheological properties of the asphalts are quite different, indicating the increased susceptibility to oxidation for AC-20b. For both aged asphalts, there is a flattening of the mastercurve with higher moduli values at all frequencies below approximately 1,000 Hz. Asphalt AC-20b has much higher moduli than AC-20a at frequencies above approximately 10 Hz.

In Figure 21, the complex modulus components for AC-20a, A-RSBS, A-SBR, A-LDPE, and A-MCR are presented at a reference temperature of 30 °C. At low frequencies, it is apparent that in order of decreasing G*, the ranking is A-LDPE > A-RSBS > A-SBR > A-MCR> AC-20a. Both of the modified asphalts exhibited increases in G′ and G″ at low frequencies and slight decreases at high frequencies. These results are consistent with the SHRP DSR findings that demonstrate larger improvements in G*/sin δ for the modified asphalts. All of the modified asphalts also show slightly decreased G* values over the unmodified AC-20a at all but the highest frequencies for A-RSBS, consistent with the BBR results. The A-LDPE material has a significantly higher modulus at low frequencies than the original asphalt AC-20a. At higher modulus at low frequencies than the original asphalt AC-20a. At high

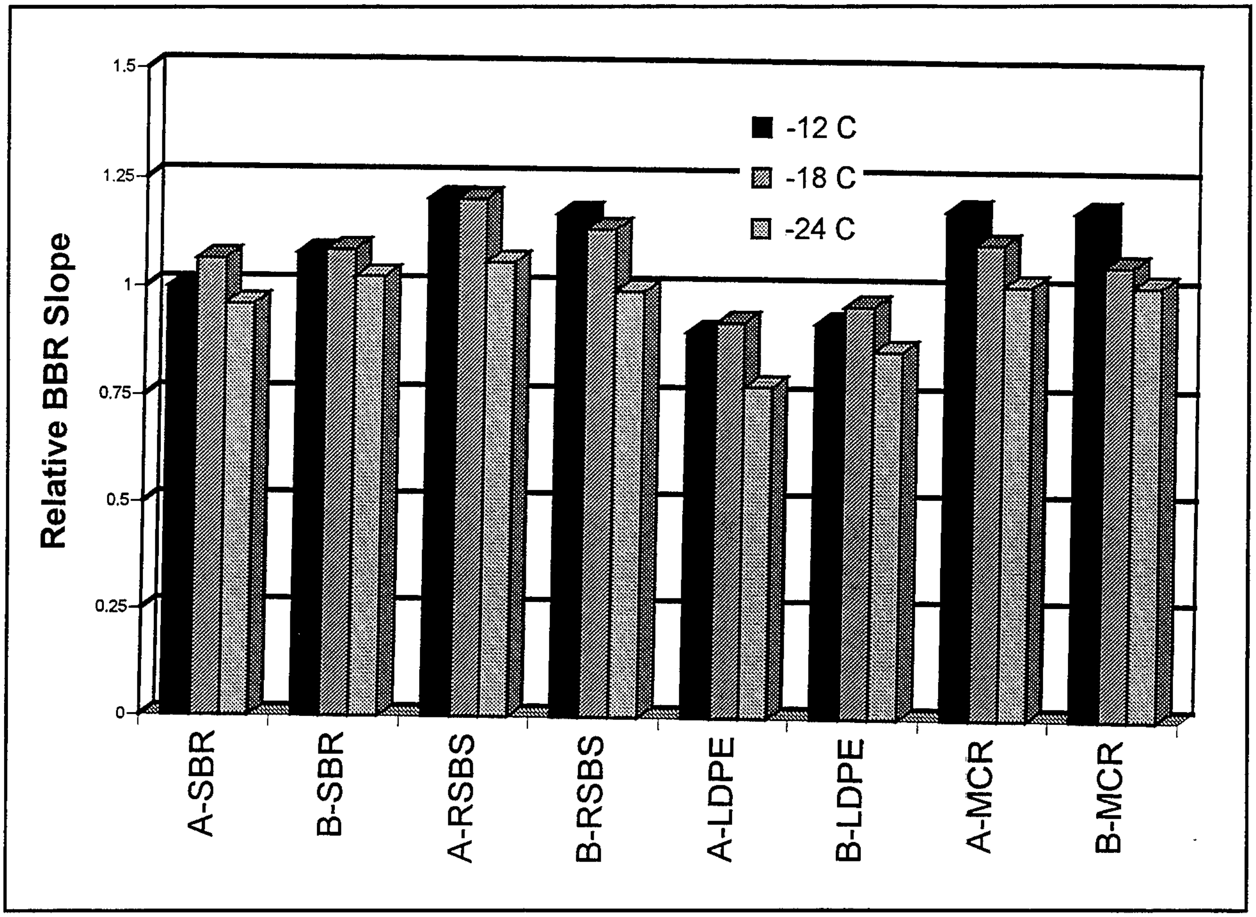

Figure 19. Slope (m) results from BBR analysis

frequencies, G* for A-LDPE is less than AC-20a which is generally inconsistent with the BBR results. The A-MCR had little effect on G″ compared to AC-20a at low frequencies; however, G″ is significantly lower at high frequencies and the peak in G″ is shifted to higher frequencies.

The trends in the slope of tan δ for the modified asphalts differ substantially from the unmodified AC-20a. The addition of LDPE modifier to AC-20a caused large changes to tan δ below $10^3$ Hz. For A-SBR, tan δ is slightly higher at high frequencies and much lower at low frequencies compared to AC-20a. The A-RSBS binder demonstrates similar behavior but with a slight peak occurring at approximately 0.5 Hz. This slight peak signifies the onset of a rubbery plateau that, in polymers, generally results from long-range chain entanglement or a low degree of crosslinking within the sample, both of which lead to some form of network formation. Close inspection of the behavior for G′ reveals an inflection in the curve in the region of the peak in tan δ. Although barely visible by inspection of G′, the onset of a rubbery plateau is clear from the behavior in tan δ. The reactive RSBS material is reported to be weakly crosslinked in situ, and this is observed in the rheological behavior. The peak for G″ is shifted for the modified binders towards higher frequencies, indicating a lowering of the glass transition temperature. For A-MCR, the

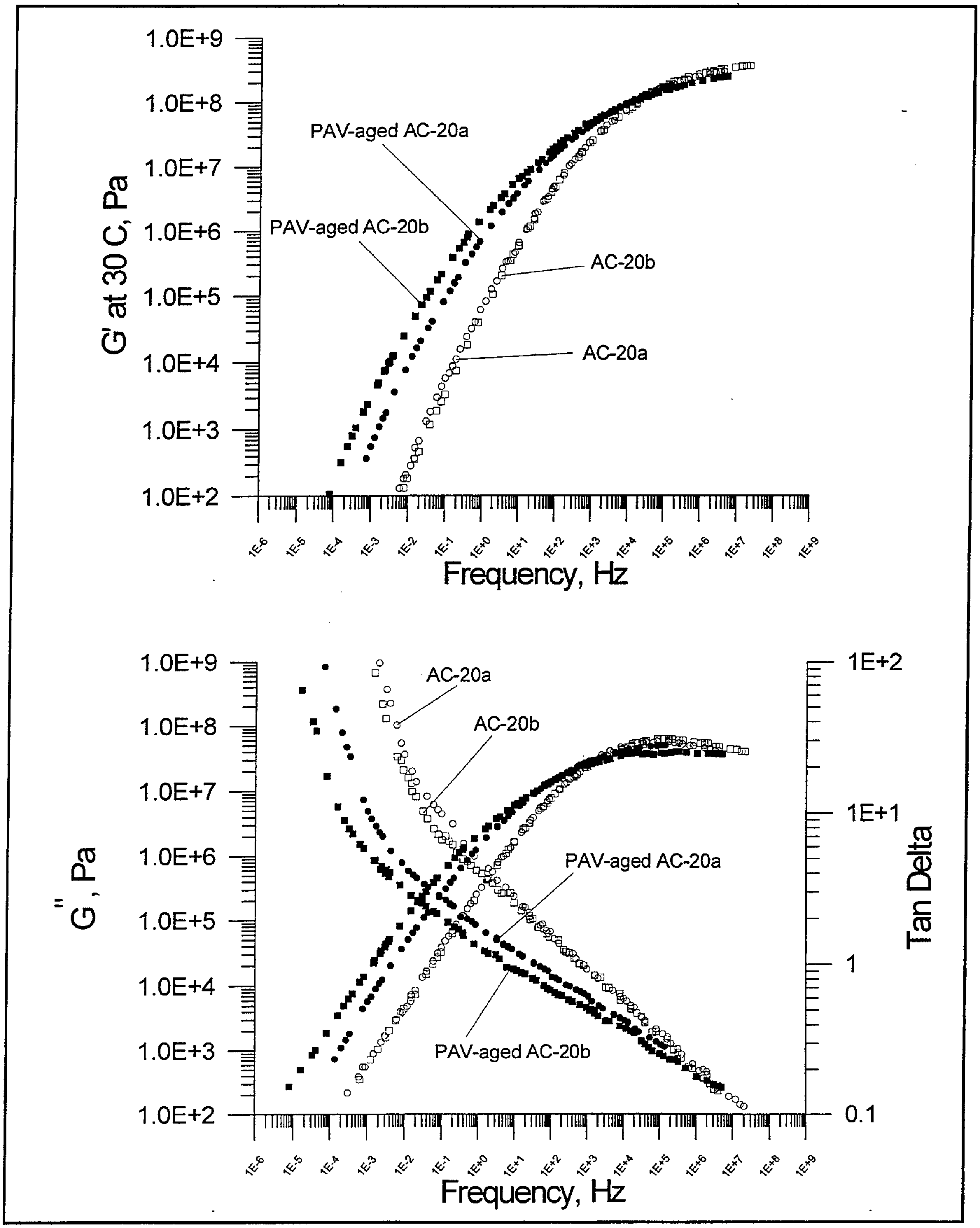

Figure 20. Comparison of tank and PAV-aged asphalts (reference temperature = 30 °C)

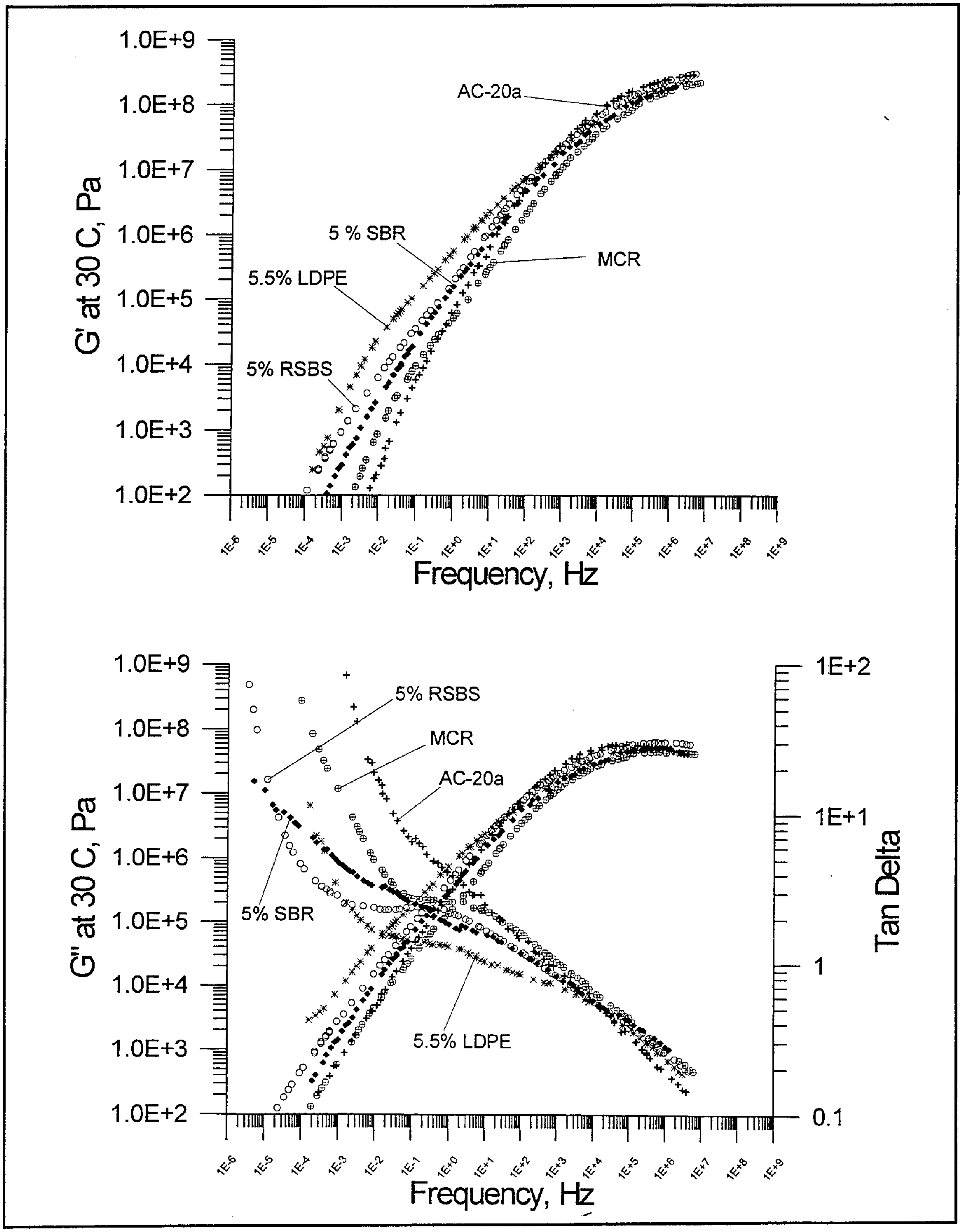

Figure 21. Modified AC-20a mastercurve (reference temperature = 30 °C)

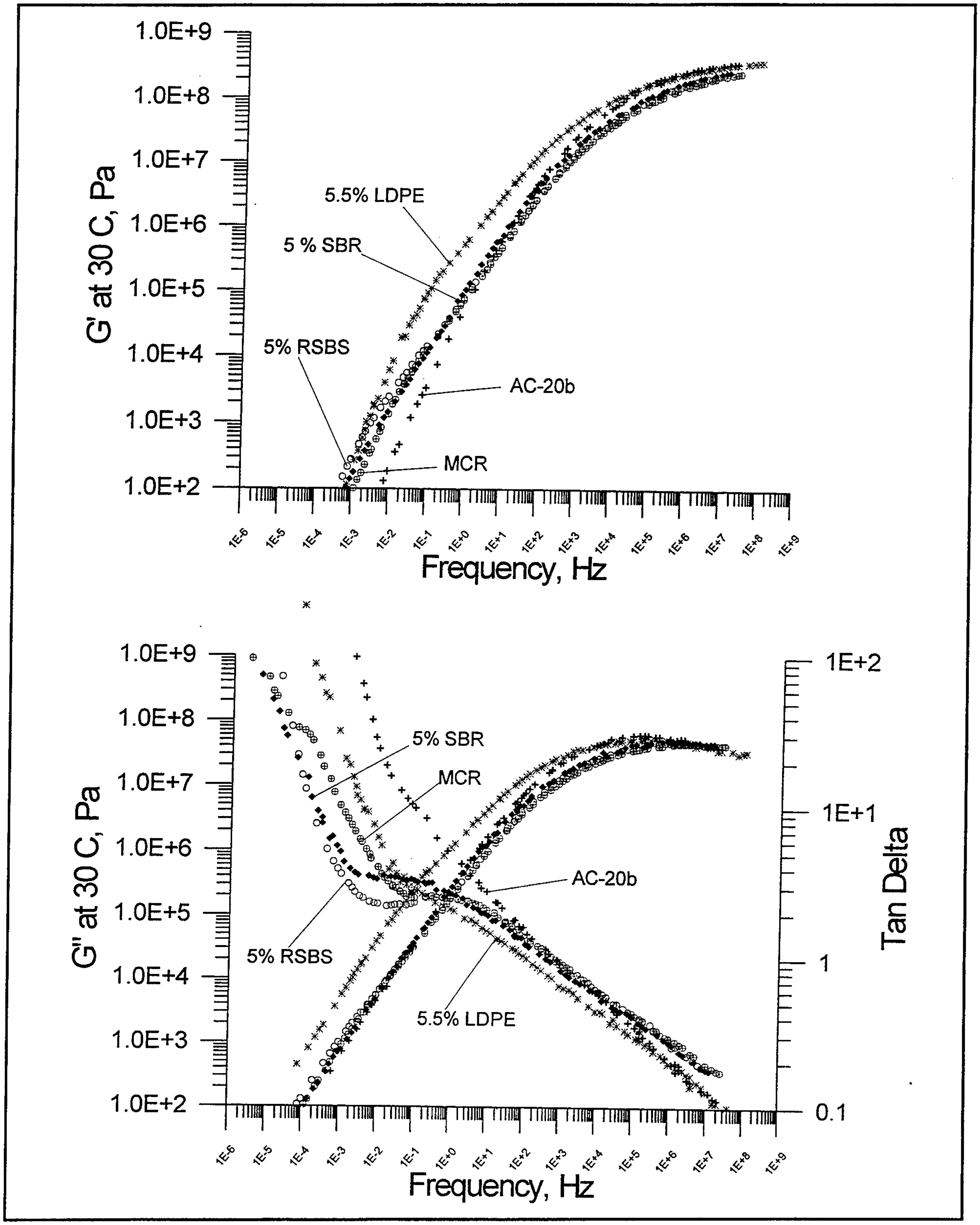

Figure 22. Modified AC-20b mastercurves (reference temperature = 30 °C)

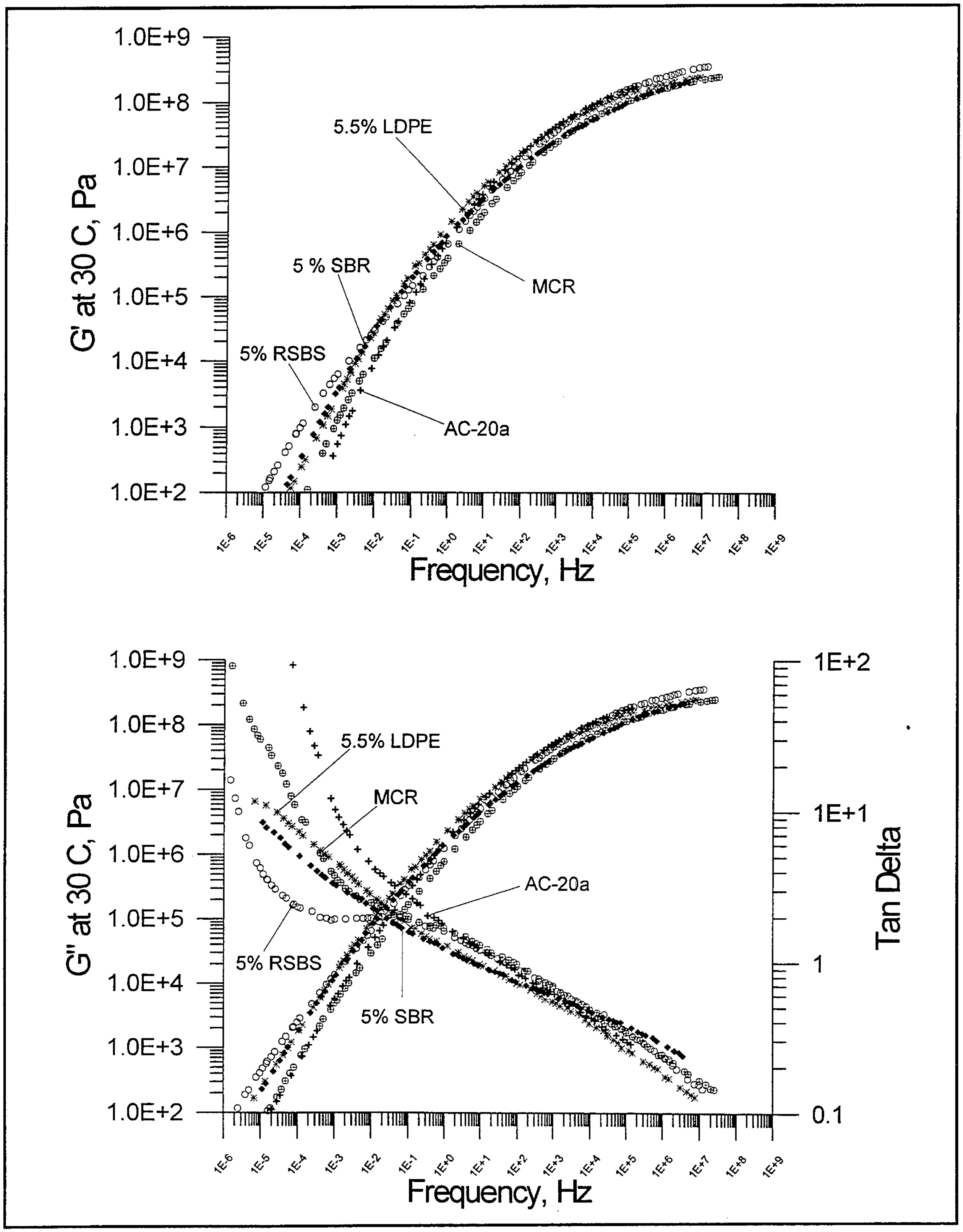

Figure 23. Modified AC-20a PAV-aged mastercurves (reference temperature = 30 °C)

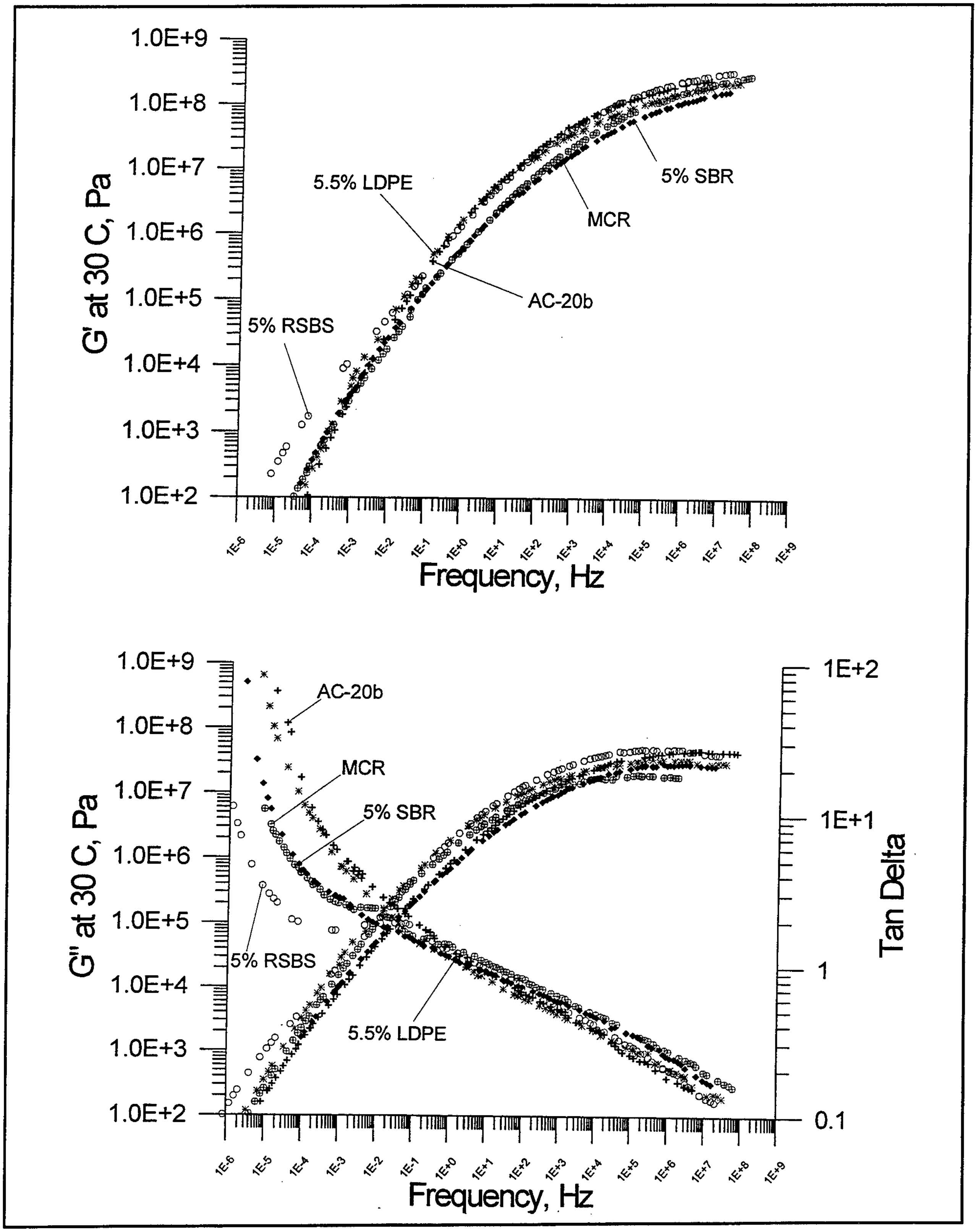

Figure 24. Modified AC-20b PAV-aged mastercurves (reference temperature = 30 °C)

onset of a rubbery plateau for G′ coincides with a sharp change in slope for tan δ. A plateau approximately 2 decades wide centered about 1 Hz for tan δ highlights the region in which molecular rearrangements are slow compared to the applied loading time.

In Figure 22, mastercurves for AC-20b, B-LDPE, B-RSBS, B-MCR, and B-SBR are shown. The rheological changes imparted to asphalt AC-20b by polymer addition are somewhat different from that observed for asphalt AC-20a. Smaller increases in G* properties at low frequencies are consistent with the SHRP DSR findings for asphalt AC-20b compared to AC-20a. The B-LDPE binder demonstrates large increases in G* compared to the base asphalt at intermediate frequencies. Ranking of the complex modulus at low frequencies yields B-LDPE > B-RSBS ≈ B-SBR ≈ B-MCR > AC-20b. At high frequencies, all of the materials have similar modulus properties. As in the case with asphalt AC-20a, a slight inflection in G′ for B-RSBS is present beginning near 1 Hz, which is reflected in the behavior for tan δ. Lower G* values for the modified binders at high frequencies are also observed as with asphalt AC-20a, consistent with BBR results. Again, as with asphalt AC-20a, the RSBS and SBR modifiers shift the maximum in G″ to higher frequencies. In contrast to A-SBR, the B-SBR material displays a plateau in tan δ centered about 0.05 Hz that begins near 1 Hz and extends to approximately 0.005 Hz. This is reflected in the behavior for G′, in which a small inflection is observed in the region of 1 Hz. The MCR modifier caused little change to G″ for asphalt B at low frequencies but had a significant effect above approximately 10 Hz, similar to that observed for A-MCR. As with A-MCR, the onset of a rubbery region in G′ is evident and is reflected in the change in tan δ.

Figure 23 details the DSR results of PAV aging on asphalt A and the modifiers blended with A. There is a flattening of the moduli curves compared to the unaged materials, and the curves are shifted to lower frequencies. The relative order of stiffness at low frequencies has also changed such that A-RSBS > A-SBR ≈ A-LDPE > A-MCR ≈ AC-20a. There is little difference in any of the materials at high frequencies. There are notable differences in the tan δ curves compared to the unaged materials as well. For instance, the A-MCR material exhibited a plateau in tan δ for the unaged material, but this has disappeared in the PAV-aged binder. The shape of tan δ for the A-RSBS material is relatively unchanged compared to the unaged binder, but is shifted to much lower frequencies. The peak in tan δ for A-RSBS is still present and for the aged material occurs at approximately 0.1 Hz, compared to 0.5 Hz for the unaged material. The changes occurring in tan δ for the A-LDPE material after aging are suspect since tan δ for the aged material is actually larger than the unaged material. This is inconsistent with the results for all of the other materials. The reasons for this are not clear but may be related to inconsistent sampling and/or aging. The PAV-aging temperature is 100 °C and the LDPE material will phase separate from the asphalt at temperatures above 80 °C. Thus, the PAV-aging technique may produce inconsistent results for binders with a propensity toward phase separation at the PAV-aging temperature. Nonetheless, after aging, the relative order of the properties of the binders has not changed drastically compared to the unaged materials, exclusive of A-LDPE.

In Figure 24, the DSR data for the PAV-aged binders derived from AC-20b are presented. Similar to the aged AC-20a binders, the modulus and tan δ curves are shifted to lower frequencies compared to the unaged AC-20b materials. The ranking of the modifiers in order of modulus properties at low frequencies has changed from that of the unaged material to B-RSBS > B-LDPE ≈ B-SBR ≈ B-MCR ≈ AC-20b. The B-LDPE PAV-aged material follows a similar trend to the A-LDPE PAV-aged material in that the modulus properties are not drastically different from that of the original, unaged LDPE binder. As observed in the A-MCR binder system, the aged B-MCR material has lost the large plateau observed in tan δ for the original B-MCR. The same is true for the B-SBR binder except that the plateau in tan δ has completely disappeared after aging. The B-RSBS material still displays a slight peak in tan δ and the onset of a plateau region is evident upon inspection of G′. At high frequencies, all of the modified materials have modulus values lower than that of the unmodified aged binder except for B-RSBS.

**Fluorescence reflection microscopy.** Both unmodified asphalts, when viewed under UV light, appear either dark gray or black. The polymers appear as bright areas in contrast to a darker background if discrete polymer particles are present. For the purpose of this work, compatibility of the modifier in the asphalt will be defined by the lack of clearly visible phase separated domains within the resolution of the microscope at 50 times magnification. Examination of sample A-RSBS at 50X magnification reveals that very few discrete polymer particles can be observed. Phase separated polymer domains, if present, were beyond the resolving power of the microscope. Sample B-RSBS is almost identical to A-RSBS. In both cases of RSBS binders, the surface appears gray and featureless.

The SBR modifier appears as long, threadlike domains of polymer, clearly phase separated but with a unique morphology. The domains do not appear to be continuous. However, in B-SBR, the material appears to very well dispersed as small spheres or individual particles cannot be distinguished at all. The polymer appears to be very well dispersed and more compatible with asphalt AC-20b than AC-20a. Examination at higher magnifications reveals regions that appear cloud like and nebulous suggesting swelling of the polymer phase. This demonstrates that different chemistry between asphalt sources clearly affects the morphology of the polymer in the asphalt cement.

The LDPE modifier appeared as dispersed, spherical particles in both A-LDPE and B-LDPE. However, the B-LDPE binder appears to have regions of swollen particles in which the asphalt binder appears “inside” of LDPE particles. The LDPE material appears have little compatibility with either of the asphalts studied here.

The MCR modifier is a mixture of SBS and ground tire rubber. Ground tire rubber is a mixture of materials that include SBR and carbon black. The material is crosslinked by vulcanization and may swell or react somewhat over time as components of the asphalt diffuse into the crosslinked rubber matrix. The CR particles are visible as dark solids in both A-MCR and B-MCR. For both asphalts with MCR, the added polymer appears as discrete separate particles. This is not surprising since the SBS concentration is only 2 percent.

## Summary

The purpose here is not to generate a ranking of which modifier system may outperform another, but to present and discuss the rheological effects imparted to the asphalts from a variety of modifier types. The modifiers selected represent a sampling of materials employed in construction. The LDPE modifier is described as recycled "plastic" and is representative of a semicrystalline polyolefin material that is generally chemically incompatible with most asphalts. An SBR rubber was chosen because it has varying degrees of compatibility with different asphalts. A proprietary reactive polymer system based on crosslinked styrene-butadiene block copolymers that is reported to ensure a compatible asphalt/polymer blend was also selected. A recycled ground tire rubber modifier was included because, at the time of project initiation, the ISTEA legislative mandate requiring states to include a portion of recycled tires in asphalt mixtures was still active.

A major focus of this study was to determine the effects on the modifiers on the durability of an asphalt/aggregate pavement. Durability, as described here, refers to the ability of the binder material to resist permanent deformation (rutting), fatigue resistance, and thermal cracking as the pavement ages. As such, the properties of the original binder and the aged material are important. Rutting and fatigue are both load-related and thermal cracking is not. Rutting problems in pavements generally occur early in the pavement life in warm climatic conditions, when the binder is less stiff. As the pavement ages and the binder becomes more brittle due to oxidation, the pavement becomes more resistant to permanent deformation. Thus, the plastic deformation tendencies of the binder prior to long-term aging are generally more critical. However, embrittlement of the binder results in pavements more susceptible to fatigue cracking because the material's ability to dissipate energy is reduced (SHRP-A-409 (1994c)). Thermal cracking also becomes more of a problem as the pavement ages because the material becomes stiffer and less able to resist thermal strains (SHRP-A-398 (SHRP 1994b) and SHRP-A-409 (SHRP 1994c)). Thus, as defined here, a more durable binder would be one that, after modification, is better able to resist permanent deformation, fatigue cracking, and thermal cracking than the unmodified material as measured in the laboratory.

The SHRP specification properties relating to performance are: $G^*/\sin\delta$, the rutting parameter; $G^*\sin\delta$, the fatigue parameter, and $S$ and $m$, the thermal cracking parameters. $G^*/\sin\delta$ is the inverse of the viscous compliance at a given temperature and 10 rad/sec applied load. Thus, the specification established a limit to the plastic deformation tendencies of unaged material. $G^*\sin\delta$ is the viscous modulus and is related to the ability of the material to dissipate energy, necessary to resist repetitive strains that lead to fatigue cracking. Fatigue cracking is generally a problem for aged pavements in which the binder has become brittle due to oxidation, thus, the specification establishes a minimum value for $G^*\sin\delta$ for laboratory-aged materials. Similarly, thermal cracking is more often observed in older pavements, and the specifications are for a minimum value of the creep rate, $m$, and a maximum stiffness, $S$, for the aged binders. These properties have been correlated to actual field performance for highway pavements (SHRP-A-398 (SHRP 1994b) and SHRP-A-409

(SHRP 1994c)) and, as such, should be taken with some caution given the differences between a heavy-duty airfield and a typical highway mixture. However, the caution would be only for the magnitude of the specification criteria and not necessarily on the parameter itself.

According the SHRP criteria, all of the modifiers impart favorable effects to the durability of the base asphalt. All of the modified binders demonstrated increased resistance to plastic deformations in the mix as evidenced by the increased G*/sin δ values for the modified binders. After PAV-aging, the effects are somewhat mixed. Except for LDPE, the modifiers reduced the limiting temperature for G*sin δ. Thus, the LDPE binders may exhibit greater fatigue cracking as the pavement ages, according to these criteria. For low-temperature properties, the MCR, RSBS, and SBR modified binders reduced the SHRP PG by at least one temperature with the LDPE material having no effect on the SHRP grades. The effects of the modifiers on the asphalts are clearly dependent on the modifier type and asphalt source.

For the RSBS modified binders studied here, significant improvements were imparted to the durability properties of the base asphalts. This modifier is chemically reacted with the asphalt to provide inherent compatibility with the base asphalt and is lightly crosslinked to further enhance favorable modulus properties. The lack of phase separated domains visible by Fluorescence Reflection Microscopy (FRM) confirms the former and the latter is verified by examination of tan δ for both asphalts. The aged asphalts also exhibit behavior in tan δ indicative of crosslinking, demonstrating that the network survives the PAV-aging process. For both of the RSBS modified binders, significant improvements in the high-temperature properties and low-temperature stiffness and creep rate were observed.

As observed with the RSBS modifier, addition of SBR to the asphalts resulted in improvements in the durability properties of the base asphalts. For the SBR binders, two different polymer morphologies are observed. In A-SBR, the polymer is phase separated and forms long, threadlike regions of polymer that upon close examination, are not continuous. However, the SBR material appears to be more compatible with asphalt AC-20b because of the "fuzzy" (swollen) appearance of the polymer particles and the lack of clearly phase separated domains as observed for A-SBR. A comparison of tan δ curves indicates a substantial inflection for B-SBR not present in A-SBR. For both of the SBR modified binders, significant improvements in the high- and low-temperature properties were observed.

The MCR modifier imparts favorable high- and low-temperature properties to both of the asphalts, improving the durability. Both of the MCR binders exhibited sharp inflections in tan δ. Based on past experience, it is doubtful that a concentration of 2 percent weight of CBS to weight of asphalt (w/w) SBS in asphalt would cause such a large change in tan δ. In addition, the FRM results clearly show the existence of noninteracting phase domains. As previously mentioned, large changes in tan δ have been observed in binders with high concentrations of CR (15 percent). The CR particles are crosslinked and would not be expected to network in the traditional manner that results from entanglement or crosslinking of polymer chains. If the CR particles were behaving as mere fillers, little change would be expected in tan δ. Whatever the mechanism for the behavior in tan δ, the CR particles are causing an event to occur in a range of intermediate frequencies that resists molecular rearrangements.

The LDPE modifier had mixed effects on the durability properties of the base asphalts. The LDPE modifier caused large increases in the high temperature modulus of the asphalts studied here. A large inflection in tan δ is noted for A-LDPE; however, no peak is observed. For B-LDPE, the changes to tan δ compared to the tank asphalt are less than those for AC-20a, but a substantial slope change is observed. As mentioned earlier, the LDPE modifier is incompatible with both asphalts. LDPE had deleterious effects on the low temperature properties, raising the stiffness and lowering the creep rate compared to the original asphalts. This is not surprising given the nature of the modifier itself. LDPE is a high-modulus, low-creep, semicrystalline solid.

## HMA and Modified HMA Characterization

The results of HMA sample preparation and characterization testing of the 20 HMA mixtures are presented in this section. The objective of the laboratory evaluation was to determine the effects of modification on HMA performance with respect to durability. To ensure that the asphalt content and aggregate gradation did not influence the test results, the aggregate gradation was held constant for all mixes and the optimum asphalt content was selected at 4 percent air voids (voids total mix), using the 75-blow Marshall mix design criteria. Once the optimum asphalt content had been selected, the remaining samples were compacted using the GTM.

### Data analyses and discussion

**Volumetric and Marshall properties.** A summary of the average Marshall mix properties for the HMA and modified HMA mixtures is provided in Table 26. The test results include unit weight, air voids, voids filled with asphalt, and the Marshall stability and flow values.

None of the modifiers had a significant effect on the optimum binder content, as determined by the criterion of 4 percent voids. Optimum binder contents for all the mixtures varied between 5.1 and 5.3 percent by total mass. The percent voids filled (and therefore the associated voids in the mineral aggregate) were also relatively insensitive to HMA modification. At optimum binder content, the percent voids filled varied between 74 and 77 percent. Based on these observations, any influences of polymer modification that are presented in other sections can be attributed effects on binder properties, rather than effects on mixture design.

All the polymer modifiers increased Marshall Stability, as shown in Figures 25 and 26. Increases in Marshall Stability ranged from 15 percent to

Table 26
Average Volumetric and Marshall Properties for the HMA Mixtures

| | Results | | | | |
|---|---|---|---|---|---|
| Test Property | Unmodified | LDPE | RSBS | SBR | MCR |
| **AC-20a** | | | | | |
| Binder Content % | 5.2 | 5.1 | 5.1 | 5.2 | 5.2 |
| Voids Total Mix % | 4.1 | 4.2 | 4.1 | 4.2 | 4.3 |
| Voids Filled % | 74.7 | 73.9 | 74.4 | 74.4 | 73.6 |
| Unit Weight Kg/m$^3$ (pcf) | 2,403.1 (150.1) | 2,404.7 (150.2) | 2,403.1 (150.1) | 2,398.3 (149.8) | 2,399.9 (149.9) |
| Stability KN (lb) | 10 (2,430) | 16 (3,650) | 15 (3,630) | 13 (3,010) | 12 (2,870) |
| Flow | 11 | 10 | 15 | 15 | 13 |
| Retained Stability % | 93 | 89 | 97 | 92 | 98 |
| **AC-20b** | | | | | |
| Binder Content % | 5.3 | 5.1 | 5.1 | 5.2 | 5.2 |
| Voids Total Mix % | 3.8 | 4.0 | 4.2 | 3.6 | 3.9 |
| Voids Filled % | 76.5 | 75.0 | 74.1 | 77.4 | 75.5 |
| Unit Weight Kg/m$^3$ (pcf) | 2,404.7 (150.2) | 2,406.3 (150.3) | 2,399.9 (149.9) | 2,411.1 (150.6) | 2,407.9 (150.4) |
| Stability KN (lb) | 9.7 (2,280) | 15 (3,590) | 17 (3,930) | 16 (3,680) | 13 (3,110) |
| Flow | 11 | 12 | 15 | 14 | 13 |
| Retained Stability % | 89 | 98 | 90 | 99 | 92 |

57 percent. The unmodified mixtures demonstrated a favorable resistance to stripping, as measured by retained Marshall Stability. Therefore, the polymer modifiers were not able to improve performance in this respect. Retained Marshall stabilities were satisfactory for all mixtures, ranging from 89 percent to 99 percent.

Generally, the polymer modifiers increased the Marshall flow values as compared to the unmodified HMAs. These increases were most substantial when RSBS or SBR was used as the modifier. Historically, high flow values for unmodified HMA indicate susceptibility to in situ permanent deformation. However, the significance of high flow values for modified HMA is questionable.

One of the reported benefits of adding a modifier to a binder is the fact that the asphalt content of the HMA can be increased without significantly effecting the mechanical properties of the HMA as compared to the unmodified HMA. This could prove to be significant from a durability standpoint because an increase in binder content should increase HMA resistance to durability type distresses. Therefore, Table 27 includes the estimated volumetric and

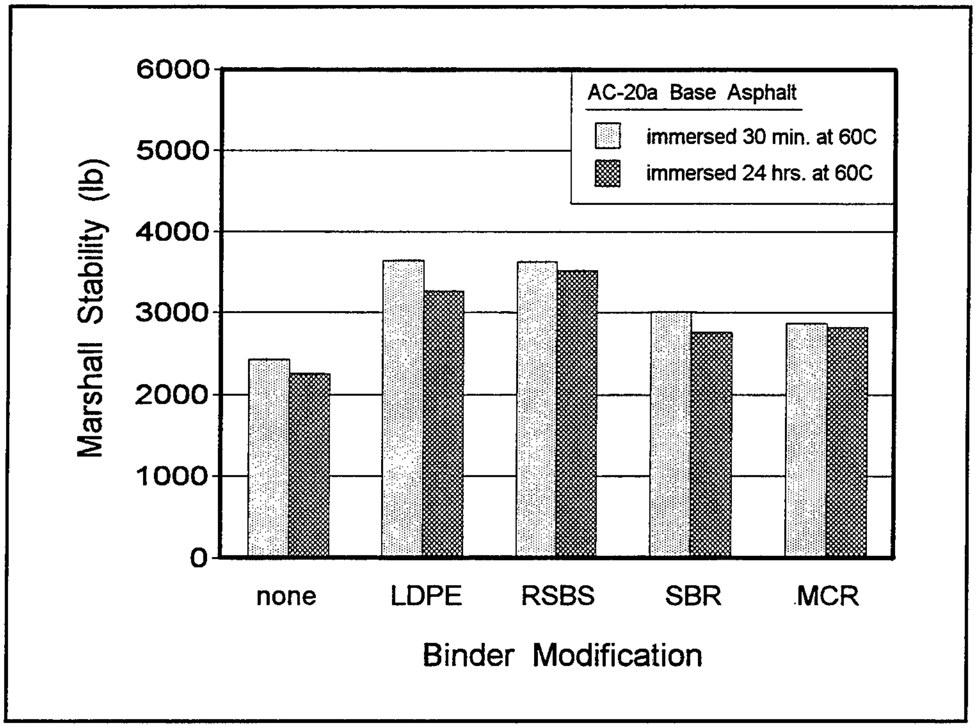

Figure 25. Effects of binder modification of AC-20a on Marshall stability results

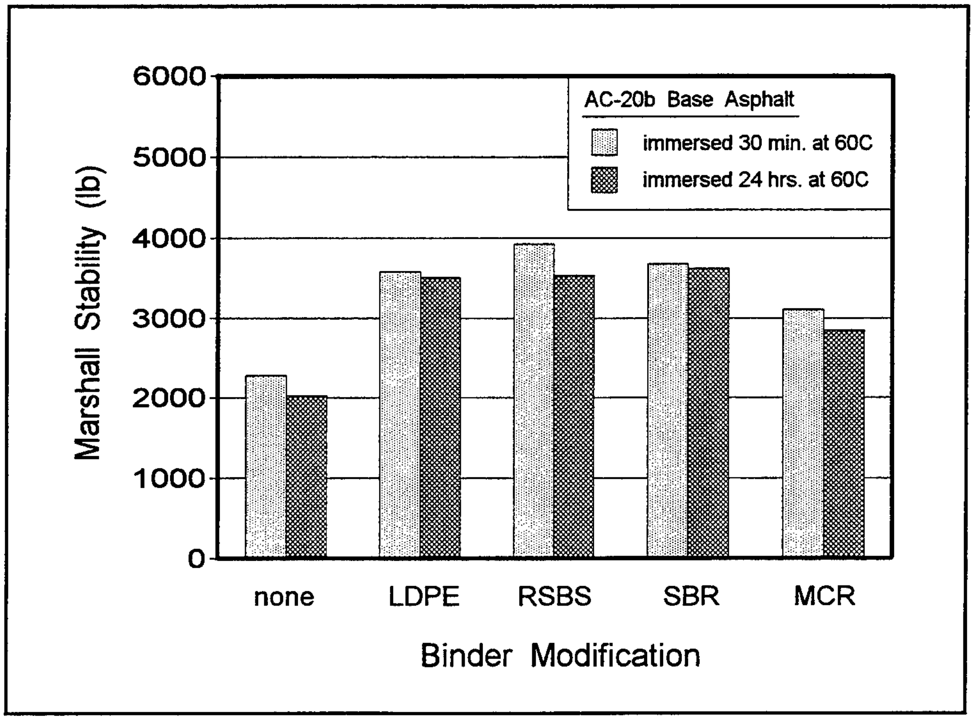

Figure 26. Effects of binder modification of AC-20b on Marshall stability results

## Table 27
## Estimated Volumetric and Marshall Properties for the HMA Mixtures

| | Results | | | | |
|---|---|---|---|---|---|
| Test Property | Un-modified | LDPE | RSBS | SBR | CR |
| AC-20a | | | | | |
| Binder Content % | 5.8 | 5.6 | 5.6 | 5.7 | 5.7 |
| Voids Total Mix % | 2.8 | 3.2 | 3.2 | 3.4 | 3.2 |
| Voids Filled % | 82.5 | 80.1 | 80.4 | 79.8 | 80.5 |
| Unit Weight Kg/m$^3$ (pcf) | 2,415.9<br>(150.9) | 2,409.5<br>(150.5) | 2,411.1<br>(150.6) | 2,398.3<br>(149.8) | 2,411.1<br>(150.6) |
| Stability KN (lb) | 11<br>(2,490) | 14<br>(3,210) | 15<br>(3,400) | 13<br>(2,880) | 15<br>(3,280) |
| Flow | 12 | 13 | 16 | 15 | 14 |
| AC-20b | | | | | |
| Binder Content % | 5.8 | 5.6 | 5.6 | 5.7 | 5.7 |
| Voids Total Mix % | 2.9 | 2.8 | 2.8 | 2.9 | 3.2 |
| Voids Filled % | 82.4 | 82.5 | 82.7 | 82.1 | 80.5 |
| Unit Weight Kg/m$^3$ (pcf) | 2,407.9<br>(150.4) | 2,417.5<br>(151.0) | 2,417.5<br>(151.0) | 2,407.9<br>(150.4) | 2,411.1<br>(150.6) |
| Stability KN (lb) | 10<br>(2,350) | 15<br>(3,430) | 16<br>(3,690) | 14<br>(3,250) | 13<br>(2,820) |
| Flow | 12 | 14 | 18 | 17 | 14 |

Marshall properties of the HMA with a 0.5 percent increase in binder content over the optimum amount. These values were generated during the development of the Marshal mix design procedures. Generally, when considering physical and mechanical properties, the increase in binder content caused similar changes for both the unmodified and modified HMA. The increase in binder content may have been too small to demonstrate the supposed benefits of polymer modification.

**Gyratory test machine (GTM) properties.** The GTM was selected to compact all of the HMA specimens because the kneading action used in the compaction process produces specimens that have aggregate particle orientation similar to that of in-place pavements. A second benefit of the Corps of Engineers' GTM was that stress-strain measurements for each specimen were obtained, thus assisting in the evaluation of the quality of the HMA. A third benefit of using the GTM was that compaction could continue for each sample until a target unit weight was attained. This was essential for proper comparisons between unaged and aged mixtures. A summary of the average GTM properties for the HMA mixtures is presented in Table 28. The GTM test results include the Gyratory Stability Index (GSI), Gyratory Elasto-Plastic Index (GEPI), and gyratory shear strength values. The HMA specimens were compacted to a selected unit weight based on the Marshall mix design values. Therefore, an additional piece of information was available for comparisons.

## Table 28
## Summarized GTM Data

| HMA Mixture | Condition | Number of Gyratory Revolutions | GEPI | GSI | $S_G$ KPa (psi) |
|---|---|---|---|---|---|
| AC-20a (Unmodified) | Unaged | 36 | 1.28 | 1.00 | 992 (144) |
| | Aged | 71 | 1.25 | 1.02 | 1,069 (155) |
| AC-20b (Unmodified) | Unaged | 35 | 1.28 | 1.00 | 820 (119) |
| | Aged | 179 | 1.26 | 1.06 | 910 (132) |
| AC-20a + LDPE | Unaged | 44 | 1.25 | 1.00 | 896 (130) |
| | Aged | 149 | 1.24 | 1.09 | 565 (82) |
| AC-20b + LDPE | Unaged | 40 | 1.27 | 1.00 | 765 (111) |
| | Aged | 162 | 1.28 | 1.11 | 593 (86) |
| AC-20a + RSBS | Unaged | 50 | 1.26 | 1.00 | 841 (122) |
| | Aged | 320 | 1.23 | 1.05 | 655 (95) |
| AC-20b + RSBS | Unaged | 43 | 1.25 | 1.00 | 772 (112) |
| | Aged | 500+ | 1.27 | 1.06 | 655 (95) |
| AC-20a + SBR | Unaged | 48 | 1.25 | 1.01 | 889 (129) |
| | Aged | 70 | 1.25 | 1.03 | 745 (108) |
| AC-20b + SBR | Unaged | 45 | 1.25 | 1.00 | 786 (114) |
| | Aged | 121 | 1.27 | 1.09 | 607 (88) |
| AC-20a + MCR | Unaged | 35 | 1.27 | 1.00 | 772 (112) |
| | Aged | 356 | 1.26 | 1.05 | 703 (102) |
| AC-20b + MCR | Unaged | 49 | 1.21 | 1.00 | 683 (99) |
| | Aged | 500+ | N/A | N/A | 545 (79) |

This information is also presented in Table 28 and may provide information concerning the workability (or compactive effort required to obtained a pre-selected density) if the mix is stored for an extended period of time in a storage silo.

The number of GTM flange revolutions required to achieve target densities, corresponding to 4 percent voids, is shown in Table 28 and in Figures 27 and

28. Aging caused the required number of revolutions to increase for each mixture type. A comparison of the two unmodified mixtures reveals that those containing AC-20b were more susceptible to changes in compactibility than mixtures containing AC-20a. A comparison of modified mixtures reveals that those containing RSBS or MCR were more susceptible to changes in compactibility than mixtures containing LDPE or SBR. Mixtures containing AC-20b and either RSBS or MCR did not reach their target densities, corresponding to 4 percent voids, despite 500 revolutions of the GTM flange (see Figures 27 and 28). Considering that 30 revolutions correlate with a 75-blow compactive effort by Marshall procedures, 500 revolutions made a significant compactive effort in attempting to reach density.

GEPI is a measure of the shear strain occurring in an asphalt mixture during compaction. Its value ranged from 1.21 to 1.28 deg for all mixtures included in this study. The range for this GTM parameter was expected to be small because it depends primarily on aggregate characteristics and the aggregate properties remained constant for all mixtures.

GSI for all unaged mixtures ranged from 1.00 to 1.01. These values indicate that the mixtures were stable as designed. The desired void content of 4 percent was attainable without mixtures "flushing." Gyratory stability indices for aged materials ranged from 1.02 to 1.11, indicating various degrees of instability. These mixtures may have flushed as a result of changes in effective particles sizes that could have occurred as a result of the aging process. A mass of aged asphalt-based binder and fines may not be broken as easily during compaction as a mass of unaged asphalt-based binder. If small masses of asphalt-based binder and fines are not dispersed, the effective particle gradation has changed, potentially causing the binder content to be different than optimum.

The mixture shear strengths were calculated strictly from roller pressures and machine dimensions. The effects of friction on the inside surfaces of the cylindrical molds were not considered in the calculations. Therefore, the magnitudes of the shear strengths should be regarded with caution. If the effects of friction are assumed to be approximately the same for all mixtures, however, the relative values of shear strength can provide some valuable comparisons between the engineering properties of the mixtures. Comparisons between all of the unaged mixtures do not reveal any great differences in strength, but some consistent trends can be found in comparisons between unaged and aged materials. For each of the unmodified mixtures, shear strength increased with aging. For each of the modified mixtures, shear strength decreased with aging. This may indicate that the unmodified binders were better able to retain their cohesive properties during aging, as compared to the modified binders.

### Indirect tensile properties

The indirect tensile test was conducted to provide an indication of the stiffness of the various HMA mixtures. The testing was conducted on a minimum of three specimens at each of two test temperatures: 25 °C (77 °F) and 40 °C

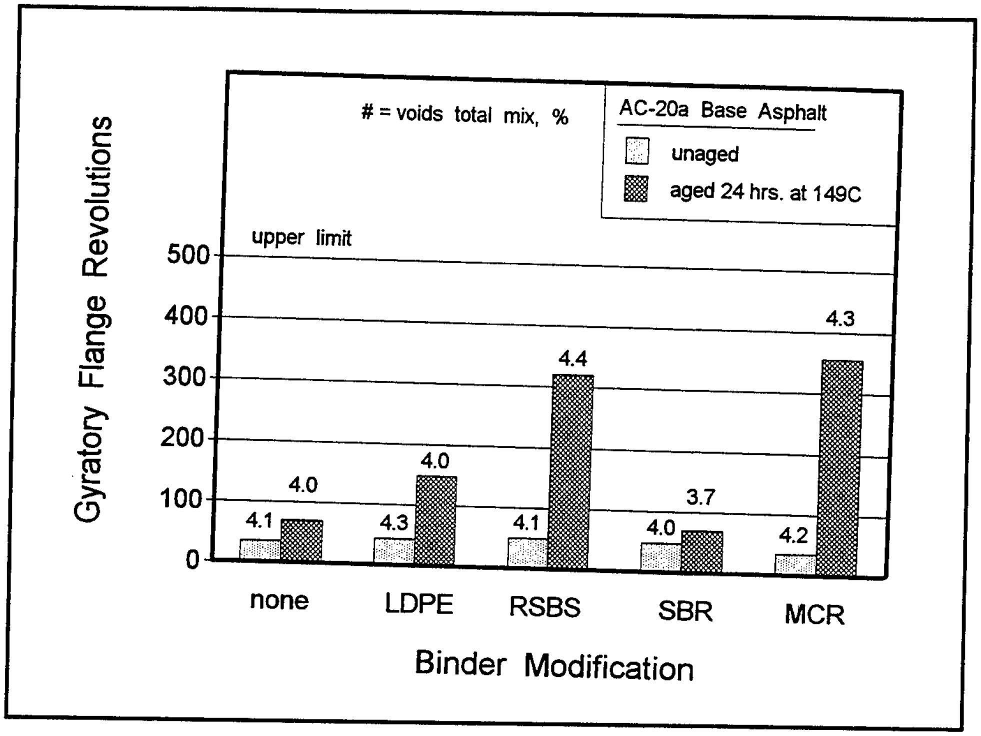

Figure 27. Gyratory revolutions to obtain the target unit weight for AC-20a

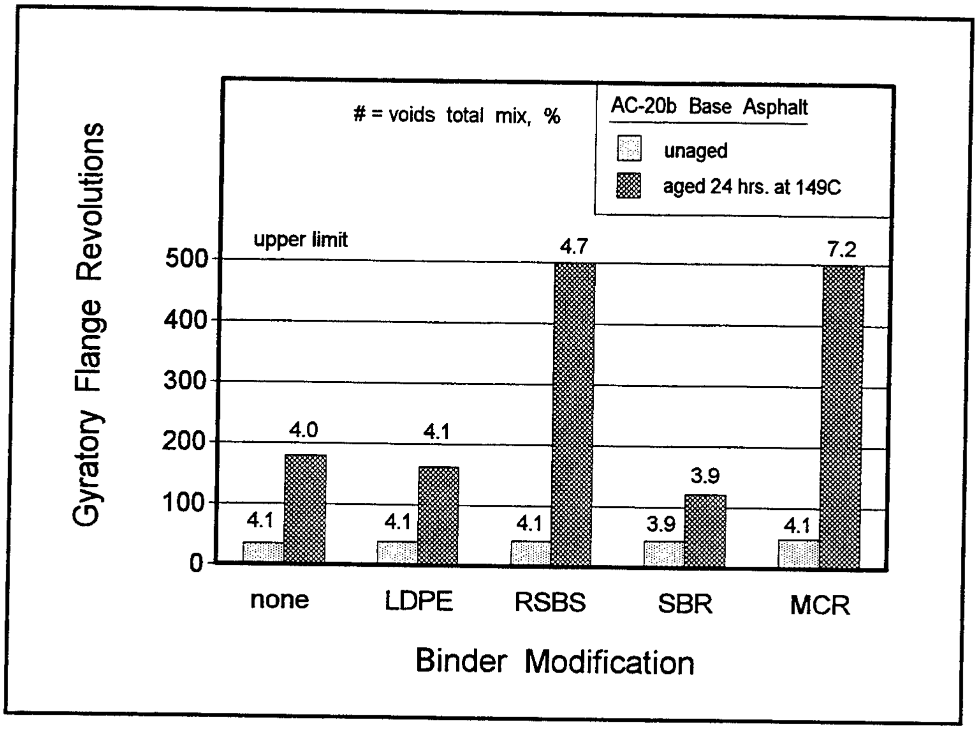

Figure 28. Gyratory revolutions to obtain the target unit weight for AC-20b

(104 °F). These temperatures were selected to evaluate the tensile strength of the HMA at typical in-service temperatures and to provide an indication of how those properties change with temperature. The HMA tensile strengths calculated according to ASTM D 4123 (ASTM 1994g) are summarized in Tables 29 and 30.

In order to consider both the strength and brittleness of materials, the indirect tension test results seemed to be best presented as "indirect tension toughness" values. These values, which were calculated by multiplying peak load by the vertical deformation at peak load, provided a measure of the ability of materials to absorb energy imposed by loads. When data are presented in this form, indirect tension tests performed at 25 °C (77 °F) show clear differences between the two base asphalt cements (Figures 29 and 30). Source AC-20a, in its unmodified condition, exhibited an increase in toughness with aging, while AC-20b showed a decrease.

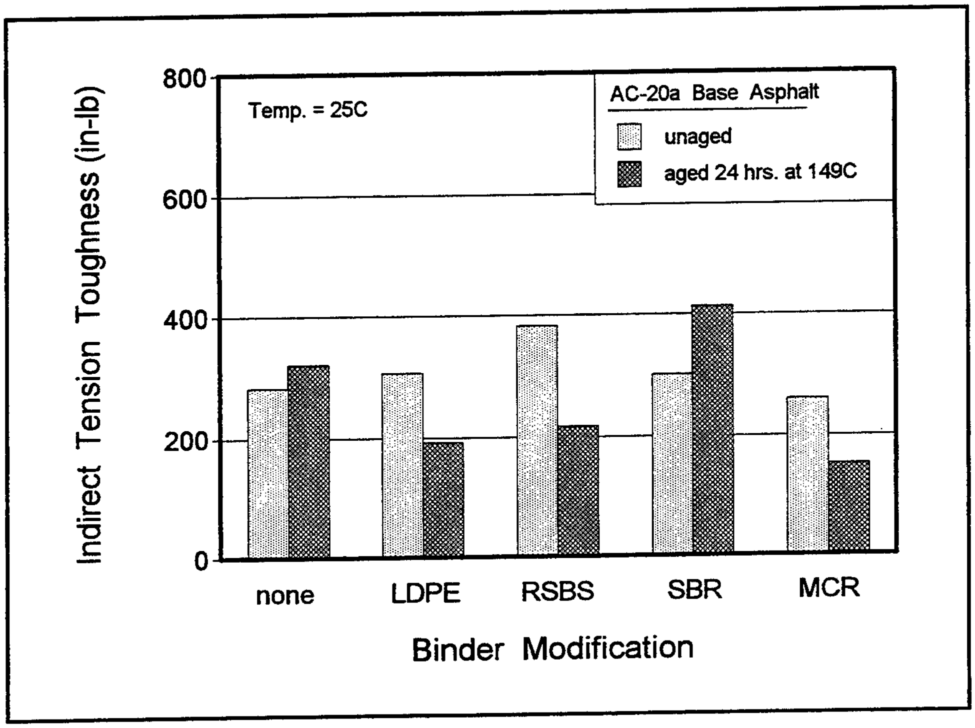

Figure 29. Indirect tensile toughness (in.-lb) of AC-20a mixtures at 25 °C (77 °F)

When all the unaged mixtures are compared, no tremendous differences are evident between unmodified and modified mixtures. However, when unaged and aged modified mixtures are compared, some differences are evident. When used in combination with AC-20a, SBR mixtures were the only polymer-modified HMA that increased in toughness with aging. When used in combination with AC-20b, SBR mixtures suffered the least loss in toughness. Mixtures containing RSBS or MCR became particularly dry and brittle with aging.

**Table 29**
**Indirect Tensile Test Data at 25 °C (77 °F)**

| HMA Mixture | Condition | Maximum Load KN (lb) | Tensile Strength MPa (psi) |
|---|---|---|---|
| AC-20a (Unmodified) | Unaged | 14.41<br>(3,230) | 1.39<br>(202) |
| | Aged | 28.0<br>(6,310) | 2.71<br>(395) |
| AC-20b (Unmodified) | Unaged | 17.4<br>(3,925) | 1.70<br>(246) |
| | Aged | 23.5<br>(5,280) | 2.28<br>(331) |
| AC-20a + LDPE | Unaged | 18.5<br>(4,160) | 1.81<br>(262) |
| | Aged | 19.4<br>(4,370) | 1.89<br>(274) |
| AC-20b + LDPE | Unaged | 21.5<br>(4,840) | 2.10<br>(304) |
| | Aged | 17.7<br>(3,980) | 1.72<br>(250) |
| AC-20a + RSBS | Unaged | 17.2<br>(3,870) | 1.68<br>(243) |
| | Aged | 20.1<br>(4,530) | 1.95<br>(283) |
| AC-20b + RSBS | Unaged | 19.7<br>(4,430) | 1.92<br>(278) |
| | Aged | 11.2<br>(2,540) | 1.09<br>(158) |
| AC-20a + SBR | Unaged | 15.4<br>(3,470) | 1.51<br>(219) |
| | Aged | 29.2<br>(6,560) | 2.83<br>(411) |
| AC-20b + SBR | Unaged | 17.9<br>(4,038) | 1.74<br>(253) |
| | Aged | 28.0<br>(6,290) | 2.72<br>(394) |
| AC-20a + MCR | Unaged | 13.5<br>(3,030) | 1.32<br>(191) |
| | Aged | 16.9<br>(3,800) | 1.63<br>(237) |
| AC-20b + MCR | Unaged | 17.2<br>(3,870) | 1.68<br>(243) |
| | Aged | 9.4<br>(2,110) | 0.90<br>(130) |

As one would expect, strength decreased as the testing temperature increased from 25 °C (77 °F) to 40 °C (104 °F). In the unaged condition, there was little difference between the mixtures produced with AC-20a as the base asphalt and those produced with AC-20b. Additionally, there was little difference when comparing the unaged modified and unaged unmodified

**Table 30**
**Indirect Tensile Test Data at 40 °C (104 °F)**

| HMA Mixture | Condition | Maximum Load KN (lb) | Tensile Strength MPa (psi) |
|---|---|---|---|
| AC-20a (Unmodified) | Unaged | 2.8<br>(620) | 2.68<br>(39) |
| | Aged | 15.5<br>(3,480) | 1.50<br>(218) |
| AC-20b (Unmodified) | Unaged | 3.0<br>(686) | 0.30<br>(43) |
| | Aged | 12.4<br>(2,800) | 1.21<br>(175) |
| AC-20a + LDPE | Unaged | 5.4<br>(1,210) | 0.52<br>(76) |
| | Aged | 11.7<br>(2,640) | 1.14<br>(166) |
| AC-20b + LDPE | Unaged | 6.6<br>(1,480) | 0.64<br>(93) |
| | Aged | 9.6<br>(2,160) | 0.94<br>(136) |
| AC-20a + RSBS | Unaged | 4.3<br>(970) | 0.42<br>(61) |
| | Aged | 14.2<br>(3,200) | 1.39<br>(201) |
| AC-20b + RSBS | Unaged | 4.9<br>(1,110) | 1.48<br>(70) |
| | Aged | 7.6<br>(1,700) | 0.73<br>(106) |
| AC-20a + SBR | Unaged | 4.7<br>(1,060) | 0.46<br>(67) |
| | Aged | 15.9<br>(3,580) | 1.55<br>(225) |
| AC-20b + SBR | Unaged | 5.3<br>(1,190) | 0.52<br>(75) |
| | Aged | 16.5<br>(3,720) | 1.61<br>(234) |
| AC-20a + MCR | Unaged | 3.2<br>(730) | 0.32<br>(46) |
| | Aged | 13.4<br>(3,020) | 1.31<br>(190) |
| AC-20b + MCR | Unaged | 4.0<br>(900) | 0.39<br>(56) |
| | Aged | 5.2<br>(1,160) | 0.48<br>(70) |

HMA. However, the LDPE-modified mixtures were consistently the strongest by a small margin.

The aging of the HMA had variable effects on the indirect tensile strength. In general, strength increased with aging. The unmodified HMA and those modified with SBR experienced the largest increases in strength. At 40 °C (104 °F); however, aging caused several AC-20b modified HMA to lose

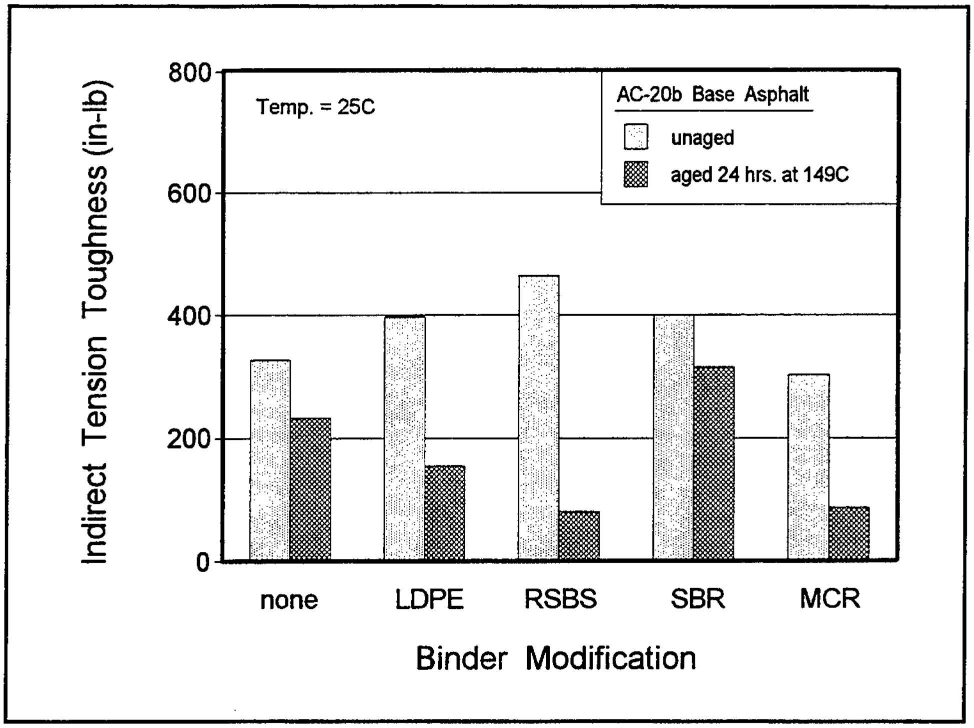

Figure 30. Indirect tensile toughness (in.-lb) of AC-20b mixtures at 25 °C (77 °F)

strength. These mixtures included the LDPE, RSBS, and MCR modified HMA.

Indirect tension tests at 40 °C (104 °F) reinforced the differences between the two base asphalt cements (Figures 31 and 32). All the mixtures containing AC-20a increased in toughness with aging. The unmodified mixture and the mixture containing SBR demonstrated particularly large increases in toughness. For HMA containing AC-20b, only two mixtures experienced increased toughness with aging: the unmodified mixture and the mixture modified with SBR.

**Confined repeated load-deformation properties.** The confined repeated load-deformation test was conducted to evaluate the HMA performance and change in performance versus age under repeated loading. The confined repeated load-deformation test has been determined to provide better evaluations of the rutting potential of HMA as compared to traditional static load creep tests. A test temperature of 60 °C (140 °F) was selected to simulate the typical maximum in-service pavement temperature.

Since the mixtures containing source asphalt AC-20b appeared to be more affected by the aging conditions, confined repeated load tests were performed on these mixtures in both unaged and aged conditions. In the unaged state, both the unmodified and modified mixtures proved to be resistant to permanent deformation. This was a result of the high-quality crushed aggregates, in combination with the high compactive effort (75-blow) mixture design. Permanent

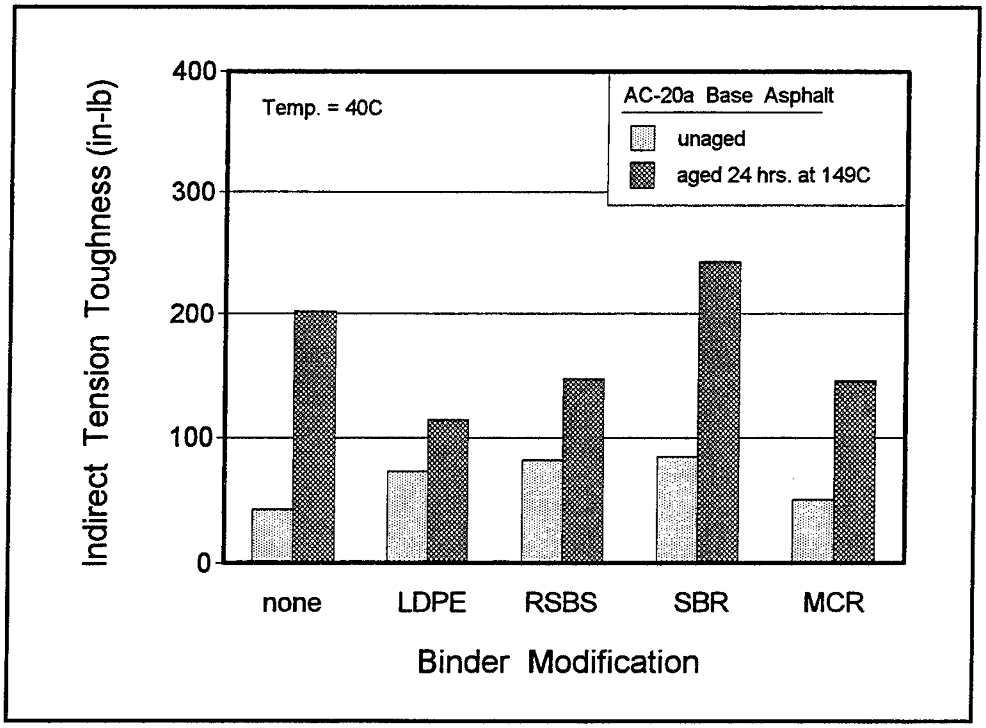

Figure 31. Indirect tensile toughness (in.-lb) of AC-20a mixtures at 40 °C (104 °F)

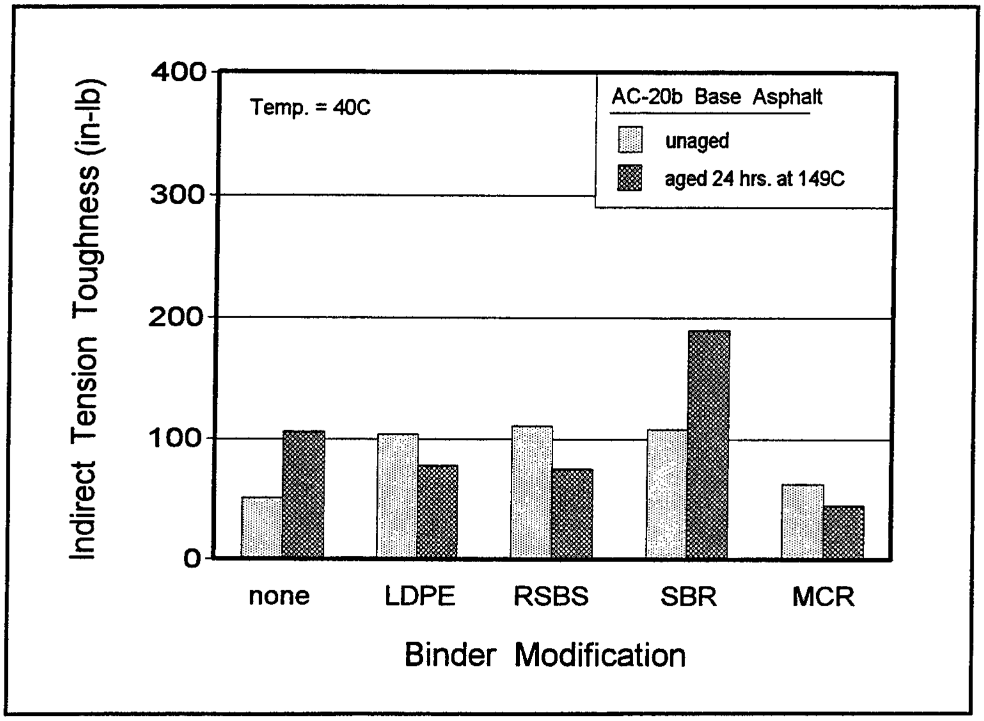

Figure 32. Indirect tensile toughness (in.-lb) of AC-20b mixtures at 40 °C (104 °F)

strains at the end of the test ranged from 1.0 percent to 1.5 percent as shown in Table 31. Generally, for the imposed test conditions, mixtures that suffer permanent strains less than 2 percent are considered favorable. Since all the mixtures were resistant to rutting in the unaged state, any increases in stiffness caused by aging were viewed as potentially detrimental. These increases in stiffness would be directly related to stiffening of the binders during aging. Creep stiffness values for mixtures containing source asphalt AC-20b are shown in Figure 33. Increases in stiffness with aging were maintained at only 50 to 60 percent with SBR and RSBS modifiers, respectively. However, increases in creep stiffness were much greater for the unmodified mixture (290 percent), the LDPE-modified mixture (190 percent), and the MCR-modified mixture (100 percent).

### Table 31
### Summarized Confined Repeated-Load Data

| HMA Mixture | Condition | Permanent Strain[1] (%) | Creep Stiffness[2] MPa (psi) |
|---|---|---|---|
| AC-20b | Unaged | 1.50 | 91.7<br>(13,300) |
| | Aged | 0.35 | 397.8<br>(57,700) |
| AC-20b + LDPE | Unaged | 1.10 | 126.2<br>(18,300) |
| | Aged | 0.34 | 405.4<br>(58,800) |
| AC-20b + RSBS | Unaged | 1.10 | 125.5<br>(18,200) |
| | Aged | 0.36 | 379.2<br>(55,000) |
| AC-20b + SBR | Unaged | 1.10 | 126.2<br>(18,300) |
| | Aged | 0.54 | 253.7<br>(36,800) |
| AC-20b + MCR | Unaged | 1.00 | 137.2<br>(19,900) |
| | Aged | 0.31 | 444.7<br>(64,500) |

[1] After recovery.
[2] Calculated using deviator stress.

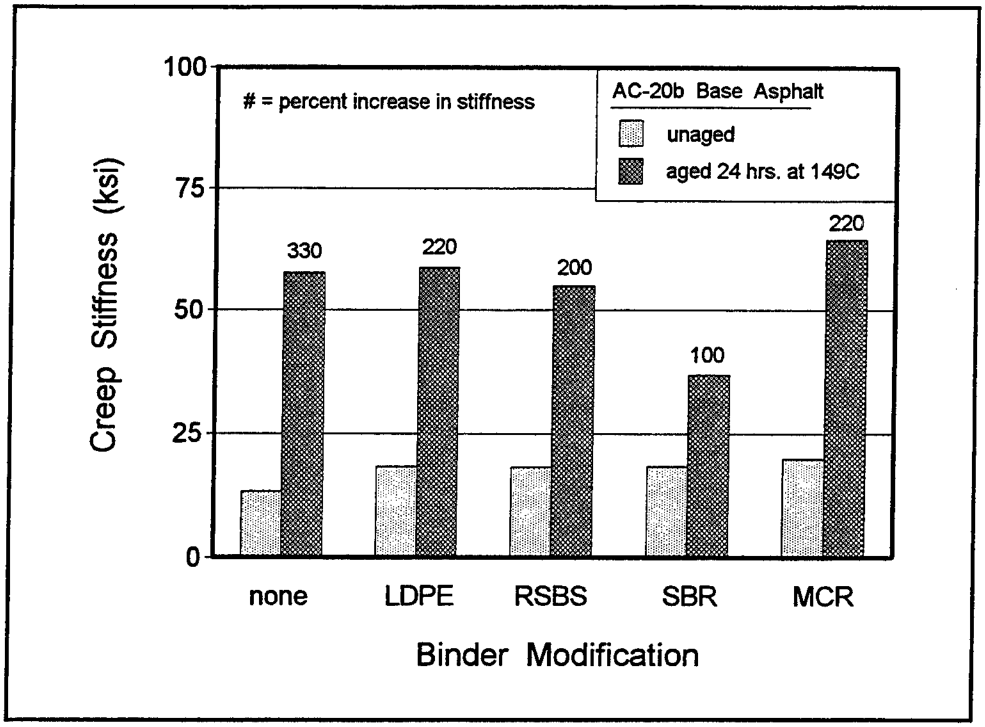

Figure 33. Creep stiffness test results

# 6 Conclusions

## General

The USDA has a considerable investment in pavement structures. The DA pavement structure consists of approximately 676,000,000 sq yd of pavement and the annual expenditures to maintain these pavements is approximately $124 million. The total quantity of pavement and the annual maintenance expenditures combined with the decrease in high quality materials for paving, the deterioration of the Nation's infrastructure, and the Nation's current environmental awareness delineate the need for research that can: determine applications of new pavement materials in military roads, airfields, and other paved surfaces; determine the degree to which waste materials can be effectively used in pavements, and determine performance-based specification requirements that can be used to simplify the specification process allowing the integration of new materials and technologies into military pavement construction projects.

The Material Utilization in Military Pavement Systems (MUMPS) project focused on several facets of the needed research. These included review of both new and waste materials with potential for use as modifiers, additives, and/or fillers in HMA pavements; durability issues; asphalt binder testing; and HMA mixture testing. Three other items were initiated in Fiscal Year 1995, but were not completed due to time constraints, funding constraints, or both (i.e., the reduction in scope and funding from the initial proposal and the segmentation of funding). These three areas were the procurement of SHRP test equipment, evaluation of numerical analysis methods to predict HMA and modified-HMA field performance, and laboratory evaluation techniques that could be used as direct input into numerical analysis procedures.

The major conclusions from the MUMPS research project are:

*a.* Research is required to quantify pavement performance improvements resulting from binder modification.

*b.* A simplified process (as compared to the current process used in military construction) to evaluate materials and technologies is required to allow the use of new and innovative materials and technologies in military pavement construction projects.

*c.* The predominant concern in relation to military airfield HMA pavements is durability related distresses. Cracking due to the effects of aging and thermal extremes are of particular concern.

*d.* Before a waste material or modifier is added to an HMA, an evaluation of the effects of the added material must be made based on engineering, economic, and environmental factors. In addition, the incorporation of a waste material or modifier into a HMA pavement must not adversely affect the performance of the pavement and preferably should enhance pavement performance.

*e.* The recyclability of an HMA pavement containing a waste material or modifier must be determined before allowing widespread use of that particular material in HMA.

*f.* Conventional binder and HMA tests have limited usefulness when evaluating modified binders and modified HMA due to their empiricism. Rules of thumb for unmodified binders and HMA, which have developed over many years of experience, have limited applicability to HMA that contain modifiers or waste products.

*g.* The highway industry is adopting SHRP binder testing criteria for use in HMA construction projects. This criteria may not be directly applicable to airfield pavements because of their unique load requirements. In the past, highway mix design methods and criteria have required modification before they could be used for airfield applications.

*h.* The gyratory test machine can be used to evaluate mixture strength and compactibility. The gyratory test machine was particularly useful for comparing unaged and aged mixtures due to its ability to compact to a designated unit weight. Aging of mixtures in a forced-draft oven was efficient and severe.

*i.* The combination of repeated-load deformation tests and dynamic testing in the linear viscoelastic region of the HMA may provide the data required for numerical analysis of the HMA material response to various aging and loading conditions.

*j.* The modifiers exhibited large differences in terms of their effects on HMA properties after aging. Aging caused some of modified mixtures to become dry and crumbly. Similar to low-temperature properties, the effects of aging were largely dependent on the base asphalt cement.

More specific conclusions related to each aspect of the MUMPS research effort are provided in the following sections.

# Modifiers, Additives, and Fillers for HMA Pavements

## Waste materials and by-products

Society produces ever increasing quantities of nondecaying waste products. As a result, there is a need to find productive uses for these waste materials. A number of the waste materials exhibit a potential for use as a construction material, especially in HMA pavements. However, there are areas that must be investigated before a waste material can be used in HMA construction.

The conclusions from the literature review on the use of waste materials and by-products are:

*a.* The paving industry has received increasing pressure to incorporate waste materials into pavement structures. Some of this pressure has come in the form of State and Federal legislation.

*b.* Most of the State DOT have investigated to some extent the applicability of various waste materials in pavement structures.

*c.* The majority of research concerning the use of waste materials has been conducted in the laboratory with limited field test sections.

*d.* The addition of a waste material to the HMA generally increases the initial cost of the HMA pavement and the benefits to the performance of the pavement have been variable at best (i.e., under one set of conditions a waste-modified pavement has performed much better than the control section while in another situation, the addition of the waste material appeared to adversely affect performance). Most reported improvements have been with respect to increased permanent deformation resistance.

*e.* The performance of a pavement containing a waste material will be dependent on the binder, the waste material, the actual traffic, the environment to which the modified HMA pavement is exposed, and any chemical/physical interaction of the waste material with constituents of the HMA.

*f.* The addition of a waste material to an HMA must be evaluated on a case-by-case basis. The ultimate decision to include a waste material in an HMA must be made based on engineering, economic, and environmental factors.

*g.* The ability to recycle an HMA pavement containing a waste material must be determined before allowing widespread use of that particular waste material in HMA.

*h.* The incorporation of a waste material into an HMA pavement must not adversely affect the performance of the pavement and preferably should enhance pavement performance.

### New or manufactured materials

New or manufactured materials are generally designed, produced, and sold as a method of improving some aspect of a binder material and thereby improving some characteristic of an HMA pavement. The modifier market is dynamic, with new manufacturers appearing frequently and experienced manufacturers adjusting their formulations to refine modifier performance. This active environment makes the development of field data concerning these modifiers difficult. Therefore, modifiers are often selected based on laboratory tests.

The conclusions based on the literature review concerning new or manufactured materials include:

*a.* The effects of modification are dependent on the modifier used and the base asphalt binder being modified. Therefore, specifications should be based on performance criteria of the modified-binder combination rather than on the modifier alone.

*b.* Current HMA mixture designs do not allow the engineer to take advantage of performance improvements realized from modification.

*c.* Considerable phase separation can occur during modified binder storage, resulting in increased variability during modified HMA placement. Additionally, some modified HMA mixtures have exhibited a tendency to tear behind the paver during placement and have a greater tendency to stick to the wheels of the roller during compaction.

*d.* The addition of modifiers to an HMA generally increase the initial cost of the HMA; therefore, life cycle costs must be determined and evaluated before using a modifier.

*e.* The recyclability of modified HMA pavements must be determined. If the HMA pavement cannot be effectively recycled at the end of the useful life, then the use of modifiers must be questioned.

*f.* Typically, the addition of polymers or modifiers have been reported to improve the high-temperature performance of asphalt binders and HMA pavements, i.e., improved rut resistance. Generally, modifiers have been less successful addressing distresses related to low-temperature properties and moisture susceptibility.

## Asphalt Durability Issues

Durability-related distresses were the predominant pavement distress area associated with airfield pavements (as determined by the MUMPS Tri-Service Selection Committee and the WES Airfield Evaluation database). Therefore, these types of distresses were given the highest priority in the MUMPS project. Load related distresses could not be completely dismissed from consideration; however, it is possible that as a binder and HMA mixture are modified to resist durability related distresses they will become more susceptible to load-related distresses.

The conclusions from the literature review on asphalt durability issues are:

*a.* The predominant mode of distress in airfield pavements is durability related. This is somewhat reversed from highway pavements where the predominant distress mode or concern is generally load related (i.e., rutting).

*b.* There are several factors that can result in durability-related cracking: use of an inappropriate asphalt grading or type, HMA mixing temperature at the plant, HMA temperature during the paving operation, filler type and/or content, compaction techniques, and climatological factors.

*c.* One of the most predominant causes of durability-related distresses is the aging process that occurs in the asphalt binder and the HMA pavement.

*d.* The SHRP research estimated that the binder contributes or is responsible for approximately 60 percent of fatigue related distresses and approximately 85 percent of thermal-related distresses. Therefore, binder improvements exhibited in fatigue and thermal distress resistance should translate into HMA pavement improvements.

*e.* Binder durability analysis has typically been conducted using chemical analysis or physical property testing. Physical property or material characterization type tests were selected for the MUMPS program because the ultimate goal of the research is to achieve performance related criteria. As a result, the chemical constituents of the modified binder are not necessarily important as long as the physical properties and changes in physical properties are adequately characterized.

*f.* SHRP testing criteria, which were developed for the highway industry, may not be directly applicable to airfield pavements because of their unique load requirements. In the past, highway mix design methods and criteria have required modification before they could be used for airfield applications.

*g.* Field test sections will be required to verify any laboratory performance criteria for airfield pavements.

## Binder Testing

The properties of the binder material have been estimated to have the greatest effect on durability related distresses. Therefore, any improvements derived through modification of the binder should ultimately improve the properties of the HMA mixture, assuming that the modified binder remains homogeneous during mixing and placement of the HMA.

The MUMPS research investigated the effects of four different modifiers (RSBS, SBR, MCR, and LDPE) on the physical characteristics of two asphalt cements (AC-20a and AC-20b). The conclusions derived from the binder testing include:

*a.* Conventional asphalt-cement tests were developed for neat asphalt cements, so their applicability to modified binders is questionable.

*b.* The effects of modification on the physical properties of the asphalt binder were dependent on both the modifier and the base asphalt cement.

*c.* The capillary tube viscometer could not be used to accurately measure the viscosity of the modified binders. This was expected from previous work at WES and from the literature review.

*d.* The modified binders exhibited variable ductility results. The LDPE imposed the most significant decrease in ductility, particularly after thin-film oven aging.

*e.* Penetration tests conducted at 25 °C (77 °F) indicated a decrease in penetration with the addition of the modifier in both the unaged and thin-film oven aged samples. The most significant decrease in penetration was exhibited by the LDPE-modified material while the SBR- and MCR-modified materials exhibited the least change from the control materials.

*f.* Penetration tests conducted at 4 °C (39 °F) exhibited higher variability than the 25 °C (77 °F) penetrations. The test did indicate that the low temperature properties of a modified binder appear to be controlled by the base asphalt cement. In general, the penetration of the AC-20a materials were higher than the AC-20b materials.

*g.* All of the modifiers reduced low-temperature binder stiffness except for LDPE with AC-20a. The effect of the modifiers on the BBR slope was mixed. Only RSBS and MCR increased the BBR slope of both asphalts at all temperatures studied. BBR slope values for blends with LDPE were lower than the base asphalt at all temperatures. The order of low-temperature properties as measured by BBR on aged binders was RSBS > SBR > MCR > LDPE.

*h.* All of the modifier binders exhibited improved modulus properties over the base asphalt at low-frequency rates of loading. The order of high-temperature modulus as measured by DSR was RSBS > LDPE > SBR > MCR.

*i.* The RSBS modifier was compatible with both asphalts. The SBR and MCR modifiers were more compatible with AC-20b than AC-20a. LDPE was not compatible with either asphalt.

*j.* The interaction and subsequent compatibility of the passive modifiers (MCR, SBR, and LDPE) with asphalt is dependent on the chemistry of the asphalt source. The compatibility of the reactive modifier RSBS was not dependent on the asphalt source for the two asphalts employed for this study.

## HMA Testing

It has been estimated that the properties of the binder have the greatest effect on durability related distresses. However, the aggregate in the HMA, the HMA properties, and the aggregate/binder interaction also contribute to the durability performance of the HMA pavement. This implies that the HMA mixture must also be appropriately characterized. Since durability issues were the primary concern of this research effort; the HMA mixtures were tested in both an unaged and aged condition. The aged testing allowed changes in physical properties to be determined and to determine how modification affected those changes.

The following conclusions were derived from the HMA testing:

*a.* Marshall testing indicated that the addition of the modifiers increased Marshall Stability and flow values. Typically, high flow values indicate that the mixture could be susceptible to rutting. However, the relationship between high flow values of a modified HMA and rutting has not been established.

*b.* One reported benefit of HMA modification has been that the asphalt content of the HMA can be increased slightly above optimum without significantly affecting the mechanical properties of the HMA. Estimated properties from the Marshall mix design at an asphalt content of 0.5 percent above optimum appeared to produce similar changes in both the unmodified and modified HMA. Increasing the binder content 0.5 percent above optimum reduced the voids total mix below the minimum 3 percent value specified in the DA requirements.

*c.* The number of gyratory test machine revolutions used to compact the aged modified HMA specimens to the target unit weight was significantly increased as compared to the unaged specimens. This indicates that storage of modified HMA in plant silos for periods of 24 hr or longer may significantly increase the required compactive effort in the field to obtain the appropriate densities. Mixtures containing RSBS or

MCR were particularly prone to increases in required compactive effort with aging.

*d.* Shear strength, as measured by the gyratory test machine, increased as each of the unmodified mixtures was aged. For each of the modified mixtures, shear strength decreased with aging. The unmodified binders seemed to be better able to retain their cohesive properties during aging.

*e.* In their unaged conditions and at 25 °C (77 °F), no tremendous differences were noted between the indirect tension toughness values for unmodified and modified mixtures. However, the RSBS-modified mixtures were toughest by a small margin, for both base asphalt cements. With aging, the unmodified AC-20a mixtures exhibited an increase in tensile toughness, while the unmodified AC-20b mixtures exhibited a decrease in tensile toughness. Among the modified mixtures, SBR was best able to promote tensile toughness after aging. The SBR-modified mixtures had the highest toughness values after aging, including all unmodified and modified mixtures.

*f.* In their unaged conditions and at 40 °C (104 °F), indirect tension toughness was consistently increased by modification with LDPE, RSBS, and SBR. With aging, the unmodified AC-20a mixtures exhibited a larger increase in tensile toughness than that exhibited by AC-20b. Among the modified mixtures, SBR was best able to promote tensile toughness after aging. Similar to the results at 25 °C (77 °F), the SBR-modified mixtures had the highest toughness values after aging, including all unmodified and modified mixtures.

*g.* The performance of all unaged mixtures in the repeated-load deformation test was favorable. This was a result of the high-quality aggregate and the HMA design for high tire pressures. Consequently, increases in stiffness upon aging were viewed as a measure of increases in binder rigidity, which could cause problems in terms of durability cracking. The mixture modified with SBR experienced the smallest increase in stiffness upon aging.

# 7 Recommendations

The original MUMPS program was proposed as a 3-year, $2.5 million per-year project. The original objectives of this project were to: determine the applicability of new pavement materials in military roads, airfields, and other paved areas; determine the degree to which waste (by-products) and recycled (i.e., ground tire and plastic) materials could be effectively used in military pavements (asphalt, concrete, and unsurfaced); evaluate requirements for performance-based specifications that would allow the use of new and waste materials in military pavement construction; and develop a simplified specification process that would allow the integration of new materials and technologies into military pavement construction projects. These objectives were to be achieved through a multifaceted project consisting of laboratory material characterization, numerical analysis, and field verification.

The scope and funding of the MUMPS project were significantly reduced to a 1-year project and $950K. As a result, the objectives were reduced to include: initiate a laboratory-based methodology that could quantify the improved resistance of HMA pavement to durability-related distresses; provide current practice information on the use of new and waste materials in HMA; and provide technical recommendations on the requirements to advance from the completion of this project phase to the original objectives.

As discussed in the previous chapter, the FY 95 MUMPS project verified the need for a methodology to quantify improved HMA pavement performance to ensure that the life-cycle cost of modified HMA pavements justifies the increase in initial cost. The FY 95 MUMPS project conclusions also indicate that the laboratory methodology used to evaluate materials for use in HMA pavements should include both binder testing and mixture testing. Based on the original MUMPS objectives, the reduced project objectives and the conclusions of the FY 95 MUMPS program, the recommendations from this research effort are:

*a.* Recyclability of modified HMA pavements is a major concern. Currently, unmodified HMA pavements can be recycled to produce a new HMA pavement. The addition of a modifier or waste material could create a situation in which the HMA could no longer be recycled. Therefore, research is required to determine if modified HMA pavements can be recycled. If they cannot be recycled, an additional cost may have to be added to the life-cycle cost.

*b.* Additional research is required to correlate the accelerated laboratory HMA aging techniques used in the MUMPS project to the aging of HMA pavements in the field. The HMA aging techniques used in the laboratory were severe and delineated differences in the modified HMA mixture.

*c.* The highway industry is adopting SHRP binder specifications which will eventually eliminate viscosity-graded asphalts. The criteria used in the SHRP binder specification is more performance-based than the current specifications. The SHRP criteria should be evaluated to determine if any modifications are required before being applied to airfield pavement projects. The evaluation must include field test sections to verify the criteria.

*d.* The combination of a modified repeated-load-deformation test and dynamic testing in the linear viscoelastic region of the HMA will provide the data required for numerical analysis. The numerical analysis could then be used to model pavement performance allowing life-cycle costs of specific modifiers to be compared for a specific set of conditions. Research should be continued to complete the development of these test procedures and numerical analysis should be initiated.

*e.* Two finite element programs were evaluated for potential use in modeling the pavement performance. It is recommended that both of these programs (PACE by SWK Pavement Engineering and ABAQUS by Hibbit, Karlsson, and Sorensen) be used to predict pavement performance. These programs will accept the data generated from the same characterizations. The use of both programs should increase confidence in the modeled results.

The results from the FY 95 MUMPS project indicate that it is possible to develop a procedure that can provide performance-related data on binder and HMA mixtures, thus allowing the use of new and/or waste materials in military pavement construction projects. To achieve the original objectives of the MUMPS project, a commitment must be made to a multiyear project which would include additional laboratory evaluations and more importantly, field evaluations. The development and verification of performance-related criteria must be made with actual field data, not assumed field conditions.

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# Appendix A
# Conventional Binder Tests

## Table A1
## Conventional Binder Tests for Unmodified AC-20a

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.95 min. |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 635 (335) min. |
| Specific gravity at 25 °C (77 °F) | -- | 1.033 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | 2,155 |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | 414 min. |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 19 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 89 min. |
| Ductility at 25 °C (77 °F), cm | -- | 150+ |
| Softening point, °C (°F) | -- | 124 (51.1) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | -0.005 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | 4,500 |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 15 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 37 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 150+ |
| Softening point, °C (°F) | -- | 128 (53.5) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A2
## Conventional Binder Tests for Unmodified AC-20b

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.9 min. |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 603 (317) min. |
| Specific gravid at 25 °C (77 °F) | -- | 1.026 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | 1,468 |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | 328 min. |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 9 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 63 min. |
| Ductility at 25 °C (77 °F), cm | -- | 150+ |
| Softening point, °C (°F) | -- | 115 (46.1) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.14 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | 3,888 |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 9 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 29 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 150+ |
| Softening point, °C (°F) | -- | 127 (52.5) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A3
## Conventional Binder Tests for AC-20a Modified with 5.5 % LDPE

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 95.96 min. |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 662 (350) min. |
| Specific gravid at 25 °C (77 °F) | -- | 1.033 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 8 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 35 |
| Ductility at 25 °C (77 °F), cm | -- | 24 |
| Softening point, °C (°F) | -- | 131 (55) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.015 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 12 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 24 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 12 |
| Softening point, °C (°F) | -- | 131 (55) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A4
## Conventional Binder Tests for AC-20a Modified with 5 % RSBS

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.77 min. |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 617 (325) |
| Specific gravid at 25 °C (77 °F) | -- | 1.032 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 15 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 45 |
| Ductility at 25 °C (77 °F), cm | -- | 100 |
| Softening point, °C (°F) | -- | 153 (67) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | -0.03 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 23 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 38 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 80 |
| Softening point, °C (°F) | -- | 142 (61) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A5
## Conventional Binder Tests for AC-20a Modified with 5 % SBR

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.73 |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 608 (320) |
| Specific gravid at 25 °C (77 °F) | -- | 1.024 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 15 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 54 |
| Ductility at 25 °C (77 °F), cm | -- | 150+ |
| Softening point, °C (°F) | -- | 138 (59) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.503 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 16 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 41 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 150+ |
| Softening point, °C (°F) | -- | 143 (61.5) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A6
## Conventional Binder Tests for AC-20a Modified with 5 % MCR

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.85 |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 581 (305) |
| Specific gravity at 25 °C (77 °F) | -- | 1.043 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 20 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 63 |
| Ductility at 25 °C (77 °F), cm | -- | 150+ |
| Softening point, °C (°F) | -- | 127 (53) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.14 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 21 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 45 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 68 |
| Softening point, °C (°F) | -- | 129 (54) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A7
## Conventional Binder Tests for AC-20b Modified with 5.5 % LDPE

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 97.06 min. |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 554 (290) |
| Specific gravity at 25 °C (77 °F) | -- | 1.027 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 6 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 34 |
| Ductility at 25 °C (77 °F), cm | -- | 3 |
| Softening point, °C (°F) | -- | 127 (53) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.16 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 13 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 16 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 0 |
| Softening point, °C (°F) | -- | 216 (102) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A8
## Conventional Binder Tests for AC-20b Modified with 5 % RSBS

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.97 |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 549 (287) |
| Specific gravity at 25 °C (77 °F) | -- | 1.019 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 16 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 50 |
| Ductility at 25 °C (77 °F), cm | -- | 100 |
| Softening point, °C (°F) | -- | 131 (55) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.08 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 9 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 37 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 69 |
| Softening point, °C (°F) | -- | 144 (62) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

## Table A9
## Conventional Binder Tests for AC-20b Modified with 5 % SBR

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.92 |
| Flash point, Cleveland open cup, °F (°C) | 450 (232) min. | 590 (310) |
| Specific gravity at 25 °C (77 °F) | -- | 1.021 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 9 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 59 |
| Ductility at 25 °C (77 °F), cm | -- | 150+ |
| Softening point, °C (°F) | -- | 129 (54) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.27 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 14 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 38 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 150+ |
| Softening point, °C (°F) | -- | 135 (57) |

[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

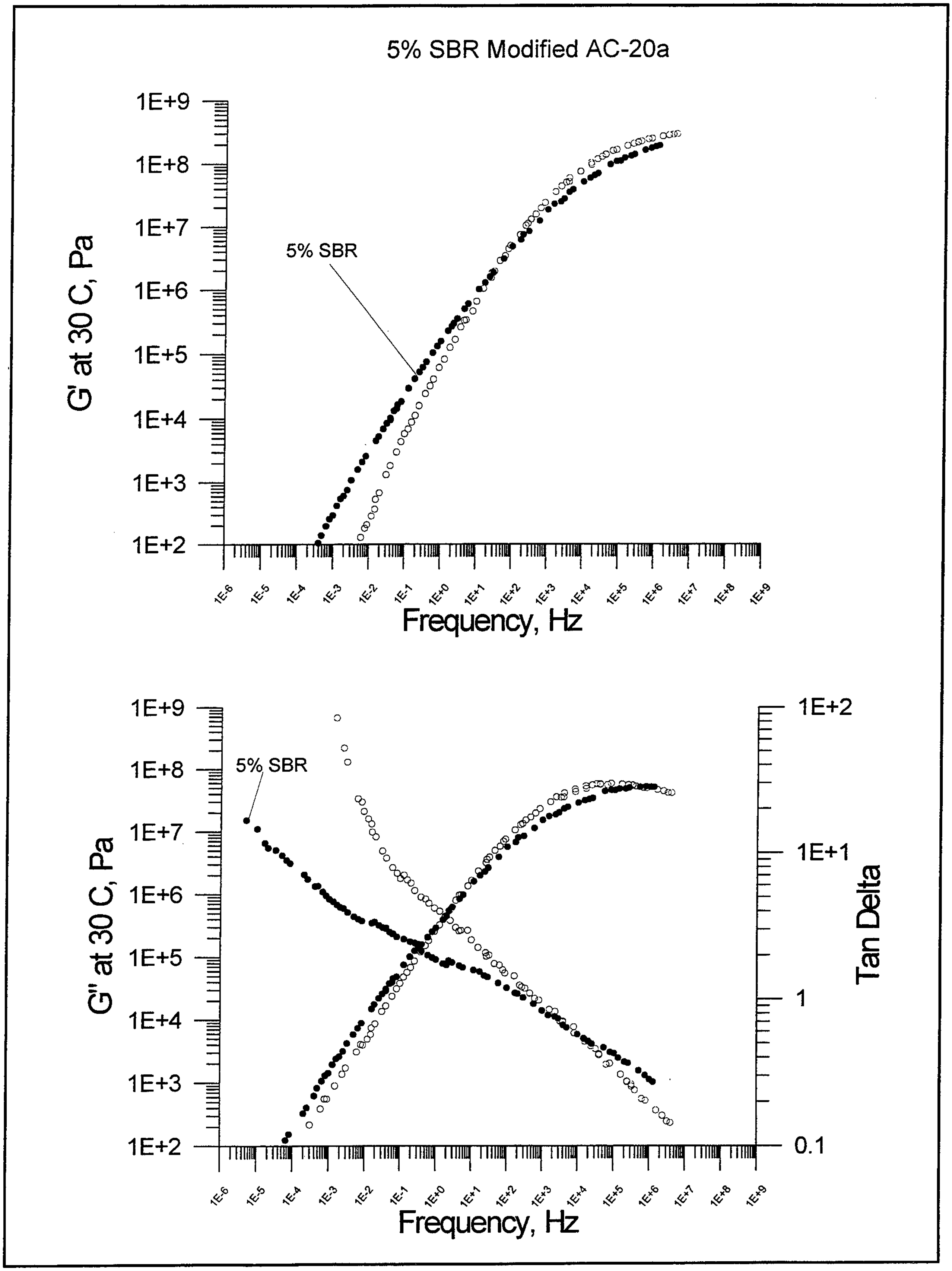
## Table A10
## Conventional Binder Tests for AC-20b Modified with 5 % MCR

| Material Conditioning/Test Procedure | Requirement[1] | Test Result |
|---|---|---|
| **Virgin Material** | | |
| Solubility in trichloroethylene, percent | 99.0 min. | 99.07 |
| Flash point, Cleveland open cup, °C (°F) | 450 (232) min. | 549 (287) |
| Specific gravity at 25 °C (77 °F) | -- | 1.042 |
| Absolute viscosity at 60 °C (140 °F), P | 2,000 ± 400 | -- |
| Kinematic viscosity at 135 °C (275 °F), cSt | 300 min. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 15 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | 60 min. | 60 |
| Ductility at 25 °C (77 °F), cm | -- | 145 |
| Softening point, °C (°F) | -- | 124 (51) |
| **Aged by Thin-Film Oven Test** | | |
| Mass loss, percent | -- | 0.20 |
| Absolute viscosity at 60 °C (140 °F), P | 10,000 max. | -- |
| Penetration (200 g, 60 s) at 4 °C (39.2 °F), 1/10 mm | -- | 15 |
| Penetration (100 g, 5 s) at 25 °C (77 °F), 1/10 mm | -- | 42 |
| Ductility at 25 °C (77 °F), cm | 50 min. | 121 |
| Softening point, °C (°F) | -- | 136 (58) |

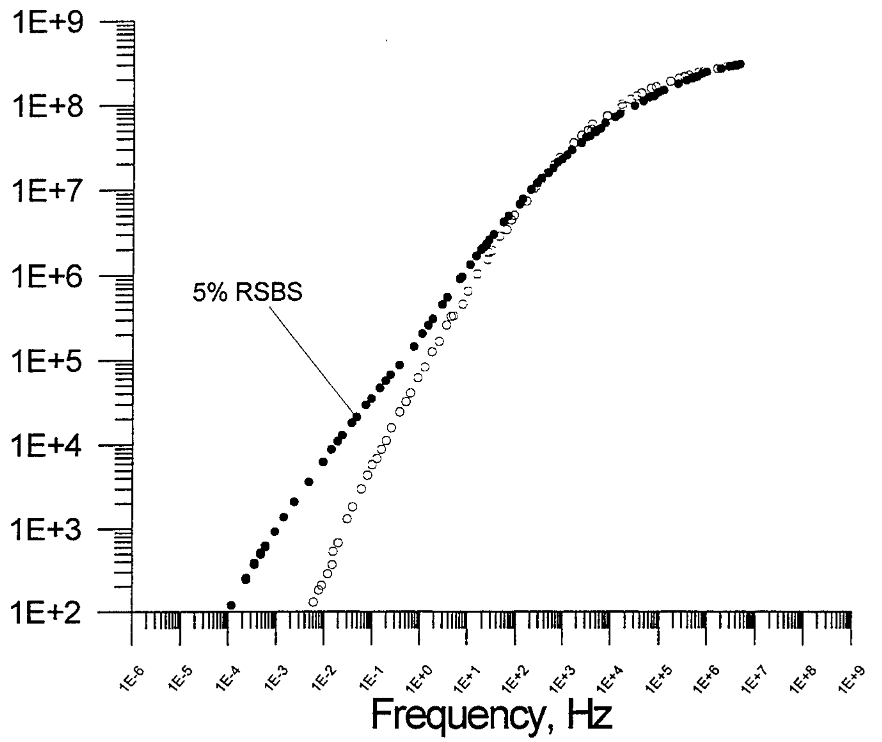
[1] Viscosity grade AC-20 (ASTM D 3381 (ASTM 1994e)).

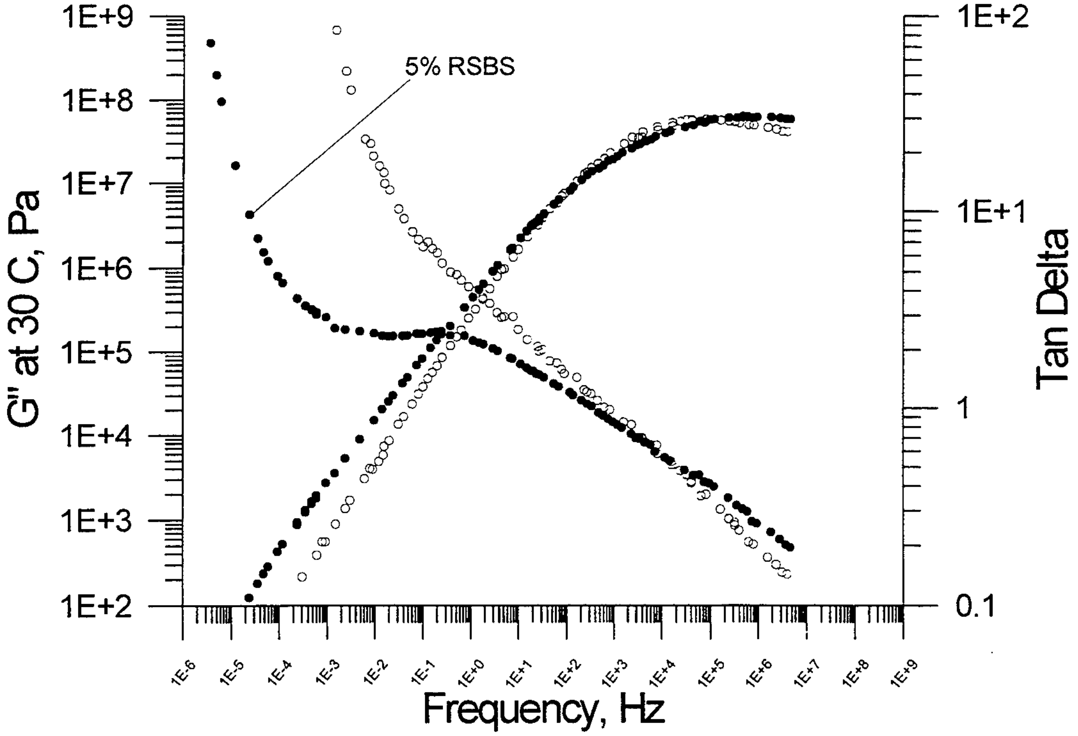
# Appendix B

# Dynamic Shear Rheometry Mastercurves of Virgin, Modified, and Aged Binders

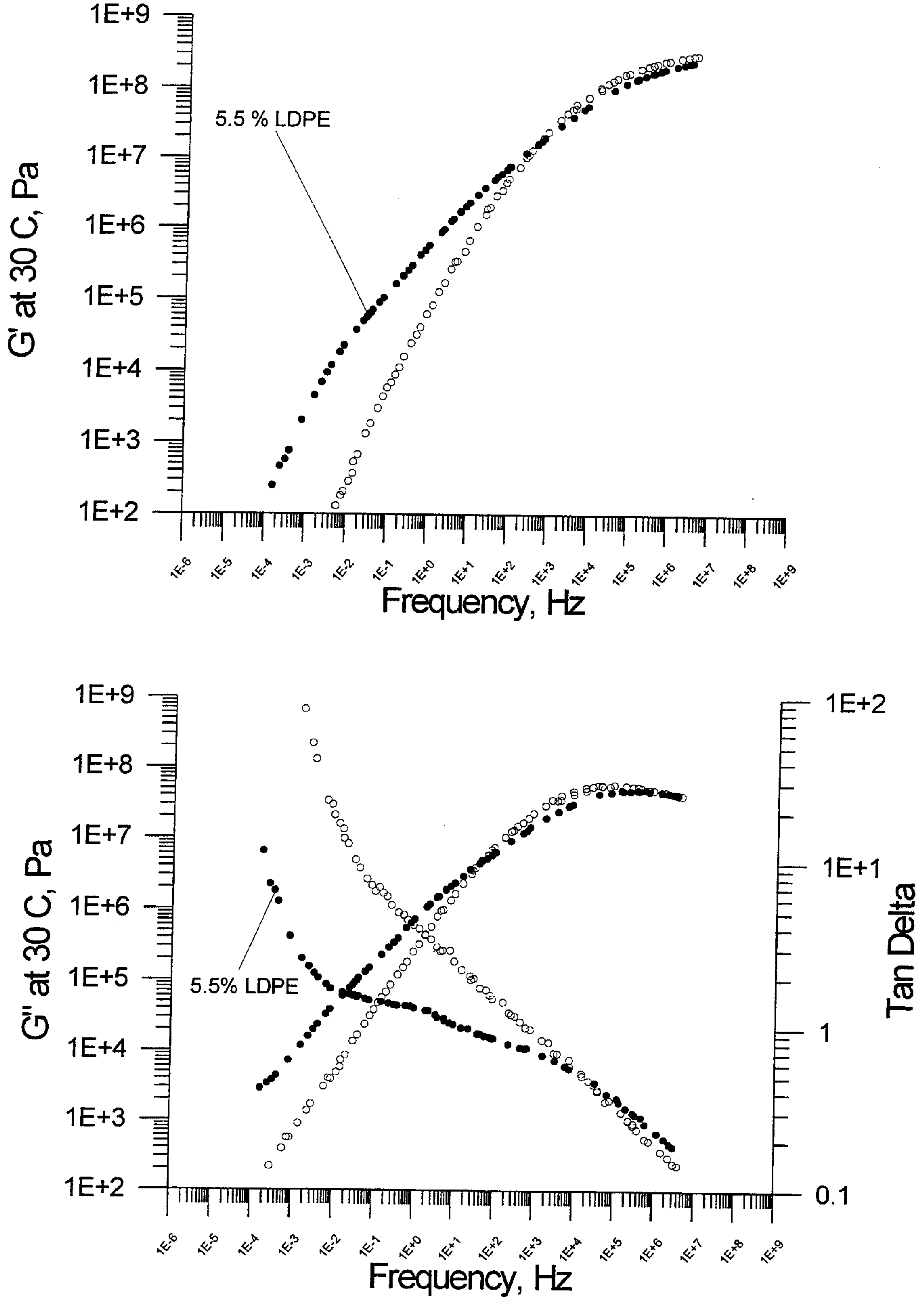

5% SBR Modified AC-20a
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
5% SBR
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz
G" at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
5% SBR
Tan Delta
1E+2
1E+1
1
0.1
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz

## 5% RSBS Modified AC-20a

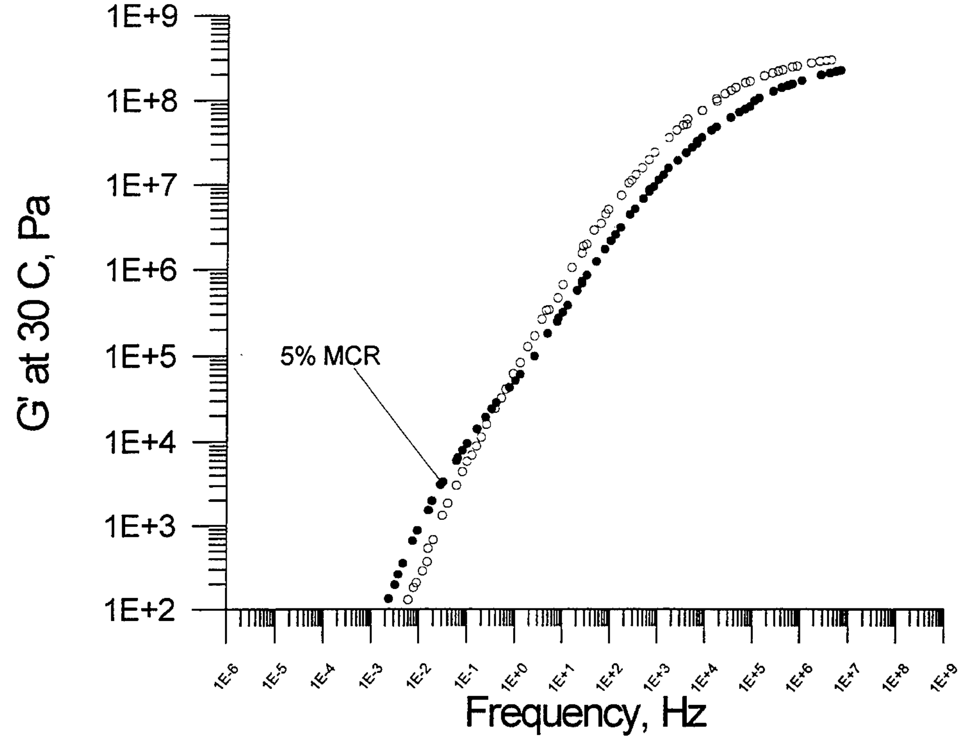


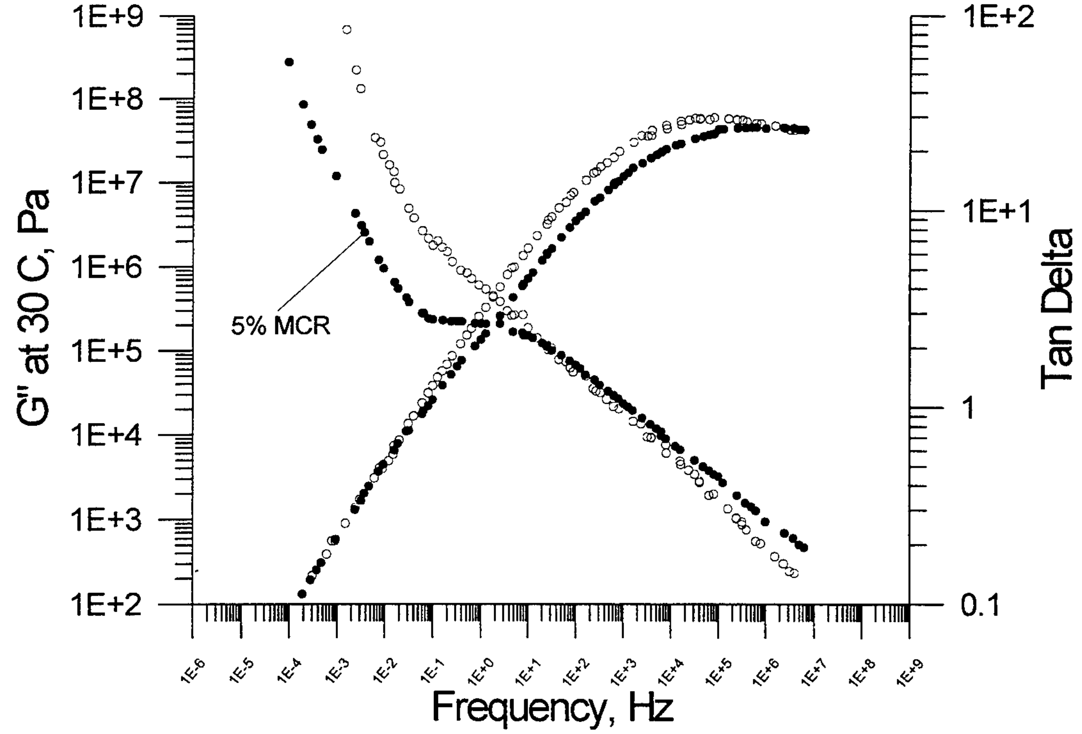


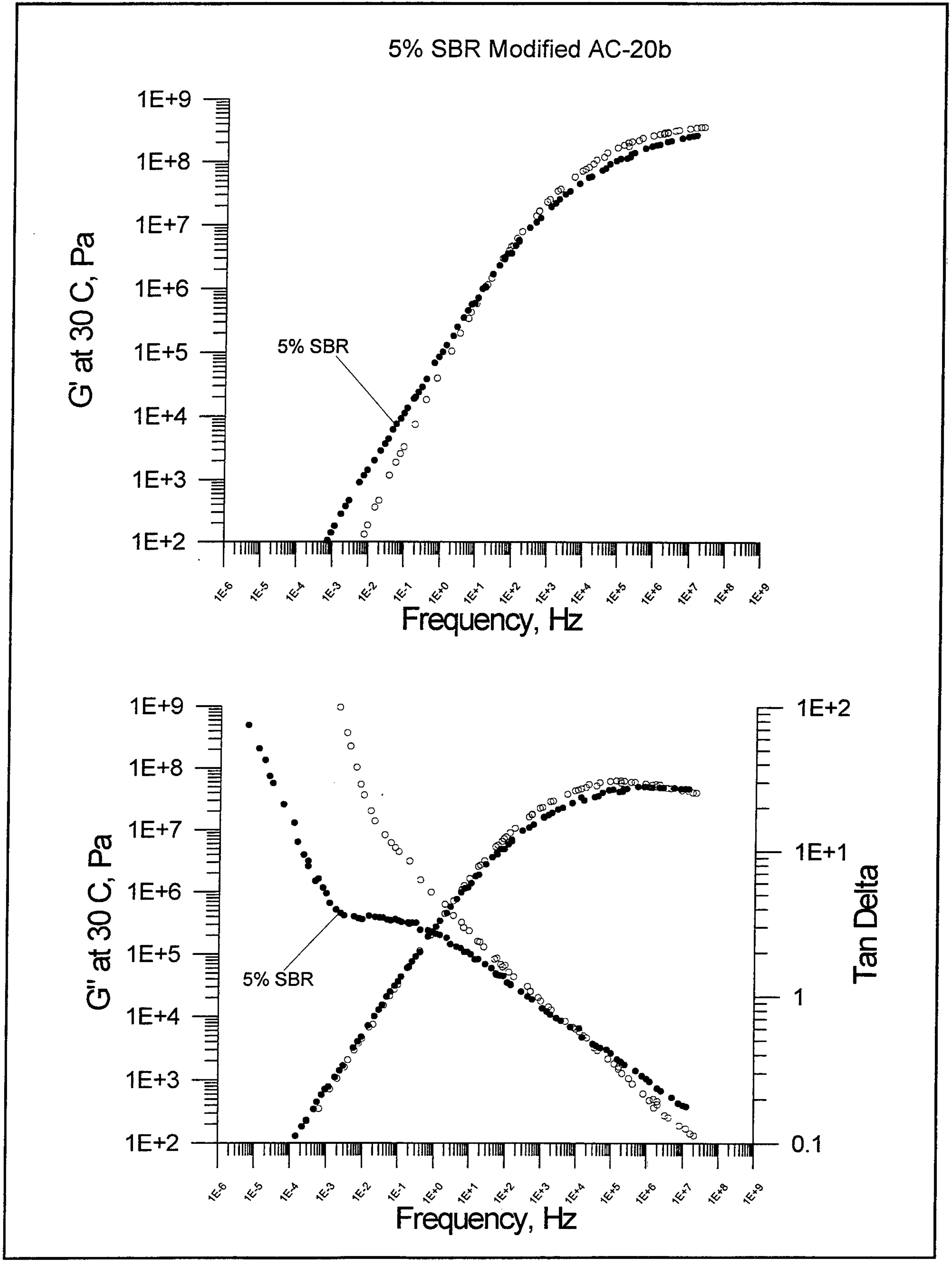
5.5% LDPE Modified AC-20a

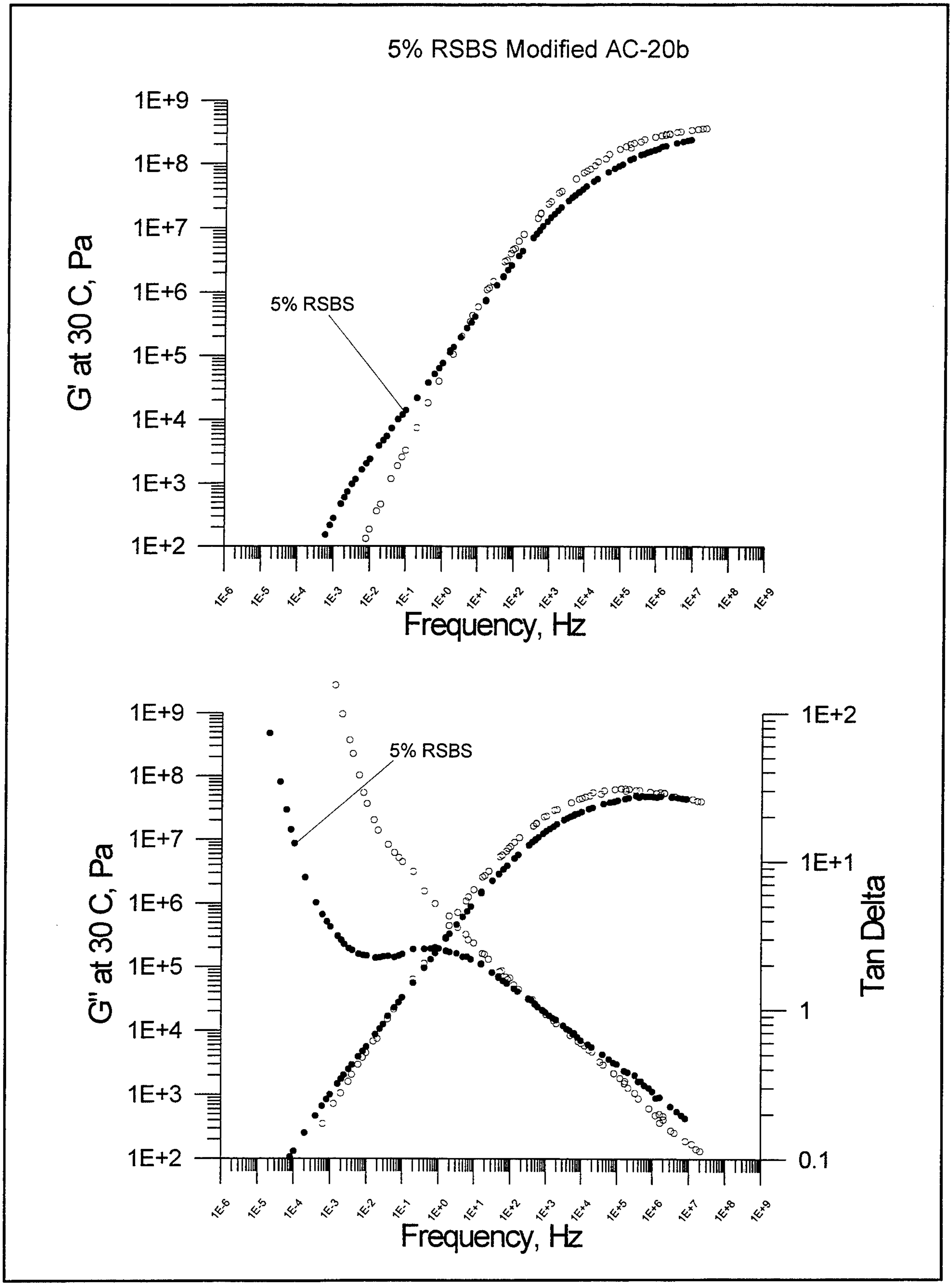


## MCR Modified AC-20a

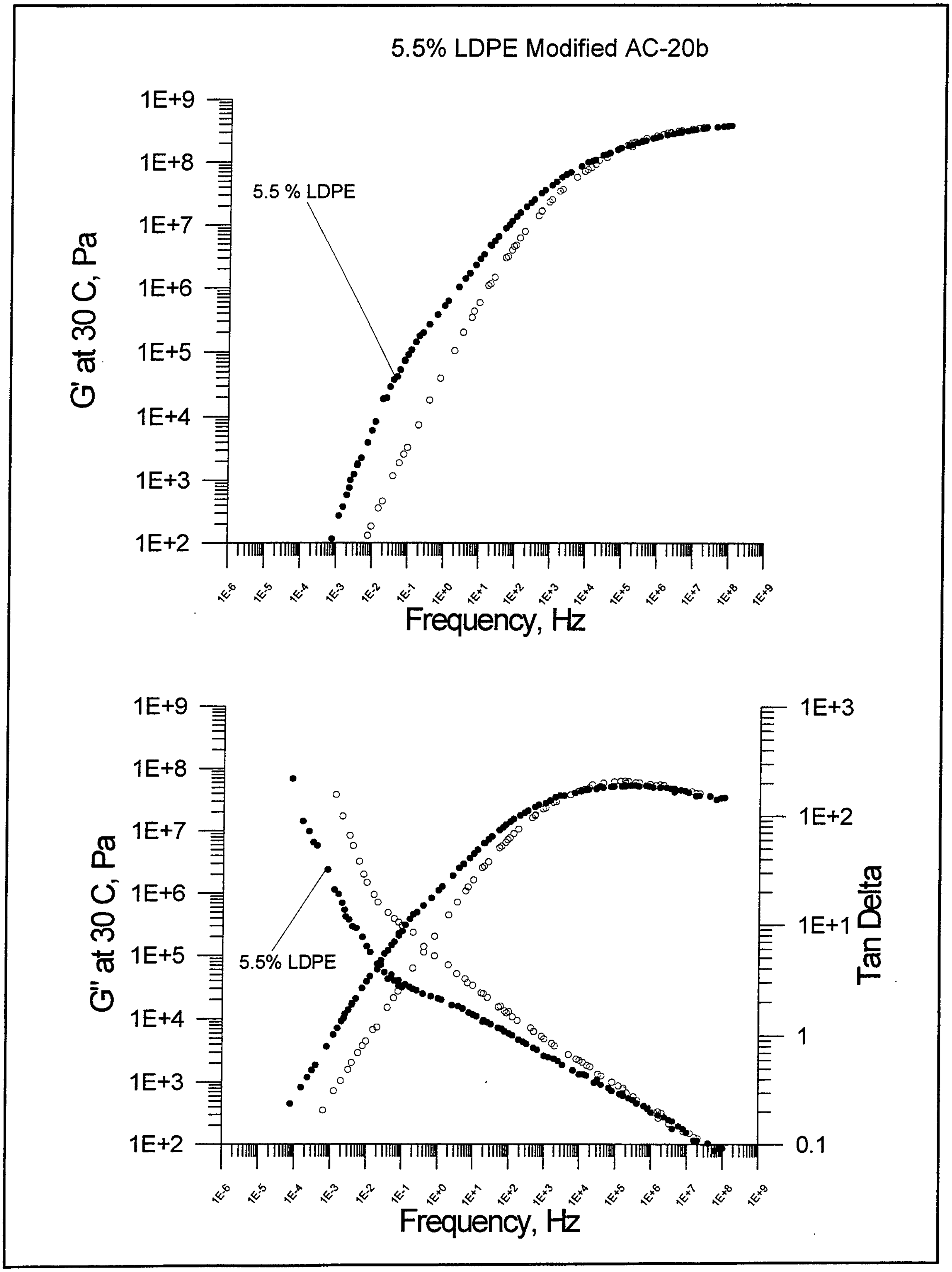





5% SBR Modified AC-20b

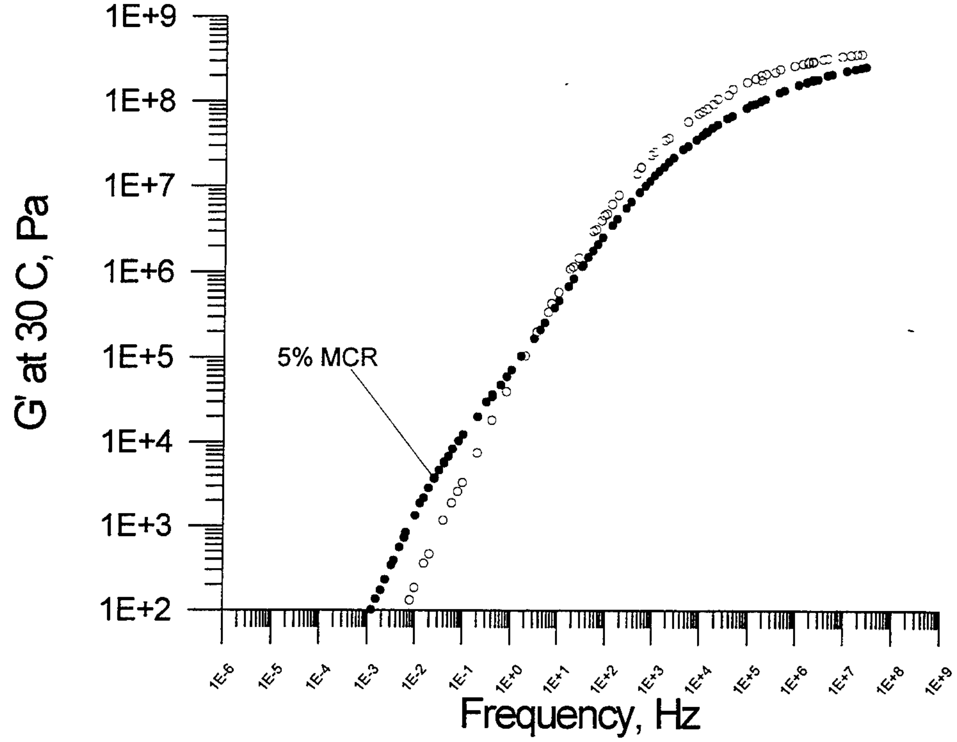


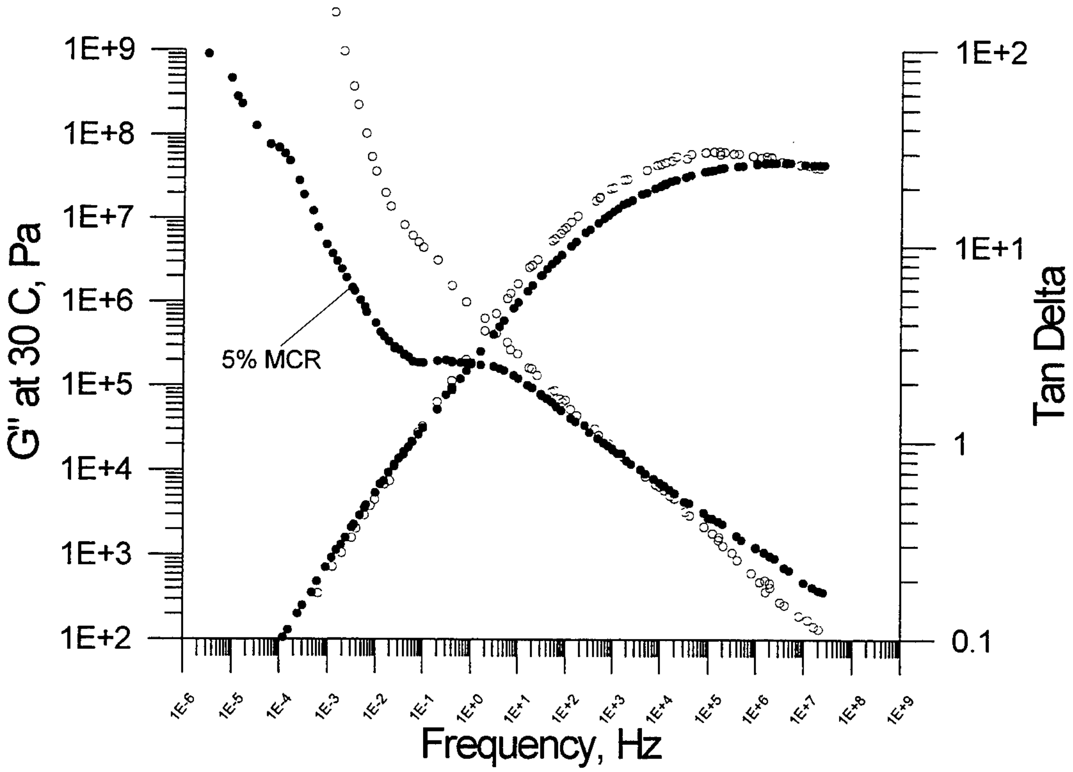
5% RSBS Modified AC-20b

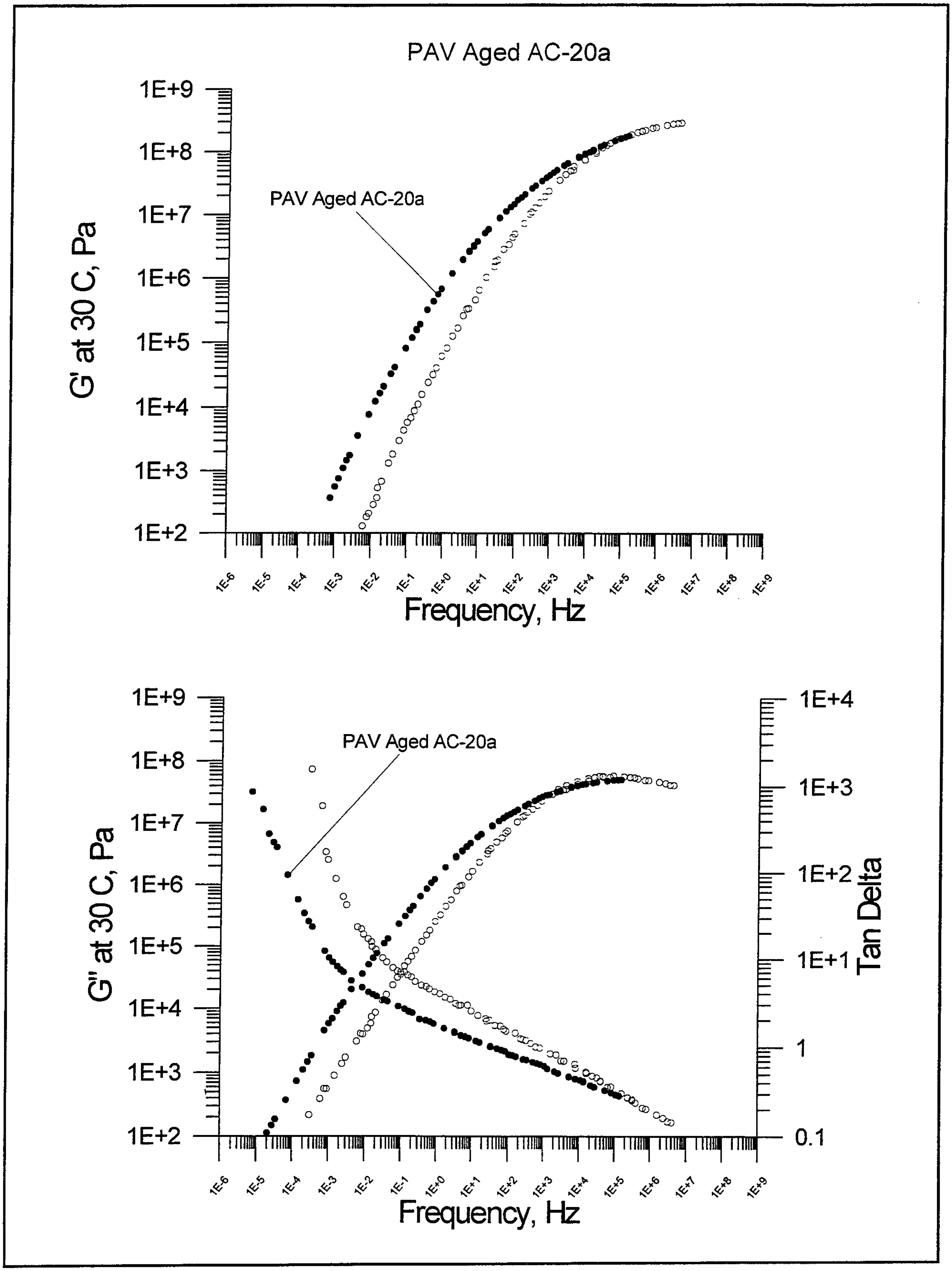


5.5% LDPE Modified AC-20b

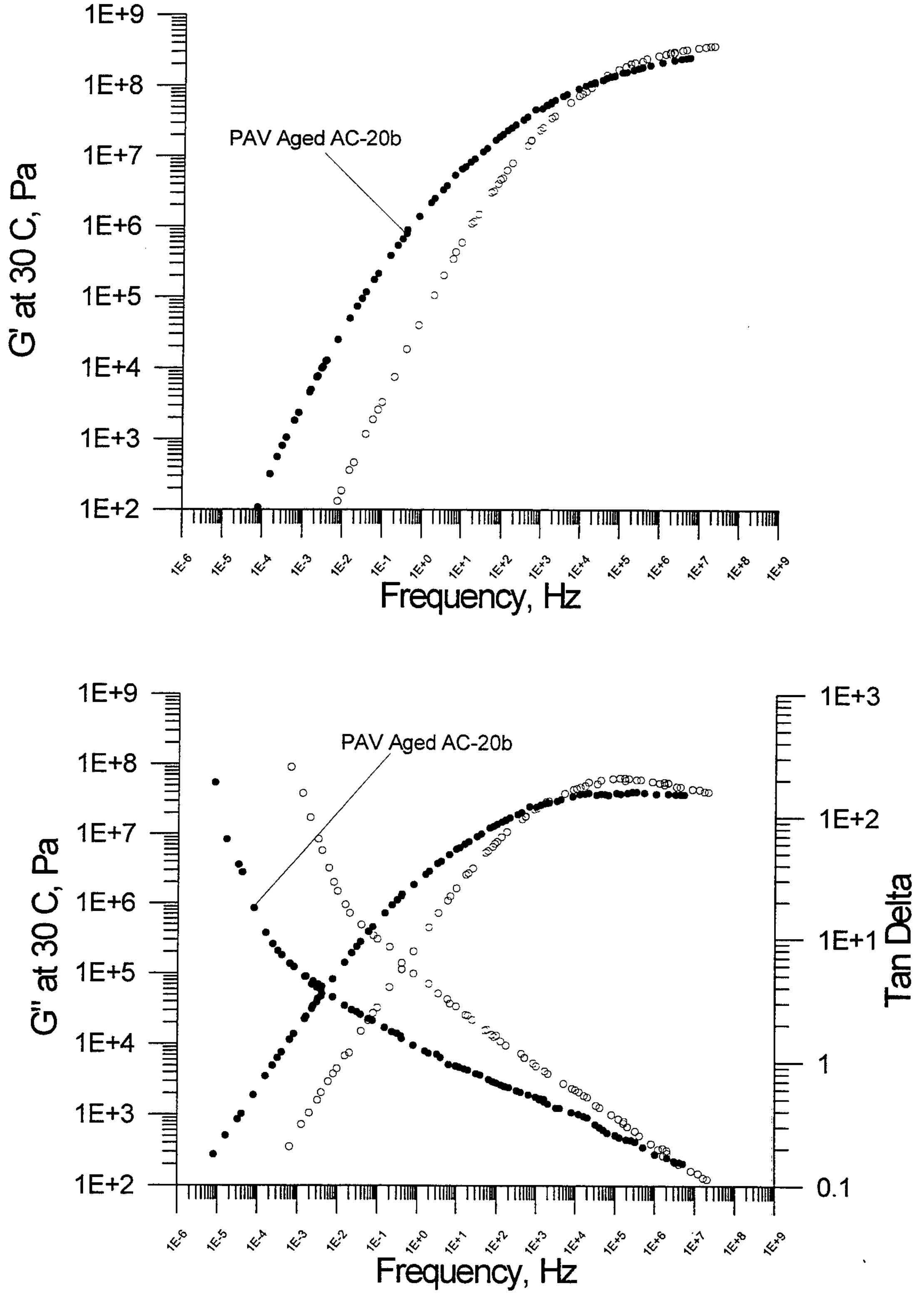
## MCR Modified AC-20b

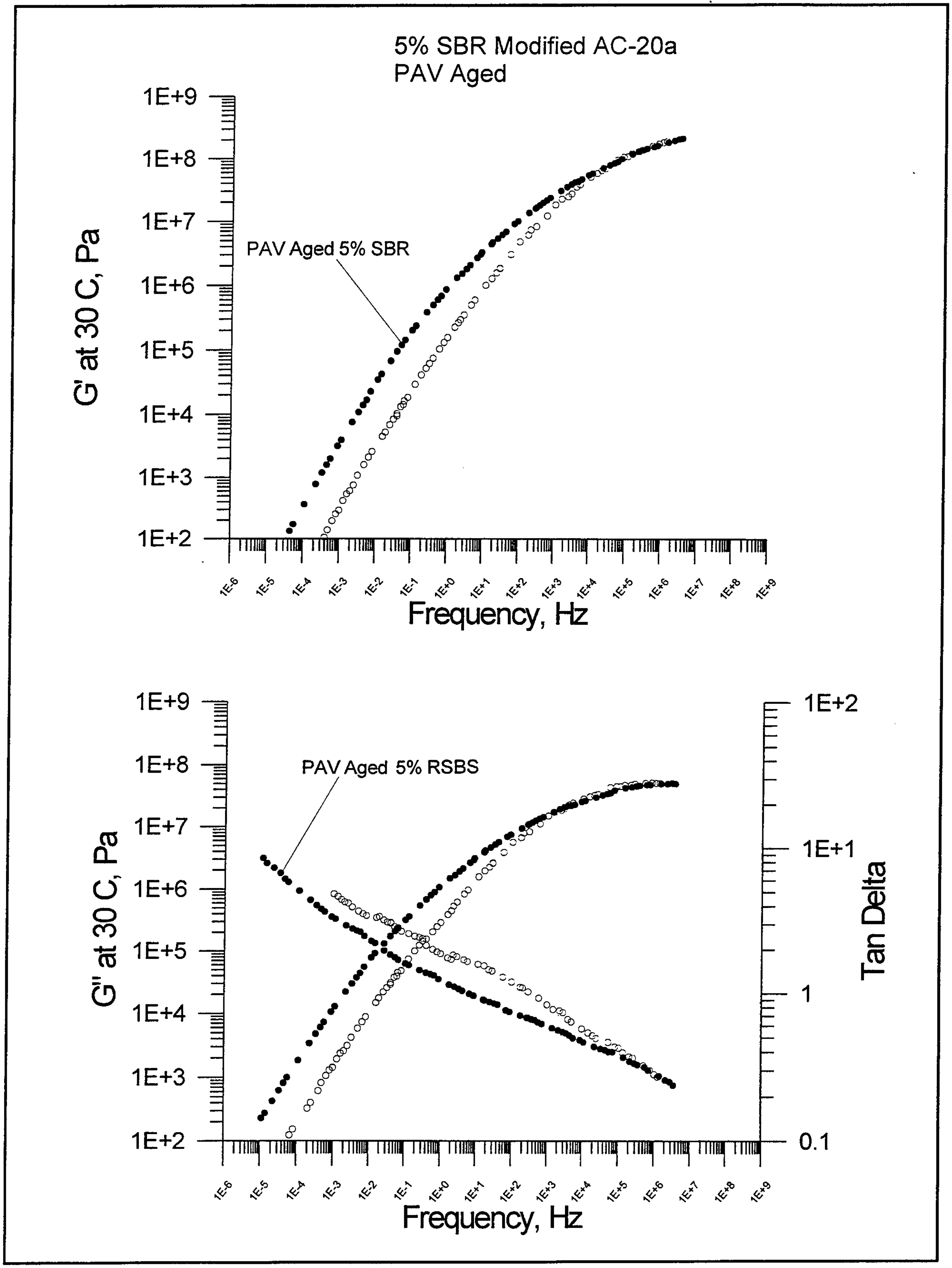




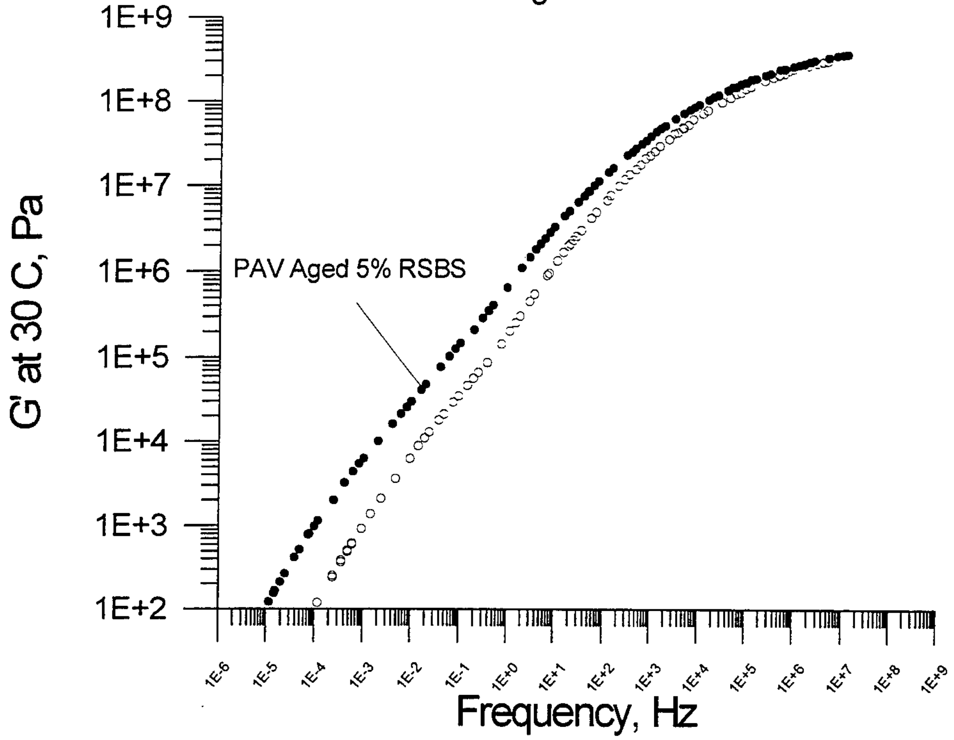

PAV Aged AC-20a
PAV Aged AC-20a
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz
PAV Aged AC-20a
G" at 30 C, Pa
Tan Delta
1E+4
1E+3
1E+2
1E+1
1
0.1
Frequency, Hz

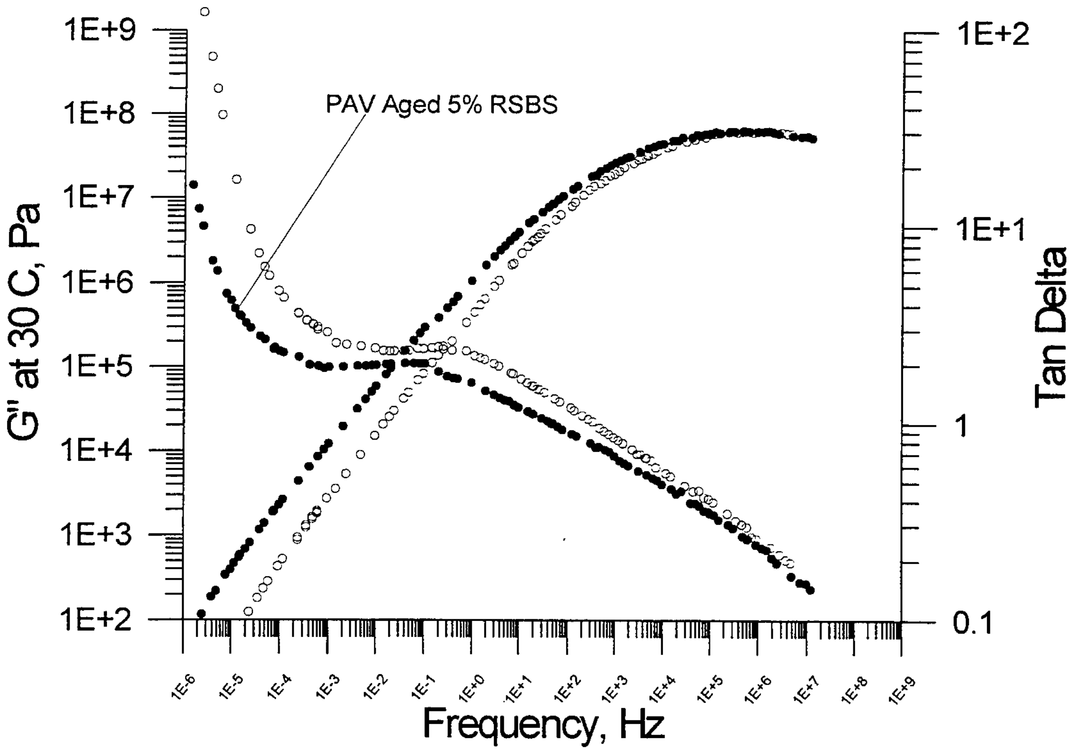
PAV Aged AC-20b

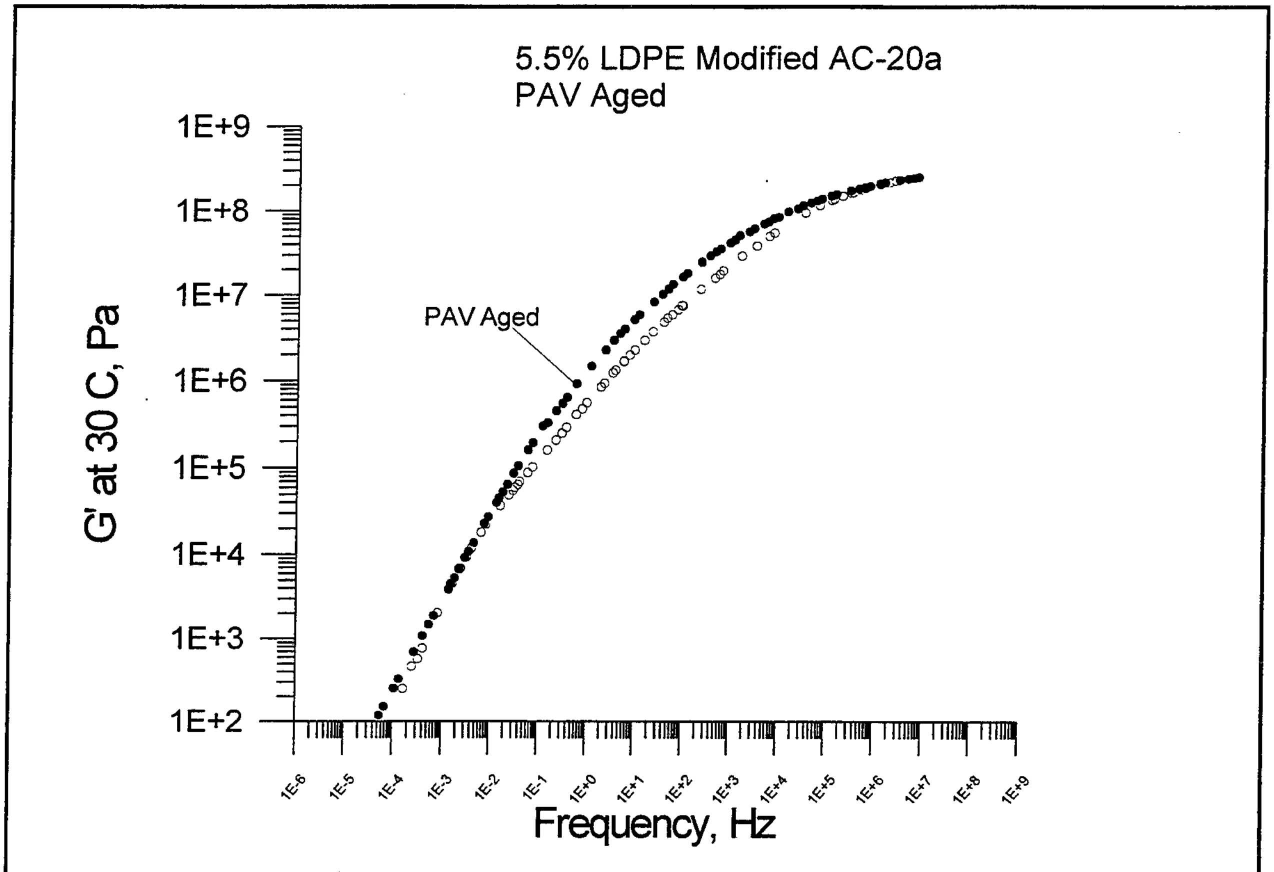


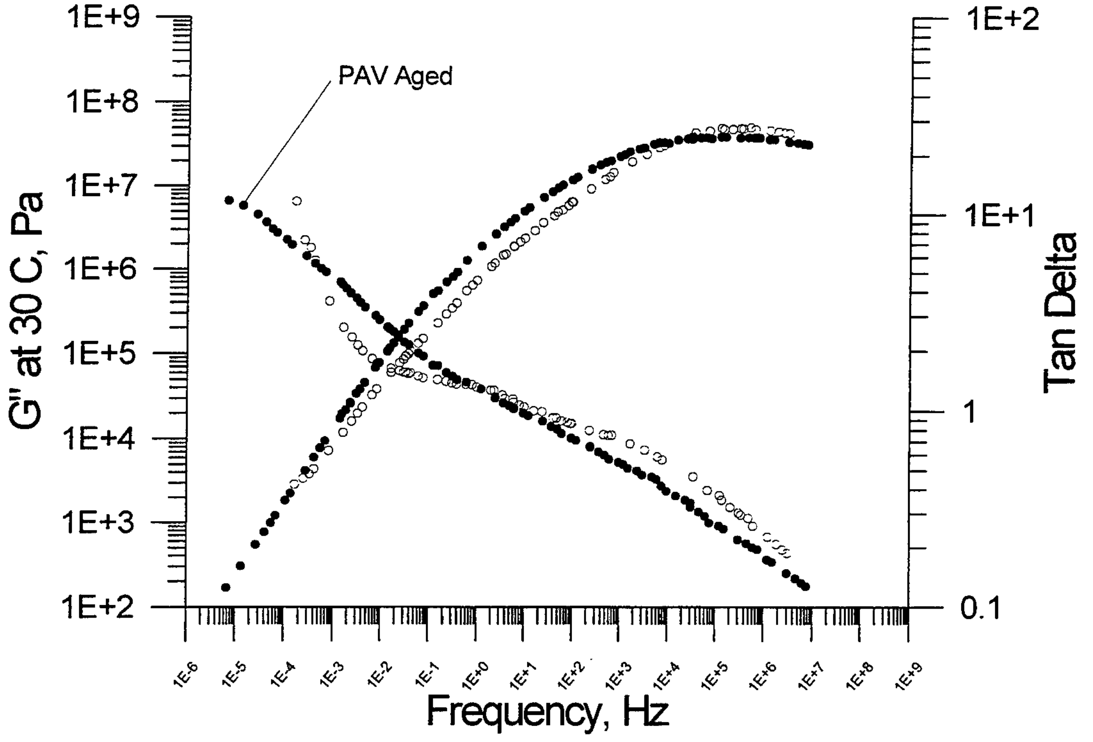

5% SBR Modified AC-20a
PAV Aged
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
PAV Aged 5% SBR
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz
G'' at 30 C, Pa
PAV Aged 5% RSBS
Tan Delta
1E+2
1E+1
1
0.1
Frequency, Hz

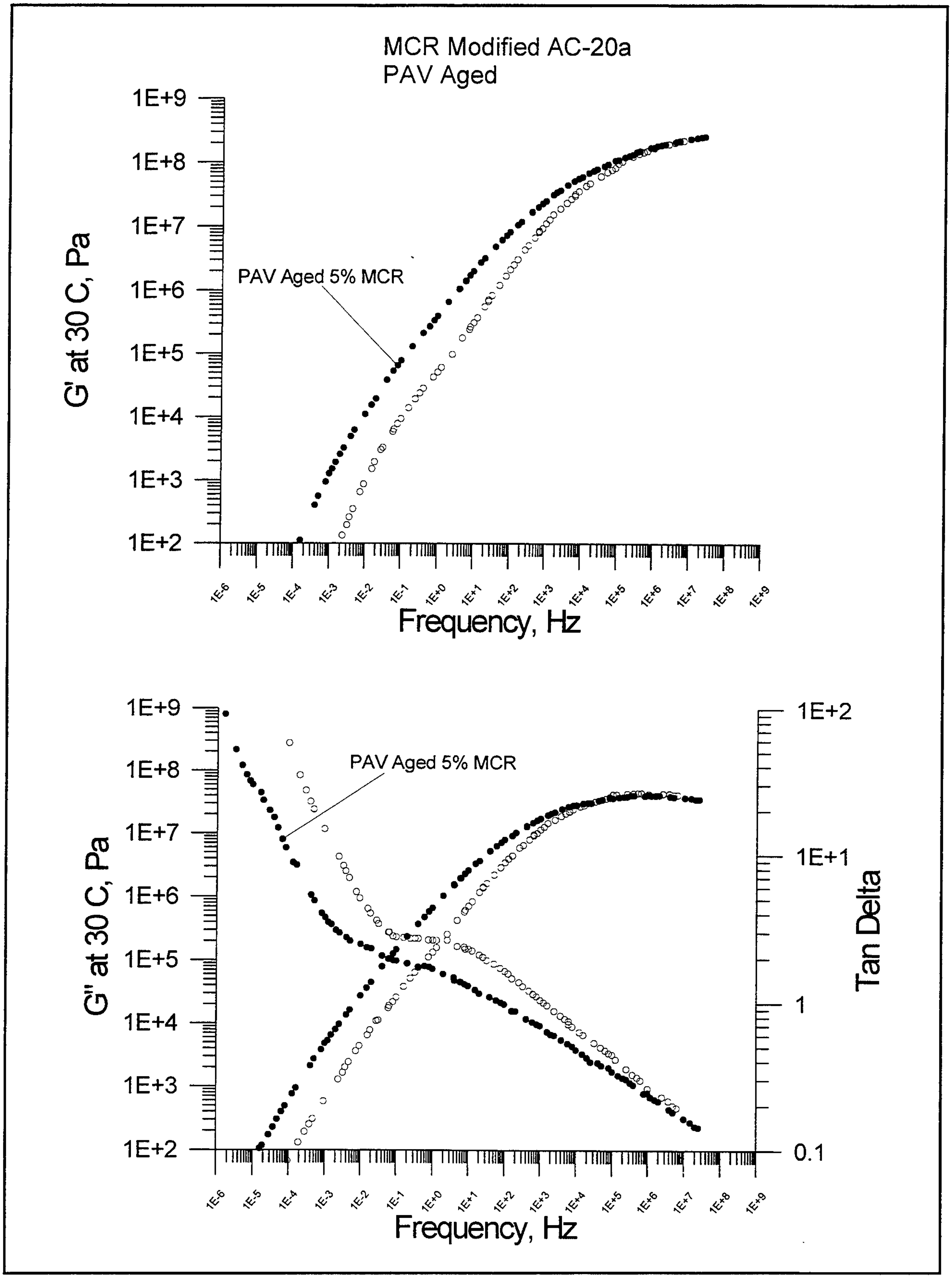
5% RSBS Modified AC-20a
PAV Aged

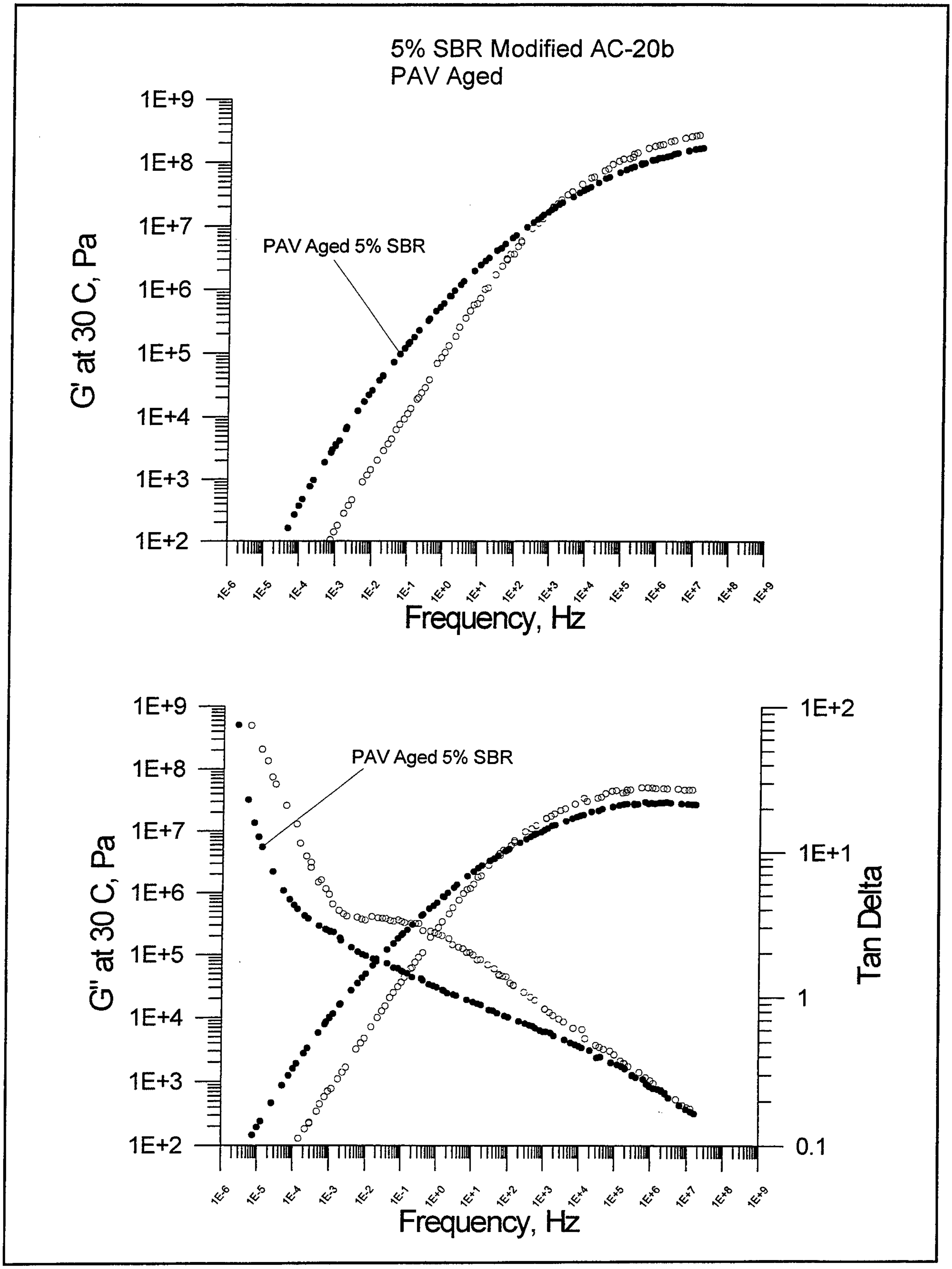


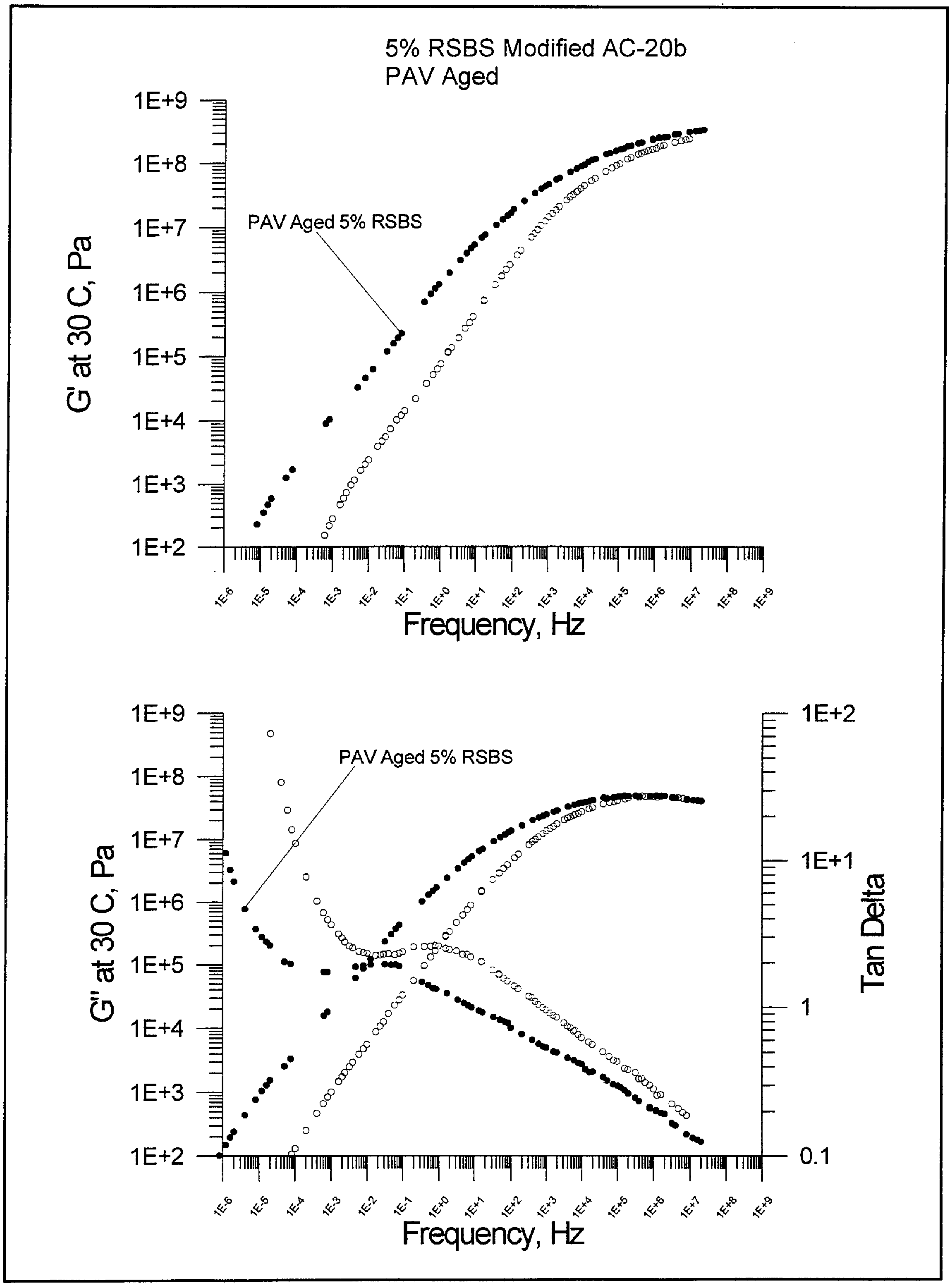


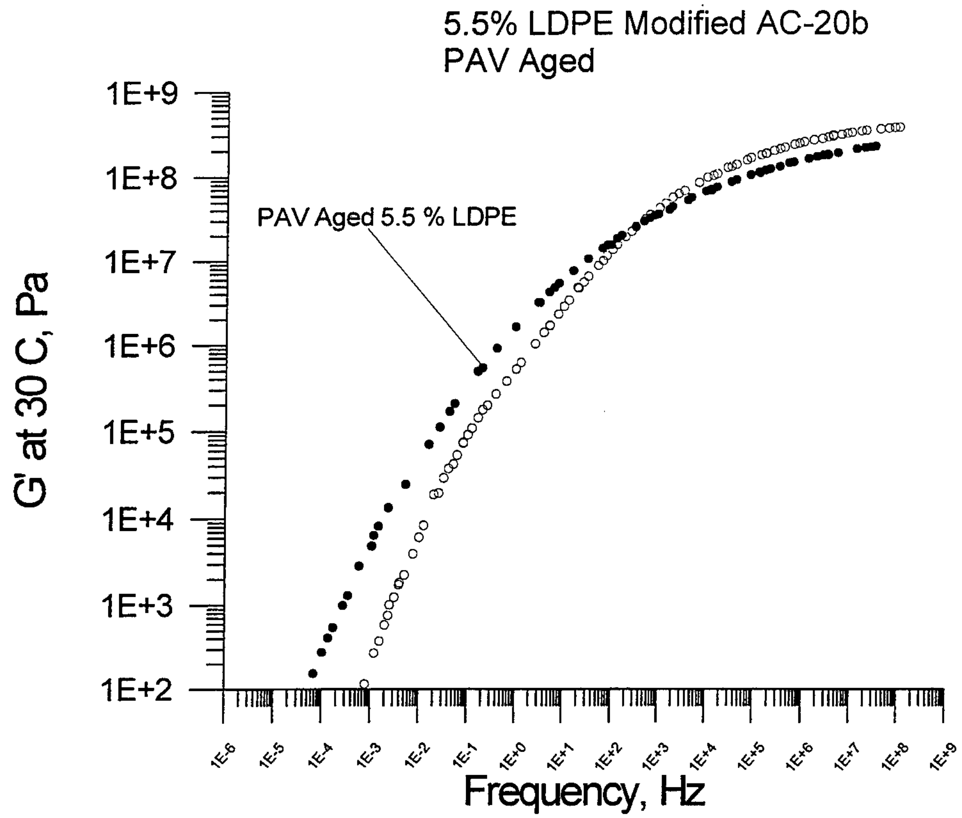

5.5% LDPE Modified AC-20a
PAV Aged
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
G' at 30 C, Pa
PAV Aged
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz

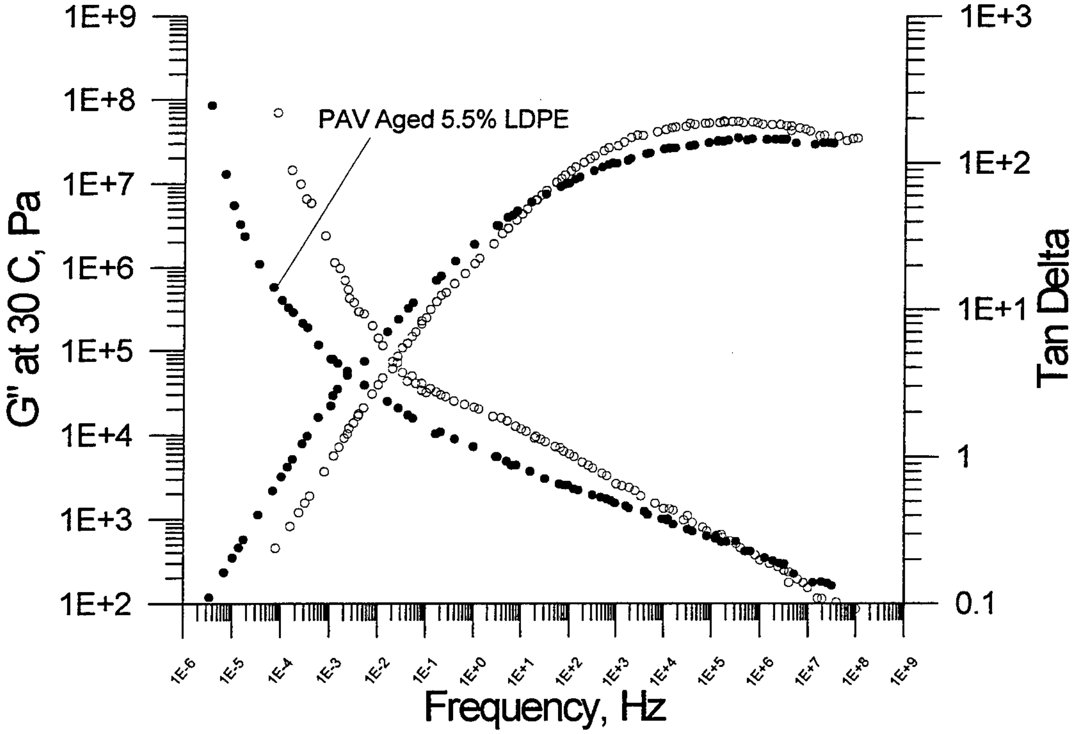

1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
G" at 30 C, Pa
PAV Aged
1E+2
1E+1
1
0.1
Tan Delta
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz

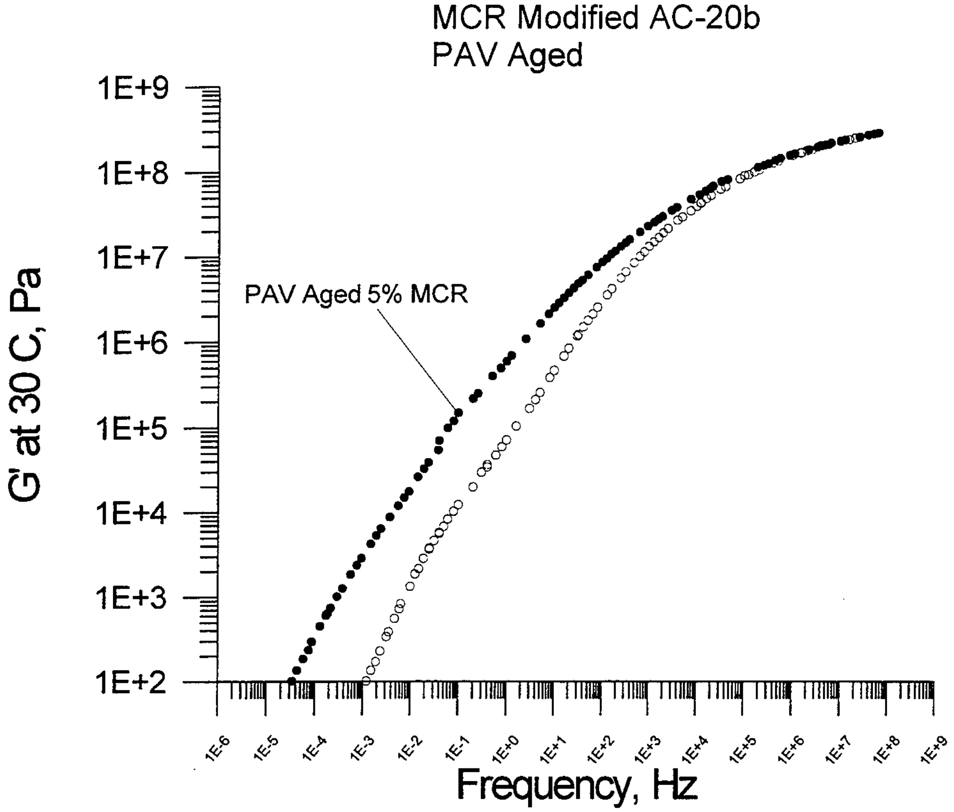

MCR Modified AC-20a
PAV Aged
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
PAV Aged 5% MCR
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz
G'' at 30 C, Pa
Tan Delta
1E+2
1E+1
1
0.1
PAV Aged 5% MCR
Frequency, Hz

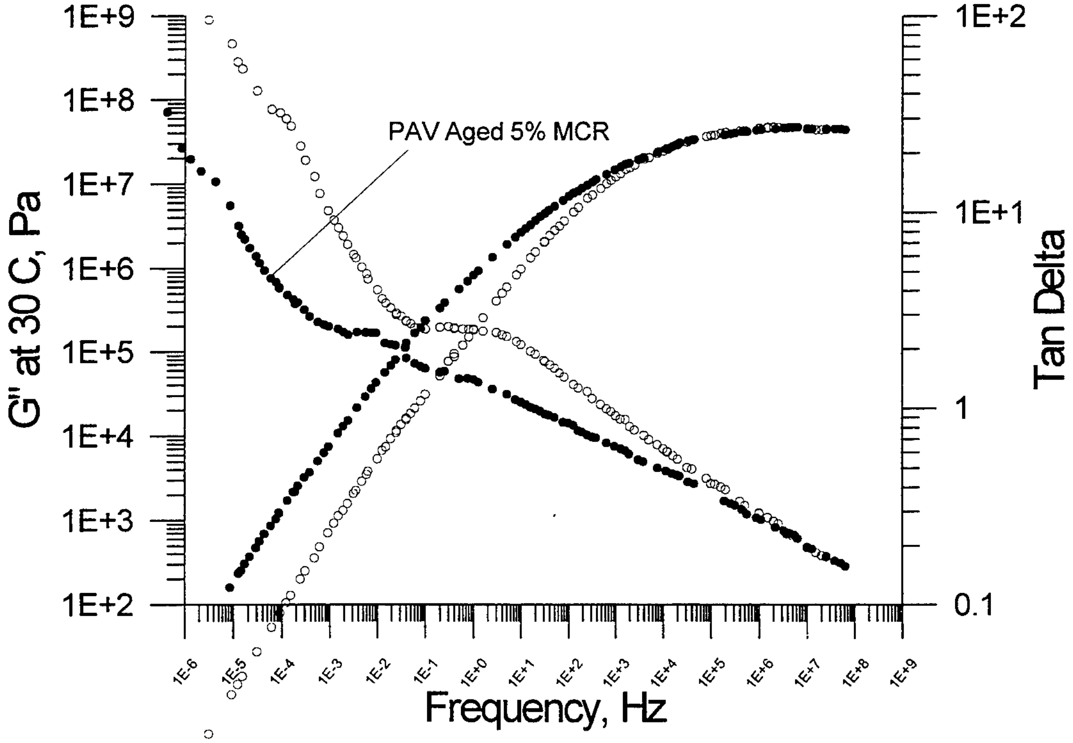

5% SBR Modified AC-20b
PAV Aged


5% RSBS Modified AC-20b
PAV Aged


5.5% LDPE Modified AC-20b
PAV Aged
PAV Aged 5.5 % LDPE
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz


PAV Aged 5.5% LDPE
G" at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
Tan Delta
1E+3
1E+2
1E+1
1
0.1
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz


MCR Modified AC-20b
PAV Aged
PAV Aged 5% MCR
G' at 30 C, Pa
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz


PAV Aged 5% MCR
G" at 30 C, Pa
Tan Delta
1E+9
1E+8
1E+7
1E+6
1E+5
1E+4
1E+3
1E+2
1E+2
1E+1
1
0.1
1E-6
1E-5
1E-4
1E-3
1E-2
1E-1
1E+0
1E+1
1E+2
1E+3
1E+4
1E+5
1E+6
1E+7
1E+8
1E+9
Frequency, Hz

# Appendix C
# Marshall Method for Mixture Design

## Table C1
## Marshall Mixture Design for Unmodified AC-20a

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.2 |
| Voids total mix, percent | 2-4 | 4.1 |
| Voids filled, percent | 75-85 | 74.7 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,404<br>(150.1) |
| Marshall stability, kN (lb) | 1,800 min. | 10.8<br>(2,430) |
| Marshall flow, mm (1/100 in.) | 16 max. | 2.8<br>(11.0) |
| Retained stability, percent | -- | 93 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C2
## Marshall Properties for Rich Mixtures Containing Unmodified AC-20a

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.7 | +0.5 |
| Voids total mix, percent | 2.8 | -1.3 |
| Voids filled, percent | 82.5 | +7.8 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,417<br>(150.9) | +0.8 |
| Marshall stability, kN (lb) | 11.1<br>(2,490) | +60 |
| Marshall flow, mm (1/100 in.) | 3.1<br>(12.1) | +1.1 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C3
## Marshall Mixture Design for Unmodified AC-20b

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.3 |
| Voids total mix, percent | 2-4 | 3.8 |
| Voids filled, percent | 75-85 | 76.5 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,406<br>(150.2) |
| Marshall stability, kN (lb) | 1,800 min. | 10.1<br>(2,280) |
| Marshall flow, mm (1/100 in.) | 16 max. | 2.7<br>(10.5) |
| Retained stability, percent | -- | 89.1 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C4
## Marshall Properties for Rich Mixtures Containing Unmodified AC-20b

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.8 | +0.5 |
| Voids total mix, percent | 2.9 | -0.9 |
| Voids filled, percent | 82.4 | +5.9 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,409<br>(150.4) | +0.2 |
| Marshall stability, kN (lb) | 10.5<br>(2,350) | +70 |
| Marshall flow, mm (1/100 in.) | 3.1<br>(12.2) | +1.7 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C5
## Marshall Mixture Design for AC-20a Modified with 5.5 % LDPE

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.1 |
| Voids total mix, percent | 2-4 | 4.2 |
| Voids filled, percent | 75-85 | 73.9 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,406<br>(150.2) |
| Marshall stability, kN (lb) | 1,800 min. | 16.2<br>(3,650) |
| Marshall flow, mm (1/100 in.) | 16 max. | 10.2<br>- (2.6) |
| Retained stability, percent | -- | 89.2 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C6
## Marshall Properties for Rich Mixtures Containing AC-20a Modified with 5.5 % LDPE

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.6 | +0.5 |
| Voids total mix, percent | 3.2 | -1.0 |
| Voids filled, percent | 80.1 | +6.2 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,411<br>(150.5) | +0.3 |
| Marshall stability, kN (lb) | 14.3<br>(3,210) | -440 |
| Marshall flow, mm (1/100 in.) | 3.3<br>(12.8) | +2.6 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C7
## Marshall Mixture Design for AC-20a Modified with 5 % RSBS

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.1 |
| Voids total mix, percent | 2-4 | 4.1 |
| Voids filled, percent | 75-85 | 74.4 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,404<br>(150.1) |
| Marshall stability, kN (lb) | 1,800 min. | 16.1<br>(3,630) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.8<br>(14.8) |
| Retained stability, percent | -- | 96.9 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C8
## Marshall Properties for Rich Mixtures Containing AC-20a Modified with 5 % RSBS

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.6 | +0.5 |
| Voids total mix, percent | 3.2 | -0.9 |
| Voids filled, percent | 80.4 | +6.0 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,412<br>(150.6) | +0.5 |
| Marshall stability, kN (lb) | 15.1<br>(3,400) | -230 |
| Marshall flow, mm (1/100 in.) | 4.1<br>(16.2) | +1.4 |

[1] Interpolated from mixture design data.

[2] Property for rich mixture less property at optimum binder content.

## Table C9
## Marshall Mixture Design for AC-20a Modified with 5 % SBR

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.2 |
| Voids total mix, percent | 2-4 | 4.2 |
| Voids filled, percent | 75-85 | 74.4 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,400<br>(149.8) |
| Marshall stability, kN (lb) | 1,800 min. | 13.4<br>(3,010) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.7<br>(14.5) |
| Retained stability, percent | -- | 91.7 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C10
## Marshall Properties for Rich Mixtures Containing AC-20a Modified with 5 % SBR

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.7 | +0.5 |
| Voids total mix, percent | 3.4 | -0.8 |
| Voids filled, percent | 79.8 | +5.4 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,400<br>(149.8) | +0.0 |
| Marshall stability, kN (lb) | 12.8<br>(2,880) | -130 |
| Marshall flow, mm (1/100 in.) | 3.8<br>(15.1) | +0.6 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

**Table C11**
**Marshall Mixture Design for AC-20a Modified with 5 % MCR**

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.2 |
| Voids total mix, percent | 2-4 | 4.3 |
| Voids filled, percent | 75-85 | 73.6 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,401 (149.9) |
| Marshall stability, kN (lb) | 1,800 min. | 12.8 (2,870) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.2 (12.5) |
| Retained stability, percent | -- | 98.3 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

**Table C12**
**Marshall Properties for Rich Mixtures Containing AC-20a Modified with 5 % MCR**

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.7 | +0.5 |
| Voids total mix, percent | 3.2 | -1.1 |
| Voids filled, percent | 80.5 | +6.9 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,412 (150.6) | +0.7 |
| Marshall stability, kN (lb) | 14.6 (3,280) | +410 |
| Marshall flow, mm (1/100 in.) | 3.6 (14.0) | +1.5 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C13
## Marshall Mixture Design for AC-20b Modified with 5.5 % LDPE

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.1 |
| Voids total mix, percent | 2-4 | 4.0 |
| Voids filled, percent | 75-85 | 75.0 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,408<br>(150.3) |
| Marshall stability, kN (lb) | 1,800 min. | 16.0<br>(3,590) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.0<br>(11.7) |
| Retained stability, percent | -- | 98.1 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C14
## Marshall Properties for Rich Mixtures Containing AC-20b Modified with 5.5 % LDPE

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.6 | +0.5 |
| Voids total mix, percent | 2.8 | -1.2 |
| Voids filled, percent | 82.5 | +7.5 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,419<br>(151.0) | +0.7 |
| Marshall stability, kN (lb) | 15.3<br>(3,430) | -160 |
| Marshall flow, mm (1/100 in.) | 3.6<br>(14.0) | +2.3 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C15
## Marshall Mixture Design for AC-20b Modified with 5 % RSBS

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.1 |
| Voids total mix, percent | 2-4 | 4.2 |
| Voids filled, percent | 75-85 | 74.1 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,401 (149.9) |
| Marshall stability, kN (lb) | 1,800 min. | 17.5 (3,930) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.8 (15.0) |
| Retained stability, percent | -- | 90.0 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C16
## Marshall Properties for Rich Mixtures Containing AC-20b Modified with 5 % RSBS

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.6 | +0.5 |
| Voids total mix, percent | 2.8 | -1.4 |
| Voids filled, percent | 82.7 | +8.6 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,419 (151.0) | +1.1 |
| Marshall stability, kN (lb) | 16.4 (3,690) | -240 |
| Marshall flow, mm (1/100 in.) | 4.5 (17.7) | +2.7 |

[1] Interpolated from mixture design data.

[2] Property for rich mixture less property at optimum binder content.

## Table C17
## Marshall Mixture Design for AC-20b Modified with 5 % SBR

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.2 |
| Voids total mix, percent | 2-4 | 3.6 |
| Voids filled, percent | 75-85 | 77.4 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,412<br>(150.6) |
| Marshall stability, kN (lb) | 1,800 min. | 16.4<br>(3,680) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.6<br>(14.3) |
| Retained stability, percent | -- | 98.6 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C18
## Marshall Properties for Rich Mixtures Containing AC-20b Modified with 5 % SBR

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.7 | +0.5 |
| Voids total mix, percent | 2.9 | -0.7 |
| Voids filled, percent | 82.1 | +4.7 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,409<br>(150.4) | +0.2 |
| Marshall stability, kN (lb) | 14.5<br>(3,250) | -430 |
| Marshall flow, mm (1/100 in.) | 4.4<br>(17.3) | +3.0 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

## Table C19
## Marshall Mixture Design for AC-20b Modified with 5 % MCR

| Property | Requirement[1] | Test Result |
|---|---|---|
| Optimum binder content, percent by weight of total mix | -- | 5.2 |
| Voids total mix, percent | 2-4 | 3.9 |
| Voids filled, percent | 75-85 | 75.5 |
| Unit weight (density), kg/m$^3$ (pcf) | -- | 2,409<br>(150.4) |
| Marshall stability, kN (lb) | 1,800 min. | 13.8<br>(3,110) |
| Marshall flow, mm (1/100 in.) | 16 max. | 3.3<br>(12.8) |
| Retained stability, percent | -- | 91.8 |

[1] Bituminous Pavements Standard Practice Manual, TM 5-822-8/AFM 88-6 (Headquarters, Departments of the Army and Air Force 1991).

## Table C20
## Marshall Properties for Rich Mixtures Containing AC-20b Modified with 5 % MCR

| Property[1] | Binder Content 0.5 percent Above Optimum | Difference[2] |
|---|---|---|
| Binder content, percent by weight of total mix | 5.7 | +0.5 |
| Voids total mix, percent | 3.2 | -0.7 |
| Voids filled, percent | 80.5 | +5.0 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,412<br>(150.6) | +0.2 |
| Marshall stability, kN (lb) | 12.5<br>(2,820) | -290 |
| Marshall flow, mm (1/100 in.) | 3.6<br>(14.1) | +1.3 |

[1] Interpolated from mixture design data.
[2] Property for rich mixture less property at optimum binder content.

# Appendix D
# Gyratory Compaction

## Table D1
## Gyratory Compaction Data for Mixtures Containing Unmodified AC-20a

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.5<br>(2.580) | 66.4<br>(2.616) |
| Height at 30 revolutions, mm (in.) | 64.2<br>(2.529) | 65.2<br>(2.566) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.277 | 1.251 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.278 | 1.278 |
| Total voids, percent | 4.1 | 4.0 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,404<br>(150.1) | 2,406<br>(150.2) |
| Number of gyratory flange revolutions | 36 | 71 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.02 |
| Gyratory shear strength ($S_G$), kPa (psi) | 993<br>(144) | 1,069<br>(155) |

[1] Average of 12 replicates.

## Table D2
## Gyratory Compaction Data for Mixtures Containing Unmodified AC-20b

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.5<br>(2.581) | 67.8<br>(2.670) |
| Height at 30 revolutions, mm (in.) | 64.3<br>(2.530) | 66.4<br>(2.614) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.277 | 1.262 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.278 | 1.345 |
| Total voids, percent | 4.1 | 4.0 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,403<br>(150.0) | 2,404<br>(150.1) |
| Number of gyratory flange revolutions | 35 | 179 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.06 |
| Gyratory shear strength ($S_G$), kPa (psi) | 821<br>(119) | 910<br>(132) |

[1] Average of 12 replicates.

## Table D3
## Gyratory Compaction Data for Mixtures Containing AC-20a Modified with 5.5 % LDPE

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.6<br>(2.584) | 67.0<br>(2.636) |
| Height at 30 revolutions, mm (in.) | 64.5<br>(2.541) | 65.8<br>(2.589) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.248 | 1.235 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.249 | 1.343 |
| Total voids, percent | 4.3 | 4.0 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,403<br>(150.0) | 2,409<br>(150.4) |
| Number of gyratory flange revolutions | 44 | 149 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.09 |
| Gyratory shear strength ($S_G$), kPa (psi) | 896<br>(130) | 565<br>(82) |

[1] Average of 12 replicates.

## Table D4
## Gyratory Compaction Data for Mixtures Containing AC-20a Modified with 5 % RSBS

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.9<br>(2.594) | 68.0<br>(2.677) |
| Height at 30 revolutions, mm (in.) | 64.6<br>(2.545) | 66.6<br>(2.624) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.255 | 1.226 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.258 | 1.281 |
| Total voids, percent | 4.1 | 4.4 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,408<br>(150.3) | 2,400<br>(149.8) |
| Number of gyratory flange revolutions | 50 | 320 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.05 |
| Gyratory shear strength ($S_G$), kPa (psi) | 841<br>(122) | 655<br>(95) |

[1] Average of 12 replicates.

## Table D5
## Gyratory Compaction Data for Mixtures Containing AC-20a Modified with 5 % SBR

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.9<br>(2.594) | 66.2<br>(2.608) |
| Height at 30 revolutions, mm (in.) | 64.7<br>(2.546) | 65.0<br>(2.561) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.250 | 1.254 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.258 | 1.293 |
| Total voids, percent | 4.0 | 3.7 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,404<br>(150.1) | 2,411<br>(150.5) |
| Number of gyratory flange revolutions | 48 | 70 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.01 | 1.03 |
| Gyratory shear strength ($S_G$), kPa (psi) | 889<br>(129) | 745<br>(108) |

[1] Average of 12 replicates.

## Table D6
## Gyratory Compaction Data for Mixtures Containing AC-20a Modified with 5 % MCR

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.4<br>(2.576) | 68.1<br>(2.681) |
| Height at 30 revolutions, mm (in.) | 64.2<br>(2.528) | 66.7<br>(2.626) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.269 | 1.259 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.271 | 1.319 |
| Total voids, percent | 4.2 | 4.3 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,404<br>(150.1) | 2,400<br>(149.8) |
| Number of gyratory flange revolutions | 35 | 356 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.05 |
| Gyratory shear strength ($S_G$), kPa (psi) | 772<br>(112) | 703<br>(102) |

[1] Average of 12 replicates.

## Table D7
## Gyratory Compaction Data for Mixtures Containing AC-20b Modified with 5.5 % LDPE

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.5<br>(2.580) | 67.4<br>(2.655) |
| Height at 30 revolutions, mm (in.) | 64.4<br>(2.537) | 66.1<br>(2.601) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.267 | 1.284 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.267 | 1.423 |
| Total voids, percent | 4.1 | 4.1 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,406<br>(150.2) | 2,404<br>(150.1) |
| Number of gyratory flange revolutions | 40 | 162 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.11 |
| Gyratory shear strength ($S_G$), kPa (psi) | 765<br>(111) | 593<br>(86) |

[1] Average of 12 replicates.

## Table D8
## Gyratory Compaction Data for Mixtures Containing AC-20b Modified with 5 % RSBS

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.8<br>(2.590) | 68.5<br>(2.696) |
| Height at 30 revolutions, mm (in.) | 64.5<br>(2.541) | 67.2<br>(2.645) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.250 | 1.273 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.255 | 1.345 |
| Total voids, percent | 4.1 | 4.7 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,403<br>(150.0) | 2,387<br>(149.0) |
| Number of gyratory flange revolutions | 43 | 500+ |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.06 |
| Gyratory shear strength ($S_G$), kPa (psi) | 772<br>(112) | 655<br>(95) |

[1] Average of 12 replicates.

## Table D9
## Gyratory Compaction Data for Mixtures Containing AC-20b Modified with 5 % SBR

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.8<br>(2.592) | 66.8<br>(2.629) |
| Height at 30 revolutions, mm (in.) | 64.6<br>(2.543) | 65.6<br>(2.581) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.246 | 1.270 |
| Final gyratory angle ($\theta_{max}$), degrees | 1.253 | 1.379 |
| Total voids, percent | 3.9 | 3.9 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,404<br>(150.1) | 2,406<br>(150.2) |
| Number of gyratory flange revolutions | 45 | 121 |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | 1.09 |
| Gyratory shear strength ($S_G$), kPa (psi) | 786<br>(114) | 607<br>(88) |

[1] Average of 12 replicates.

## Table D10
## Gyratory Compaction Data for Mixtures Containing AC-20b Modified with 5 % MCR

| Parameter[1] | Unaged | Aged |
|---|---|---|
| Height at 15 revolutions, mm (in.) | 65.8<br>(2.591) | 70.2<br>(2.763) |
| Height at 30 revolutions, mm (in.) | 64.6<br>(2.543) | 68.7<br>(2.706) |
| Minimum gyratory angle ($\theta_{min}$), degrees | 1.206 | [2] |
| Final gyratory angle ($\theta_{max}$), degrees | 1.208 | [2] |
| Total voids, percent | 4.1 | 7.2 |
| Unit weight (density), kg/m$^3$ (pcf) | 2,406<br>(150.2) | 2,327<br>(145.3) |
| Number of gyratory flange revolutions | 49 | 500+ |
| Gyratory compactibility index (GCI) | 0.98 | 0.98 |
| Gyratory stability index (GSI) | 1.00 | [2] |
| Gyratory shear strength ($S_G$), kPa (psi) | 683<br>(99) | 545<br>(79) |

[1] Average of 12 replicates.

[2] Material became dry and crumbly, and mold tended to slip; necessary periodic adjustments prevented proper measurement of these values.

# Appendix E

# Indirect Tensile Test

## Table E1
## Indirect Tensile Test Data for Mixtures Containing Unmodified AC-20a

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 14.4<br>(3,230) | 28.1<br>(6,310) |
| Tensile strength, kPa (psi) | 1,390<br>(202) | 2,720<br>(395) |
| Deformation at maximum load, mm (1/100 in.) | 2.2<br>(8.8) | 1.3<br>(5.1) |
| Indirect tensile toughness, N-m (in.-lb) | 32.1<br>(284) | 36.4<br>(322) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 2.76<br>(620) | 15.5<br>(3,480) |
| Tensile strength, kPa (psi) | 269<br>(39) | 1,500<br>(218) |
| Deformation at maximum load, mm (1/100 in.) | 1.8<br>(6.9) | 1.5<br>(5.8) |
| Indirect tensile toughness, N-m (in.-lb) | 4.8<br>(43) | 22.8<br>(202) |

## Table E2
## Indirect Tensile Test Data for Mixtures Containing Unmodified AC-20b

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 17.5<br>(3,925) | 23.5<br>(5,280) |
| Tensile strength, kPa (psi) | 1,700<br>(246) | 2,280<br>(331) |
| Deformation at maximum load, mm (1/100 in.) | 2.1<br>(8.3) | 1.1<br>(4.4) |
| Indirect tensile toughness, N-m (in.-lb) | 36.8<br>(326) | 26.3<br>(232) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 3.05<br>(686) | 12.5<br>(2,800) |
| Tensile strength, kPa (psi) | 296<br>(43) | 1,210<br>(175) |
| Deformation at maximum load, mm (1/100 in.) | 1.9<br>(7.4) | 1.0<br>(3.8) |
| Indirect tensile toughness, N-m (in.-lb) | 5.7<br>(51) | 12.0<br>(106) |

## Table E3
## Indirect Tensile Test Data for Mixtures Containing AC-20a Modified with 5.5 % LDPE

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 18.5<br>(4,160) | 19.4<br>(4,370) |
| Tensile strength, kPa (psi) | 1,810<br>(262) | 1,890<br>(274) |
| Deformation at maximum load, mm (1/100 in.) | 1.9<br>(7.4) | 1.1<br>(4.4) |
| Indirect tensile toughness, N-m (in.-lb) | 34.8<br>(308) | 21.7<br>(192) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 5.38<br>(1,210) | 11.7<br>(2,640) |
| Tensile strength, kPa (psi) | 524<br>(76) | 1,145<br>(166) |
| Deformation at maximum load, mm (1/100 in.) | 1.5<br>(6.0) | 1.1<br>(4.3) |
| Indirect tensile toughness, N-m (in.-lb) | 8.2<br>(73) | 12.8<br>(114) |

## Table E4
## Indirect Tensile Test Data for Mixtures Containing AC-20a Modified with 5 % RSBS

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 17.2<br>(3,870) | 20.2<br>(4,530) |
| Tensile strength, kPa (psi) | 1,680<br>(243) | 1,950<br>(283) |
| Deformation at maximum load, mm (1/100 in.) | 2.5<br>(9.9) | 1.2<br>(4.8) |
| Indirect tensile toughness, N-m (in.-lb) | 43.3<br>(383) | 24.6<br>(217) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 4.32<br>(970) | 14.2<br>(3,200) |
| Tensile strength, kPa (psi) | 421<br>(61) | 1,390<br>(201) |
| Deformation at maximum load, mm (1/100 in.) | 2.2<br>(8.5) | 1.2<br>(4.6) |
| Indirect tensile toughness, N-m (in.-lb) | 9.3<br>(82) | 16.6<br>(147) |

### Table E5
### Indirect Tensile Test Data for Mixtures Containing AC-20a Modified with 5 % SBR

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 15.4<br>(3,470) | 29.2<br>(6,560) |
| Tensile strength, kPa (psi) | 1,510<br>(219) | 2,830<br>(411) |
| Deformation at maximum load, mm (1/100 in.) | 2.2<br>(8.7) | 1.6<br>(6.3) |
| Indirect tensile toughness, N-m (in.-lb) | 34.1<br>(302) | 46.7<br>(413) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 4.72<br>(1,060) | 15.9<br>(3,580) |
| Tensile strength, kPa (psi) | 462<br>(67) | 1,550<br>(225) |
| Deformation at maximum load, mm (1/100 in.) | 2.0<br>(8.0) | 1.7<br>(6.8) |
| Indirect tensile toughness, N-m (in.-lb) | 9.6<br>(85) | 27.5<br>(243) |

### Table E6
### Indirect Tensile Test Data for Mixtures Containing AC-20a Modified with 5 % MCR

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 13.5<br>(3,030) | 16.9<br>(3,800) |
| Tensile strength, kPa (psi) | 1,320<br>(191) | 1,630<br>(237) |
| Deformation at maximum load, mm (1/100 in.) | 2.2<br>(8.6) | 1.0<br>(4.0) |
| Indirect tensile toughness, N-m (in.-lb) | 29.4<br>(261) | 17.2<br>(152) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 3.25<br>(730) | 13.4<br>(3,020) |
| Tensile strength, kPa (psi) | 317<br>(46) | 1,310<br>(190) |
| Deformation at maximum load, mm (1/100 in.) | 1.8<br>(7.0) | 1.2<br>(4.8) |
| Indirect tensile toughness, N-m (in.-lb) | 5.8<br>(51) | 16.4<br>(145) |

## Table E7
## Indirect Tensile Test Data for Mixtures Containing AC-20b Modified with 5.5 % LDPE

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 21.5<br>(4,840) | 17.7<br>(3,980) |
| Tensile strength, kPa (psi) | 2,100<br>(304) | 1,720<br>(250) |
| Deformation at maximum load, mm (1/100 in.) | 2.1<br>(8.2) | 1.0<br>(3.9) |
| Indirect tensile toughness, N-m (in.-lb) | 44.8<br>(397) | 17.5<br>(155) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 6.58<br>(1,480) | 9.61<br>(2,160) |
| Tensile strength, kPa (psi) | 641<br>(93) | 938<br>(136) |
| Deformation at maximum load, mm (1/100 in.) | 1.8<br>(7.0) | 0.9<br>(3.6) |
| Indirect tensile toughness, N-m (in.-lb) | 11.7<br>(104) | 8.8<br>(78) |

## Table E8
## Indirect Tensile Test Data for Mixtures Containing AC-20b Unmodified with 5 % RSBS

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 19.7<br>(4,430) | 11.3<br>(2,540) |
| Tensile strength, kPa (psi) | 1,920<br>(278) | 1,090<br>(158) |
| Deformation at maximum load, mm (1/100 in.) | 2.7<br>(10.5) | 0.8<br>(3.2) |
| Indirect tensile toughness, N-m (in.-lb) | 52.6<br>(465) | 9.2<br>(81) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 4.94<br>(1,110) | 7.56<br>(1,700) |
| Tensile strength, kPa (psi) | 480<br>(70) | 731<br>(106) |
| Deformation at maximum load, mm (1/100 in.) | 2.5<br>(10.0) | 1.1<br>(4.4) |
| Indirect tensile toughness, N-m (in.-lb) | 12.5<br>(111) | 8.5<br>(75) |

## Table E9
## Indirect Tensile Test Data for Mixtures Containing AC-20b Modified with 5 % SBR

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 18.0<br>(4,038) | 28.0<br>(6,290) |
| Tensile strength, kPa (psi) | 1,740<br>(253) | 2,720<br>(394) |
| Deformation at maximum load, mm (1/100 in.) | 2.5<br>(9.9) | 1.3<br>(5.0) |
| Indirect tensile toughness, N-m (in.-lb) | 45.2<br>(400) | 35.5<br>(315) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 5.29<br>(1,190) | 16.5<br>(3,720) |
| Tensile strength, kPa (psi) | 520<br>(75) | 1,610<br>(234) |
| Deformation at maximum load, mm (1/100 in.) | 2.3<br>(9.1) | 1.3<br>(5.1) |
| Indirect tensile toughness, N-m (in.-lb) | 12.2<br>(108) | 21.4<br>(190) |

## Table E10
## Indirect Tensile Test Data for Mixtures Containing AC-20b Modified with 5 % MCR

| Test Condition/Test Result | Unaged | Aged |
|---|---|---|
| **Testing Temperature of 25 °C (77 °F)** | | |
| Maximum load, kN (lb) | 17.2<br>(3,870) | 9.39<br>(2,110) |
| Tensile strength, kPa (psi) | 1,680<br>(243) | 900<br>(130) |
| Deformation at maximum load, mm (1/100 in.) | 2.0<br>(7.8) | 1.0<br>(4.1) |
| Indirect tensile toughness, N-m (in.-lb) | 34.1<br>(302) | 9.8<br>(87) |
| **Testing Temperature of 40 °C (104 °F)** | | |
| Maximum load, kN (lb) | 4.00<br>(900) | 5.16<br>(1,160) |
| Tensile strength, kPa (psi) | 390<br>(56) | 480<br>(70) |
| Deformation at maximum load, mm (1/100 in.) | 1.8<br>(7.0) | 1.0<br>(3.9) |
| Indirect tensile toughness, N-m (in.-lb) | 7.1<br>(63) | 5.1<br>(45) |

# Appendix F
# Confined Repeated-Load Deformation

## Table F1
## Confined Repeated-Load Deformation Test for Mixtures Containing Unmodified AC-20b

| Test Time (seconds) | Strain (in./in.) | |
|---|---|---|
| | Unaged Condition | Aged Condition |
| 0 | 0 | 0 |
| 5 | 0.0046 | 0.0014 |
| 15 | 0.0057 | 0.0019 |
| 30 | 0.0065 | 0.0020 |
| 60 | 0.0074 | 0.0020 |
| 120 | 0.0084 | 0.0024 |
| 240 | 0.0096 | 0.0025 |
| 480 | 0.0109 | 0.0029 |
| 900 | 0.0120 | 0.0030 |
| 1,800 | 0.0135 | 0.0034 |
| 3,600 | 0.0150 | 0.0037 |
| 4,500 | 0.0150 | 0.0035 |

## Table F2
## Confined Repeated-Load Deformation Test for Mixtures Containing AC-20b with 5.5 % LDPE

| Test Time (seconds) | Strain (in./in.) | |
|---|---|---|
| | Unaged Condition | Aged Condition |
| 0 | 0 | 0 |
| 5 | 0.0039 | 0.0012 |
| 15 | 0.0045 | 0.0016 |
| 30 | 0.0053 | 0.0018 |
| 60 | 0.0060 | 0.0022 |
| 120 | 0.0066 | 0.0023 |
| 240 | 0.0076 | 0.0026 |
| 480 | 0.0083 | 0.0028 |
| 900 | 0.0089 | 0.0028 |
| 1,800 | 0.0099 | 0.0030 |
| 3,600 | 0.0112 | 0.0035 |
| 4,500 | 0.0109 | 0.0034 |

### Table F3
### Confined Repeated-Load Deformation Test for Mixtures Containing AC-20b with 5 % RSBS

| Test Time (seconds) | Strain (in./in.) | |
|---|---|---|
| | Unaged Condition | Aged Condition |
| 0 | 0 | 0 |
| 5 | 0.0041 | 0.0023 |
| 15 | 0.0046 | 0.0024 |
| 30 | 0.0058 | 0.0027 |
| 60 | 0.0066 | 0.0027 |
| 120 | 0.0074 | 0.0030 |
| 240 | 0.0081 | 0.0034 |
| 480 | 0.0088 | 0.0034 |
| 900 | 0.0096 | 0.0035 |
| 1,800 | 0.0100 | 0.0035 |
| 3,600 | 0.0108 | 0.0038 |
| 4,500 | 0.0110 | 0.0036 |

### Table F4
### Confined Repeated-Load Deformation Test for Mixtures Containing AC-20b with 5 % SBR

| Test Time (seconds) | Strain (in./in.) | |
|---|---|---|
| | Unaged Condition | Aged Condition |
| 0 | 0 | 0 |
| 5 | 0.0042 | 0.0023 |
| 15 | 0.0052 | 0.0030 |
| 30 | 0.0058 | 0.0034 |
| 60 | 0.0066 | 0.0036 |
| 120 | 0.0071 | 0.0040 |
| 240 | 0.0078 | 0.0044 |
| 480 | 0.0086 | 0.0046 |
| 900 | 0.0094 | 0.0048 |
| 1,800 | 0.0099 | 0.0052 |
| 3,600 | 0.0111 | 0.0056 |
| 4,500 | 0.0109 | 0.0054 |

## Table F5
## Confined Repeated-Load Deformation Test for Mixtures Containing AC-20b with 5 % MCR

| Test Time (seconds) | Strain (in./in.) | |
|---|---|---|
| | Unaged Condition | Aged Condition |
| 0 | 0 | 0 |
| 5 | 0.0037 | 0.0016 |
| 15 | 0.0046 | 0.0017 |
| 30 | 0.0050 | 0.0019 |
| 60 | 0.0057 | 0.0022 |
| 120 | 0.0064 | 0.0024 |
| 240 | 0.0072 | 0.0026 |
| 480 | 0.0079 | 0.0026 |
| 900 | 0.0086 | 0.0029 |
| 1,800 | 0.0093 | 0.0029 |
| 3,600 | 0.0093 | 0.0032 |
| 4,500 | 0.0100 | 0.0031 |

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**13. ABSTRACT (Maximum 200 words)**

The Department of Defense (DOD) approved a Tri-Service Civil Engineering Science and Technology Plan that included the subarea of "Airfields and Pavements." The impetus of this research was to sustain and extend the life of DOD pavements. The Department of the Army (DA) pavement system consists of approximately 676 million square yards of pavements and annual maintenance expenditures of approximately $124 million. The DA pavement system includes approximately 46,000 miles (74,230 km) of roadways (comparable in size to the Interstate system), 2,500 miles (4,023 km) of rail which is comparable in size to the Amtrak system, and approximately 150 airfields.

It has been estimated that increasing the service life of a pavement by 1 year will reduce maintenance costs by 20 percent and construction costs by 10 percent. Currently, it costs approximately $2.5M to reconstruct a 10,000-ft runway using asphalt concrete. A 1-year extension in the service life would provide a $250K savings in construction costs alone, not including maintenance costs and the costs of relocating the mission during the reconstruction. If the service life was extended by 5 years, the construction cost savings would increase to $1.2M. This would be a savings totaling almost half of the initial reconstruction cost. As each runway at the 150 locations were reconstructed, the savings would rapidly increase and could reach approximately $180M. This figure only includes one runway at each location. It does not include additional runways, taxiways, or parking apron pavements. If these pavements were included, the savings would exceed $824M for the Department of the Army alone.

(Continued)

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The major pavement distress area for DOD airfield pavements, as determined by a Tri-Service selection committee, was durability related distresses in asphalt pavements. Based on this major distress area and the original objectives described in HR 103-516 (which was the impetus for this research), the project objectives were to: develop a methodology based on laboratory testing that can quantify the improved resistance of asphalt pavements to durability distresses resulting from binder modification, provide current practice information of new or manufactured and waste or by-product materials used as modifiers and/or fillers in asphalt pavements, provide technical recommendations on requirements to advance from the conclusions of this research to the original objectives outlined in HR 103-516.

The generalized approach to achieve the objectives included: obtaining two different asphalt cements and four different modifiers for laboratory evaluation including unaged and aged binder testing and unaged and aged mixture testing; and determining the current industry practice involving asphalt binder modification through a market survey and literature review. The general conclusions from the MUMPS research program were:

a. Significant strides were made in this project toward developing the methodology required to develop a simplified process (as compared to the current process used in military construction) to evaluate materials and technologies thus allowing the use of new and innovative materials and technologies in military pavement construction projects.

b. The recyclability of an HMA pavement containing a waste material or modifier must be determined before allowing widespread use of that particular material in HMA.

c. The use of a waste material or modifier in an HMA must be based on engineering, economic, and environmental factors, not legislated mandates.

d. The incorporation of a waste material or modifier into an HMA pavement must not adversely affect the performance of the pavement and preferably should enhance pavement performance.

e. Conventional binder and HMA tests have limited usefulness when evaluating modified binders and modified HMA due to their empiricism.

f. The highway industry is adopting SHRP binder testing criteria for use in HMA construction projects. These criteria may not be directly applicable to airfield pavements because of their unique load requirements (in the past, highway mix design methods and criteria have required modification before they could be used for airfield applications).

g. The predominate concern in relation to military airfield HMA pavements is durability related distresses.