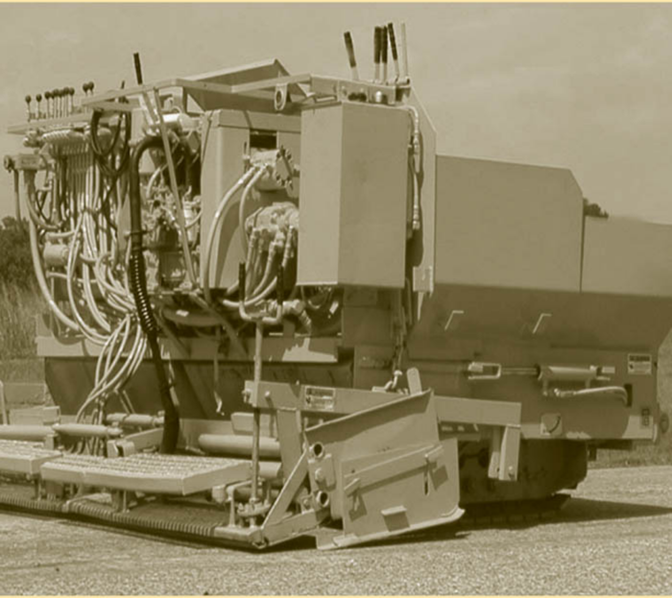
Asphalt Mix Design and Construction

Past, Present, and Future

Edited by K. Wayne Lee, Ph.D., P.E.
and Kamyar C. Mahboub, Ph.D., P.E.



ASCE

ASPHALT MIX DESIGN AND CONSTRUCTION

Past, Present, and Future

A SPECIAL PUBLICATION OF THE 150TH ANNIVERSARY OF ASCE

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Preface

When its twelve Founders gathered in New York City on November 5, 1852, and agreed to incorporate American Society of Civil Engineers and Architects, one can only wonder if they perhaps dreamed of the profound significance and long-lasting impact it would have on the overall development of society. During the week of November 3, 2002, in Washington D.C., we paused to honor our forebears in our American Society of Civil Engineers (ASCE), which has grown to include more than 125,000 civil engineers. Then, we looked forward to building a better world.

The ASCE Bituminous Materials Committee (BMC) was established with the purpose to advance the technology through the systematic collection, assessment, and dissemination of information relating to research, development, processing, applications, evaluations, and performance of bituminous materials under the Materials Engineering Division in 1995. In the year 2000, the Construction Division and Materials Engineering Division of ASCE were joined into a new partnership, and it established the Construction Institute (CI) at the ASCE Annual Conference in Seattle. Representing a major segment of members in ASCE, the Institute seeks to bring state-of-the-art practices and policies to its members to enhance related engineering practice, education, and research.

The BMC sponsored a special session for the Washington D.C. Conference to celebrate the 150th Anniversary of the ASCE and to perform its new mission in ASCE on November 3, 2002. The special technical session CC-1/2 "Evaluation of Bituminous Materials Mix-Design & Superpave" invited three outstanding speakers on past, present, and future of asphalt mix-design practices for construction industries. Furthermore, the CI Executive Committee approved the plan of BMC for the publication of proceedings. Since one of three authors could not provide a manuscript, two BMC members volunteered providing manuscripts on state-of-the-art modified asphalt binder and quality control and assurance. Their contributions and efforts are sincerely appreciated. Also, Mr. Matthew Whelan, a graduate student at University of Kentucky, offered valuable assistance with document format adjustment issues.

Finally, we want to pay special thanks to all of BMC members, and ASCE CI and Publication staffs for their many valuable suggestions and efforts in making this Special Publication possible.

K. Wayne Lee Ph.D., P.E., Member ASCE, and Kamyar C. Mahboub, Ph.D., P.E., Fellow ASCE, September 2005

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State of the Art: Bituminous Materials Mix Design

Carl L. Monismith, P.E., Honorary Member, ASCE[1]

Prepared for Session on Evaluation of Bituminous Materials Mix-Design and Superpave Civil Engineering Conference and Exposition
Washington, D.C.
November 3-7, 2002

INTRODUCTION

Design of an asphalt-aggregate mix consists of the following basic steps:

1. Select the *type* and *gradation* of the mineral aggregate.

2. Select the *type and grade* of asphalt binder, with or without modification.

3. Select the *amount* of asphalt binder to satisfy the project-specific requirements for mix properties.

These steps have been incorporated into a general design framework (Figure 1). Proper selection of the mix components and their relative proportions, i.e., asphalt or "binder" content,[2] requires a knowledge of the significant properties and performance characteristics of asphalt paving mixes and how they are influenced by the mix components. Table 1 contains a listing of the mix properties which should be considered for specific design situations together with a summary of the factors which influence these properties.

Mix design is the selection of the components to achieve a desirable balance in these properties for the specific pavement application. Selection of the components and their relative proportions is also influenced by the pavement section in which the mix will be incorporated. For pavements subjected to traffic, Figure 1 emphasizes the fact that the designer must be cognizant of the fact that *mix-design and pavement-design are interrelated and, therefore, must be considered together.*

It is the purpose of this paper to briefly summarize the state-of-the-art mix design technology in the United States at this time (2002).

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[1] Robert Horonjeff Professor of Civil Engineering Emeritus and Director, Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley

[2] Asphalt or binder content is expressed either as a percent of the total weight of mix or of the aggregate. It is important to state the basis for binder content, either aggregate or total weight of mix basis since the actual amount of asphalt in the mix differs with the basis; it is higher for the same asphalt content on a total weight of mix basis as compared to that for the aggregate basis.

Table 1. Properties of Asphalt-Aggregate Mixes

Property	Definition	Examples of Mix Variables Which have an Influence
Stiffness	$S_{mix}(t, T) = \sigma / \in$ Relationship between stress and strain at a specific temperature and time of loading	Aggregate gradation Asphalt stiffness Degree of compaction Water sensitivity Asphalt content
Stability	Resistance to permanent deformation (usually at high temperatures and long times of loading – conditions of low S_{mix})	Aggregate surface texture Aggregate gradation Asphalt stiffness Asphalt content Degree of compaction Water sensitivity
Durability	Resistance to weathering effects (both air and water) and to the abrasive action of traffic	Asphalt content Aggregate gradation Degree of compaction Water sensitivity
Fatigue Resistance	Ability of mix to bend repeatedly without fracture	Aggregate gradation Asphalt content Degree of compaction Asphalt stiffness Water sensitivity NOTE: Selection of mix components and or AC thickness dependent on structural pavement section design.
Fracture Characteristics	Strength of mix under single tensile stress application	Aggregate gradation Aggregate type Asphalt content Degree of compaction Asphalt stiffness Water sensitivity
Skid Resistance (Surface friction characteristics)	Ability of mix to provide adequate coefficient of friction between tire and pavement under "wet" conditions	Aggregate texture and resistance to polishing Aggregate gradation Asphalt content
Permeability	Ability of air, water, and water vapor to move into and through mix	Aggregate gradation Asphalt content Degree of compaction
Workability	Ability of the mix to be placed and compacted to specified density	Asphalt content Asphalt stiffness at placement Aggregate surface texture Aggregate gradation

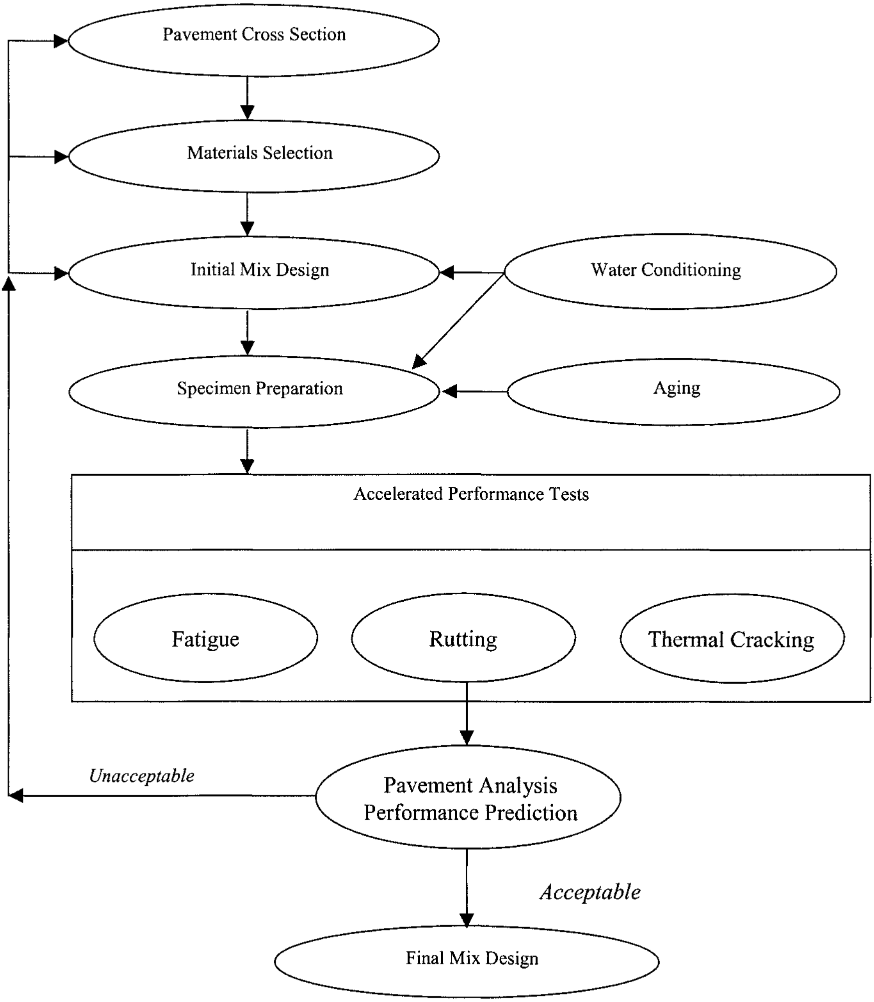

Figure 1. Schematic of mix design and analysis framework.

This paper includes a brief summary of mix design procedures developed during the 1930's and 40's for highway and airfield pavements and which are still in use today by highway and other government agencies. These are: (Highway Research Board 1949) the U.S. Army Corps of Engineers (USACE) and Federal Aviation Administration (FAA) methodology for airfield pavements (which includes the Marshall Stability Test) and modifications thereof for highway pavements (Highway Research Board 1949, White 1985, Department of the Army, Kandhal and Koehler 1985, Asphalt Institute 1994); and (2) the State of California mix design procedure (referred to sometimes as the Hveem method) for highway pavements (Stanton and Hveem 1934, Hveem, 1942, Vallerga and Lovering 1985, California Department of Transportation). The latter procedure is used in modified form by a number of the Western States in the U.S. (Kandhal and Koehler 1985). An example is provided of the use of combination of the two procedures for airfield pavements subjected to heavy aircraft loading (Vallerga et al. 2000, Monismith et al. 2000). Also included is a brief summary of the Superpave[3] volumetric design procedure developed during the Strategic Highway Research Program (SHRP) (Cominsky et al. 1994). It is now in use by a number of states. Associated with this methodology is the "simple performance test" under development in the National Cooperative Highway Research Program (NCHRP) Project 9-19. Developments associated with the Superpave methodology are discussed in more detail in the papers by Witczak and Bonaquist and Harman *et al.* as a part of this symposium.

Other approaches which have successfully been used for mix design are also briefly summarized. These include: (1) a methodology developed by Shell utilizing the creep test and applied to a mix design for an airfield pavement in Saudi Arabia (van de Loo 1976, Finn et al. 1983); and (2) the use of the SHRP-developed technology including the simple shear, flexural fatigue, and thermal stress restrained specimen tests for mix design and analysis purposes (Jung and Vinson 1994A, Deacon et al. 1994, Sousa et al. 1994). Also included are discussions pertaining to laboratory test specimen preparation, specimen size, importance of temperature and traffic loading conditions and test variability. These latter items must be reflected in any new method of asphalt mix design which is utilized.

In concluding these introductory remarks it is important to note that this state of the art presentation is somewhat limited in scope, particularly in summarizing the many developments that have taken place in materials characterization, and mix analysis and design in the past 25 years. Two excellent sources that provide information in addition to that included herein are:

1. *Symposium – Asphalt Mix Design*, chaired by L.E. Santucci and published in the 1985 *Proceedings* of the Association of Asphalt Paving Technologists (Santucci 1985). References (White 1985, Kandhal and Koehler 1985, Vallerga and Lovering 1985) are included in that Symposium;
2. The paper by R. B. Leahy and R. B. McGennis entitled "Asphalt Mixes: Materials, Design and Characterization" published in the special 75th

[3] SUPERPAVE – An acronym for *Su*perior *Per*forming Asphalt Concrete *Pave*ments

Anniversary Volume of the Association of Asphalt Paving Technologists in 1996 (Leahy and McGennis 1999).

Both of these documents provide excellent summaries of existing methodologies as well as summaries of developments to the dates of the publications; in addition, both include a wealth of references on this subject.

GENERAL FRAMEWORK FOR A COMPREHENSIVE SYSTEM

Figure 1 illustrates the general framework for a comprehensive design system. The system consists of a series of subsystems in which the mix components, asphalt (or binder) and aggregate and their relative proportions are selected in a step by step procedure to produce a mix which can then be tested and evaluated to insure that it will attain a desired level of performance in the specific pavement section in which it is to function. The influence of environmental factors, the effects of traffic loading, and the consequence of the pavement structural section design at the selected site are also included in this evaluation.

It is important to note in Figure 1 that an evaluation for water sensitivity of the mix is scheduled in the trial design phase. Satisfactory resolution of this problem prior to examination of the response of the trial mix to the three modes of distress shown in Figure 1, i.e. *fatigue, rutting, and thermal (low temperature) cracking,*[4] will allow full concentration on these evaluations.

Depending on the conditions of exposure to climatic and loading factors to which the specific pavement is to be subjected, any or all of the distress modes may be evaluated. For example, in a hot, dry climate, it may not be necessary to examine the potential for thermal cracking whereas, because of the potential for fatigue and rutting associated with the specific environmental conditions, it would be essential to evaluate these latter two modes.

While satisfactory resolution of the water sensitivity problem is desirable in the initial design phase, it may not always be possible to completely preclude the deleterious influence of water and/or water vapor. Accordingly, provision may also be included in the distress evaluation phase to define the mix characteristics which reasonably reflect the influence of this factor. In addition, the effects of long term mixture aging must be considered. For example, as the mixture ages, its stiffness increases leading, in turn, to increased propensity for thermal cracking. Both considerations are shown in Figure 1 as input at the appropriate place in the mixture design process. It should be noted that both water sensitivity and mix aging were also evaluated in the SHRP program (Terrel and Al-Swailmi 1994, Bell et al. 1994). However, only the results of the short-term aging research have been incorporated in the Superpave mix design procedure discussed herein.

It must be emphasized that provision must also be included in the mix design system to "temper" the design by such factors as material availability, local

[4] These three forms of distress, considered to be the primary modes contributing to reduction in asphalt pavement serviceability, were the focus of the mix performance evaluation phase of the SHRP asphalt research program.

experience, cost, and risk options associated with the facility and resulting from specific distress modes.

Finally, the system should include considerations of construction control. Levels of compliance must be established to insure that the mix will achieve the desired performance objective to the requisite level of reliability. Assessment of the influence of noncompliance can also be determined, and appropriate pay factors associated with different levels of performance relative to the target design established.

EXAMPLES OF MIX DESIGN METHODS WHICH HAVE BEEN USED DURING THE PAST 50 YEARS

In this section, brief summaries of two methods which have been used in the United States for many years are included. The first is a procedure developed by the U.S. Army Corps of Engineers for airfield pavements utilizing the Marshall Stability Test (Highway Research Board 1949).[5]

This method has also been used extensively by highway agencies with adaptations to their specific conditions; e.g. in 1984 this methodology was used by 38 states (Kandhal and Koehler 1985).[6] The second is the procedure developed in California under the direction of F.N. Hveem (Vallerga and Lovering 1985); ten states were listed as using this methodology in 1984.

U.S. Army Corps of Engineers (FAA) Method of Mix Design

This method was developed during World War II by the U.S. Army Corps of Engineers for asphalt pavements for airfields. The procedure was based on correlation of laboratory tests with performance of a test track at Vicksburg, Mississippi, subjected to airplane loadings with tires inflated to a pressure of approximately 100 psi (Highway Research Board 1949). At the outset of the Korean War, jet aircraft were introduced which operated with tires at inflation pressures of approximately 200 psi. These aircraft necessitated some changes in design criteria and procedures relative to the original method. Thus, the present procedure now consists of two sets of criteria, one for aircraft operating at tire pressures in the vicinity of 100 psi and the other for aircraft with tire pressure of approximately 200 psi (White 1985, Department of the Army).

Mix design is based on the following concepts: (1) use of aggregate gradations resulting in dense mixes; (2) use of crushed coarse aggregate with 75 percent of the particles having two or more fractured faces; fine aggregate must be angular; (3) use of as soft of an asphalt cement as possible consistent with traffic and environment and as much as possible for durability; and (4) use of void criteria in selecting the design asphalt content.

A step by step summary of the procedure is as follows:

1. Select crushed aggregate conforming to specifications for particular usage.

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[5] The methodology is a part of the mix design requirements for mixes used both in commercial and general aviation airports receiving Federal Aviation Administration (FAA) funds.

[6] With the advent of Superpave, a number of states have shifted from the so-called Marshall or Hveem procedures to the use of the Superpave methodology.

Select grade of asphalt cement.
Estimate design asphalt content (based on total weight of mixture).
Prepare five sets (at least three each) of trial mix specimens 4 in. in diameter by 2-½ in. high. For example, assume that the design asphalt content was estimated to be 5.0 percent. Then, in addition to three specimens at 5.0 percent, additional mixes would be prepared at asphalt contents of 4.0, 4.5, 5.5, and 6.0 percent asphalt.

a. Mixing – either by hand or with mechanical mixer, asphalt cement heated to 250-280°F and aggregate to 350-375°F.
b. Compaction – impact compaction using Marshall hammer, at a temperature of at least 225°F, hammer consists of 10 lb weight falling through a height of 18 in., Figure 2.
   i. for mixes to be subjected to tire pressure of 100 psi – 50 blows on both ends of specimen.
   ii. for mixes to be subjected to 200 psi tire pressures – 75 blows on both ends of specimen.

Density and void analysis – from density determinations, plot average data and draw smooth curve as shown in Figure 4. From the curve, select unit weight values corresponding to the various asphalt contents and determine: (a) percent air voids; and (b) percent voids filled with asphalt.
Perform Marshall Stability Test, Figure 3 - test performed at 140°F at a load rate of 2 in. per min. from which is determined the maximum load (termed *Marshall Stability* and measured in lb) and deformation corresponding to maximum load (termed *flow value* and measured in 0.01 in.).
Prepare curves of data similar to those illustrated in Figure 4 and select design asphalt content according to criteria presented in Tables 2a and 2b.

The following illustrates the use of the procedure for a mix consisting of a crushed limestone aggregate containing an 85-100 penetration asphalt cement to be used as a surface course for an airfield subjected to aircraft with tires inflated to 200 psi. From laboratory tests on mix specimens, compacted with 75 blows on each end of the spectrum, the data shown in Table 3 were obtained.

When the data is plotted as shown in Figure 4, the following analysis is used to select the design asphalt content (Table 2a).

1.	Asphalt content corresponding to peak of unit weight curve	4.6
2.	Asphalt content corresponding to 4 percent air voids	4.9
3.	Asphalt content corresponding to 75 percent voids filled	4.9
4.	Asphalt content corresponding to peak of Marshall Stability curve	4.4
	Average of 4 values =	4.7 percent (by total weight of mix)

This value must then be checked to determine whether or not the mix satisfies all of the criteria stipulated in Table 2a.

1. Marshall Stability – minimum requirement of 1800 lb.; actual mix at 4.7 percent asphalt content – 2030 lb.; therefore satisfactory.

2. Percent Air Voids – 3 to 5 percent; actual mix at 4.7 percent contains 4.2 percent; therefore, satisfactory.

3. Percent Voids Filled – 70 to 80 percent; actual mix at 4.7 percent contains 73 percent; therefore satisfactory.

4. Flow Value – maximum value of 16; actual mix at 4.7 percent, has value of 11; therefore satisfactory.

Thus, in this example the design (or optimum) asphalt content would be 4.7 percent by *weight of total mix*. If the asphalt content determined by the averaging technique failed to meet one of the abovementioned requirements, it would be modified slightly in order to satisfy all conditions.

It should be noted that in this procedure the averaging process was originally developed to provide asphalt contents in the laboratory specimens that corresponded to the best performance in a test track subjected to 100 psi tires. The procedure for 200 psi tires is also an attempt to do the same thing.

Modifications to the above procedure are required for channelized traffic areas, areas subjected to tires with pressure greater than 200 psi, and for high temperatures as measured by the Pavement Temperature Index. Table 4 provides a summary of these modifications. Generally, for the type of conditions enumerated above, the optimum asphalt content, based on the 200 psi criteria, may be reduced by as much as 20 percent.

The Corps of Engineers also utilizes a mechanical gyratory compactor for mixes designed for very heavy load pavements, as noted in Table 4 (McRae and McDaniel 1958, McRae and Foster 1959).

A similar approach is used for mixes for highway pavements. For example, the Asphalt Institute has published a procedure in which the mix, in addition to meeting the criteria for Marshall Stability, flow value, percent air void, and percent voids filled with asphalt, must also satisfy a minimum void in the mineral aggregate (VMA) requirement (Asphalt Institute 1994).

State of California Department of Transportation (Hveem) Method of Asphalt Mix Design.

Initial development by F.N. Hveem, formerly Materials and Research Engineer of the California Division of Highways, of the stabilometer as a mix design test took place in the 1930's. References (Stanton and Hveem 1934, Hveem 1942, Vallerga and Lovering 1985) provide the basis for Hveem's philosophy for mix design, the evolution of the stabilometer as a mix evaluation procedure, and some of the tests performed to develop mix design criteria, particularly those used for binder content selection based on stabilometer test results.

Hveem recognized early on the importance of compacting mixes in the laboratory so that the resulting aggregate structure would be representative of that produced in the mix when compacted in-situ by steel and pneumatic-tire rollers. He introduced the concept of *kneading* compaction for laboratory-prepared specimens. Through a collaborative effort with B. A. Vallerga, then at UC Berkeley, the current method of kneading compaction evolved. The resulting compaction equipment, termed the Triaxial Institute Kneading Compactor, was introduced in 1949 (Vallerga 1951).[7] By comparing the densities of mixes in situ after trafficking for about one year, the current compaction procedure of 150 tamps at 500 psi tamping pressure at a temperature of 230°F was established.

The essential elements of Hveem's philosophy of mix design are as follows:

1. Stability is a function primarily of surface roughness of aggregate particles.
2. Quantity of asphalt is a function of surface area, surface roughness, and porosity of aggregate and viscosity of asphalt.
3. If required, the design asphalt content is adjusted to leave approximately 4 percent calculated air voids to avoid bleeding or possible loss in stability.

References (Endersby and Vallerga 1952, Vallerga 1955) provide additional background on mix evaluation using the "Hveem" method as it evolved during the early 1950's.

Examples of the use of the procedure as developed by Hveem during the period 1957-1963 and as currently practiced are briefly summarized in the following paragraphs.

[7] This mechanical compactor is still in use by a number of agencies although servo-hydraulic equipment has replaced it, Figure 5.

Table 2a. Bituminous Pavement Design Criteria for Use in Conjunction with ASTM Apparent Specific Gravity (Department of the Army)

For use with Aggregate Blends Showing Water Absorption up to 2.5 Percent.

For Determining Optimum Bitumen Content				For Determining Satisfactoriness of Mix			
Test Property	Type of Mix	Point on Curve		Test Property	Type of Mix	Criteria	
		50 Blows	75 Blows			50 Blows	75 Blows
Marshall Stability	Bituminous-concrete surface course	Peak of curve	Peak of curve	Marshall Stability	Bituminous-concrete surface course	500 lb. or higher	1800 lb. or higher
	Bituminous-concrete binder course	Peak of curve*	Peak of curve		Bituminous-concrete binder course	500 lb. or higher	1800 lb. or higher
	Sand asphalt	Peak of curve*	**		Sand asphalt	500 lb. or higher	**
Unit Weight	Bituminous-concrete surface course	Peak of curve	Peak of curve	Unit Weight	--	Not used	Not used
	Bituminous-concrete binder course	Not used	Not used				
	Sand asphalt	Peak of curve	**				
Flow	--	Not used	Not used	Flow	Bituminous-concrete surface course	20 or less	16 or less
					Bituminous-concrete binder course	20 or less	16 or less
					Sand asphalt	20 or less	**
Percent voids total mix	Bituminous-concrete surface course	4	4	Percent voids total mix	Bituminous-concrete surface course	3-5	3-5
	Bituminous-concrete binder course	5	5		Bituminous-concrete binder course	4-6	5-7
	Sand asphalt	6	**		Sand asphalt	5-7	**
Percent voids filled with bitumen	Bituminous-concrete surface course	80	75	Percent voids filled with bitumen	Bituminous-concrete surface course	75-85	70-80
	Bituminous-concrete binder course	70*	60*		Bituminous-concrete binder course	65-75	50-70
	Sand asphalt	70	**		Sand asphalt	65-75	**

* If the inclusion of bitumen contents at these points in the average causes the voids total mix to fall outside the limits, then the optimum bitumen content should be adjusted so that the voids total mix are within the limits.

** Criteria for sand asphalt to be used in the designing pavements for 200 psi tires have not been established.

Table 2b. Bituminous Pavement Design Criteria for Use in Conjunction with Bulk Impregnated Specific Gravity (Department of the Army)

For use with Aggregate Blends Showing Water Absorption Greater Than 2.5 Percent.

For Determining Optimum Bitumen Content				For Determining Satisfactoriness of Mix			
Test Property	Type of Mix	Point on Curve		Test Property	Type of Mix	Criteria	
		50 Blows	75 Blows			50 Blows	75 Blows
Marshall Stability	Bituminous-concrete surface course	Peak of curve	Peak of curve	Marshall Stability	Bituminous-concrete surface course	500 lb. or higher	1800 lb. or higher
	Bituminous-concrete binder course	Peak of curve*	Peak of curve		Bituminous-concrete binder course	500 lb. or higher	1800 lb. or higher
	Sand asphalt	Peak of curve*	**		Sand asphalt	500 lb. or higher	**
Unit Weight	Bituminous-concrete surface course	Peak of curve	Peak of curve	Unit Weight	--	Not used	Not used
	Bituminous-concrete binder course	Not used	Not used				
	Sand asphalt	Peak of curve	**				
Flow	--	Not used	Not used	Flow	Bituminous-concrete surface course	20 or less	16 or less
					Bituminous-concrete binder course	20 or less	16 or less
					Sand asphalt	20 or less	**
Percent voids total mix	Bituminous-concrete surface course	3.0	3.0	Percent voids total mix	Bituminous-concrete surface course	2-4	2-4
	Bituminous-concrete binder course	4.0	5.0		Bituminous-concrete binder course	3-5	4-6
	Sand asphalt	5.0	**		Sand asphalt	4-6	**
Percent voids filled with bitumen	Bituminous-concrete surface course	85	80	Percent voids filled with bitumen	Bituminous-concrete surface course	80-90	75-85
	Bituminous-concrete binder course	75*	65*		Bituminous-concrete binder course	70-80	55-75
	Sand asphalt	75	**		Sand asphalt	70-80	**

* If the inclusion of bitumen contents at these points in the average causes the voids total mix to fall outside the limits, then the optimum bitumen content should be adjusted so that the voids total mix are within the limits.

** Criteria for sand asphalt to be sued in the designing pavements for 200 psi tires have not been established.

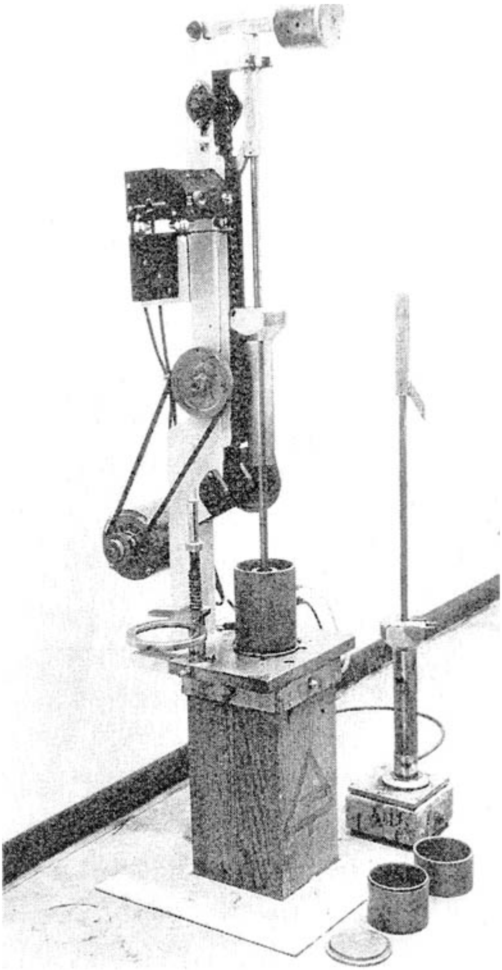


Figure 2. Pedestal, hammer (mechanical) and mold used in preparing Marshall test specimens. (Asphalt Institute 1994)

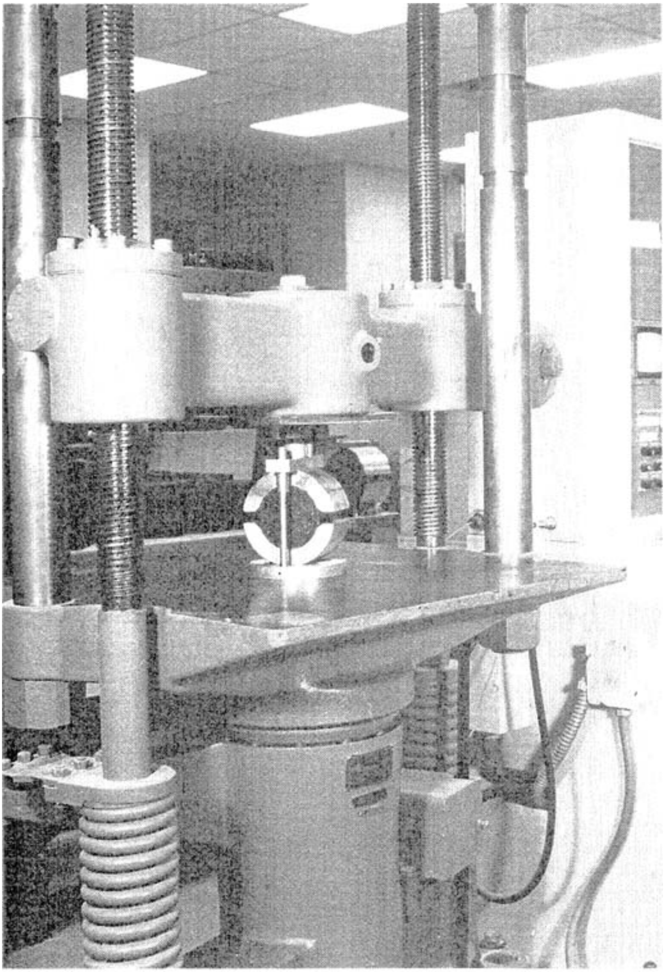


Figure 3. Marshall stability and flow test, using an automatic recording device. (Asphalt Institute 1994)

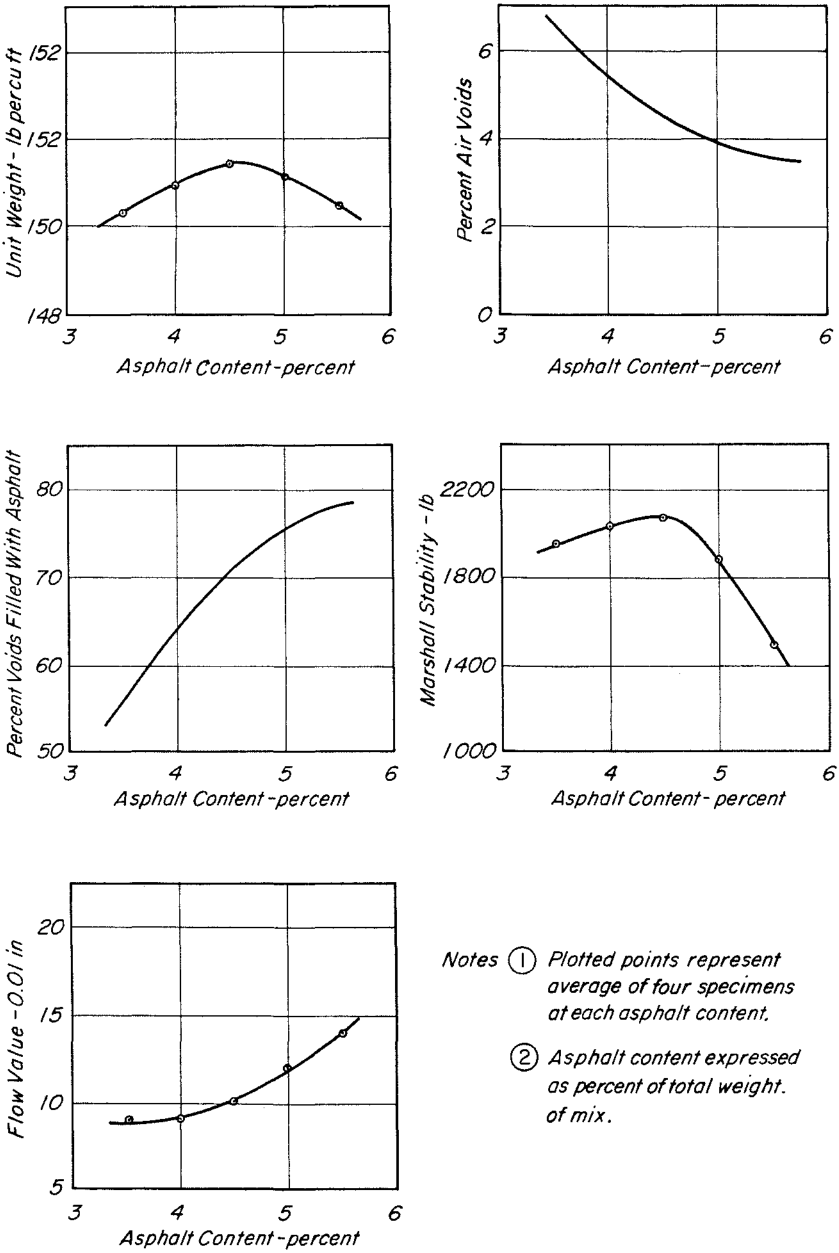


Figure 4. Example of Army Corps of Engineers method of asphalt mix design.

Table 3. Laboratory Test Data Summary, 75 blow Marshall compaction procedure.

Asphalt content (Percent of Total Weight Mix)	Unit Wt. lab/cu. ft. (average of four specimens)	Percent Air Voids (calculated from values taken from unit weight curve)	Percent voids Filled with Asphalt (calculated using same unit weight values as in voids determination)	Marshall Stability – lb. (average of four specimens)	Flow Value – 0.01 in. (average of four specimens)
3.5	150.3	6.6	55.7	1955	9
4.0	150.9	5.4	63.8	2037	9
4.5	151.4	4.5	70.4	2075	10
5.0	151.4	3.9	75.3	1884	12
5.5	150.5	3.6	78.3	1494	14

Table 4. Bitmumen Content and Penetration Grade of Asphalt for Various Temperature Index Ranges (Department of the Army)

Pavement Temp. Index	Asphalt Pen. Grade	Bitumen Content by Traffic Areas								
		Type A Traffic Areas			Type B and C Traffic Areas			Type D Traffic Areas[2]		
		Light Load Pavements	Intermediate Load Pavements[1]	Heavy Load Pavements	Light Load Pavements	Intermedi-ate Load Pavements	Heavy Load Pavements	Light Load Pavements	Intermediate Load Pavements	Heavy Load Pavements
Negative	120-150	---	Optimum	[3]	Opt. +10%	Opt. +10%	Optimum	---	Opt. +10%	Opt. -10%
0-40	100-120	---	Optimum	[3]	Optimum	Optimum	Opt. – 10%	---	Opt. +10%	Opt. –10%
40-100	85-100	---	Opt. –10%	[3]	Optimum	Optimum	Opt. –20%	---	Opt. +10%	Optimum
Above 100	60-70	---	Opt. –20%	[3]	Optimum	Opt- -10%	[3]	---	Optimum	Optimum

[1] Intermediate load pavements, for the purposes of this tabulation, include those for the twin bicycle, twin tricycles, and twin-tandem tricycle gear configurations for which design criteria are included in this manual.
[2] Blast zones within overrun areas are included with Type D traffic areas.
[3] Design bitumen content to be furnished by OCE at time of airfield design.

Pavement Temperature Index:

The sum, for a one-year period, of the increments above °75F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration, the record for the hottest year should be used. A negative index results when no monthly average exceeds 75°F. Negative indices are evaluated merely by subtracting the largest monthly average from 75°F.

To stimulate the long-term densities obtained under traffic loading, Vallerga and Zube introduced the concept of subjecting a specimen, compacted by the regular procedure at the design asphalt content, to additional compactive effort (Vallerga and Zube 1953). By testing this specimen in the stabilometer one could assess whether or not expected traffic compaction would have a deleterious effect on stability. Their work is briefly summarized as are some additional studies to evaluate the effects of traffic compaction on mix performance as measured by the stabilometer test on specimens subjected to additional compactive effort.

Mix Design Procedure (as developed by Hveem)

A brief summary of the methodology is as follows and associated California Test Methods utilized are included in Table 5.

1. Select type of aggregate form from a consideration of stability and grading of aggregate for workability, pavement texture, perviousness, and economy.
2. Select type and grade of asphalt.
3. Calculate surface area from grading (on a volume basis) and surface area factors.
4. Evaluate surface roughness and porosity of aggregate by CKE test procedure (California Test Method No. 303)
   a. kerosene equivalent (fine aggregate)
   b. oil equivalent (coarse aggregate)

Determine the preliminary design asphalt content by surface area procedure from CKE nomographs (CTM 303).

Note: Theoretically, the asphalt content obtained from the CKE test procedure and accompanying nomographs should be satisfactory since the calculation procedure has been correlated with field performance and actual laboratory tests. However, unless there has been previous experience with the particular aggregate, one does not know whether the mix will be stable or unstable, i.e., be able to support the anticipated traffic loading. In addition, from the CKE test procedure one does not know how the material will behave with small variations in asphalt content, a factor quite necessary from a field control standpoint, since it is possible that mixes leaving the asphalt plant will have variations of at least ±0.3 percent from the design value. Thus, the CKE procedure provides a starting point for laboratory testing of mixes.

Table 5. Caltrans (Hveem) Mix Design Procedure [Mixes with Coarse and Medium Gradations, Section 39, Standard Specifications (California Department of Transportation 1999)]

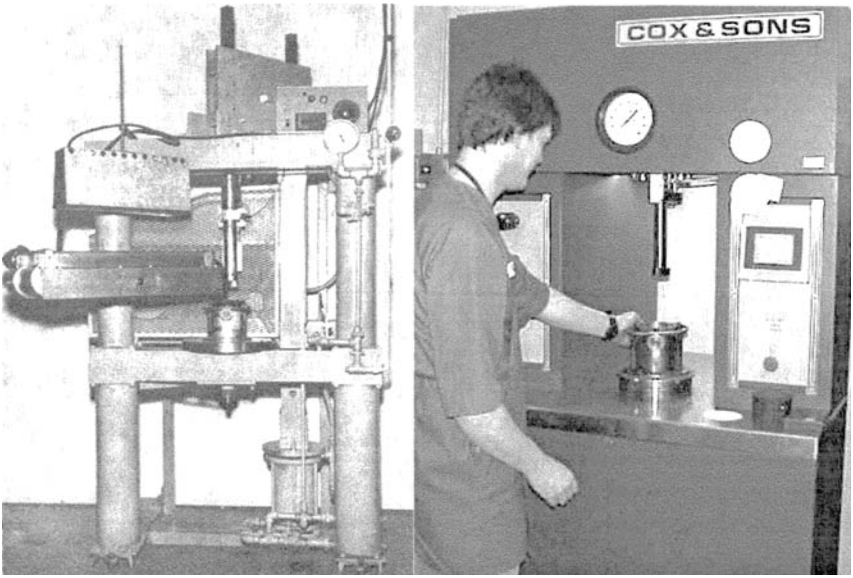
Material	Test	California Test Method
Aggregate	Sieve and Wash Analysis	202
	Sand Equivalent	217
	Los Angeles Abrasion	211
	Percent Crushed Aggregate	205
	Sodium Sulfate Soundness	214
	Specific Gravity (coarse aggregate)	206
	Specific Gravity (fine aggregate)	208
	CKE and Oil Equivalent	303
Asphalt	Absolute Viscosity	346 (AASHTO T202)
Mix	Approximate Bitumen Ratio (ABR)	303
	Specimen Preparation / Kneading Compaction	304
	Hveem Stabilometer	366
	Bulk Specific Gravity Compacted Mix	308
	Optimum Bitumen Content (OBC)	367
Other	Cohesiometer	306
	Swell	305
	Moisture Vapor Susceptibility	307
	Surface Abrasion Test	360

5. Prepare three trial mix specimens, one at the value indicated by the CKE procedure, one 0.5 percent above, and one 0.5 percent below the CKE value.
   a. Mixing – vertical bowl mixer, 5 minutes for specimens containing asphalt cements. Actual maximum mixing temperatures may vary from 250°F for 150-300 penetration asphalt cements up to 325° for 60-70 penetration asphalt cements.[8]
   b. Curing – 15 hours at 140°F.
   c. Compaction to prepare specimens 4 in. in diameter by 2½ in. high – kneading compaction (California Type Kneading Compactor, California Test Method 304), Figure 5 - 150 tamps at 500 psi tamping

[8] Mixes containing modified binders may be mixed at temperatures higher than 325°F.

pressure and 230°F compaction temperature for specimens containing asphalt cements. This compactive effort produces densities in laboratory specimens comparable to values now obtained in the field at the time of construction if good compaction procedures are followed.

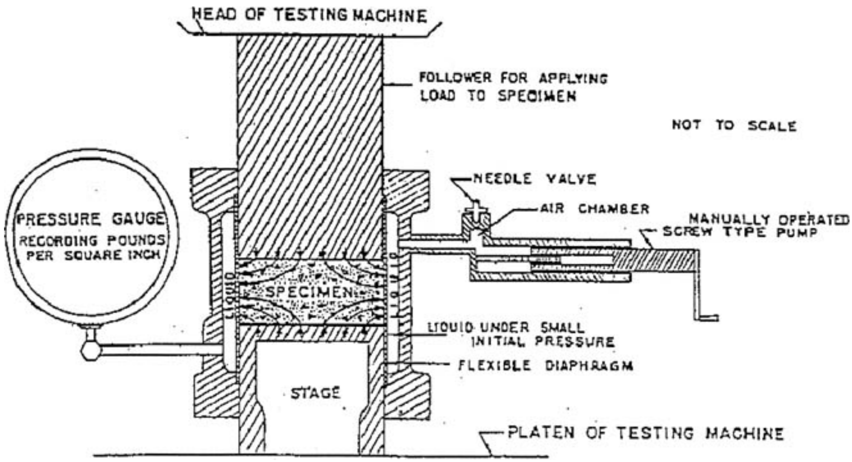
6. Laboratory testing to determine desirable mix properties.
   a. Hveem Stabilometer – closed system triaxial compression test (California Tests Method No. 366), Figure 6. In this test the lateral pressure is measured as the vertical stress is increased at a slow rate of deformation (0.05 in. per min). In asphalt mixes, the lateral pressure corresponding to a vertical pressure of 400 psi is used as a measure of stability. The test is performed at 140°F to simulate average high temperatures occurring in pavements during hot summer months.



A. Triaxial Institute Mechanical Compactor. 1949 -

B. Servo-hydraulic kneading Compactor.

Figure 5. Kneading Compactors.



NOTE: Specimen given lateral support by flexible side wall which transmits horizontal pressure to liquid. Magnitude of pressure may be read on gauge.

Figure 6. California (Hveem) stabilometer.

Since the test is conducted at this high temperature and at a slow rate of deformation, it measures primarily the frictional resistance of the aggregate mass and how this resistance is reduced due to the addition of asphalt. From the test results, a measure of stability is obtained from the following empirical equation:

$$S = \frac{22.2}{\frac{P_h \cdot D}{P_v - P_h} + 0.22} \qquad (1)$$

where

S	=	Relative stability (0 for water, approximately 90 for steel
P_h	=	Laterally transmitted pressure, psi,
P_v	=	Applied vertical pressure, usually 400 psi,
D	=	Number of turns of displacement pump handle (measure of roughness of surface of test specimen; D=2 for smooth specimen). This may be thought of as a correction to be applied to the laterally transmitted pressure. Due to the nature of the test, the roughness of the surface of the specimen influences the laterally transmitted pressure. Also it serves to standardize the instrument.

The state of California has set a minimum value of stability, depending on the type of mix [Section 39, Standard Specifications of State of California Department of Transportation (California Department of Transportation 1999)]:

Type A asphalt concrete – minimum value 37
Type B asphalt concrete – minimum value 35

These limits have been set on basis of correlation of laboratory tests with field performance.

b. Hveem Cohesiometer – bending or breaking test to determine the cohesive or tensile resistance of an asphalt mixture (California Test Method No. 306). This test is also conducted at 140°F. The cohesiometer value, C, is calculated from the equation:

$$C = \frac{L}{0.0800H + 0.178H^2} \quad \mathbf{(2)}$$

where

L	=	Load in grams required to fracture specimen,
H	=	Specimen height in inches

While not specified, a minimum cohesiometer value of 50 has been recommended for fresh mix to preclude surface disturbance caused by tires turning in place.*

c. Density determinations for the laboratory prepared mixes (California Test Method No. 308). From the density determinations, percent air voids in the compacted mixes are calculated. Design is adjusted if necessary to allow not less than 4 percent calculated air voids to preclude bleeding or possible loss in stability.

d. Tests for durability (resistance to water action):
   i. Swell Test – maximum value of 0.030 in. in 24 hours for dense graded aggregates (California Test Method No. 305)
   ii. Moisture Vapor Susceptibility (MVS) Test – for dense graded aggregate at design asphalt content (California Test Method No. 307). Minimum value of stability after MVS Test depends on mixture type:

      Type A asphalt concrete – minimum value 30
      Type B asphalt concrete – minimum value 25

   iii. Film Stripping Test – for open graded aggregate (California Test Method No. 302)

e. Section 39 of the Standard Specification also contains design criteria for mixes to be used as base courses and for open graded surface courses.

* Most mixes containing asphalt cements have "C" values considerably larger than 50. Hence in most instances this test is no longer performed.

Mix Design Example.

This example illustrates the selection of the design asphalt content for a mix containing a granite aggregate (rough surface texture). According to the CKE procedure a preliminary binder content, selected according to the surface area procedure (CKE procedure), is 5.2 percent (by weight of aggregate). This binder content is based on the following parameters:

Surface area	28.6 sq. ft. per lb.
C Value	2.65 gr. oil per 100 gr. coarse aggregate
F value	3.0 gr. kerosene per 100 gr. fine aggregate
Aggregate, sp. gr., G_{aggr}	2.92
Asphalt	AR-4000 asphalt cement

Five specimens were prepared and the following data were obtained from the laboratory tests:

Specimen No.	1	2	3	4 (MVS)	5 (Swell)
Asphalt Content – Percent	4.7	5.2	5.7	5.2	5.2
Relative Stability	46	41	34	32	-
Cohesiometer Value	220	250	260	-	-
Percent Air Voids	6.3	4.5	3.1	-	-
Swell – in.	-	-	-	-	0.008

The stability data are plotted in Figure 7. From this plot a value of 5.2 percent is selected. Actually, a value of 5.5 percent could be used since the stability corresponding to this asphalt content is 37. However, due to plant variations, some mixes leaving the plant could conceivably have stabilities less than 37. Thus, to allow for this variation, a value of 5.2 percent is selected. At this asphalt content the calculated percent air voids is approximately 4.5 percent, Figure 8; hence, no further adjustment is necessary. It will also be noted that the results of the swell and MVS tests are satisfactory. Therefore, the recommendation is adequate to satisfy these requirements as well.

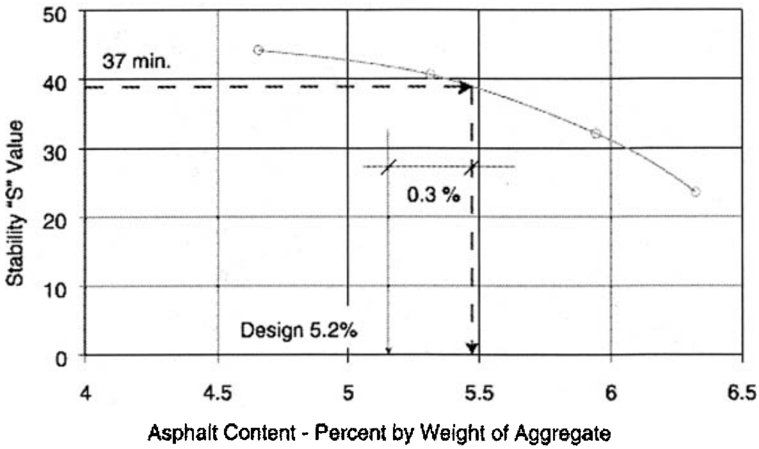


Figure 7. Stabilometer "S" value vs. asphalt content

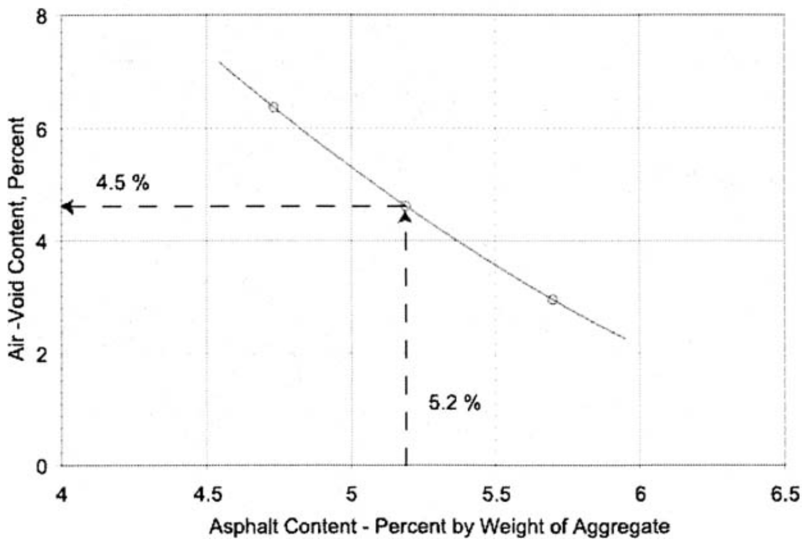


Figure 8. Percent air void vs. asphalt content.

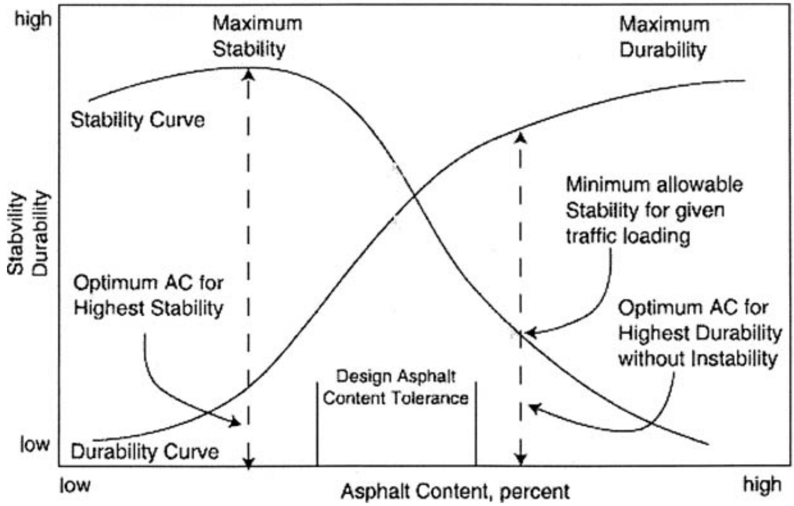


Figure 9. Schematic illustrating the philosophy of selecting the design binder content.

In this procedure, the basic philosophy of mix design is adhered to in that as much asphalt as possible is incorporated into the mix without reducing the stability below a minimum desirable value consistent with traffic and also taking into consideration production variations. This concept is illustrated in Figure 9.

Current Method – Design Asphalt Content Selection

As noted earlier, subsequent to the retirement of Hveem, the method for binder content selection was modified somewhat. California Test Method 367 reflects this modification and is sometimes referred to as the "pyramid method." Figure 10 from California Test Method 367 illustrates the selection of the design binder content termed optimum content (OBC).

In this revised procedure, four rather than three specimens are prepared as described earlier including: 1) the calculated value obtained from California Test Method 303, 2) two above in 0.5 percent increments, and 3) one 0.5 percent below the calculated value.

The added feature in this method (California Test Method 367), as compared to that described in the previous section, is the observation of surface flushing at the conclusion of kneading compaction. Surface flushing and/or bleeding is considered *slight* (acceptable), *moderate* (unacceptable), or *heavy* (unacceptable) and guidelines are provided to assist the person performing the testing.

The following example illustrates the modified procedure based on a calculated binder content of 5.5 percent (dense-graded aggregate, surface area of 32.3 sq. ft. per lb. and an AR-2000 asphalt cement).

Specimen No.	1	2	3	4
Asphalt Content – percent	5.0	5.5	6.0	6.5
Relative Stability	48	45	36	25
Percent Air Voids	4.5	3.6	3.0	1.8
Observing Flushing During Compaction	None	None	Slight	Moderate

Following the procedure in Figure 10, the asphalt contents are entered into the pyramid as shown below using the criteria for a Type A mix ($S \geq 37$).

5.0

5.0 5.5

5.0 5.5 6.0

5.0 2	5.5 2	6.0 2	6.5 2

Guidelines are provided in Method 367 for selecting the recommended range for binder contents called for in Figure 10. Following the procedure described in the previous section the recommended value would be 5.3 percent (based on a minimum air void content of 4.0 percent) as seen in Figure 11; the recommended range for this example would be 5.0 to 5.3 percent.

Modification – Added Compactive Effort.

As a part of an investigation of different design methods for asphalt mixes, Vallerga and Zube subjected a mix for which they had field density measurements, to different levels of compactive effort in the kneading compactor (Vallerga and Zube 1953). The results of this investigation are shown in Figure 12. It will be noted that increased members of tamps result in both an increase in density and an increase in stability for this mix up to a point beyond which the stability drops off with further increase in density.

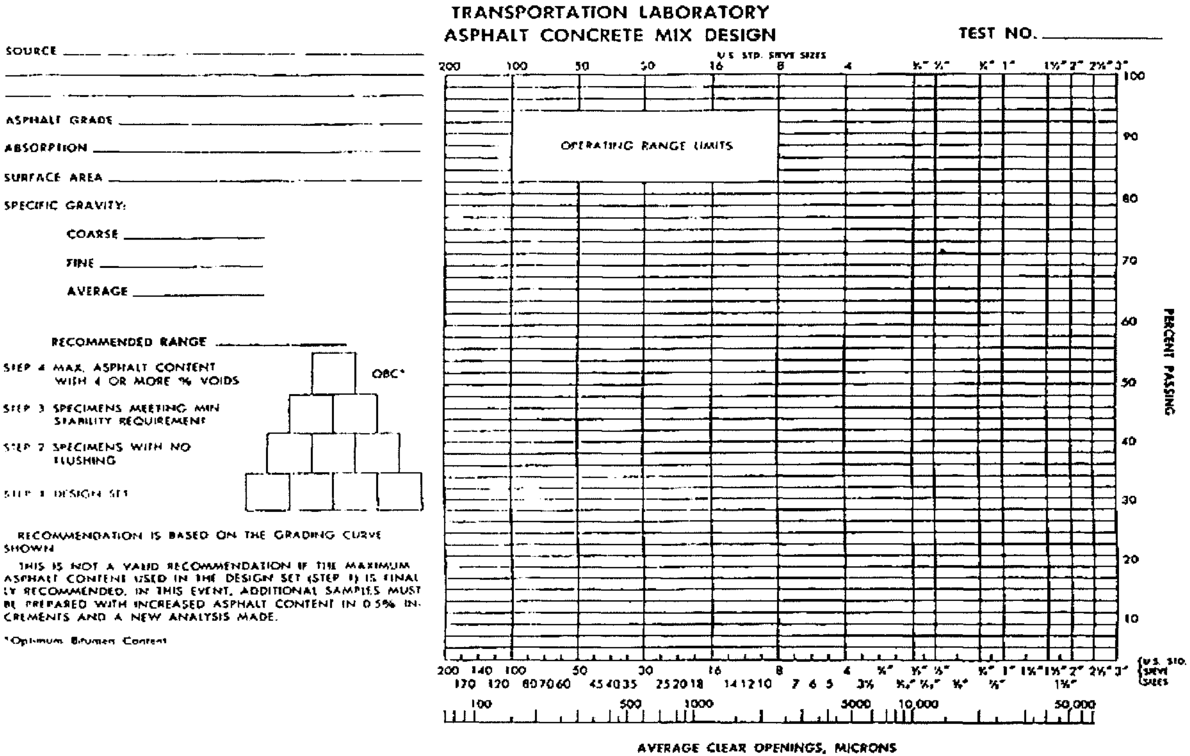


Figure 10. Asphalt content selection – California Test Method 367.

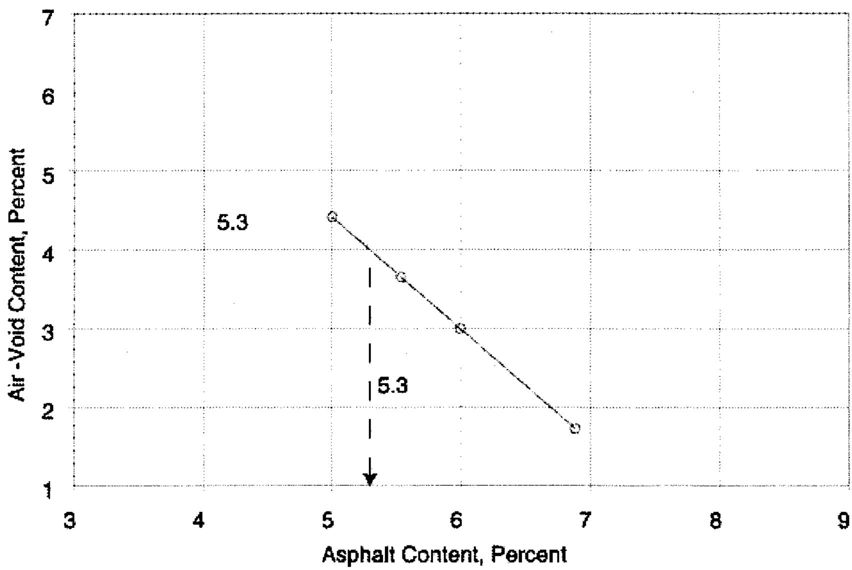


Figure 11. Air -void content vs. asphalt content.

As seen in Figure 13, which is based on the results of triaxial compression tests on specimens prepared by kneading compaction (Monismith and Vallerga 1956), when the volume of air, V_{air}, of air in the mix is reduced below about 2.0 to 2.5 percent, the stability (as measured by the stress at 2 percent strain) drops off significantly.

The data presented in Figure 12 suggested that to evaluate the effects of additional traffic it would be appropriate to subject a compacted specimen at the design asphalt content to additional compactive effort. Vallerga and Zube recommended that 500 tamps at 500 psi and a temperature of 140°F (60°C) would be an appropriate combination of compactive effort and temperature to use.

The importance of added compaction is illustrated in Figure 14 (Tayebali 1990). In this instance, mixes with a single aggregate type and gradation were prepared over a range of asphalt contents. Based on stabilometer criteria and depending on the traffic site, a suitable mix design would likely range from about 5.0 to 4.2 percent. If the traffic were very heavy and the environment quite hot, then mix behavior at 1000 to 1200 tamps might be the appropriate and the design asphalt content would be less than 4.7 percent. Reference (Vallerga et al. 1995) illustrates this point for a pavement in the Dubai area.

Combination of USACE/FAA and Hveem Procedures

At times the two procedures have been used together. A recent example is that for mixes used on the taxiways at the San Francisco International Airport (Vallerga et al. 2000, Monismith et al. 2000). In this instance, because of rutting due to the stop-and-go operations and slewing action of fully-loaded Boeing 747-400 aircraft, requirements for mixes meeting the current FAA specifications (San Francisco International Airport 1996) were modified to include the Hveem Stabilometer test results and the added compaction consideration described above.

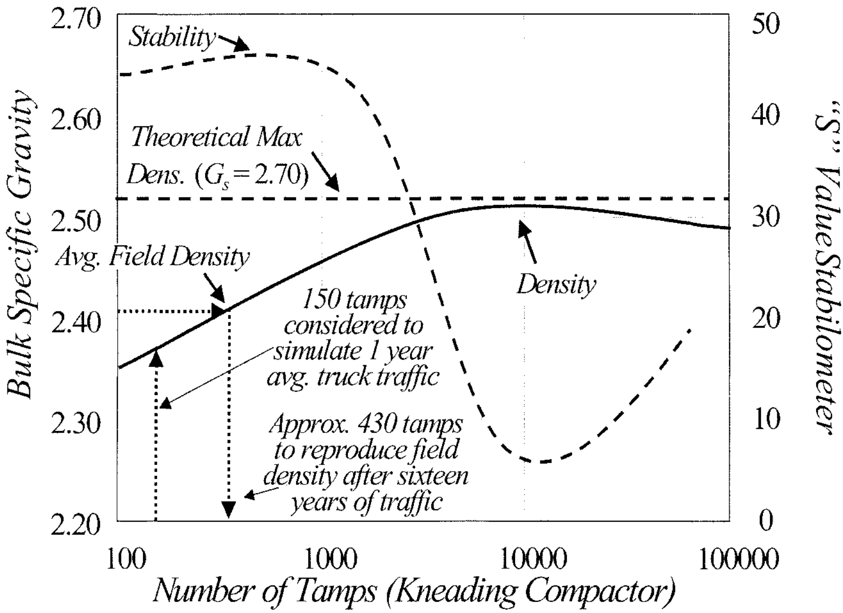


Figure 12. Effect of prolonged kneading compaction on density and stability.(Vallerga and Zube 1953)

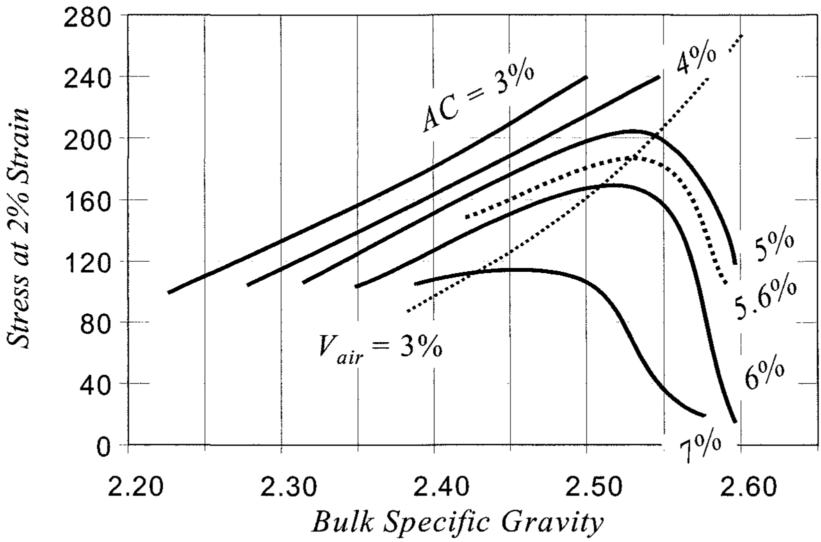


Figure 13. Relationship between build specific gravity and stress at 2 percent strain for constant asphalt content.

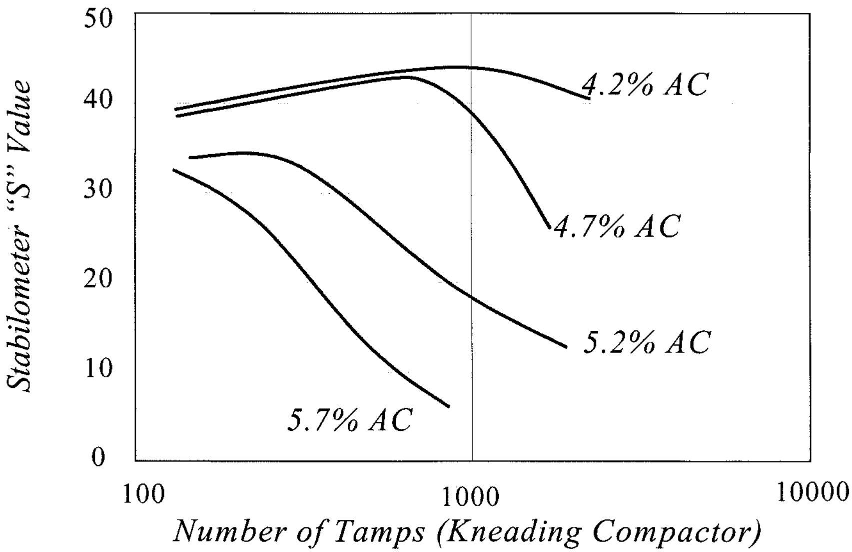


Figure 14. Effect of asphalt content and the compaction effort in the Triaxial Institute Kneading Compactor on the Stabilometer "S" value. (Tayebali 1990)

Mixes, termed *High Stability* mixes, used for taxiways and aprons are now specified as follows (San Francisco International Airport 1996):

For High Stability mixes, verification of the Job Mix Formula (JMF), developed by the Marshall Method of Mix Design, shall be performed on 4 in. by 2.5 in. test specimens:

a. Fabricated from Marshall JMF mix by kneading compaction at each of two levels of compaction as follows:
   - Level 1 – 150 tamps at 230°F (110°C)
   - Level 2 – 150 tamps at 230°F (110°C) followed by 1000 tamps at 140°F (60°C)

b. Tested in the Hveem Stabilometer in accordance with Hveem Method of Design in Manual Series MS-2* shall be required to meet the following criteria:

Compaction Level	"S" value, minimum
Level 1	40
Level 2	35

SUPERPAVE* and RELATED METHODOLOGY

The Superpave mix design procedure is one of the products of the Strategic Highway Research Program (SHRP) (Kennedy et al. 1994). As originally developed, the method included both a volumetric design procedure and performance tests on the mix or mixes resulting from the volumetric design (Cominsky 1994, Cominsky et al. 1994). At this time (2002) only the volumetric method is being used by a number of states and other agencies, as discussed by Harman in this symposium. The performance tests, to predict in-situ mix performance, have either been modified [e.g. low temperature cracking (Roque et al. 1995)] or new test methodology is under development [e.g. permanent deformation / rutting (Witczak et al. 2000)]. The new test methodology includes development of a simple performance test or tests; developments in the area of performance prediction and the associated tests are discussed by Witczak and Bonaquist. References (Asphalt Institute 1997, Asphalt Institute 1994) are excellent sources of information on both the binder and mix requirements and tests associated with Superpave.

Superpave Volumetric Mix Design

The volumetric mix design procedure includes three steps: (a) selection of component materials; (b) selection of design aggregate structure; and (c) selection of the design asphalt content. Upon completion of these three steps the resulting mix design is checked for water sensitivity, in effect following the process shown in Figure 1.

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* Manual Series MS – 2, (The Asphalt Institute 1994) .

* The material presented in this section on Superpave is adapted from material included in the *Hot-Mix Asphalt Paving Handbook 2000* prepared under the aegis of the Transportation Research Board and published by the U.S. Army Corps of Engineers and the Federal Aviation Administration (Transportation Research Board 2001).

Selection of Asphalt and Aggregate. This step requires selection of the appropriate asphalt/binder performance grade (PG) and aggregate with the requisite characteristics for the traffic applied.

The asphalt/binder requirements are defined by the PG specifications (AASHTO-MP-1) (American Association of State Highway and Transportation Officials 1997). These specifications help in selecting a binder grade that will limit the contribution of the binder to low-temperature cracking and permanent deformation (rutting) and may assist in controlling fatigue cracking under some circumstances within the range of climate and traffic associated with the project site.

As compared to earlier asphalt specifications [based on penetration (American Association of State Highway Transportation Officials A) or viscosity (American Association of State Highway Transportation Officials B)], the physical properties included in the PG specifications remain constant; however, the temperatures at which the properties must be achieved vary depending on the climate in which the binder is expected to serve. Consider, for example, a binder with the designation PG 64-22; this binder is expected to provide adequate service for a properly designed mix in an environment in which the average

7-day maximum pavement design temperature is 64°C (147°F) or lower, and the minimum pavement design temperatures is -22°C (-8°F) or higher. Reference (Asphalt Institute 1997) contains an excellent discussion of this system.

To control the surface texture and angularity of the aggregate, the *coarse aggregate angularity* and *fine aggregate angularity* tests have been included in the aggregate specifications (Asphalt Institute 1994). In addition the specifications require a control of clay content [as measured by the *Sand Equivalent Test* (American Association of State Highway and Transportation Officials C)] and of the amount of flattened elongated particles (American Society for Testing and Materials 2002).

Design requirements for the parameters are more stringent as the traffic, expressed in equivalent 80-kN (18000 lb.) single-axle loads (ESALs), increase.

Aggregate gradation is specified using the aggregate gradation chart developed by the Bureau of Public Roads [now the Federal Highway Administration (FHWA)] (Goode and Lufsey 1962). The chart together with the requirements added to the chart for gradation control are shown in Figure 15a for a 19 mm (3/4 in.) maximum size while Figure 15b shows an aggregate grading conforming to these requirements. Details of the basis for the requirements are discussed in Reference (Asphalt Institute 1994).

Selection of Design Aggregate Structure and Design Asphalt Content. At least three trial blends of aggregate are selected which meet the grading requirements (which are a function of the nominal maximum size of the aggregate used). Selection of the design aggregate structure is then made on the basis of the properties of the mixes compacted using the Superpave gyratory compactor, Figure 16.*

For each of the blends, the trial asphalt content (mix basis) is used that is either calculated to produce 4 percent air voids at a design number of gyrations in the

* This equipment was patterned after the original Texas gyratory compactor (Philippi) which served not only as the basis for the SHRP gyratory compactor, but also for the Corps of Engineers and LCDC gyratory compactors.

compactor or selected based on experience. The design number of gyrations, N_{design}, is established as a function of traffic (design ESALs) and climate (air temperature).

Heavily trafficked pavements require a relatively high N_{design}, while low-volume pavements require low N_{design}. Because the asphalt content used during this step is merely a trial value, 4 percent air voids is rarely achieved at N_{design}. Accordingly, the compacted properties of each trial blend are evaluated to estimate an asphalt content that would produce 4 percent air voids. The following parameters are then estimated for each of the trial blends: (1) VMA at N_{design}; (2) VFA at N_{design}; (3) percent of maximum theoretical density at $N_{initial}$; (4) percent of maximum theoretical density at $N_{maximum}$; and (5) dust proportion P_{200} /effective Pw_{asp}.

The parameter $N_{initial}$ is calculated from N_{design}. $N_{initial}$ represents mix response during initial compaction, as in breakdown rolling. A high density at $N_{initial}$ is generally considered undesirable since it is likely that the mix would compact very easily, and thus could be susceptible to rutting. Although some data indicate this, it is not always true.

A high density at $N_{maximum}$ is also considered undesirable since $N_{maximum}$ represents a traffic level much higher than that for which the project is designed. By limiting the density at $N_{maximum}$, it is expected that the mix will not densify to extremely low air voids with unexpectedly high traffic. Figure 17 illustrates schematically mix response in the gyratory compaction test; Figure 17a shows the relationship between mix density (expressed in terms of the theoretical maximum density) and gyrations while Figure 17b shows the relative positions of $N_{initial}$ and $N_{maximum}$ to N_{design}. Figure 17c illustrates the influence of aggregate structure and binder content on the compaction relationship.

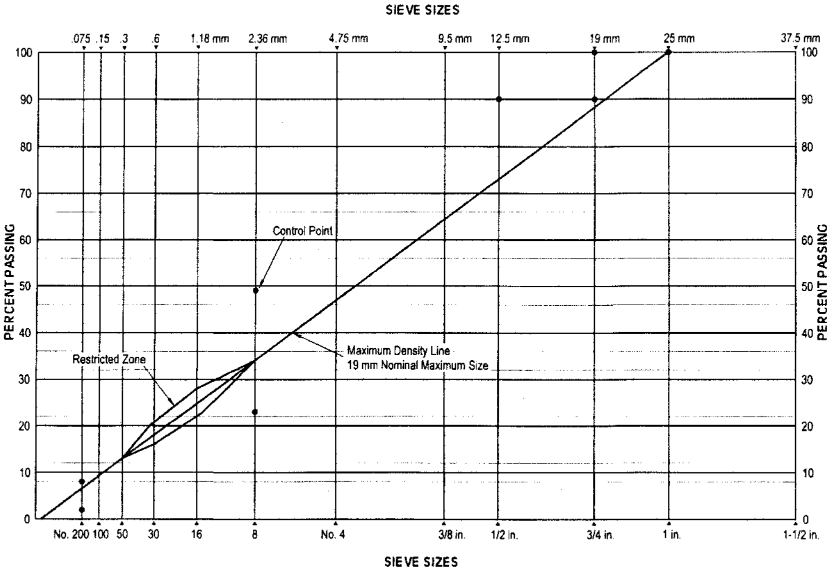
The trial blends are then compared with established criteria, and a blend estimated to meet the criteria is selected. This blend is termed the design aggregate structure. To determine the design asphalt content, trial specimens are compacted at N_{design}, with the design aggregate structure at four different asphalt contents bracketing the estimated asphalt content (usually duplicates at each asphalt content).

Volumetric properties of the compacted mix (e.g., air voids, VMA) are determined for the four asphalt contents. The design asphalt binder content is selected to achieve 4 percent air voids at N_{design}. Usually, the design asphalt binder content is within 0.1 to 0.2 percent of the estimated binder content from the previous step. Reference (Jester 1997) provides a summary of experience and applicability of the Superpave system through 1996. It includes discussions of field and laboratory investigations pertaining to both binders and mixes.

Water Sensitivity Evaluation

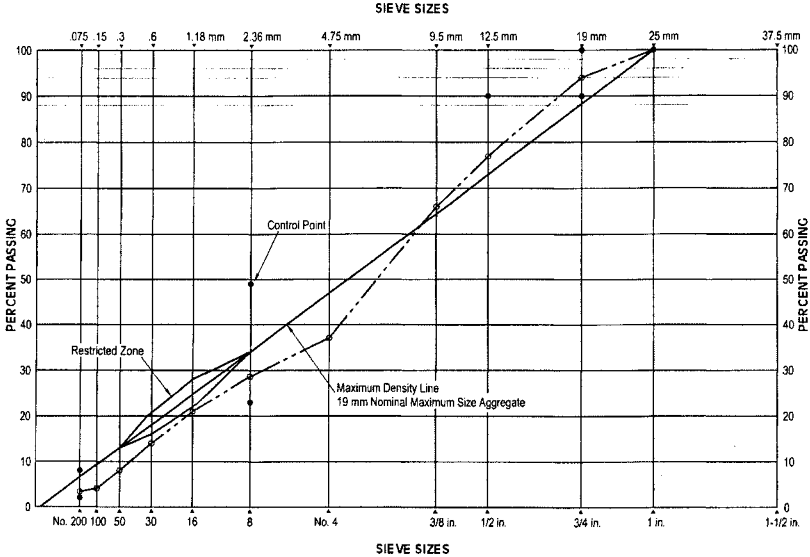
When the design mix has been selected, it is subjected to a water (moisture) sensitivity evaluation. For this purpose the AASHTO T-283 procedure (American Association of State Highway and Transportation Officials D) is utilized.

Specimens are compacted to approximately 7 percent air voids. One subset of three specimens are considered control specimens. The other subset of three specimens are conditioned. Conditioned specimens are subjected to partial vacuum saturation followed by an optional freeze cycle, followed by a 24 hour thaw at 60°C. All specimens are tested to determine their indirect tensile strengths. Moisture sensitivity is determined as a ratio of the average tensile strengths of the conditioned subset divided by the average tensile strengths of the control subset. The Superpave criterion for tensile strength ratio is 80 percent, minimum (Cominsky et al. 1994, Asphalt Institute 1996).



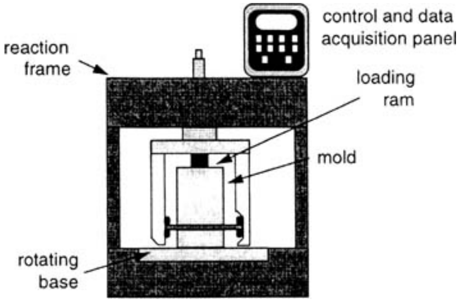
a. **Superpave gradation chart showing control points and restricted zone for aggregate gradation with 19-mm nominal maximum size.**

Figure 15a. Superpave gradation chart; grading requirements and example aggregate grading, 19 mm (3/4 in.) nominal maximum size.

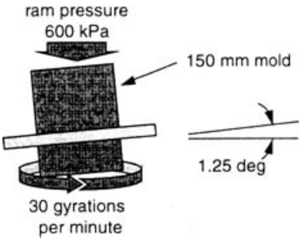


b. Aggregate grading meeting Superpave criteria and passing below restricted zone.

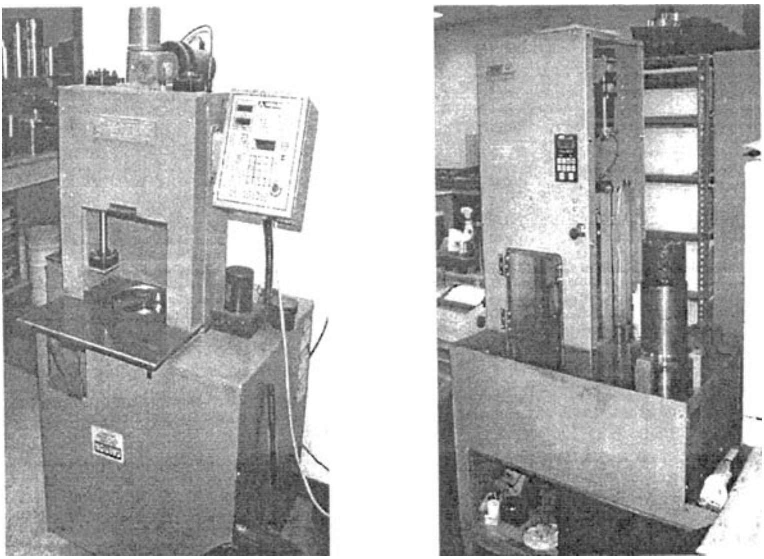
Figure 15b. Superpave gradation chart; grading requirements and example aggregate grading, 19 mm (3/4 in.) nominal maximum size.



a. Compactor Schematic



b. Compaction Mold Configuration



c. Compaction Equipment Examples

Figure 16. Superpave Gyratory Compactor

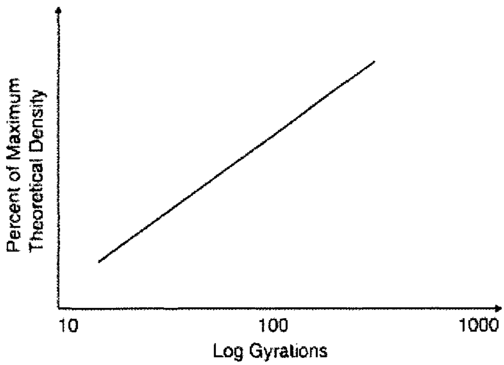


Figure 17a. Schematic representations of gyratory compactor data.

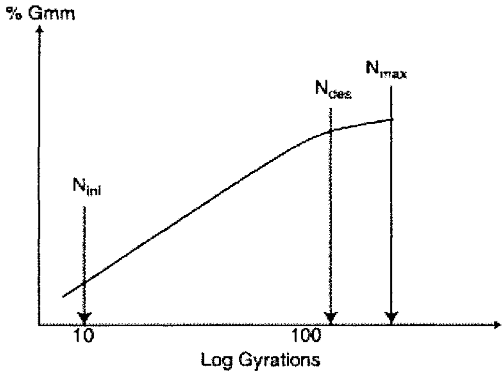


Figure 17b. Schematic representations of gyratory compactor data.

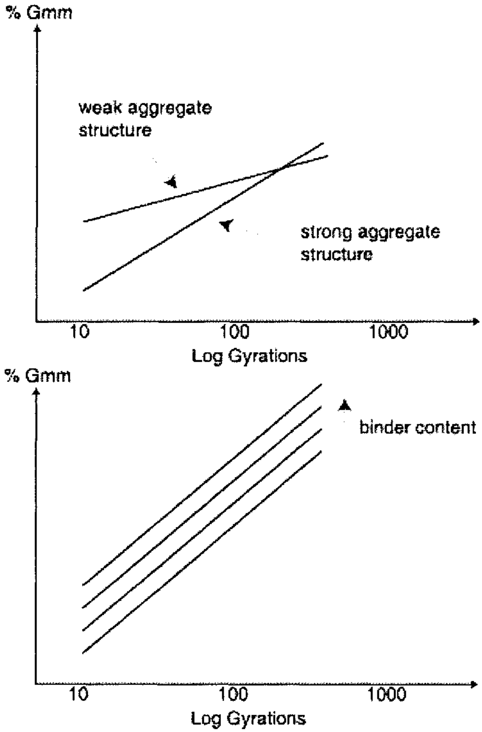


Figure 17c. Schematic representations of gyratory compactor data.

EXAMPLES OF APPROACHES USING PERFORMANCE-RELATED TESTS

With the advent of mechanistic-empirical pavement design and rehabilitation, performance-related tests have been developed not only for structural thickness design but also for asphalt mix design. While there have been a number of such procedures presented, e.g., References (Finn 1967, van de Loo 1978, Viljoen and Meadows 1981, Cooper et al. 1985, Bonnot 1986, Monismith et al. 1989, Von Quintus et al.1991, Lytton et al. 1993, University of California at Berkeley et al. 1994), only a few examples will be illustrated herein. These include a procedure developed by Shell researchers to predict rutting in asphalt concrete layer using the results of static creep tests (Asphalt Institute 1994) and applied to mix design/analysis in Saudi Arabia and the Southwestern United States (Whiteoak 1990), and three SHRP developed procedures – one for rutting using the repeated simple shear test at constant height (RSST-CH) (Sousa at al. 1994), a second for flexural fatigue (Deacon et al. 1994), and a third for low temperature cracking evaluation using the thermal stress restrained specimen test (TSRST) (Jung and Vinson 1994A).

Rut Depth Prediction using Static Creep Test

An approach to rut depth prediction was developed by the Shell researchers [e.g., Reference (Finn 1967)] and incorporated in the Shell pavement design procedure. The purpose was to permit the designer to compare the permanent deformation characteristics of different mixes as measured in creep and to select a mix or mixes in which the estimated rutting for specific traffic loading and environmental conditions would not exceed some predetermined value.

The rut depth at the pavement surface due only to permanent deformations in the asphalt-bound layer is determined from the following expression:

$$\Delta h_1 = C_M \cdot h_1 \cdot \frac{\sigma_{av}}{S_{mix}}, \ mm \tag{3}$$

where:

C_M = correction factor for the so-called "dynamic effect" which takes account of differences between static (creep) and dynamic (rutting) behavior. This factor is dependent on the type of mix and has been found empirically to be in the range 1–2;

h_1 = design thickness of asphalt layer, m;

σ_{av} = the average stress in the pavement under the moving wheel, N/m^2

$\sigma_{av} = Z \cdot \sigma_0, N/m^2$

Z is a parameter dependent on the stiffness of the pavement layers; σ_0 is the contact stress between tire and pavement;

S_{mix} = the value of the stiffness of the mix at $S_{bit} = S_{bit,\ visc}$.

If it is to be desired to subdivide a thick asphalt-bound layer into a number of sub layers, equation (3) can be stated:

$$\Delta h_1 = C_M \sum_{i=1}^{n} \left[h_{i-1} \cdot \frac{(\sigma_{ave})_{i-1}}{(S_{mix})_{i-1}} \right] \tag{4}$$

Figure 18 shows an example of creep test equipment for asphalt mixes as utilized by the Shell investigators (Whiteoak 1990). To illustrate the use of this methodology, examples are included for mix evaluation for an airfield pavement in Saudi Arabia and for mixes used in street intersections in Las Vegas, NV (Finn et al. 1983). For the Saudi Arabian project, mixes for the taxiways and runways for a major new airport were designed by the US Army CE procedure (200 psi tire criteria). Hveem stabilometer tests and creep tests were also performed.

Based on the results of the US Army CE procedure, a preliminary design binder content (60–70 pen asphalt) of 6 percent by weight of mix was selected. Hveem stabilometer results, however, suggested that the stability characteristics of the mix were very sensitive to asphalt content as seen in Figure 19 with a change in binder content from 4.5 to 5 percent resulting in a decrease in the stabilometer "S" value of about 30 points.*

The Shell procedure was selected to obtain a measure of the relative permanent deformations of mixes containing 4.5 and 6.0 percent binder. Creep tests were performed by kneading compaction in the temperature range 70°F (21°C) to 100°F (38°C). Results of these tests are shown in Figure 20 and results of compactions using the Shell procedure [Equation (4)] are shown in Figure 21. Based on the results of these analyses, test sections of the mix were constructed and subjected to proof rolling immediately after construction (March 1981) and during a period with high ambient temperature (August 1981). Results of these tests are summarized in following tabulation:

Time	Coverages	Cumulative Deformation (in.)			
		Average		Maximum	
		4.9 Percent	5.4 Percent	4.9 Percent	5.4 Percent
March 1981	1,150	0.05	0.08	---	---
August 1981	1,224	0.05	0.04	0.09	0.12

* This mix would be considered a critical mix, i.e., a small increase in asphalt content causes a significant reduction in stability.

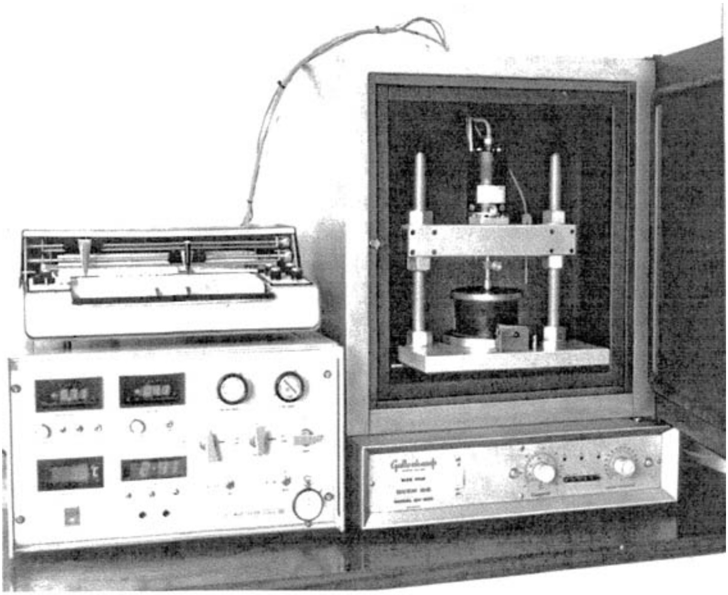


Figure 18. Example of creep test equipment. (Whiteoak 1990)

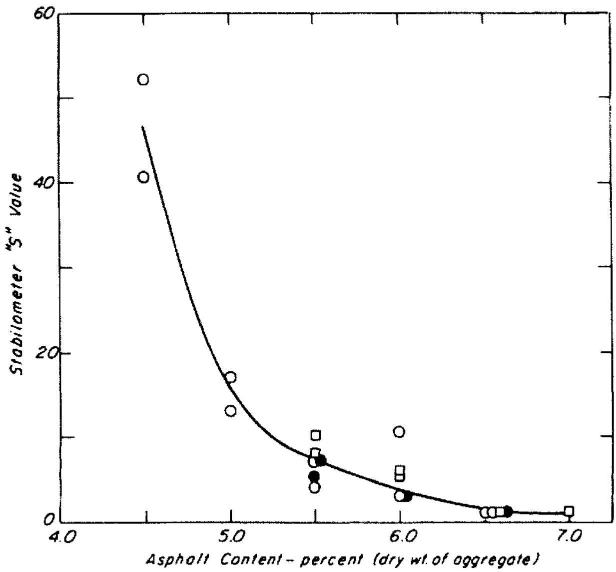


Figure 19. Stability (stabilometer "S" value) versus asphalt content relationship.

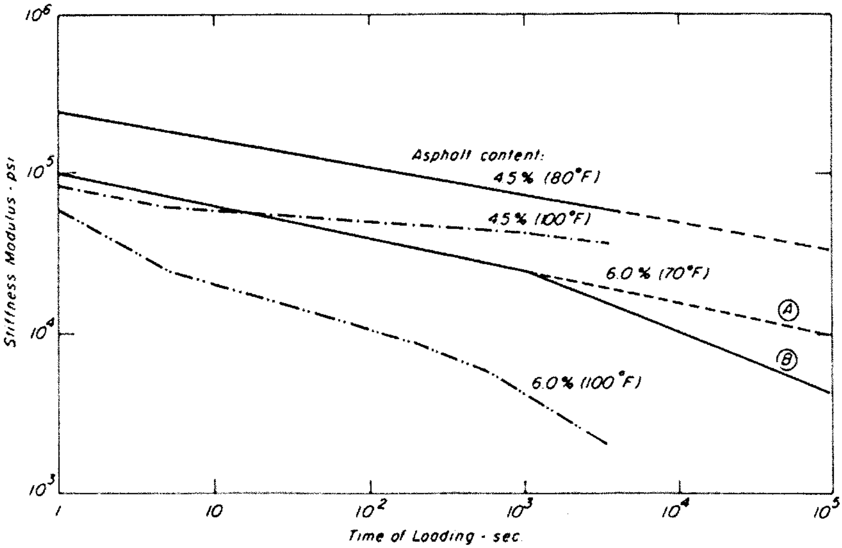


Figure 20. Average creep curves for surface course mix at asphalt contents of 4.5 and 6.0 percent (by weight of mix).

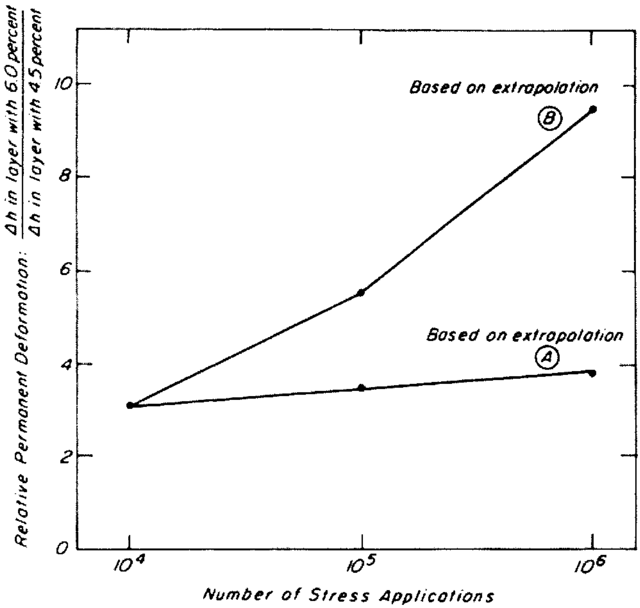


Figure 21. Relative permanent deformation versus number of load applications.

Based on the results of the new test program, a design binder of 4.9 percent content was selected. Results of performance over at least a 10-year period (through 1991) corroborated this selection. The second project involved investigations of a number of mixes, some of which contained absorptive aggregates and others non-absorptive aggregates. These mixes were in use in the Las Vegas, Nevada area and had been designed using the Marshall stability test and following the criteria established by the Asphalt Institute (Asphalt Institute 1994).

Four of the mixes were evaluated, two with absorptive aggregates (Projects 1 and 2) and two with non-absorptive aggregates (Projects 3 and 4). Results of Marshall stability and flow values for remolded cores specimens and stabilometer "S" values for both cores and remolded specimens are summarized in Table 6. From these data as well as other information included in Reference (Whiteoak 1990), it was concluded that:

1. High Marshall stability values do not assure adequate rutting resistance;
2. Stabilometer "S" values decrease significantly as the air-void content is reduced below 4.0 percent.
3. Stabilometer "S" values are generally less than the values recommended for highway design, i.e., S = 35 min.
4. Flow values from the Marshall test are high, i.e., values in range 15 to 23 for four of the eight conditions shown in Table 6.

Creep tests like those described above were performed at 100°F (38°C) on core specimens. Creep moduli were ascertained at different times after the initial stress application. In this study only the creep moduli obtained after one hour of load application were reported. Results of the creep tests are included in Table 7. Also shown in this table are the mean stabilometer "S" values. In general, it will be noted that as the stabilometer "S" value increases, the creep modulus also increases. If the creep data were plotted versus the "S" values, it would be noted that a creep modulus of 34,500 psi would correspond to an "S value of 35.

Interestingly, the airport pavement data reported earlier exhibited similar trends, i.e., for an average stabilometer "S" value of 46 (4.5 percent asphalt) the creep modulus at 100°F and one hour loading time was 37,000 psi; whereas for a stabilometer "S" value of 1 (6.0 percent asphalt), the creep modulus for the same conditions is 2,000 psi.

Based on the result of this test program, modifications were recommended to the mix design procedure and subsequently incorporated by the local government involved.

Table 6 Stability Data – In Situ Mixes

Project	Performance	Marshall Test Data		Stabilometer "S" Value	
		Stability lb.	Flow (0.01 in.)	Core	Remolded Specimen
1	Acceptable	5,480	10	31	20
	Unacceptable	4,570	12.5	7	10
	Nontrafficked (I)				29
	Nontrafficked (II)				18
2	Acceptable	4,590	16	29	12
	Unacceptable	4,100	17	1	5
	Nontrafficked			17	8
3	Acceptable	4,540	10	45	22
	Unacceptable	3,595	15	6	2
	Nontrafficked			42	21
4	Acceptable	3,435	11	22	16
	Unacceptable	2,625	23	3	3
	Nontrafficked			24	7

Table 7 Creep Moduli and Stabilometer "S" Values Associated with Different Levels of Performance

Performance	Creep Modulus – psi			Stabilometer	
	Mean	Standard Deviation	Number of Specimens	Mean "S" Value	Number of Specimens
Acceptable	29,000	7,900	8	8	8
Unacceptable	13,400	5,300	9	9.0	14
Nontrafficked Upper Section	32,600	11,600	6	28,36	-
Nontrafficked Lower Section	20,900	4,800	6	----	-

While this approach, that is the use of the creep test to define S_{mix}, has worked satisfactorily for mixes containing conventional asphalt cements, Valkering, et al. (1990) reported that when the procedure was used for mixes containing non-conventional binders "a correction is required to take account of the different relationship between rutting and binder viscosity. The dynamic creep test has shown potential for a more universal applicability, extending to include those asphalts based on a *modified* binder…The greater suitability of the dynamic test for rating the effect of the binder modification is ascribed to the recovery effects of the test."

In another publication (Lizenga 1997), the work of the Shell investigators suggest that the use of creep test data may overpredict rutting for mixes containing some modified binders.

The research of Tanco (1992) on the pavement deformation response of conventional and modified asphalt-aggregate mixes under simple and compound loading conditions supports the work of the Shell investigators. He found that repetitive load tests were more responsive to the presence of modified binders in AC mixes than static constant load (creep) tests. Similar research had been reported earlier by Tayebali (1991).[1]

[1] It should be noted that Tayebali and Tanco presented convincing evidence that a conventional test like the Hveem Stabilometer, while adequate for mix design with conventional asphalt binders, is not suitable for mixes containing modified binders.

SHRP-Developed Methodologies

During the SHRP program a number of mix performance-related tests were developed including those to measure: (1) shear stiffness and permanent deformation characteristics (Sousa et al. 1994); (2) low temperature cracking potential (Jung and Vinson 1994A); (3) flexural stiffness and fatigue response (Deacon et al. 1994) and (4) water sensitivity (Terrel and Al-Swailmi 1994). In this section the shear, fatigue, and low-temperature cracking tests will be briefly discussed together with their use for mix evaluation.

Constant Height Simple Shear Test. The objective of the constant height shear test is to measure properties associated with shear response of HMA. Analyses (Weissman 1997, Harvey et al. 2001) suggest that the primary contributor to rutting is shape distortion which results from shear stresses. For the shear test, the majority of testing in current practice is done with cylindrical specimens with a diameter of 150 mm (6.0 in.) and a height of 50 mm (2.0 in.).

An approach to mix design and analysis using the RSST-CH was presented in Reference (Sousa et al. 1994) and has been used in a number of instances for this purpose (Monismith et al. 2000, Harvey et al. 1995, , Monismith et al. 2001, Monismith et al. 2002). Essentially the procedure consists of testing a mix over a range in binder contents and selecting the highest binder content which will permit the mix to accommodate the design traffic at a critical temperature* without exceeding some predetermined level of rutting, e.g. 12 to 13 mm (0.5 in.).

The RSST-CH test (Figure 22) is conducted for 5,000 stress repetitions, or a permanent shear strain of 5 percent if this occurs at less than 5,000 repetitions. Figure 23 illustrates a relationship between permanent shear, γ_p, and stress repetitions, N, for a mix containing PBA-6a* (PG 64-40) binder tested at 50°C. Results of tests for this mix over a range in binder contents are plotted as shown in Figure 24. Also shown on the figure are results for a mix containing the same aggregate and grading with an AR-8000 (PG 64-16) asphalt cement.

The tests are usually done with specimens prepared by rolling wheel compaction to an air-void content of 3 to 3.5 percent. The test is performed at the critical temperature, 50°C for the data shown in Figure 22 (Deacon et al. 1994).

The design procedure is illustrated schematically in Figure 25. As seen in this figure, the results of the laboratory tests, termed N_{supply} (the repetitions corresponding to 5 percent shear strain), must satisfy the following:

$$N_{supply} \geq M \cdot N_{demand} \tag{5}$$

In this expression, N_{demand} is determined from the estimate of the design ESALs, a temperature conversion factor (TCF) which converts the traffic applied year round to an equivalent number applied at the critical temperature, and a shift

* The critical temperature is defined as the temperature at a 50 mm (2 in.) depth at which the maximum permanent deformation occurs, assuming in this case that the truck traffic is applied at a uniform rate throughout the year.

factor (SF) which converts the repetitions applied in the field to an equivalent number in the laboratory; that is:

$$N_{demand} = Design\ ESALs \cdot TCF \cdot SF \qquad (6)$$

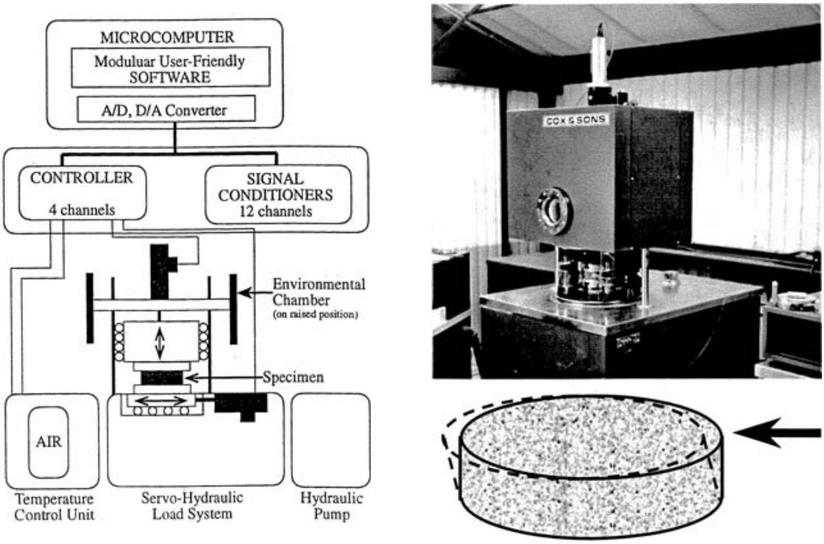


Figure 22. Shear test.

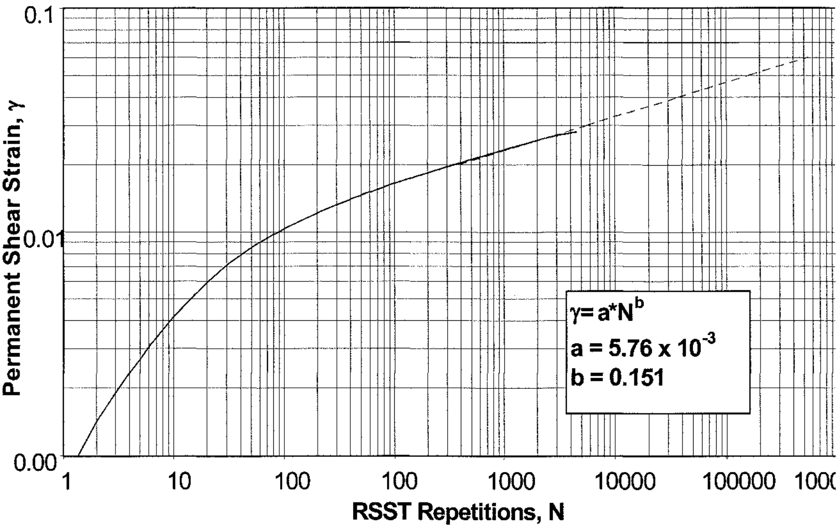


Figure 23. Permanent shear strain versus stress repetitions in RSST-CH at 50°C; PBA-6a* mix, 4.7 percent binder control.

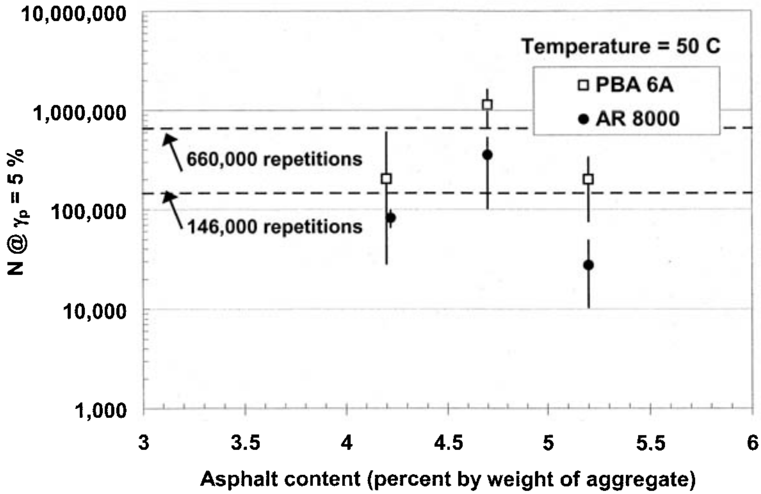


Figure 24. Repetitions to a permanent shear strain of 5 percent versus binder content; tests performed at 50°C (122°F) (McRae and Foster 1959).

For the example shown in Figure 24, the design ESALs were estimated to be 30×10^6, the TCF = 0.11, the SF = 0.04, and N_{demand} = 660,000 repetitions.

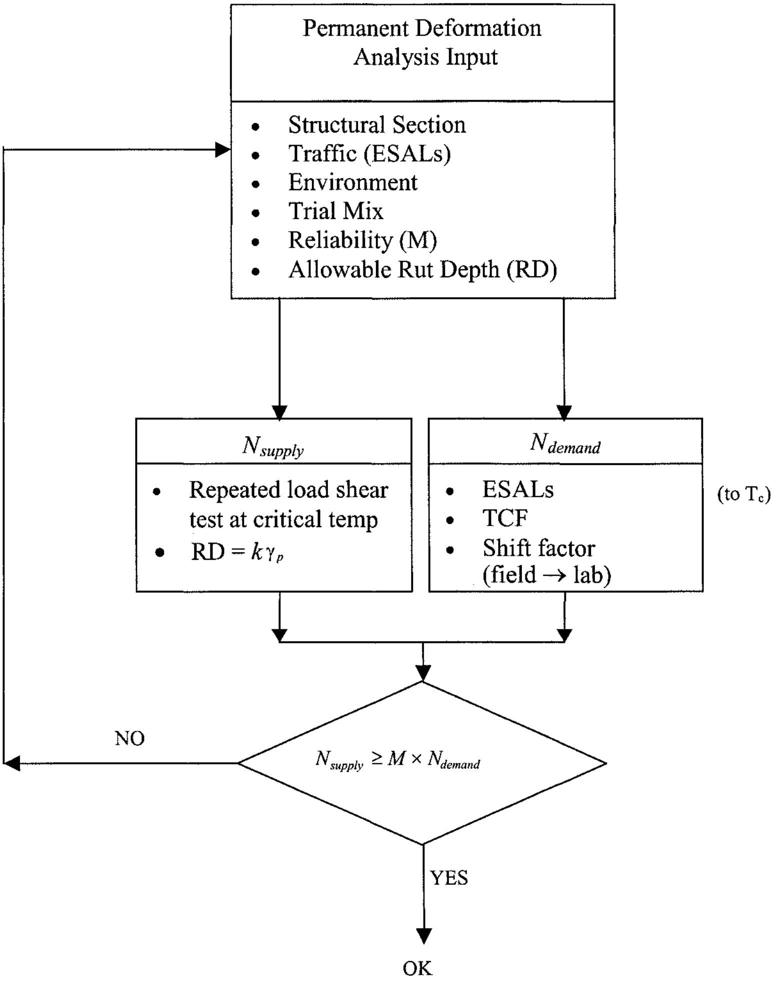


Figure 25. Permanent deformation mix design/analysis system.

The term M in Equation 5 represents a reliability multiplier which reflects the test variance and the estimated variance in the ln(ESALs) for a specified level of reliability. In the example, M was determined to be equal to 5.0. In Figure 24 it will be seen that the mix with the PBA-6a* asphalt will satisfy the design requirements at a binder content of 4.7 percent (by weight of aggregate).[1]

As a part of the CAL/APT program, rutting studies have been performed using the Heavy Vehicle Simulator (HVS) on a number of different mixes. Results of studies on a dense-graded asphalt concrete (DGAC) containing an AR-4000 asphalt cement and a gap-graded asphalt rubber hot mix (ARHM-GG) subjected to channelized trafficking in the HVS at a temperature of 50°C [at a 50-mm (2-in.) depth in the mixes] are shown in Figure 26 (Harvey and Popescu 2000). These two mixes were used as overlay in an asphalt concrete pavement which had been subjected to the HVS loading prior to placement of the overlays (Harvey et al. 1999). For these mixes, RSST-CH test results at 50°C were also available, with both mixes sustaining about 3×10^3 repetitions to 5 percent permanent strain in the shear test.

To evaluate the results of the mix design suggested by the data presented in Figure 22, a section was constructed with the PBA-6a* binder using the same aggregate as used in the RSST-CH tests. Results of the HVS test on that mix under the same loading and temperature conditions are also shown in Figure 26. As seen in Figure 24, the PBA-6a* mix sustained about 1×10^6 repetitions at a shear strain of 5 percent. From the results presented in Figure 26 the HVS rutting data reflect the results of the RSST-CH tests and provide support for the recommended design.

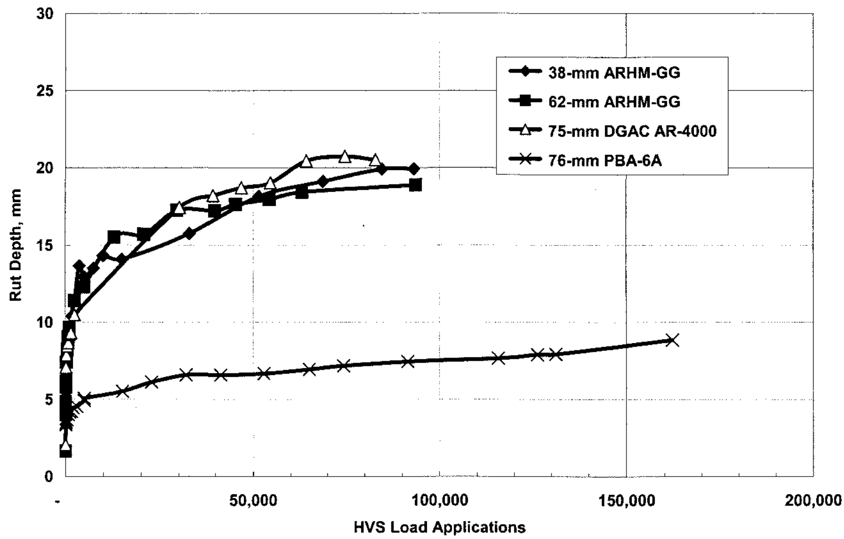


Figure 26. HVS rutting study results (40 kN load, 50°C at 50 mm depth).

[1] It should be noted that the response of the PBA-6a* mix at 4.2 percent binder content is less than the 660,000 repetitions. However, the air-void content of this mix was at about 6 percent. Had it been in the range 3 to 3.5 percent, it would have been exhibited at least the same behavior as the mix at 4.7 percent binder content.

Flexural Fatigue Test. When the design binder content has been selected, the performance of the mix in a selected structural section is evaluated to insure that the anticipated traffic for the design period can be carried so that the level of fatigue cracking will not exceed some prescribed level such as 10 percent in the wheel paths (Deacon et al. 1994). The approach to considering fatigue is similar to that for rutting and is shown schematically in Figure 27. SHRP-developed flexural fatigue equipment is shown in Figure 28.

For fatigue, N_{demand} is determined from the following expression:

$$N_{demand} = Design\ ESALS \cdot TCF \cdot \left(\frac{1}{SF} \right) \qquad \textbf{(7)}$$

All of the terms in this equation have the same meaning as those in Equation (6). Reference (Harvey et al. 1997) contains a discussion of the values for the TCF and SF.

N_{supply} is determined from a relationship between applied strain and cycles to failure. This relationship generally has the form:

$$N = a\left(\frac{1}{\varepsilon_t}\right)^b \left(\frac{1}{S_{mix}}\right)^c \qquad \textbf{(8)}$$

and may include parameters reflecting mix properties such as bitumen content and air-void content, e.g., References (*17, 71, 72*).

Analysis of the selected structural section in which the mix is to be used provides the value of strain to use in Equation (8) which provides N_{supply}. Combining N_{demand} with N_{supply} using Equation (7) allows the designer to determine whether the mix will provide an adequate service life. If the particular mix does not satisfy Equation (7), a number of choices are available. These include, but are not limited to, the following: 1) increase the thickness of the asphalt concrete layer to reduce the strain to a level which will provide a value of N_{supply} to satisfy Equation (7); or 2) change the mix to provide improved fatigue response.

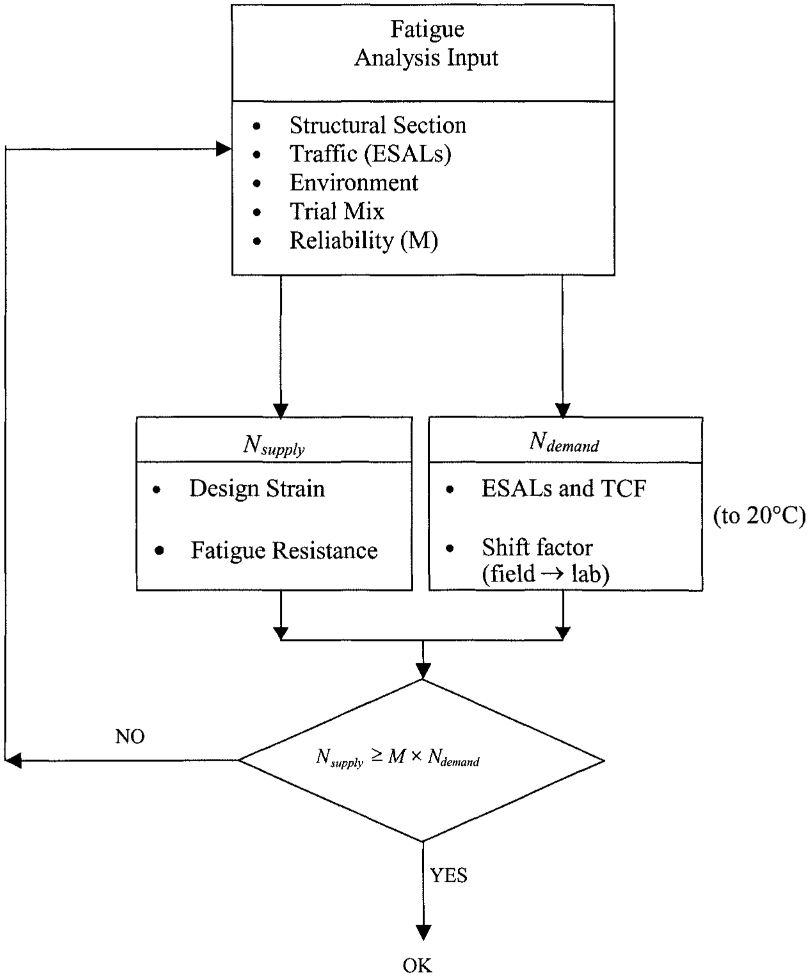


Figure 27. Fatigue mix analysis/design system.

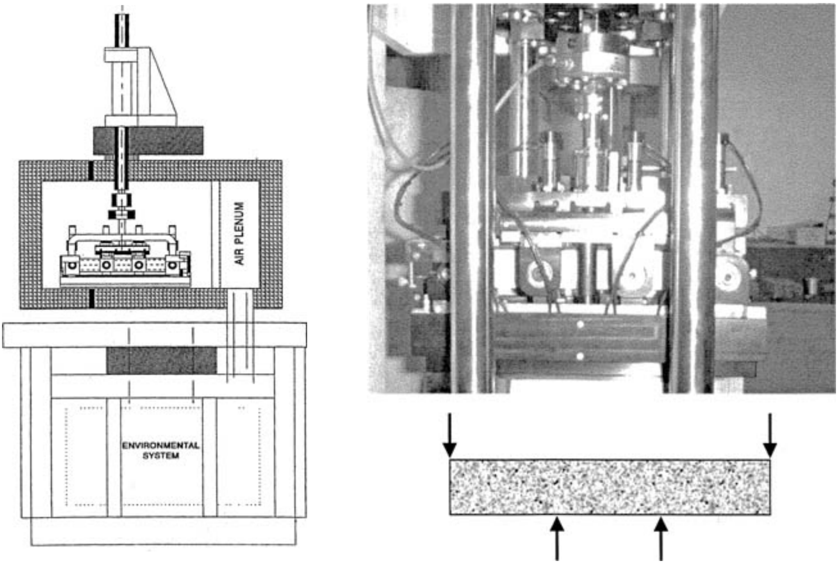


Figure 28. Flexural fatigue test equipment.

Thermal Stress Restrained Specimen Test, TSRST. To assess the propensity of a mix to low temperature cracking the thermal stress restrained specimen test (TSRST) was developed as part of the SHRP (Jung and Vinson 1994A). The mechanism of low temperature cracking is associated with a reduction in temperature of the asphalt concrete pavement in the low temperature regime, usually less than about 10°F. As the temperature of the asphalt concrete pavement is reduced below this level, tensile stresses develop in the mix being largest at the surface. With continued decrease in temperature, this tensile stress eventually exceeds the fracture strength of the mix and cracking takes place. This phenomenon is illustrated schematically in Figure 29.

The TSRST, shown in Figure 30, attempts to simulate this process using an HMA specimen maintained at constant length as its temperature is reduced at a constant rate. Eventually the specimen breaks and the fracture temperature is measured as seen in Figure 29. The measured information is shown as the bold portion of the thermal stress versus temperature relationship.

Results for tests on laboratory compacted specimens representative of mixes used in the WesTrack pavement test (Rao Tangella et al. 1990) are shown in Figure 31. The binder used in this pavement was a PG-64-22.

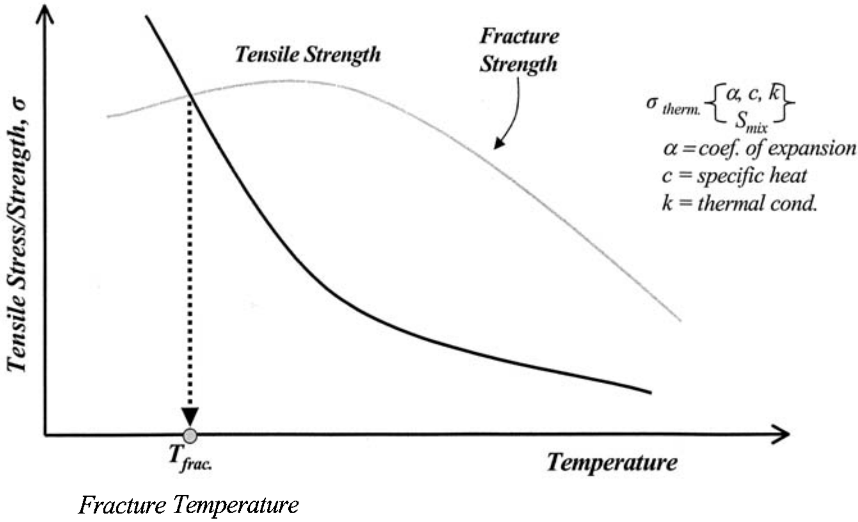


Figure 29. Schematic of low temperature cracking – TSRST.

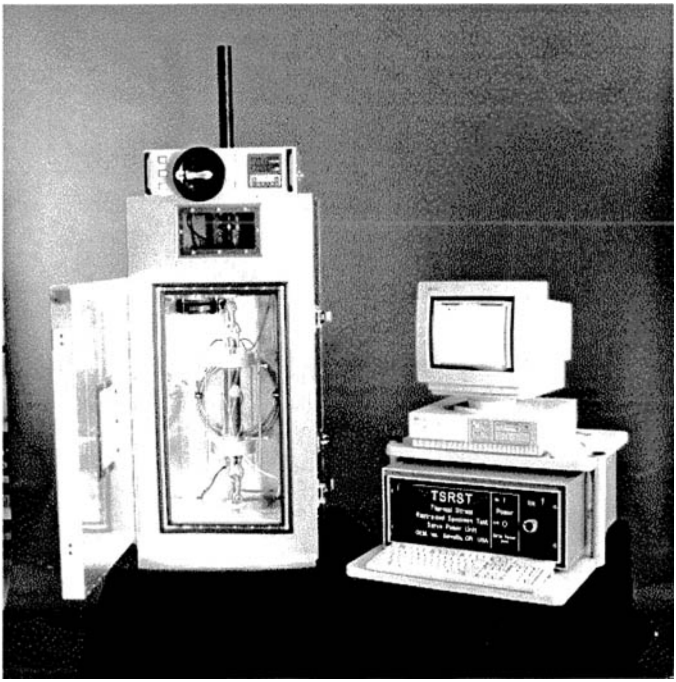


Figure 30. TSRST.

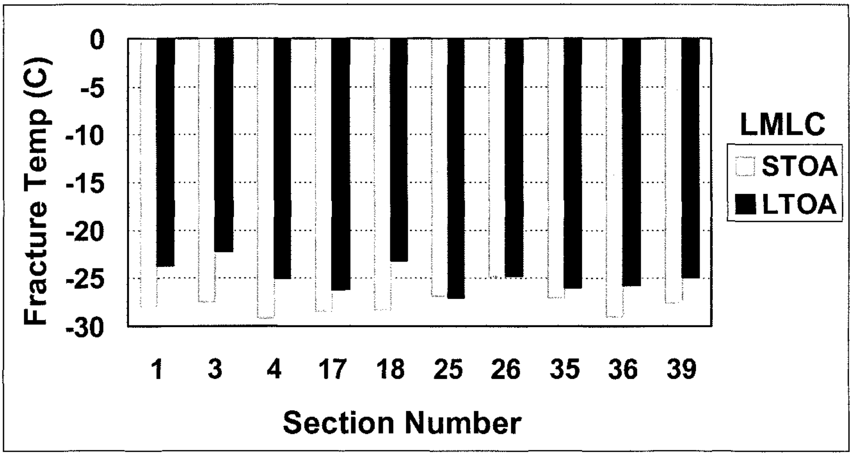


Figure 31. Schematic of low temperature cracking – TSRST.

It will be noted that fracture temperatures of the various mixes after long time aging (LTOA) are generally less than -22°C indicating the AASHTO MP-1 specification for low temperature cracking is appropriate for HMA containing conventional binders, a fact established during the validation study for the test (Jung and Vinson 1994B).

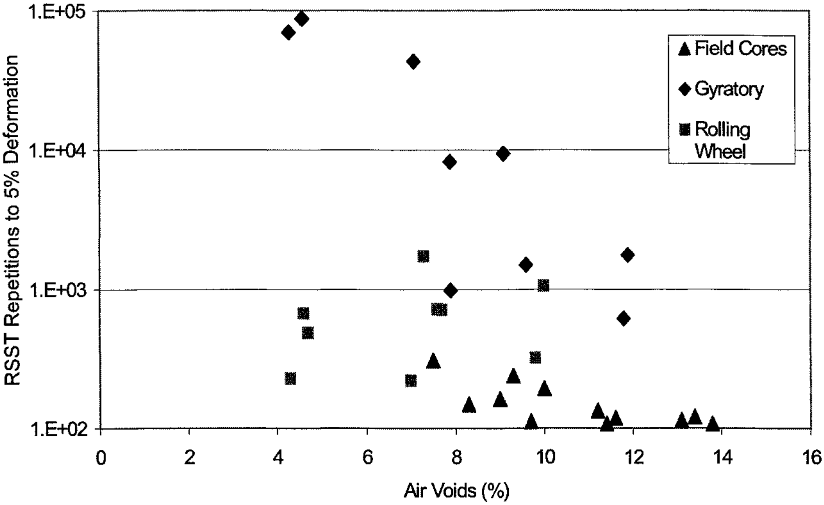
TEST SPECIMEN PREPARATION

Over the years, a number of different procedures have been developed to compact specimens for laboratory testing. These have included: static compaction (Vallerga 1951), impact compaction (Highway Research Board 1949), kneading compaction (Endersby and Vallerga 1952), gyratory compaction (Phillipi, McRae and Foster 1959, Cominsky et al. 1994), and rolling wheel compaction (van Grevenynghe 1986, Bonnot 1986, Sousa et al 1991). In recent years considerable data have developed emphasizing that when preparing specimens for permanent deformation evaluation in laboratory tests, it is important that the aggregate structure of the laboratory-compacted mix be about the same as that of the mix compacted in situ by conventional compaction. Hveem was one of the first asphalt researchers to recognize this; his efforts along with those of B. A. Vallerga led to the development of the Triaxial Institute kneading compactor (Vallerga 1951, Endersby and Vallerga 1955, Vallerga 1955).

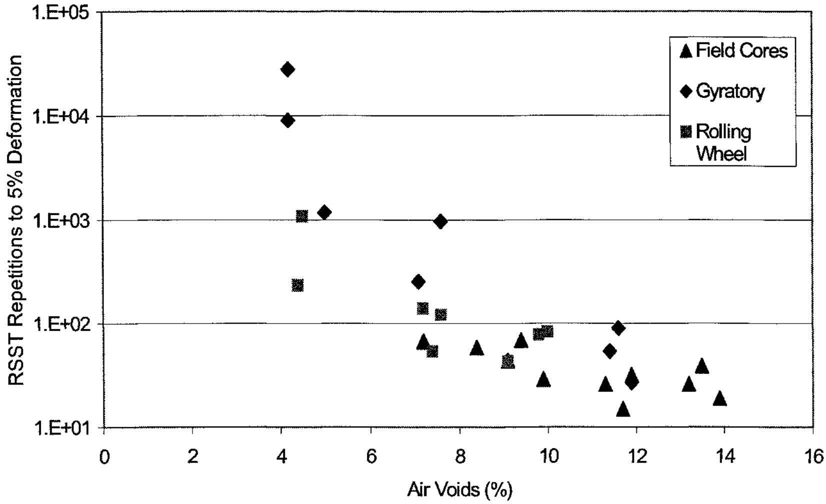
Laboratory Central Ponts et Chaussees (LCPC) conducted a study of specimens prepared by a number of different compaction procedures soon after the introduction of their gyratory compactor developed to evaluate the compactability of mixes. It was observed that the rolling wheel compactor developed by the LCPC produced specimens that best reflected performance on comparable specimens compacted in situ (van Grevenynghe 1986). The LCPC does not use gyratory-compacted specimens for permanent deformation evaluation; rather they use a form of rolling wheel compaction (Bonnot 1986).

During the Strategic Highway Research Program an extensive study was conducted of the influence of compaction method on the permanent deformation response of mixes. The compaction procedures included a mechanized version of the Texas Gyratory Compactor [150 mm (6 in.) diameter mold], the Triaxial Institute Kneading Compactor and a form of rolling wheel compaction (Harvey 1991). Results of the study supported the work of the LCPC suggesting that some form of rolling wheel compaction was most suitable for laboratory specimen preparation.

Recently, through the CAL/APT program, it was possible to compare the permanent deformation characteristics of field and laboratory compacted cores obtained from two overlay pavements, one containing a conventional dense-graded aggregate with an AR-4000 asphalt cement and the other a gap-graded material with an asphalt rubber cement. The pavements were constructed according to Caltrans procedures. The laboratory specimens were prepared with rolling wheel compaction (Harvey 1991) and the Superpave gyratory compactor (Cominsky et al. 1994). Simple shear tests (RSST-CH) were performed according to AASHTO TP7-94 at 40, 50 and 60°C (104, 122, and 140°F) (American Association of State Highway and Transportation Officials 1995). Results of these tests are summarized in Figure 32 for the asphalt rubber mix at 50 and 60°C (122 and 140°F). Similar tests were observed for the conventional dense-graded AC. The specimens prepared by the SHRP gyratory compactor exhibit greater resistance to permanent deformation than the field cores. This difference is due in a large part to the difference in aggregate structure created by the SHRP gyratory compactor as compared to rolling wheel compaction (Harvey et al. 1995). Also, the data suggest that the specimens prepared by rolling wheel compaction are similar in response to the field cores.



(a) 50°C



(b) 60°C

Figure 32. Influence of compaction method on behavior of mixes in the RSST-CH at 50°C and 60°C; gap-graded asphalt-rubber hot mix.

LABORATORY TEST SPECIMEN SIZE AND LOADING CONSIDERATIONS

Laboratory specimen size is an important factor to be considered in material testing. Laboratory testes are usually developed around theories (e.g. continuum mechanics and constitutive relationships), and are designed to identify specific parameters associated with the models. An important question is whether the theory stipulated is applicable to a specific test. In particular, many models in common use in the field of mechanics of materials are based on homogenization of properties across heterogeneous media. Thus, it is important to have enough material for the homogenizing process to provide "reasonable" properties.

For AC mixes, the question of scale between the particle size and the dimensions of the specimen is important because the maximum aggregate size may not be much smaller than the specimen size. Thus it is important to verify at what minimum specimen dimensions continuum mechanics, or any other theory based on homogenization, becomes applicable. This specimen dimension is referred to as the representative volume element (RVE), defined as *the smallest volume large enough so that the global characteristics of the material remain constant, regardless of the location of the RVE.*

When specimens smaller than the RVE are tested, much variability is observed. Consequently, the mean value of the results must be obtained from a large number of test specimens to arrive at a statistically meaningful value (Hashin 1983). On the other hand, it is likely that less variable test results will be obtained when specimens larger than the RVE are used.

Using specimens smaller than the RVE has two major disadvantages. First, a large number of specimens may be required. Second, an averaging process ignores any bias in the test procedure, which may result in large errors. Bias might occur, for example, because of the mix compaction method and selection of test specimens from a specific part of the compacted mix. In view of these limitations, the use of specimens larger than the RVE is recommended. However, in some cases, the use of specimens smaller than the RVE may be unavoidable; for example, in a mix containing larger aggregates. In such cases, statistically meaningful results can be obtained by testing a large number of replicates, although the above limitations should be noted. Results of typical laboratory tests of AC mixes show the classic indications noted in the literature for specimens smaller than the RVE (e.g., Weissman et al. 1999, Harvey et al. 1999).

Current laboratory procedures typically use only two to four replicates. Thus, if specimens are smaller than the RVE, there is no guarantee that the average result obtained predicts a statistically meaningful value of the material property. References (Weissman et al. 1999) and (Harvey et al. 1999) provide data to support the above discussion.

Dimensions of the RVE are dependent on aggregate size, shape, and orientation. Accordingly, the RVE for mixes containing different aggregates with the same nominal-sized aggregate may differ. Finally, because of aggregate shape and construction procedure, the dimensions of the RVE may differ in the three characteristic dimensions, particularly at higher temperatures (Weissman et al. 1999). The RVE dimension also depends on temperature and rate of loading. This is due to the rate of loading and temperature dependence of the material properties of the mastic (asphalt and fine aggregate), whereas the aggregate properties are relatively insensitive to these effects. As a result, at low temperatures the properties of the two components are closer, whereas at elevated temperatures the aggregate may be orders of magnitude stiffer than the mastic.

Thus, larger size specimens are required at high temperatures than for tests on the same mix at lower temperatures. Additionally, dynamic tests may require smaller specimens than static (creep) tests because the properties of the aggregate and mastic are closer at higher frequencies of loading.

In addition to RVE considerations, loading conditions will also influence specimen size. Consider for example, the simple shear test discussed above. In this test, a major imperfection results from mixing tractions on the leading and trailing edges of the specimen, as indicated in Figure 33. This introduces boundary layers near these edges that may affect the solution. Fortunately, the width of this boundary layer is independent of the specimen length and instead depends on the specimen height. Therefore, the relative contribution of these boundary layers can be diminished if the length-to-height ratio of the specimen is increased.

To demonstrate the effect of the length-to-height ratio, a series of three-dimensional finite element simulations were conducted. In this simulations, a 50 mm (2.0 in.) high and 100 mm (4.0 in.) wide specimen, with a length that varied between 25 and 500 mm (1.0 and 20.0 in.), was used. Results are shown in Figure 34; these are based on an elastic model discussed in Reference (Weissman et al. 1999).

This figure highlights two important findings. First, 10 percent error, or less, in the predicted shear modulus (G) can be expected for specimens with a length-to-height ratio greater than 3. Second, $G_{measured}$ converges to G monotonically from below. This provides a conservative value for G. In general these results indicate that the level of error can be reduced by increasing the length-to-height ratio. Thus, assuming the specimen height is prescribed by the RVE requirements, it is possible to select a specimen length that would result in an error level smaller than a specified value. Plane-strain finite element simulations of the simple shear test reported in Weissman et al. (1999) support the results of the analyses shown in Figure 34. The results of these analyses also suggest that the reliability of the test could be improved by using rectangular parallel sided rather than cylindrical specimens. For a length (or diameter) to height ratio of 3, the critical dimension for RVE is the height. Accordingly, the height of the shear specimens should probably be about 75 to 100 mm for a 19-mm nominal size aggregate. With most compaction methods, the nominal size of the aggregate will be oriented in the vertical direction (i.e., the longer side will be oriented parallel to the surface) and the aggregate length to height ratio is not important.

A similar analysis (Weissman et al. 1999) suggests that the height of a specimen subjected to axial loading should be at least twice the diameter plus the RVE in the vertical direction. The RVE within the specimen length should be maintained free of end effects, which necessitates the addition of the two-diameter requirement and an RVE length in the vertical direction. As a result, specimen dimensions for a 19-mm nominal aggregate, the type evaluated in this study, may have to be about 125 mm (5.0 in.) in diameter and as much as 350 mm (14.0 in.) tall.

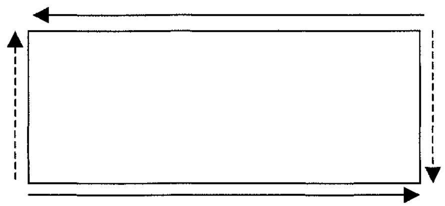


Figure 33. Simple shear test; traction represented by dotted arrows is not applied.

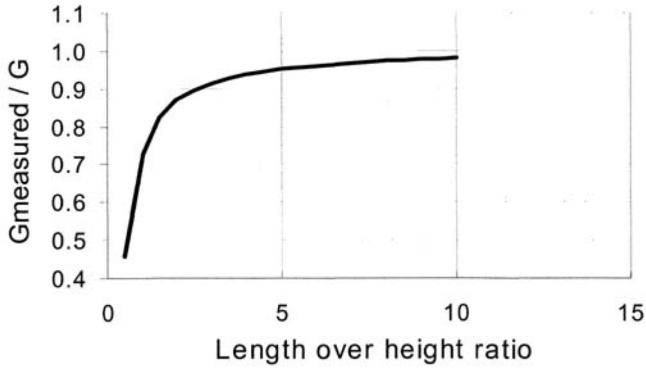


Figure 34. Convergence of the measured *G* with increased ratio of length to height.

Such a large specimen poses severe problems for testing field cores because the compacted AC layer of interest is usually less than 200 mm thick. It must be emphasized that these dimensions are subject to the limitations of the analysis and should be validated by laboratory testing. Some validation testing has been recently performed in the axial loading mode (Harvey et al. 1999), although only to a temperature of 40°C, which is below typical temperatures at which rutting occurs.

These results suggest that smaller specimens may be used for the standard repeated simple shear test at constant height than are required for an equivalent creep test in axial loading. Regardless of specimen type, however, the results emphasize the importance of considering the size of the specimen relative to the maximum size of aggregate used, the loading configuration axial or shear (for permanent deformation evaluation), load rate, and test temperature.

TEMPERATURE AND TRAFFIC CONSIDERATIONS

Selection of loading and temperature conditions for mix evaluation should reflect the environment in which the mix will be used. While a number of approaches have been developed (Asphalt Institute 1982, Cominsky et al. 1994, Deacon et al.

1994) only one will be discussed herein; i.e. the use of a *temperature conversion factor* (TCF). Reference (Deacon et al. 1994) describes this approach in detail and its use has already been illustrated in an earlier section.

The TCF is a multiplicative factor which converts the number of design load repetitions occurring in the mixed temperature environment in situ to its equivalent at a single temperature. The ability to make such a conversion is critically important for routine work, because it reduces the requisite laboratory testing and structural analysis to a single temperature. The TCF has been shown to be dependent on both the pavement structure and the thermal environment. At the same time, it has been hypothesized that the TCF may be relatively unaffected by mixture characteristics, particularly for asphalts of normal temperature sensitivity. It remains for future study to determine the limits within which this hypothesis is valid.

Reference (Deacon et al. 1994) provides values for the TCF for two hypothetical pavement structures located within nine geographic regions of the United States.

To illustrate the results for a specific area, in this case California, three locations representative of a variety of climatic conditions were examined including: Blue Canyon in Placer County (mountain environment); Daggett Airport in San Bernardino County (desert environment); and Santa Barbara airport in Santa Barbara county (coastal environment). At each location, two hypothetical pavements were examined having 102 mm (4.0 in.) and 203 mm (8.0 in.) surface courses (Harvey et al. 1997).

The asphalt mix was thought to be representative of those used in California, and laboratory stiffnesses and fatigue lives were measured and characterized in an earlier investigation (Harvey et al. 1996). Pavement temperature profiles were simulated using the climatic-materials-structural (CMS) pavement analysis model developed at the University of Illinois (Herlache et al. 1996). Results of these computations are shown in Figure 35 and represent factors necessary to convert traffic to a temperature of 20°C, a temperature convenient for fatigue testing of asphalt mixes.

A similar approach can be used for permanent deformation testing, in this instance defining a test temperature in the range 40° to 60°C where rutting is likely to occur; examples are included in References (Monismith et al. 2000, Harvey et al. 1995, Monismith et al 2001).

At this stage in time, to enable direct comparisons between field and laboratory traffic estimates, a *shift factor* must be applied to the traffic forecast. This shift factor is intended to account for traffic wander, construction variability, differences between laboratory and actual (field) states of stress, frequency of loading, as well as other factors.

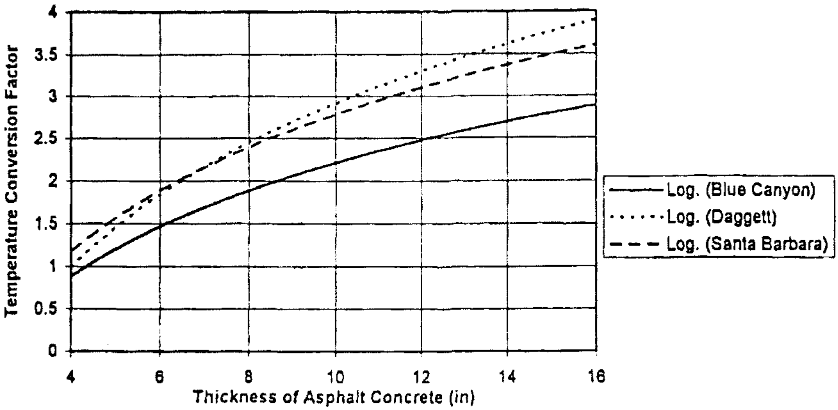


Figure 35. Effect of location and surface thickness on temperature conversion factor for fatigue life estimation.

In the case of fatigue, for example, crack propagation time as well as many of the above listed factors, highway pavements have been found to sustain from less than 10 to as much as 100 times the number of load applications based on measured laboratory mix response for specific values of calculated strains.

One investigation (Harvey et al. 1996) has suggested that the shift factor for fatigue follows an equation of the form:

$$SF = a \in^{-b} \tag{9}$$

where:

$\in$ = simulated strain on the underside of the asphalt concrete layer for standard wheel load

a, b = coefficients determined by comparisons of calculated and measured degrees of cracking

This relationship appears reasonable since the rate of crack propagation, a principal difference between in-situ and strains laboratory behavior, is affected by strain level. Small strains, for example, not only increase the number of load repetition to crack initiation but also slow the rate of crack propagation as well. The opposite is expected for large strains.

For permanent deformation an example of the development of a shift factor to relate shear strains in the laboratory to estimate for field loading conditions is described in Reference (Sousa et al. 1994).

RELIABILITY AND VARIABLITY

Decisions about anticipated mix performance cannot be made with absolute certainty. Although large safety factors can reduce the likelihood of error, their cost consequences can be considerable. Reliability analysis ensures an acceptable level of risk in mix design without the cost of excessively large safety factors.

Reliability is considered, herein, to be the probability that the mix will provide satisfactory performance throughout the design period. The reliability level for each specific mix design is set by the designer. Larger levels of reliability reduce the chances of accepting deficient mixes; however, the tradeoff is the potentially larger cost associated with reducing the number of acceptable materials or mixes or increasing the thickness of the asphalt concrete.

Reliability can be introduced in the mix design and analysis system by a reliability multiplier, *M*, which is calculated as follows:

$$M = e^{Z\sqrt{\mathrm{var}(\ln N) + \mathrm{var}(\ln ESALs)}} \qquad \textbf{(10)}$$

in which e = the base of natural or Naperian logarithms, Z = a factor depending solely on the mix design reliability, var (ln *N*) = the variance of the logarithm of the laboratory mix performance, e.g. fatigue life estimated at the in-situ strain level under the standard 40 kN (9,000-pound) wheel load, and var (ln *ESALs*) = the variance of the estimate of the logarithm of the design ESALs. Z is related to design reliability as follows:

Design Reliability (percent)	Z
95	1.64
90	1.28
80	0.84
60	0.253
50	0.000

The variability associated with forecasts of design ESALs is not well defined. However, as a point of reference, the AASHTO design guide (American Association of State Highway and Transportation Officials 1993) suggests that actual traffic may be 1.6 times that predicted at a one-standard-deviation level. Assuming a log normal distribution, this corresponds to a var (ln *ESALs*) of about 0.22.

The parameter var (ln *N*) can be considered to be the sum of three components as follows:

$$\mathrm{var}(\ln N) = s_t^2 + s_m^2 + S_s^2 \qquad \textbf{(11)}$$

in which s_t^2 = the testing variance, s_m^2 =the mix variance, and s_s^2 = the structure variance.

References (Deacon et al. 1994, Sousa et al. 1994) contain values of *M* as a function of var (ln *N*) and var (ln *ESALs*).

To illustrate the testing variability, for fatigue testing this variability reflects a combination of factors including the inherent variability in fatigue measurements (associated both with specimen preparation as well as testing equipment and procedures), the nature of the laboratory testing program, and the extent of extrapolation necessary for estimating fatigue (using a least-squares, best-fit line) at the design strain level. Var (ln *N*) is calculated as follows (Harvey et al. 1997):

$$\operatorname{var}(\ln N) = s^2\left(1+\frac{1}{n}+\frac{(X-\bar{x})^2}{q\sum(x_p-\bar{x})^2}\right) \quad (12)$$

in which s^2 = the variance in logarithm of fatigue-life measurements, n = number of test specimens, X = ln (in-situ strain) at which ln (N) must be predicted, $\bar{x}$ = average ln (test strain), q = number of replicate specimens at each test strain level, and x_p = ln (strain) at the pth test strain level.

Similar information can be develop for other mix tests [e.g. Hveem Stabilometer test data (Benson 1996), RSST-CH test data (Sousa et al. 1994)], so that the reliability of the mix design can be selected consistent with pre-selected levels risk dependent on the pavement under consideration.

SUMMARY

This paper has attempted to cover some of the developments in asphalt mix design and analysis during the past 60 years. The initial part has concentrated on two methodologies; (1) the so-called *Marshall* procedure developed by the U. S. Army Corps of Engineers (USACE/WES) during the early 1940's for airfield HMA design and modified subsequently for mixes to be used in highway pavements; and (2) the *Hveem* procedure developed in California in the 1930's and subsequently used in some form by a number of states in the U. S. As noted in the paper, both methods are still in use, with the USACE-developed procedure utilized, for example, by the USACE for military pavement applications and by the FAA for commercial and general aviation airfields while the Hveem-developed procedure is used by some state highway authorities, particularly in the Western U. S.

While both methodologies have been used successfully, changes in loading conditions particularly, have necessitated modifications of the procedures. Also, as noted above, the procedures may not successfully accommodate new materials or further changes in loading conditions; hence the reason for SHRP.

With the advent of the Superpave volumetric design procedure, the Marshall method has been replaced by it in a large number of states in the U. S. It should be noted, however, that a mix performance test to supplement the volumetric procedure is still under development (2002). While Superpave is still limited to volumetric mix design, there have been performance-related tests developed over the years and during SHRP. A few of these have been discussed herein.

The Shell procedure, with a creep test to assess mix rutting potential as a part of mix selection, has been successfully used for mixes with conventional asphalt binders. For mixes containing non-conventional or modified binders, however, it has been demonstrated that some form of repeated load test is required.

During SHRP a number of performance-related tests were developed, including the RSST-CH, flexural fatigue, and the TSRST, to define permanent deformation, fatigue, and low temperature responses, respectively. These tests have been applied successfully to mix design and analysis.

In performing mix design and analysis, using the improved test methods developed in the past 20 years, examples of which have been described herein, does require careful consideration of a number of factors. These include: (1) specimen preparation-especially for permanent deformation measurements; (2) specimen size and shape including the effects of aggregate size; (3) test temperature and loading conditions; (4) the relationship between laboratory and actual traffic load applications; and (5) reliability that reflects variability in both traffic and testing estimates.

Asphalt/binder aggregate mixes used in paving as well as in other engineering applications are complex; simple test methods (including specimen preparation) may not be suitable to define mix characteristics necessary to reflect expected performance. Accordingly, we should not settle for something simplistic in the interest of expediency. It is important that technical merit rather than simplicity and ease of implementation drive the selection of test methods and analysis techniques. This should be tempered at times by the need to transfer technology to the new generation of engineers and from research into practice. Thus, a degree of simplification may be required to communicate the fundamental principals in order to provide a proper basis for exploiting the extensive research knowledge that has been developed in the past 20 years.

Today we have analysis tools that were not available in the 1930-1960 period. Moreover, we have improved understanding of materials behavior and the ability to measure requisite characteristics. In addition, the ability to validate aspects of pavement behavior through accelerated pavement testing provides the opportunity to more quickly introduce new materials and innovative design concepts.

With all of this said, it is important not to discard past knowledge accrued since at least the 1930's, but rather to build on it to improve our mix design and analysis capabilities. This will permit us to respond to changed loading conditions and to be able to take advantage of new materials as well as more effective use of existing materials which have been used successfully for many years.

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The Future of Asphalt Pavements

By

Thomas Harman[1], John D'Angelo[2], John Bukowski[3]

Abstract

This paper discusses the future of asphalt pavement design and construction. It poses the following question: how do we make long-life pavements? In addressing this question, the answer is framed around three areas of asphalt technology: 1-Structural design; 2- Materials; and 3- Construction. Additionally, the paper presents the current trends in transportation environment, and relationships with technological advances and their impact on the asphalt pavement industry.

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Introduction

What is the cost of poor performance? In April of 2001, The Road Information Program (TRIP) released a report quantifying the individual vehicle operating cost (VOC) attributable to poor roadway condition, Figure 1. In the United States, on average we spend individually an additional $222 each year to maintain and operate our vehicles due to poor roads. As a nation, this represents a significant annual expenditure of $41.5 billion.

TRIP reports a wide range of operating costs: ranging from a high of $432 to a low, in a State with an aggressive maintenance program, of only $23 per year.

Another major cost imparted on the driving community is linked to congestion and user delay. From the period of 1980 to 1998 in the United States, we have seen a 72% increase in traffic volume with only a 1% increase in capacity (road miles).

So what can we as asphalt technologist do to reduce VOC's and congestion? Simply stated, we need to build long-life pavements. In other words, "Get in, Stay in, Get out, and Stay out." Improving performance will reduce VOC's and reduce frequency and the number of work zones, which significantly contribute to congestion.

So, how do we make long-life pavements? In addressing this question, the answer is framed around three areas of asphalt technology: 1. Structural design; 2. Materials; and 3. Construction, Figure 2. However, before discussing these areas it is important to discuss the current transportation environment or relationships and the keys associated with technological advancement and success.

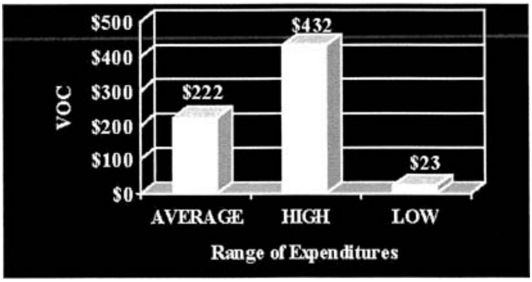

Figure 1. TRIP Findings on VOC.

"So, how do we make long-life pavements?"

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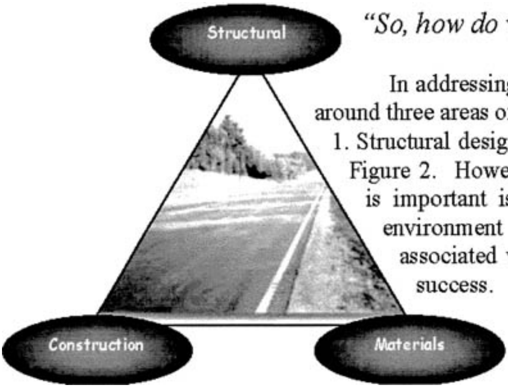


Figure 2. Three Areas of Asphalt Technology

Relationships

In the highway community, information flows principally in one direction, Figure 3. The structural designers request information from the traffic group and obtain information about the project's environment, in turn providing a cross-sectional design to the materials and construction groups. The Materials designers obtain information from the suppliers and provide the construction group with a mixture design. And then the construction group builds the pavement. A separate group typically maintains the sections and long-term performance is only looked on at a network level through pavement management systems.

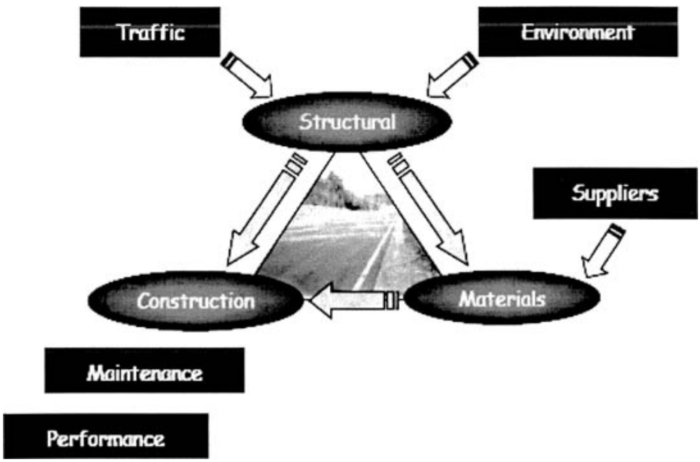


Figure 3. Typical Relationships

In this process, important "what if" questions are not addressed. "What if materials are weaker or stronger than the structural design assumptions?" "What if certain materials are easier to maintain?" "What if the supplier or contractor wants to try something new?"

Innovation and accountability are lost in the process. Ideally, what we need is two-way communication, feedback, and accountability, Figure 4.

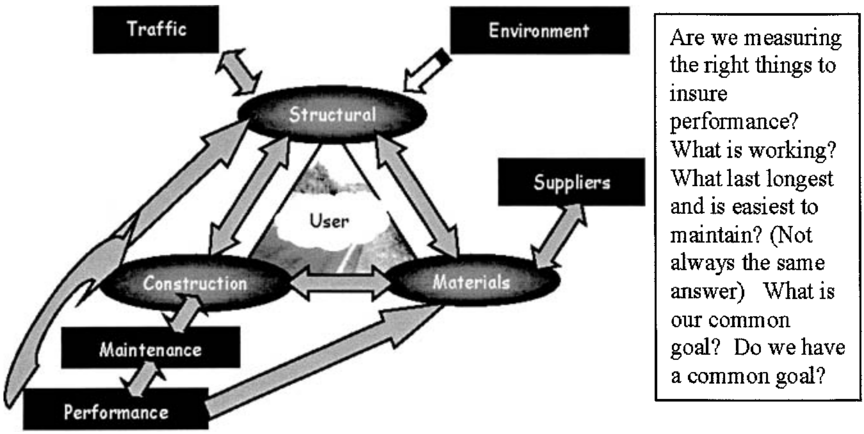


Figure 4. Idealize Communication Model

The Three Keys to Success

There are three keys to achieving our common goal of long-life pavement:

1. Communication,
2. Innovation, and
3. Fostering Expertise.

To reach this goal, a vibrant and dynamic asphalt research, development, and technology (RD&T) program is needed. One view of the RD&T process as a linear model, Figure 5, where researchers identify and work on a problem and then the developers are brought in. The developers engage the technology transfer (T2) specialists and then the users implement the innovation. The approach is very slow and all too often leaves the researchers looking for things that do NOT address the users' needs.

Effective RD&T is not that simple, Figure 6. It involves total involvement throughout the process, often called stakeholder involvement. Users and producers have to be involved at the identification and inception of research. Continuous involvement of equipment manufacturers and other developers as well as T2 specialists greatly increases the likelihood of success and reduces the time to implementation. This is not a new concept to the asphalt industry. During the Strategic Highway Research Program (SHRP), User-Producer groups were fostered to vet research ideas.

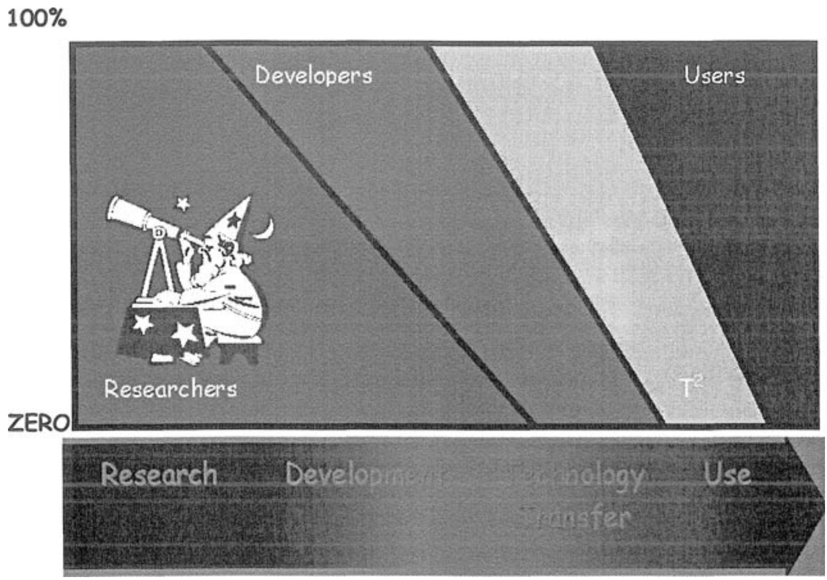


Figure 5. Linear Model of the RD&T Continuum

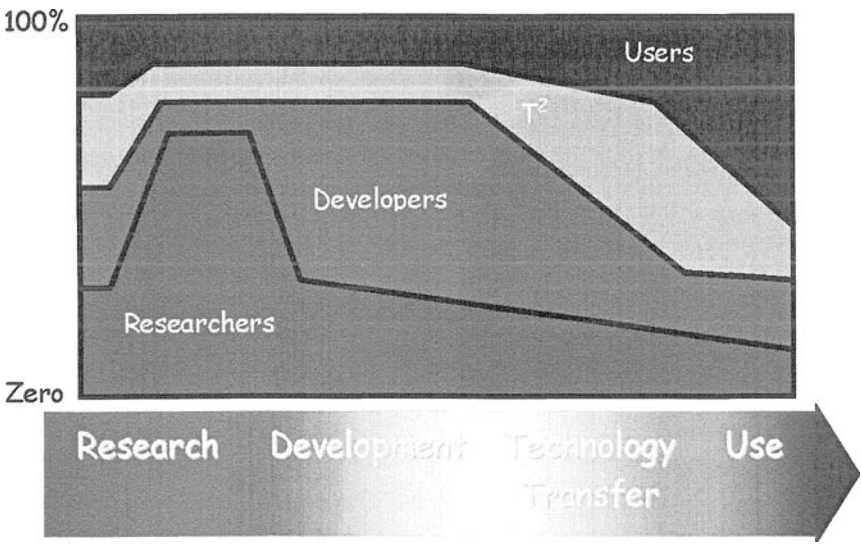


Figure 6. Model of Effective RD&T.

Superpave and Stone Matrix Asphalt (SMA) are excellent examples of how to do RD&T right!

There are three basic rules that govern the ease of the RD&T technology transfer process. For timely implementation and lasting impact, products should fulfill three simple little rules:

1. Easy to set up,
2. Easy to run, and
3. Easy to analyze.

This does not mean products cannot be technologically complex. But laborious procedures and overly complex equipment with elaborate criteria do not lend themselves to daily use in the highway community. As we look to the future, what research and products hold the greatest promise in helping us achieve our goal of long-life pavements?

LOOKING TO THE FUTURE: STRUCTURE / MIXTURE /CONSTRUCTION

Structural Design (The right cross-section)

Several major efforts are coming to fruition in the area of structural design. Most significant are two projects being managed by the National Cooperative Highway Research Program (NCHRP). Project 1-37(A) charged with developing a new mechanistic-empirical pavement design guide and Project 9-19 developing a framework for advanced performance modeling. Both of these projects will be providing the paving community with new tools (models) in the area of structural analysis and design.

MODELS? Simply stated, a model is a mathematical equation that predicts something you want to know based on something else you can easily measure. For example, we can use parameters like traffic volume, asphalt stiffness, and aggregate gradation to predict mixture rutting resistance.

These improved models offer more flexibility in how we analyze our pavement systems. In the continuum of pavement response modeling, Figure 7, we are moving closer to the state-of-the-art. One of the main catalysts behind the shift in the continuum is the recent dramatic increase in computer computational power.

A critical element, being proposed, is a common material characterization test for asphalt mixtures for both structural analysis and mixture design - the dynamic modulus, /E*/. This provides the "missing link" between these two areas. /E*/ may also lend itself to field quality control; fully closing this aspect of the technology loop.

In the future, we also need to better understand the loading being applied to our pavements. How we measure and monitor Traffic is a critical element in the modeling equation. In addition, our models need to focus more on the users' perspective. The proposed pavement design guide relates common distress modes (rutting and cracking) to roughness (international roughness index, IRI). Roughness or ride is definitely focused on the user, however, can we develop models or develop designs accounting more for: safety, smoothness, noise, or even appearance?

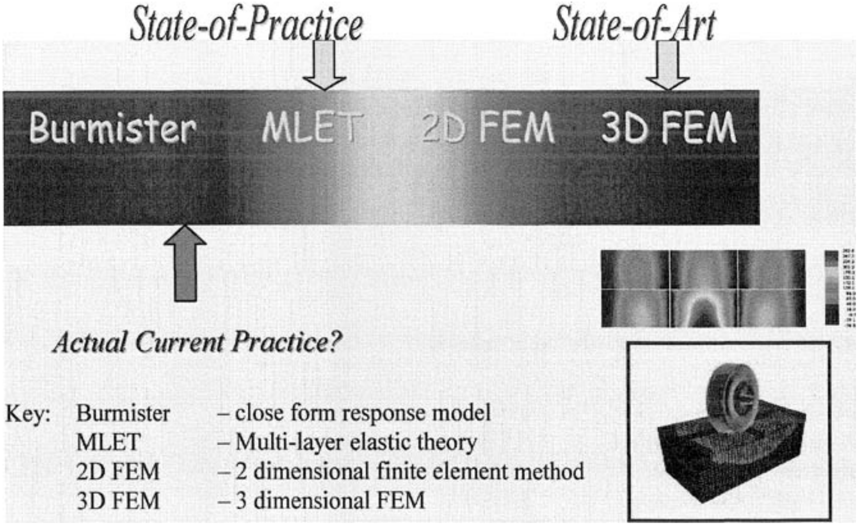


Figure 7. The Continuum of Pavement Response Models.

Materials (Optimum Performance)

How should we design our materials? To maximize performance or meet a required design life? How can faster, better, more economical equipment and procedures for design be developed? And how do we tie material properties more closely to Structural Design and Construction? The answer to all these questions has its roots in Superpave®.

It is impossible to talk about asphalt mixture design with out saying the word Superpave. Developed under the SHRP program, Superpave provides a performance-related binder purchase specification coupled with a volumetrically based mixture design system that employs gyratory compaction. In the very near future, Superpave will also include a simple performance test (SPT). The simple performance test will use specimens compacted in the gyratory and loaded uniaxially, Figure 8.

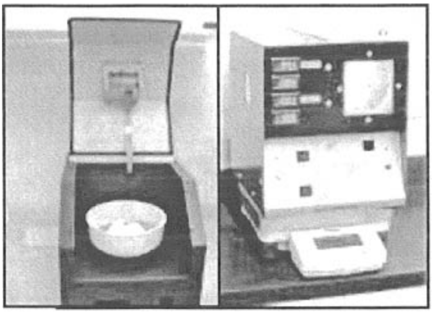
Figure 8. Uniaxially Loaded Specimen

In Superpave the binder specification, grade selection is a function of environment and traffic level. Binders are tested at different temperatures, loading conditions, and levels of aging to simulate the changes that occur throughout a pavement's service-life and assess resistance to distress, such as rutting, fatigue cracking, and low-temperature cracking.

Under the guidance and direction of the Transportation Research Board's (TRB) Superpave Binder expert task group (ETG), the Superpave binder specification is continuing to evolve. On the horizon are significant changes in the way we condition and test our materials. These changes are primarily driven by the current specification inability to fully capture the benefits of modified materials.

In the area of conditioning, considerations are being given to replace the rolling thin film oven test (RTFOT) with either the German rolling flask or a new distillation system developed at Texas A&M University. Several proposed changes to the test procedures are being considered, including using the dynamic shear rheometer (DSR) to assess rutting by measuring a new parameter called "zero shear viscosity (ZSV)." In the end, Superpave binders will still be performance graded based on two temperatures, ex. PG 64-22. However, the properties we measure to determine the 64 and -22 will be different and will be truly performance-related and blind to modification.

Over in the mix lab changes in both aggregate characterization and laboratory compaction are in sight. New procedures are being developed to more effectively measure the specific gravity (density) of the constituent materials, Figures 9 and 10. Also, imaging technology to enhance characterization of aggregates and better link their physical properties to performance is being evaluated, Figure 11.



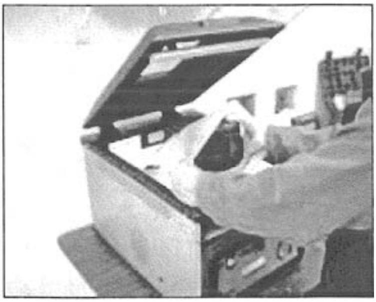


Figure 9. Automated Specific Gravity Devices Figure 10. Corelok^{TM} Device

Improvements are also being sought to increase the precision and repeatability of existing practices. Issues of gyratory compaction calibration have lead to the development of the dynamic angle validation (DAV) kit, Figure 12. However, DAV only addresses one issue on the compaction horizon. The proposed simple performance test (SPT) requires taller specimens than ever envisioned by the SHRP researchers and, in addition, the NCHRP 9-16 has further developed the concept of measuring shear resistance during compaction. The demands of the SPT and the findings of NCHRP have prompted the TRB Mixture/Aggregate ETG to form a new task group to explore the development of the next generation of laboratory compaction.

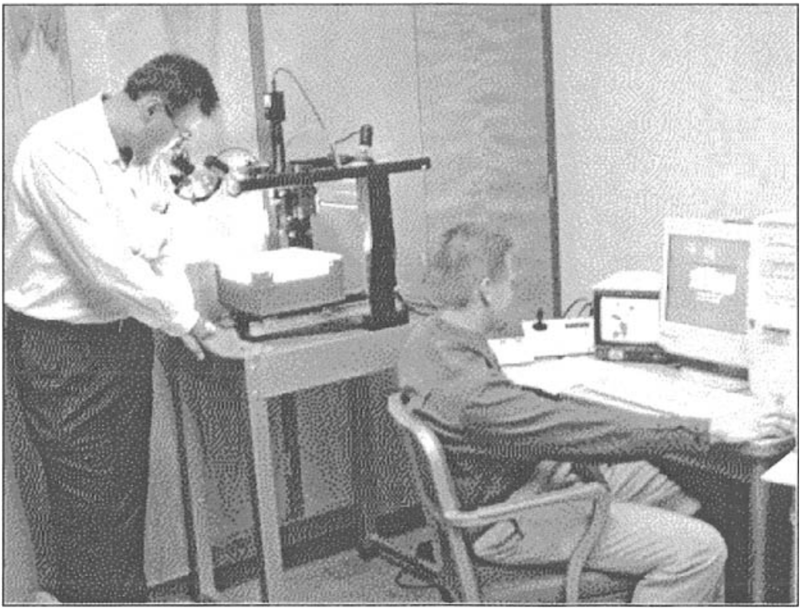


Figure 11. Researchers at the Washington State University use an Automated Image System (AIMS) to quantify aggregate shapes and texture properties.

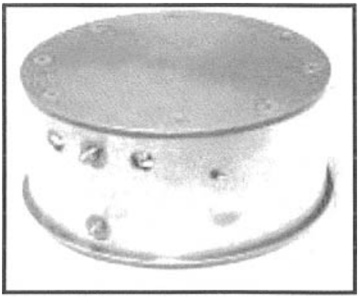


Figure 12. FHWA DAV

Construction (Get in. Stay in. Get out. Stay out.)

What is the best way to effectively place the pavement? How do we best monitor performance? What is the most effective way to conduct field management: quality control and quality acceptance (QC/QA)? What innovative contracting tools do we need? What innovations are occurring in testing and placement equipment? And how do we best insure work zone safety?

Monitoring

We need to effectively monitor performance to allow us to further develop models, including calibration and validation. This will allow for better optimization of materials and more cost effective layer thickness designs. The effective use of accelerated performance data to refine specifications in a compressed timeframe and validate innovative concepts is needed, Figure 13.

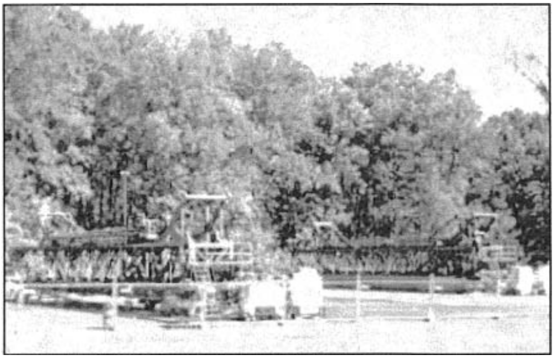


Figure 13. FHWA Test Facility, featuring 2 Accelerated Lad Frames (ALF's)

Contracting

Statistically based quality-control and quality-assurance (QC/QA), performance-related specifications (PRS), and warranties offer a mix of contractual tools to ensure performance. PRS is helping us to identify what is most important during the construction process; by linking testing to actual performance. The performance indicators, identified by PRS, are changing what we control in standard QC/QA contracting and are identifying what is important to contractors warranting their work. Not surprisingly, proper compaction is one of the most critical parameters in achieving performance.

Equipment

Much of the new testing equipment being developed in the mixture laboratory lends itself to use in the field. In addition we are seeking new ways to measure density and uniformity of the pavement mat; using means of thermal imaging and non-nuclear density gauges. Specifying agencies are also placing more of the responsibility of testing on the contractor. This has raised some questions of accountability. Smart testing equipment that automatically tracks location and number of testing is being investigated. What if we tie a global positioning system (GPS) to a nuclear gauge and track all readings?

Innovations in production equipment and placement equipment continue to advance. Technology is allowing vibratory rollers to get smarter. This trend will continue in the future.

Equipment manufactures are rising to the challenge of increase production rates and paving speeds, while achieving the smoothest pavements possible.

If we turn back the clock fifty years what do we see in the area of construction and data analysis at the AASHO Road Test, conducted in Ottawa, Illinois, what do we see? (Figures 14, 15, 16, 17, 18)

If we turn the clock forward 10, 20, 30 years, what advancements will we see? How will computers, GPS, telecommunications, the internet, shape our future? One thing is certain, we will be asked to do more with less and we will have to look to technology and innovation to find the solutions.

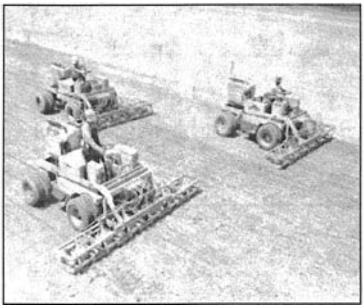


Figure 14. Subgrade Prep

Figure 15. Bituminous Laydown

AASHTO Road Test – "State-of-the-Art" Data Analysis

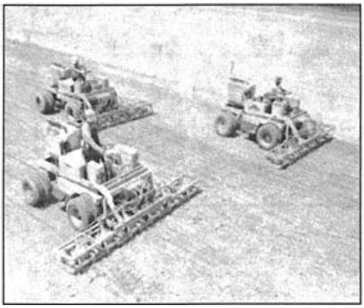


Figure 16. IBM tape-to-card printing punch

Figure 17. Chart Reader Used with Longitudinal Profilometer

Figure 18. Bendix G 15-D Computer

Fostering Expertise in the Perfect Storm

As we move to the future, the environment of the asphalt paving industry is changing. The realization of research, development, and technology transfer activities continues to provide a wide range of new tools. We have made significant advancements in the way we characterize and design our materials; analyze and design our pavement cross-sections; test and predict our pavement performance; and manage the construction process.

This new technology comes at a critical time for the agencies entrusted with the stewardship of our nation's infrastructure. Recent years have been accompanied with changes in contracting practices that have shifted the responsibility of understanding and technology application from the specifying agencies to the contractors. Along with this shift, we see a loss of expertise and institutional knowledge due to retirement and downsizing occurring in many of the specifying agencies, and the inability of many contractors to identify and retain personnel to meet the new demands being placed upon them.

This is the perfect storm: new technology, shifting responsibility, and diminishing expertise. How can we continue to be good stewards and still meet the needs of the motoring public?

Simply stated, business as usual cannot continue on; partnerships are changing. Needs are growing along with the demands of technology. The transportation industry must adapt, remove barriers, and focus our actions.

The Strategic Highway Research Program, along with developing products, nurtured and rekindled lines of communication between users and producers. The Federal Highway Administration and the Association of State Highway Transportation Officials through the Transportation Research Board continue to nurture and strengthen the communication infrastructure essential to the continued success of our nation's pavement infrastructure.

The Storm mounting around us is surmountable, but only with your help. Our relatively small community of transportation specialists and paving technologists must continue to rise to this occasion. Innovation, communication, and partnerships will shore the sheets of our sails and allow us to weather the storm.

Modified Asphalt Binders for Paving Applications

H. Bahia, Member ASCE

The University of Wisconsin Asphalt Research Group

Section One: Introduction

Modification of asphalts is prompted mainly by the limitation of the conventional refining practices used today in production of asphalts from crude petroleum. The chemical composition of asphalt and, in consequence, its properties are largely dependent on the crude source and the refining process. Asphalt production in most refineries is a secondary process that cannot compete with fuel and other products in revenue generation. Therefore production of better performing asphalts is not one of the common strategies in planning refining strategies. When the produced asphalt does not meet he climate, traffic and pavement structure requirements, modification has been used as one of the attractive alternatives to improve asphalt properties.

In effect, what is called conventional asphalts or straight run asphalts have a range of rheological and durability properties that are not sufficient for resistance on distresses caused by the increase in traffic and total loading on current highways. Modification by specialized refining practices, chemical reaction, and/or additives has been found to improve contribution of asphalt binders to resistance of asphalt mixtures to various modes or pavement distress. This improvement is recognized to result in life cycle cost savings and thus use of modified asphalts has been steadily increasing for the last 20 years or so.

In a recent survey of the State Highway Agencies in the United States 35 out of 47 agencies that responded indicated that they plan on increasing the use of modified binders in road construction, 12 were expecting to use the same amount of modified asphalt, and none indicated that they plan on reducing amount of modified asphalts. The majority of the agencies have cited premature distress such as rutting and fatigue cracking as the main reason for justifying the use of modified binders, which are on average increase the initial cost of construction (Bahia et al. 1997).

Asphalt modification using additives dates back to the last century (King et al. 1999). Patents for using polymers to modify asphalts date back to 1823 (Isacsson and Lu 1998). Test projects were placed in Europe in the 1930's and in North America in the 1950's. In the early and mid 1980's the introduction of newer polymers and the European technologies resulted in proliferation of asphalt modification in the United States. By 1982 more than 1000 technical articles had been published on polymer modified asphalts or mixtures (Zeneke, 1985) and there is a great emphasis continuing on this subject (Isacsson and lu 1998).

Visco-elasticity of Asphalts and Modification Strategies

At any combination of time and temperature, visco-elasticity of asphalt binders, within the linear range, is best characterized by two properties: The total resistance to deformation under load and the relative distribution of that deformation between elastic and viscous part. Although there are many methods of characterizing visco-elastic properties, cyclic (oscillatory) testing and creep testing are two of the best techniques to represent the uniqueness of the behavior of this class of materials (visco-elastic).

What is very unique about asphalt binders is their high sensitivity to pavement temperatures within the range of applications. The stiffness of asphalt can vary by as much as eight orders of magnitude, and their phase angle (relative distribution of response between elastic and viscous) by as much as 85 degrees, between peak summer and peak winter conditions. It cam also vary by similar amounts in response to standing traffic and high-speed traffic. Similar to visco-elastic properties, failure properties and damage accumulation properties of asphalt are also very sensitive to temperature and loading rate. Stress and strain at failure can change by an order of magnitude by a change of only 10°C (Dongre 1995, Nam et al. 2001). Fatigue life of binders can change by orders of magnitude in response to a change of a few degrees of temperature or by changing loading frequency within the application range (fast versus slow traffic) (Bonnette et al., 2002).

The successful modification of asphalt binders would usually be designed to improve one or more of the basic asphalt properties that are related to one or more of the pavement distress modes. The basic properties that have been targeted include:

Rigidity: Total resistance to deformation which can be measured by complex moduli like G* under dynamic loading or by creep stiffness, S(t), under quasi-static loading. Higher rigidity is favorable at high temperatures or low loading rates to resist rutting while lower rigidity is favorable at intermediate- and low-temperatures to resist fatigue and thermal cracking, respectively.

Elasticity: Recovery of deformation using stored energy applied. It can be measured either by the phase angle (0) or by the logarithmic creep rate (m). To resist rutting and fatigue damage more elasticity is favorable. To resist thermal cracking, less elasticity and more ability of relaxing stress by flow is favorable.

Brittleness: Failure at low strains is the best definition of brittleness. To improve resistance for fatigue and thermal cracking, brittleness should be reduced by enhancing strain tolerance or ductility.

Storage Stability and Durability: Oxidative aging, physical hardening, and volatilization are key durability properties. Resistance to all of these changes is favorable.

Resistance to Accumulated Damage: Rutting and fatigue damage are recognized as the two most important load induced types of distress. They represent progressive failure mode, which is not necessarily measured using small stress or small strain testing. Many asphalt additives are considered as reinforcements at the micro-or macro level that would later damage progression favorable in asphalts.

A modifier can be selected to improve one or more of these main properties. Also, different modifiers that affect different properties can be combined to improve several properties. There have been numerous tests used to quantify each of the properties and measure the effectiveness of certain additives to improve asphalt properties. This chapter covers some of the key techniques to measure properties of modified asphalts and gives the background of these techniques. Ideally, a modifier will change rheological properties to match requirements as defined by resistance to pavement distresses as shown in the first part of Figure 1. It would also change failure properties such that binders would tolerate higher stresses and strains before failure due to static or repeated loading as shown in the second part of Figure 1.

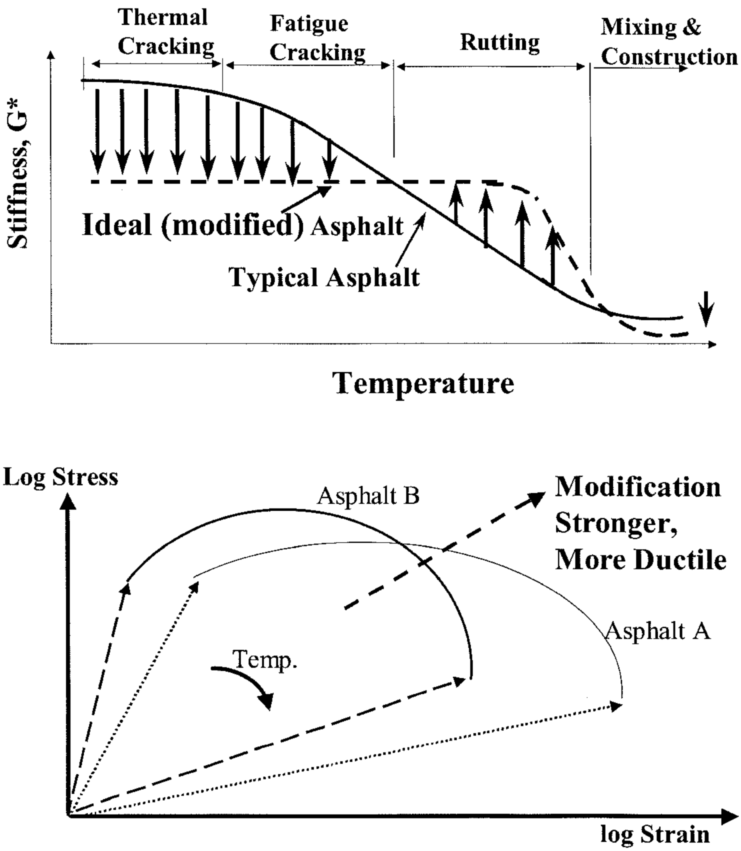


Figure 1: Schematics shown the target change in rheological and failure properties expected from Modification.

Asphalt Modifiers Currently Used

There are currently a large number of modifiers used for paving grade asphalts. Table 1 lists several published surveys of asphalt modifiers and identifies the general types of modifiers that each study had identified.

Table 1. Recent Surveys of Asphalt Modifiers

Modifier	**Reference**						
	Terrel and Epps, 1989 (2)	Peterson, 1993 (3)	Romine, et al., 1991 (4)	Moratzai and Moulthrop 1993 (5)	McGennis, 1995 (6)	Isacsson & Lu 1995 (8)	Banasiak & Geistlinger, 1996 (7)
Thermoplastic Polymers	X	X	X	X	X	X	X
Thermoset Polymers	X	X	X	X	X	X	X
Fillers/Reinforcing Agents/Extenders	X	X	X	X	X	X	X
Adhesion Promoters	X		X	X	X	X	X
Catalysts or Chemical Reaction Modifiers	X	X	X	X	X	X	X
Aging Inhibitors	X	X	X	X	X	X	X
Others	X		X	X	X		
Total number of existing brands or types	-	46	-	82 (27ASA)*	48	31	62

*ASA: Anti-stripping Additives

Asphalt modifiers can be classified based on the mechanism by which the modifier alters the asphalt properties, based on composition and physical nature of the modifier, or based on the target asphalt property that needs to be modified. For the survey in the NCHRP 9-10 project, the review results was used to prepare a list of modifiers classified based on the nature of the modifier and the mechanism by which it alters asphalt properties. A total of fifty-five modifiers classified in seventeen generic classes were identifies as shown in Table 2. The target distress shown in Table 2 corresponds to the main distress the additive is expected, or claimed, to affect favorably. The information given is based on interpretation of the published information for brands of modifiers that belong to the modifier classes shown. In many cases, the reported effects are based on limited data, meaning that they cannot be generalized to all asphalt sources.

The information gathered about these modifiers indicates that they vary in many respects. Some modifiers are particulate matters while others disperse completely or dissolve in the asphalt. The modifiers range from organic to inorganic materials, some of which react with the asphalt while others are added as inert fillers. The modifiers generically vary in their specific gravity as well as other physical characteristics. They are expected to react differently to environmental conditions, such as oxidation and moisture effects. With such diversity in asphalt modifiers, the current MP1 specification might be too simplistic to characterize all the possible varieties of modified asphalts.

TABLE 2. Generic Types of Asphalt Modifiers

Modifier Type		Effect on Distress				
	Class	PD[a]	FC[b]	LTC[c]	MD[d]	AG[e]
Fillers	Carbon black	x				x
	Mineral: Hydrated lime	x				x
	Fly ash	x				
	Portland cement	x				
	Baghouse fines	x				
Extenders	Sulfur	x	x	x		
	Wood lignin				x	
Polymers – Elastomers	Styrene butadiene diblock (SB)	x		x	x	
	Styrene butadiene triblock/radial block (SBS)	x	x	x		
	Styrene Isoprene (SIS)	x				
	Styrene ethylbutylene (SEBS)					
	Styrene butadiene rubber latex (SBR)	x		x		
	Polychloroprene latex	x	x			
	Natural Rubber	x				
	Acrylonite butadiene styrene (ABS)	x				
Polymers – Plastomers	Ethylene vinyl acetate (EVA)	x	x			
	Ethylene Propylene diene monomer (EDPM)	x				
	Ethylene acrylate (EA)	x				
	Polyisobutylene	x				
	Polyethylene (low density and high density)	x		x		
	Polypropylene	x				
Crumb rubber	Different sizes, treatments, and processes	x	x	x		
Oxidants	Manganese compounds	x				
Hydrocarbons	Aromatics			x		
	Napthenics					
	Paraffinics/wax			x		
	Vacuum gas oil			x		
	Asphaltenes: ROSE process resins	x				
	SDA asphaltenes	x				
	Asphaltenes: DEMEX asphaltenes	x				
	Shale oil				x	x
	Tall oil					
	Natural asphalts: Trinidad	x	x	x	x	
	Gilsonite	x			x	
Antistrips	Amines: Amidoamines				x	
	Polyamines				x	
	Polyamides				x	
	Hydrated lime				x	
	Organo-metallics				x	
Process-based	Air blowing					
	Steam distillation					
	Propane de-asphalted (PPA)					
	Acid Based (Poly-phosphoric)					

TABLE 2. Generic Types of Asphalt Modifiers (Cont'd)

Modifier Type	Class	Effect on Distress				
		PD[a]	FC[b]	LTC[c]	MD[d]	AG[e]
Fibers	Polypropylene	x	x	x		
	Polyester	x		x		
	Fiberglass					
	Steel	x	x	x		
	Reinforcement	x	x	x		
	Natural: Cellulose	x				
	Mineral	x				
Antioxidants	Carbamates: Lead			x		x
	Zinc			x		x
	Carbon black	x				x
	Calcium salts					x
	Hydrated lime				x	x
	Phenols					x
	Amines				x	x

[a] Permanent Deformation
[b] Fatigue Cracking
[c] Low Temperature Cracking
[d] Moisture Damage
[e] Oxidative Aging

Not all these types of modifiers are being currently used. The frequency of use varies significantly depending on marketing of the modifiers, experience of contractors and agencies, and cost. In a survey conducted in 1997, State Highway Agencies indicated that polymer modifiers are among the most widely known and used in practice. Table 3 depicts the results of the survey.

Table 3. Modifiers most commonly used by State Highway Agencies

Type	Class	No. Of Agencies	Target Distress/Property (No. Of Agencies)				
			PD[1]	FC[2]	LTC[3]	MD[4]	AR[5]
Polymer – Elastomer	Styrene Butadiene Styrene (SBS)	28	18	8	10	3	6
	Styrene Butadiene (SB)	16	13	5	5	0	2
	Styrene Butadiene Rubber Latex (SBR)	17	10	4	4	1	2
	Tire Rubber	3	1		1		
Polymer – Plastomer	Ethyl Vinyl Acetate (EVA)	6	3				1
Anti-Stripping Agents	Fatty Amidoamines	8				4	
	Polyamines	6				4	
	Hydrated Lime	4				3	
	Others	7				3	
Hydrocarbons	Natural Asphalts	6	5				
Fibers	Cellulose	12	3		1		1
	Polypropylene	7	4		1		
	Polyester	6	4		1		1
	Mineral	3	1	1	1		
Processed-Based	Air Blowing	4	2				
Mineral Fillers	Lime	4				1	
Anti- Oxidants	Hydrated Lime	7				4	
Extenders	Sulfur	4					

[1] Permanent Deformation [2] Fatigue Cracking [3] Low Temperature Cracking [4] Moisture Damage [5] Aging existence

Table 3 shows that the elastomeric polymer SBS is the most frequently used to target a variety of distresses among which permanent deformation is the most common. The other modifiers that are listed in Table 2 but are not shown in Table 3 were either not identified by any state highway agency or were identified by only one or two agencies. As indicated in Table 3, many agencies did not indicate the specific target distress for which they are using the modifier.

To identify the potential for the future usage of modified binders, the agencies were asked indicate the target distress(es) that will justify the use of more modifiers in the future. The results of the state/provincial agency survey questionnaires lead to the following findings:

Rutting	39
Low temperature cracking	28
Fatigue Cracking	21
Aging	14
Moisture Damage	14

It appears that modified binders are meeting the expectations of State Highway Agencies because the majority of agencies (47/52) are planning on using the same or more of modified binders. The justification for use of modifiers was mainly for rutting and low-temperature thermal cracking. It appears that fewer agencies are using modified asphalts for fatigue, aging, or moisture damage.

Section 2. Critical Properties of Modified Asphalts

Measuring properties of modified binders has in general followed the testing technology used for un-modified asphalts but in many specification expanded to capture some unique properties that modifiers were used to induce or enhance in the base asphalt. Based on critical properties measured they can be classified into two main groups: Mechanical properties and compatibility or storage stability properties. Testing of mechanical properties can be further sorted, based on progress in testing technologies into three groups: Traditional rheological, linear visco-elastic, and damage resistance characterization. The following sections describe the different properties measured or modified binders throughout the years and give some details about the value of these tests and trends observed as a result of using selected modifiers.

2.1 Traditional Rheological Properties

Traditional or index rheological properties include many standardized tests that were mostly used for many years around the world before the introduction and standardization of rheometers after the completion the Strategic Highway Research Program in the early 1990's. Table 4 includes a list of these tests.

Table 4. Standardized Tests prior to Rheometers

Test	Standard Protocol	Effect of modification
Index (Empirical) Tests		
Softening point (R&B)	ASTM D36	An index of consistency at very high temperatures. Instability of MAB could affect test significantly. Used widely around the world for compliance but not considered reliable for assessing the high temperature performance in paving applications.
Penetration @ 25 C (dmm)	ASTM D5, NF T 66-004	Considered an index for consistency of binder at intermediate temperature. Generally poses no problem for asphalts modified with SBS and EVA but could be problematic for high contents and is known to be affected by thermal history or by nature of modifier (Piarc-C8 report). Although test is questionable and of little value, it is used in many Modified Asphalt Binder (MAB)specifications in Europe, Australia and Japan. The effects of modifiers can be an increase or a decrease in penetration.
Frass Breaking Point	IP 80	An index of resistance to cracking at low temperatures under repeated flexing and reducing temperature. Test is mostly used in Europe as an indicator of brittleness. Modifiers are known to result in decreasing Frass point, particularly for elastomers.

Table 4. Standardized Tests prior to Rheometers (cont'd)

Test	Standard Protocol	Effect of Modification
Viscosity (resistance to flow) Tests		
Rotational Viscosity @ 135-165 C	ASTM D402	A measure to resistance to flow at production and construction temperatures of HMA. Used widely to measure workability of MABs
Absolute Viscosity @ 60 C	ASTM D2170	Measured using the capillary tube under vacuum. Because many modified asphalts are non-Newtonian, it has been used with caution and is not used in specification widely. Modifiers are known to increase the value of this viscosity and also increase shear rate dependency.
Cone and Plate Apparent Viscosity @ 25-60 C	ASTM D3205	Method was developed to measure creep response under increasing creep loads to estimate apparent viscosity at intermediate to high temperature. Not used widely and standard was discontinued in 2000. Most polymer modifiers are expected to increase viscosity.
Tensile (extensional) Properties		
Ductility @ 4 C and @ 25 C	ASTM D113	An index of flexibility at low temperatures when used at 4C. Also considered an index of compatibility particularly when used at 25 C. Effect of modifiers on ductility varies significantly depending on nature of modifier. Elastomeric modified tends to increase ductility at these temperatures, plastomers have minimal or negative effect, and chemical or oxidation tends to decrease ductility particularly at low temperatures. Used in Europe at various temperatures. Also used in North America in some states
Forced Ductility @ 4 C	Not a standard	An index of tensile strength and energy required for complete failure. Specifically developed for polymer modified asphalts and used widely in North America. Response usually include an initial and a secondary peak of the stress which is used to calculate a ratio showing effect of the modifier. Test taken from joint sealant testing field.
Elastic Recovery @ 25 C	Not a Standard	An index of the capability of modified asphalt for elastic recovery. Measured using the conventional ductility set up but sample is stretched and then cut to measure recovery of cut ends. One of the most widely used to determine if modified binder includes elastomers. Used in North America, Australia and Europe. Method has been modified several times and is run using sliding plate rheometer, ARRB Elastometer, Consistometer, and torsional loading set-up.
Toughness and Tenacity @ 25 C	Not a standard test	An index of energy to failure used to detect modifiers and assess their contribution to toughness. A hemispherical head is inserted in an asphalt container and then pulled out. The area under the load deformation curve is divided into an initial peak area and a terminal tenacity area. The sum is the toughness. Elastomeric modifiers could have a significant effect on tenacity and on toughness particularly if they are cross linked.

2.2 Linear Visco-elastic Properties

Although using rheological concepts to characterize asphalts dates back to more than 50 years, the cost and availability of equipment slowed down the spread of using visco-elasticity to qualify asphalts and study effect of modifiers. During the early 1990's SHRP put a major emphasis on development of testing equipment and/or test protocols to standardize the use of such properties for asphalt testing that is relatively blind to the composition of the binders. The test protocols, although developed with the intention of being suitable to un-modified and modified binders, focused on linear visco-elasticity as a compromise between the difficulty of rheological testing and the need to produce practical and realistic test protocols for an industry that used mainly simplistic index testing to qualify asphalts.

As a result of the Strategic Highway Research Program (SHRP), a new set of testing techniques and a new grading system has been introduced. The testing and grading systems are based on measuring fundamental properties that are related in a more rational way to pavement performance. The visco-elastic properties of asphalt binders are measured at application temperatures and grading criteria are based on sound understanding of pavement failure mechanisms.

NATURE OF ASPHALT VISCO-ELASTIC PROPERTIES

At any combination of time and temperature, visco-elastic behavior within the linear range is best characterized by two properties: The total resistance to deformation and the relative distribution of that resistance between an elastic part and a viscous part. Although there are many methods of characterizing visco-elastic properties, dynamic (oscillatory) testing is one of the best techniques that can represent the uniqueness of the behavior of this class of materials. In the shear loading mode, the complex modulus (G^*) and the phase angle (δ) are measured. G^* represents the total resistance to deformation under load, while δ represents the relative distribution of this total response between an in-phase component and an out-of-phase component. The in-phase component is an elastic component and can be related directly to energy stored in the sample for every loading cycle while the out-of-phase component represents the viscous component and can be related directly to energy lost per loading cycle in permanent flow or damage. The relative distribution of these components is a function of composition of the material, loading time or frequency, and temperature.

Asphalt rheological properties are very sensitive for temperature and time of loading; within the range of pavement application (temperature range of -40 to 80 °C and loading rate of static to 100 rad/s), typical asphalt changes its modulus by more than seven orders of magnitude and its phase angle by approximately 90 degrees. At low temperatures, or high frequencies, asphalts tend to approach a limiting value of G^* of approximately 1.0 GPa and a limiting value of 8 of 0.0 degrees. The 1.0 GPa reflects the rigidity of the carbon hydrogen bonds as the asphalts reach their minimum_thermodynamic equilibrium volume. The 0.0 value of δ represents the completely elastic nature of the asphalts at these temperatures. As the temperature increases or as the frequency decreases, G^* decreases continuously (indicating softening) while δ increases continuously (indicating less elasticity). The first reflects a decrease in resistance to deformation (softening) while the second reflects a decrease in elasticity or stored energy. The rate of change is however dependent on the composition of asphalt. Certain asphalts will show a very rapid change with temperature or frequency, others will show gradual change. Within this range, asphalts may show significantly different combinations of G* and 0. At high temperatures the 0 values approach 90° for all asphalts, which reflects the approach to complete viscous behavior or complete dissipation of energy in viscous flow. The G* values, however, vary significantly reflecting the different consistency (viscosity) of the asphalts.

In addition to the pre-failure properties of asphalts as measured by rheology, their failure properties need to be characterized. Asphalts failure behavior is also highly dependent on temperature and time of loading. They are brittle at low temperatures with a plateau zone showing a strain at failure that is relatively small (limiting value of approximately 1.0 percent strain). As temperature increases, a transition from brittle to ductile failure can be observed which, at high temperatures, converts into a flow zone. The most critical part of this behavior for pavement applications is the temperature and loading rate at which the transition from the brittle to the ductile behavior occurs. For many unmodified asphalts, there is some correlation between stuffiness measured at small strains (rheological pre-failure properties) and this transition. The correlation, however, does not hold for modified asphalts or specially produced asphalts (Anderson et al. 1991).

From the above discussion of asphalt properties, it is expected that without measuring the rheological and failure properties at the temperature and loading frequency ranges that correspond to pavement climatic and loading conditions, selection of asphalt binders for better performing pavements and selection of modifiers that can improve the properties of these binders is very difficult.

Testing of the base and modified binders included full characterization using the dynamic shear rheometer at different temperatures and frequencies. The binders were characterized by running frequency sweeps of 1 to 100 rad/s at temperatures ranging between -30 and 60°C. The testing geometries deviated from the standard geometries later selected for testing neat asphalts. For parallel plate geometry, 2.0-mm gap was used for all binders at high and intermediate temperatures. At temperatures below 5°C, torsion bar geometry were used to cover the range of high moduli measured for the different binders.

In addition to dynamic shear rheometer, the bending beam rheometer was used to measure creep properties at several low temperatures. Also the direct tension test device at low temperatures. Oxidative aging was done using the TFOT (AASHTO T179) and the pressure aging vessel. No changes were made in the standard procedures for the creep, failure, and aging tests except for taking extra care to prepare specimens and ensure uniform dispersion of the additives. Selected data are used in this article to present the important points observed during the study.

EFFECT OF MODIFICATION ON RHEOLOGY AND FAILURE PROPERTIES OF ASPHALTS

To show the general trends in effect of modifiers on asphalt visco-elastic properties, there types of additives are used: polymer, crumb rubber, and mineral fillers. Polymer modifiers included Styrene-Butadiene (SB) based modifiers and polyethylene- based modifiers (PEl to PE5). Crumb rubber modifiers (CRM) included ambient shredded crumb rubber (RB3), cryogenic grinded (RB2), and a crumb rubber-plastic composite (RBI). All crumb rubbers were produced from whole tire stock with maximum particle size of 1.0-mm. Mineral fillers included manufactured quartz and natural calcite of maximum p article size of 75 microns. The polymer modifiers were pre-blended by manufacturers at concentrations varying between 3 and 6 percent. The crumb rubber modifiers were mixed at 15 percent by weight of binder in the laboratory using a blender at 160° C for one hour. The mineral fillers were mixed using the same technique used for the crumb rubber at a ratio of 0.50 filler to asphalt by volume. Although mineral fillers are not recognized as modifiers, they are used here to show the extreme behavior of a relatively rigid, non-interacting, additive.

The dynamic shear rheometer was used to develop and compare master rheological curves of the modified binders with their base asphalts. The following discussion is divided into sections according to type of modifier or additive used.

Polymer Modification

Figure 2 depicts rheological master curves measured using a dynamic shear rheometer for an asphalt before and after modification with the SB polymer at two different concentrations (c=3 and 2c=6 percent). Changes in both G* and δ as a function of temperature are shown. The effects of this modifier show favorable trends of change: At high temperatures, G* is higher while 0 is lower. This indicates an increase in rigidity and in elasticity, which results in better resistance to permanent deformation. At intermediate temperatures (0 to 30°C) lower values of G* can be observed while 0 values remain indifferent. The reduction in G* values is favorable for fatigue cracking under strain controlled conditions, typical of conditions for thin pavements. At low temperatures (-20 to 0.0°C), a similar, or more pronounced, reduction is observed for G* and a minor increase in 0 is seen. Both these effects are favorable since they make the binder less rigid and less elastic or more prone to stress relaxation under load. The changes shown appear to improve the properties with respect to pavement performance at all temperatures.

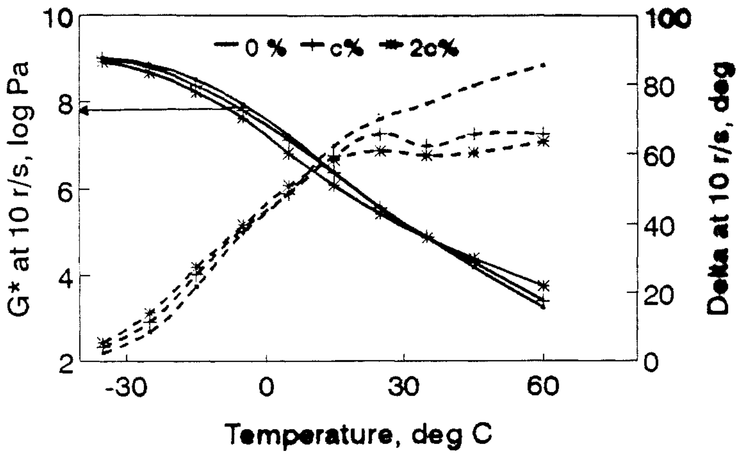


Figure 2. Isochronal rheological curves for an asphalt before and after SB modification.

Considering the relative changes in the G* and δ, it is evident that the main effect is the change in the rigidity of the binder as measured by G*. The data shown in figure 2 indicate that while the G* value is increased at 60°C by 100 to 200 percent, the δ value is reduced by approximately 16 to 30 percent. At low temperatures the same trend can be observed; the G* value is reduced by 40 to 50 percent while the δ value is increased by only few degrees. Similar trends in changes were observed for the other types of polymers that were used in the study. Considering the fact that energy dissipation and rate of relaxation of binders are functions of sin 0 or tan 0, it appears that effects of these commonly used polymeric additives on binders at small strains or stresses are mainly caused by change in rigidity while only secondary effects are caused by changes elasticity.

CRM Modification

Figure 3 depicts master rheological curves for an asphalt before and after modification with a crumb rubber modifier (CRM), at 15 percent weight concentration. The figure is in terms of loading frequency rather than temperature. As discussed earlier, frequency and temperature are interchangeable; the effect of high temperature corresponds to that of low frequencies, and visa versa. Changes in master curves are similar to the changes observed for polymer modification shown in figure 2. G* values increase at low frequencies (high temperatures) while they decrease at intermediate and high frequencies (intermediate and low temperatures). The 0 values are lower at low frequencies but higher at high frequencies. The relative changes of either parameter are of the same order of magnitude as for the polymer modification. The effects of CRM can, therefore, be also described as mainly changes in rigidity of the asphalt.

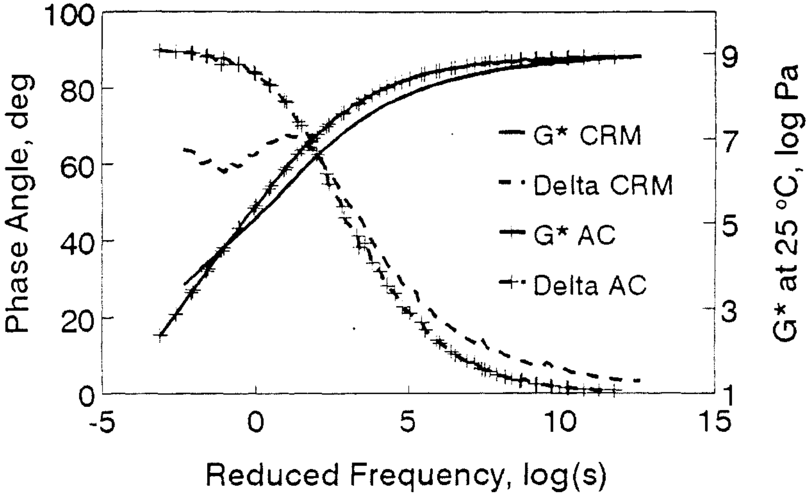


Figure 3. Rheological master curve for an asphalt before and after CRM modification (15% CRM)

The mechanism by which CRM changes properties is, however, different; while for most polymer modifiers the polymer is completely dispersed in the asphalt and causes changes in the molecular structure of the asphalt, the CRM is observed to keep its physical identity and behave as a flexible particulate filler in the asphalt. The overall effect of CRM on rheological master curve is reduction of dependency of G* and δ on frequency. This effect is similar in nature to the effect of polymer modification despite the difference in the nature of material. Polymer modification usually results in a more homogeneous binder, which is more favorable than the non-homogeneous CRM modification. The trade-off however, is the relatively higher cost of the polymer modifiers compared to the CRM.

Mineral Fillers

Figure 4 depicts master rheological curves for an asphalt before and after addition of two mineral fillers. Unlike the previous modifiers the effect of mineral fillers results in increasing the G* value and decreasing the 0 value at all frequencies (temperatures). This distinct change is expected when the rigid nature of the mineral fillers is considered. While most polymers and CRM have moduli that are lower than that of typical asphalts at intermediate and low temperatures, mineral fillers continue to show moduli that are much higher than asphalt even at very low temperatures where asphalts reach their glassy modulus. Furthermore, since mineral fillers lack the visco-elastic nature, they do not impart any significant changes in the phase angle (0). The upward shift in the G* curves seen in figure 4 is simply the effect of the addition of this rigid body of particles that increases the moduli at all temperatures and more so at high temperatures (low frequencies) where the asphalt moduli are even lower. The effect at low temperatures (high frequencies) is not favorable since it indicates an increase in modulus and decrease in the ability of relaxing stresses. At high temperatures the effect is favorable and, for the fillers shown, much more pronounced than the effect of either modifiers considered earlier.

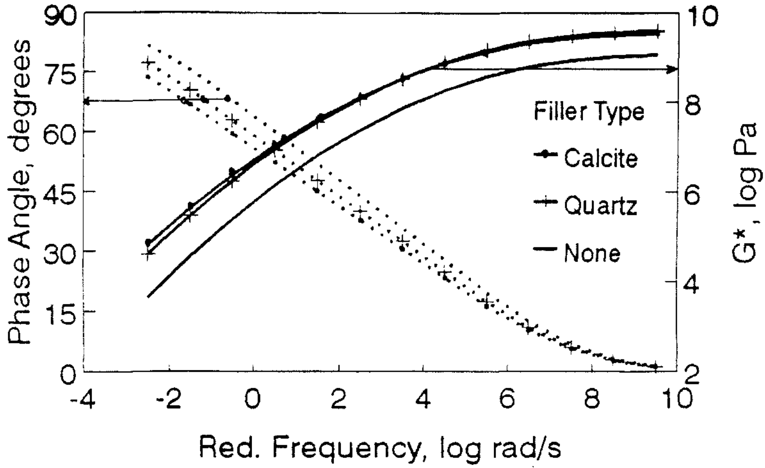


Figure 4. Rheological master curves for an asphalt before and after addition of mineral.

One of the similarities_between the effects of fillers and the other modifiers is the reduction in dependency of G* and d on temperature or loading frequency. Also, it is evident that the rheological behavior of binders with fillers as well as the two other modifiers remains relatively simple in nature: At low temperatures a glassy modulus asymptote is reached where the response is completely elastic and at high temperatures a viscous asymptote is reached where behavior is mainly, if not completely, viscous.

EFFECTS OF MODIFICATION ON FAILURE PROPERTIES

Using the direct tension test developed by SHRP, the binders modified with the different additives were tested at temperatures ranging between -30 and 0.0 °C. The tests were conducted at a deformation rate of 1.0 mm/min in three replicates and the stress and strain at failure were calculated. To evaluate the effect of the modifiers, the stress and strain values at failure of the base and the modified binders are compared.

Polymer Modification

Figure 5 shows strain at failure and stress at failure plots as a function of temperature for an asphalt before and after modification with 3 and 6 percent of the SB-based polymer. The strain curves show that the polymer increases the strain at failure within the brittle and the brittle - ductile zone but converge to the same values as the flow zone is approached. The effect can be considered as shifting the strain at failure curve horizontally to lower temperatures without significant changes in shape of curve. The effect of polymer addition is favorable as it tends to increase the strain at failure within the critical region. The results shown also indicate that the effect is more favorable with higher concentration of the polymer. The stress at failure curves are similar for all binders which indicate that the polymer does not result in significant changes in strength of binders.

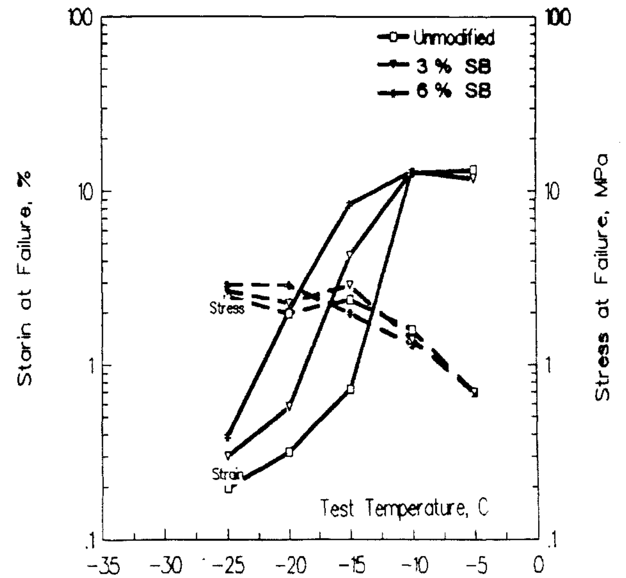


Figure 5. Failure strain isochronal curves for an asphalt before and after SB modification.

The results shown in figure 5 may not apply to all types of polymer modifiers. The effect of different polymers on failure properties is expected to depend largely on the type of interaction between the asphalt and polymer, the molecular nature of the polymer additives, and the way it is dispersed in the asphalt. The effect of polymers on failure properties can be hypothesized in different ways. One hypothesis is that polymers form some kind of molecular network inside the asphalts resulting in more strain tolerant material. Another hypothesis is that the dispersed polymer particulates may serve as reinforcements, arresting micro crack propagation and increasing toughness of binders. The typical trend that can be observed from review of polymer modification works, however, is that not many polymer used currently improve low temperature failure properties. This may be attributed to the fact that until recently there has been no simple technique to measure the brittle failure of asphalt, and also to the fact that none of the used binder specifications address the brittleness of asphalt in a rational and fundamental form. These issues did not encourage many polymer modifier producers to concentrate on designing a modifier to mainly enhance low temperature failure properties.

CRM Modification

Figure 6 depicts failure plots for an asphalt before and after modification with crumb rubber at 10 percent (CRM1) and 20 percent (CRM2) concentration by weight of total binder. The effect of the CRM is similar to the polymer modification with respect to the strain at failure values; higher strains are observed at low temperatures but *similar* strains are observed as the flow region is reached by the binders. The effect also represents a shift of the failure curve along the temperature scale toward lower temperatures. The shift is larger for the higher CRM content. The stress at failure curves, however show a trend different than the polymer modification.

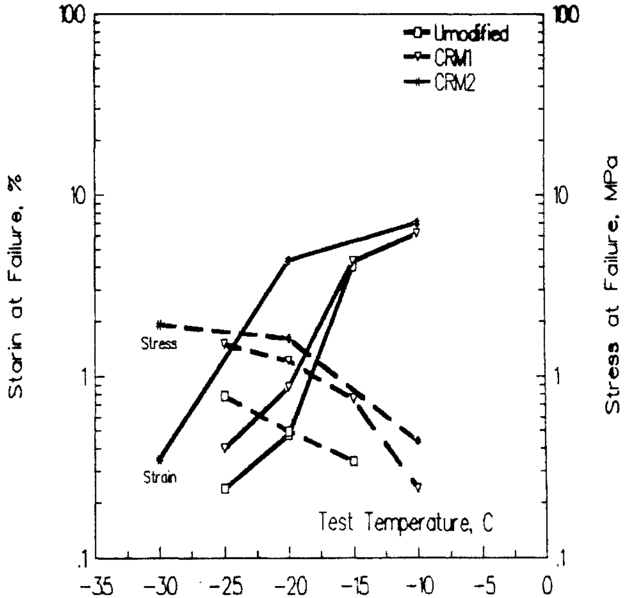


Figure 6. Failure strain isochronal curves before and after CRM modification.

The CRM results in stress values that are significantly higher than the unmodified asphalt at all temperatures. This behavior can be attributed to the reinforcing effect of the rubber particles. The crumb rubber particles do not dissolve in asphalt; the particles maintain their integrity and tend to swell in asphalts resulting in effective volumes that are larger than their initial volume (Bahia & Davies 1994, Oliver 1982, Chehoveits et al. 1982). It is speculated that the swelling results in selective absorption and/or adsorption of certain components of the asphalt. Such interactions are expected to reinforce the matrix of the binder and result in higher strength, as observed in the figure. The increase in strain and stress at failure is favorable for paving grade asphalts, particularly when it is accompanied by a reduction in stiffness as shown in figure 3.

Mineral Fillers

Figure 7 depicts a bar chart for failure stress and strain of an asphalt before and after addition of two types of mineral fillers (C and Q) at 50 percent volume concentration. As shown the addition of both fillers results in significant increase in strain at failure at all temperatures. It also results in approximately equal or higher stress at failure. Although the fillers may be considered less reactive with asphalt compared to the rubber and polymer additives, their presence appears to result in an important reinforcing effect. From a fracture mechanics consideration, the fillers may serve to arrest cracks or result in longer crack paths. This effect is comparable to the' effect reported of fillers used in polymer composites to enhance toughness of polymers. The addition of fillers, as shown in this study, can result in certain improvements in low temperature failure properties of asphalts despite of their adverse effects on rheological properties. The effects, however, are dependent on asphalt type and test temperature. Because of the different nature of mineral fillers, their effects continue to be important even within the ductile flow region. These rigid filler particles are expected to enhance resistance to flow within the ductile region and to increase the peak stress and strain at these conditions.

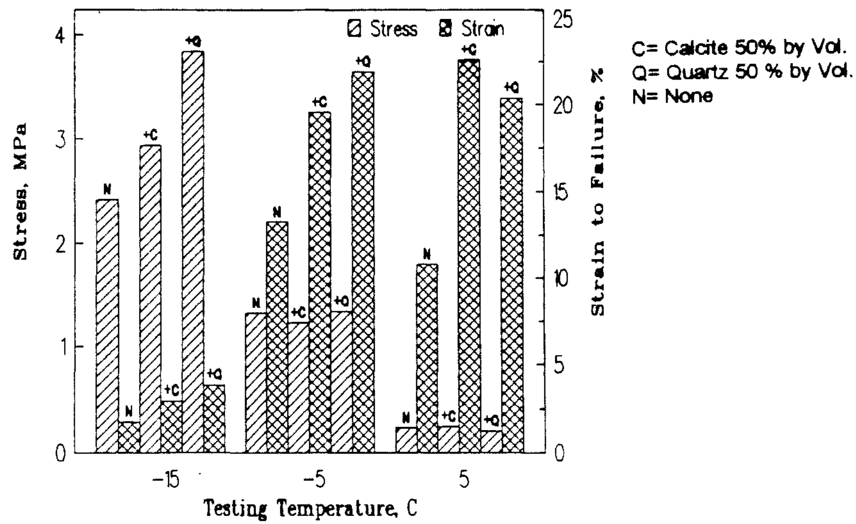


Figure 7. Effect of mineral fillers on strain and stress at failure at selected temperatures

EFFECT OF MODIFICATION ON SHRP GRADING PARAMETERS

The research efforts of the SHRP binder program have resulted in the introduction of several response parameters that are indicators of the contribution of binders to pavement performance. These parameters were derived by addressing each type of pavement failure, understanding the failure mechanism, understanding the contribution of the binder to resistance of that failure, and selecting the required measure that will best reflect that contribution of the binder (Anderson et al. 1991). The new binder specification is based on climatic conditions: The criteria that a binder has to meet does not change but the temperature at which the property is measured depends on the specific field climate, and on the failure mode being considered (Anderson & Kennedy 1993).

Three failure modes were identified as critical pavement distress modes in which the binder plays an important role: Rutting, fatigue cracking, and thermal cracking. Oxidative aging and physical hardening were considered as durability factors that result in changes in properties of binders and thus affect performance. For each distress mode, a testing procedure and a parameter have been defined to measure contribution of binder to resistance for that failure mode.

Effect of Polymers on SHRP Parameters

Figure 8 depicts a bar chart of the ratios of the 8HRP parameters for an asphalt after modification with three different concentrations of the 8B-based polymer. The figure indicates that there is a favorable trend in changes of all the performance parameters: G*/sin δ is increased by a ratio of 1.2 to 3.4, G*sinδ is reduced by 10 to 50 percent, m(60) is also lowered significantly and m(60) is increased slightly. For this polymer, the modification is extended also to the strain at failure which is increased by ratios between 3.4 and approximately 6. Figure 9 shows the ratios of parameters for five other types of Polyethylene-based polymers. At the high temperatures, G*/sinδ ratios all show favorable values ranging between 2.2 and 6.3. At intermediate and low temperatures, however, the ratios of G*sinδ, 8(60), and m(60) do not show significant change, and, for some of these polymers an unfavorable change can even be seen. The strain at failure also shows only minor changes.

The data shown in figures 8 and 9 indicate that polymer modification can have major effects on the parameter used to measure contribution of binder to rutting resistance at high temperatures. The changes in the parameters related to fatigue and thermal cracking are, however, marginal except for the strain at failure with one type of polymer. The effects on strain at failure should not be exaggerated; a look at figures 5 and 6 indicates that strain at failure is very sensitive to temperature and the transition from brittle to ductile failure occurs over a narrow temperature range. Thus a minor shift in the strain curve as a result of modification may show very high strain ratios. The data here agrees previous experience reported for polymers (Collins et al. 1990, King et al. 1992). For many years, polymer modifiers have been accepted and marketed as the cure for the problems of pavement rutting.

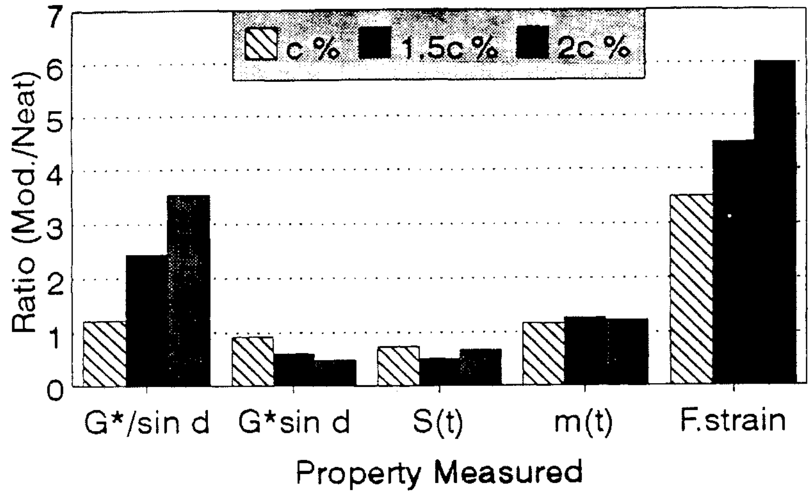


Figure 8. Relative change in SHRP performance related parameters after modification with SB-based modifiers (c=3)

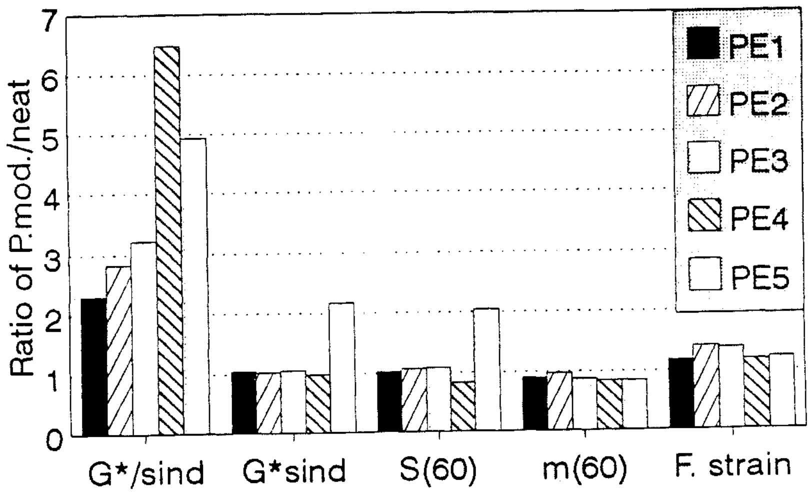


Figure 9. Relative change in SHRP performance related parameters after modification with polyethylene based modifiers

Effect of Crumb Rubber on SHRP Parameters

Figure 10 depicts similar bar charts for the CRMs used in the study. The main difference between these modifiers, as explained in an earlier section, is not the size or raw material, but the process by which they are manufactured. The ratios look similar to the polymer modification with few changes. While the ratios for the rutting parameter (G*/sin δ) are higher than for the polymer modification. The ratios of G*sin δ is much lower than the polymer modified asphalts and even lower than for the unmodified asphalt (ratios <1). 8(60) ratios are also lower, while the ratios of m(60) and strain at failure are very close to the value of 1.0 (no change). The data in figure 10 show that similar to polymer modification, crumb rubber modification shows its main effects at high temperatures. This is expected when the nature of the CRM modifier is considered. Crumb rubber acts mainly as a flexible filler; at high temperatures it is more stiff than the asphalt and thus contributes significantly to the increased moduli With decreasing temperatures, the asphalt becomes stiffer while the crumb rubber properties do not change significantly. At a certain temperature, the asphalt may become stiffer than the crumb rubber and thus a reduction in stiffness can be observed for the modified binder. Crumb rubber, at moderate concentrations that are used in practice (10 - 20 percent), however, cannot reduce stiffness by large margins because of its own relatively high stiffness at low temperatures. It is, therefore, expected that the main effects of crumb rubber remain to be seen at higher temperatures and to affect mainly the rutting parameter.

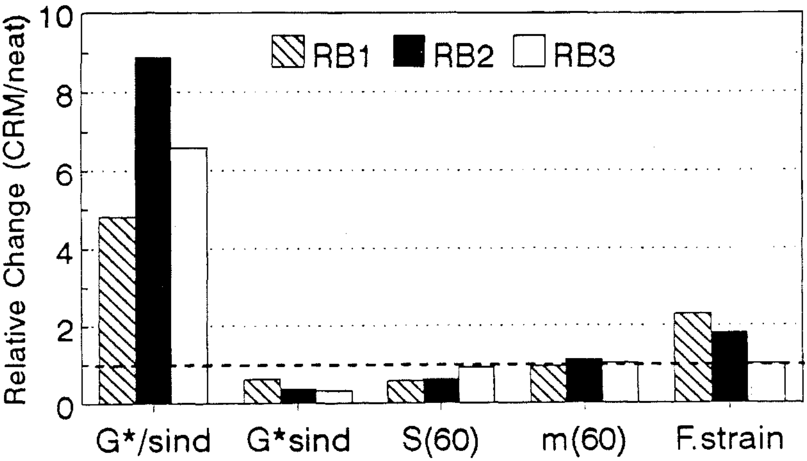


Figure 10. Relative change in SHRP performance related parameters after modification with CRM.

<u>Effect of Mineral Fillers on SHRP Parameters</u>

Figure 11 depicts ratios for two asphalts after modification with two types of fillers. Ratios of G*sin δ ranges between 6.5 and 12 for the filler volume concentration of 50 percent. These increase in G*sin δ are favorable and certainly indicate that the addition of mineral fillers can increase the contribution of binder to resistance of pavement rutting. The ratios of G*sin δ, however, do not show a favorable trend; ratios of 3.1 to 4.7 are shown which indicates that fillers can be detrimental with respect to fatigue damage under strain controlled loading conditions because of the increase in G*sin δ . The effects are even less favorable with respect to low temperature properties; S(60) ratios range between 4.5 and 6.7 and m(60) ratios are all less than 1.0. Strain ratios show some improvements for certain combinations but a value of 0.3 is shown for one of the modified binders. As mentioned earlier, fillers are not expected to modify intermediate and low temperature properties where a softer binder is more favorable. These fillers are rigid particles with stiffness values that far exceed those of conventional asphalt cements, even at very low temperatures. They are therefore expected to always result in stiffer binders that may not meet the needs for paving applications.

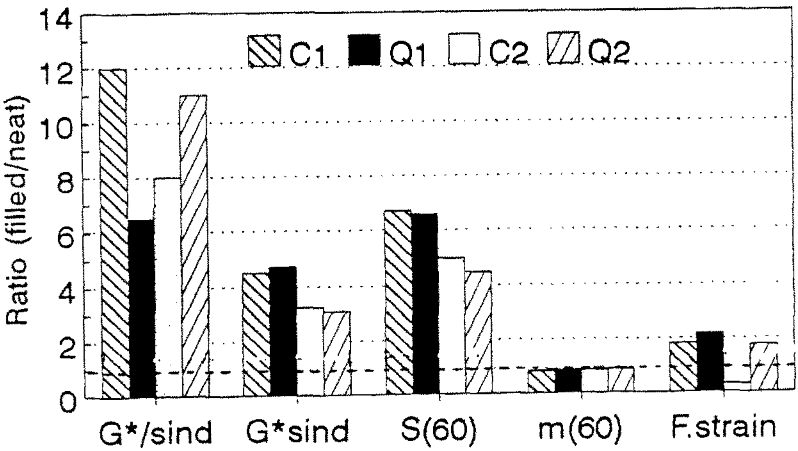


Figure 11. Relative change in SHRP performance related parameters after addition of mineral fillers.

Summary of Effect of Modification on Rheological Properties:

The effect of polymeric additives, crumb rubber additives, and mineral fillers on performance-related properties of asphalt cements have been analyzed using data collected for a number of binders. The analysis included rheological and failure properties measured using new characterization techniques developed by the Strategic Highway Research Program (SHRP).

1. Polymer modification of paving grade asphalts can result in improved rheological and failure properties. The effects of modification, which may vary significantly in their levels depending on asphalt properties and polymer type, result in reducing the sensitivity of rheological properties (G* and 0) to temperature and loading frequency. The main effects are measured at high temperature (low frequencies) where the

polymers result in a higher stiffness and a lower phase angle. At intermediate and low temperatures, the effects are less pronounced, and can be unfavorable.

2. Crumb rubber modification has similar effects to polymer modification. The major changes can be observed at high temperatures with increase in stiffness that can be higher than levels normally achieved by polymers. Crumb rubber modifiers also result in reduction of dependency on temperature and loading frequency. Effects at intermediate and low temperatures can be marginally favorable depending on the properties of the asphalt. These modifiers can result in reduction of stiffness at the intermediate and low temperatures if their stiffness is less than the stiffness of the asphalt matrix: Crumb rubber modifiers remain as particulates after mixing with asphalts; they mainly function as interactive fillers.

3. Mineral fillers result in effects similar to the other modifiers with respect to reduction in dependency of asphalt rheology on temperature and loading frequency. They, however, result in increased stiffness at all temperatures and frequencies. This indicates that, although their effects at high temperatures are favorable, their effects at intermediate and low temperatures are not favorable and can be detrimental with respect to fatigue and thermal cracking.

4. Using the performance parameters introduced by SHRP, the effects of the different additives on the contribution of binders to resistance of the distress mechanisms can be summarized as follows;

. Rutting Resistance: Polymers, crumb rubbers, and mineral fillers result in significant increase in G* and decrease in sin δ. These effects are favorable for rutting resistance because they indicate higher resistance to total deformation under load and higher elasticity of response.

. Resistance to Fatigue Damage: Polymers and crumb rubbers can result in marginal improvements by reduction in G* and sin δ. These effects are considered favorable because they indicate softer and more elastic response. Such response results in less energy dissipated under strain controlled fatigue. Mineral fillers, however, result in significantly higher G* values, which is not favorable for strain controlled fatigue.

. Thermal Cracking Resistance: Certain polymers result in lower S(t), higher m(t), and higher strains at failure. Such favorable effects could not be observed for all polymers evaluated in this study. Crumb rubbers generally have S(t) values that are less than S(t) of most conventional asphalts at critical low temperatures. They can therefore result in reduction of S(t). Crumb rubber effects on m(t) are not significant. Crumb rubbers are observed to cause significant increase in strain and stress at failure. Mineral fillers are not expected to have favorable effects on thermal cracking resistance; higher S(t) and low m(t) are observed for all tested systems in this study. Mineral fillers can, however, increase the strain at failure and stress at failure depending on the properties of asphalt and filler characteristics.

Section 3. Classification of Modified Binders Into Simple and Complex binders

One of the main objectives of the Strategic Highway Research Program (SHRP) was to develop test methods for characterization of asphalts that are equally applicable to unmodified or modified asphalt cements, collectively called asphalt binders (Anderson et al. 1994). The objective of developing these test methods was to develop a specification that is "blind", applying to all asphalt binders without exceptions and without the need to ask producers what was done to produce the Performance Graded (PG) asphalt. The argument that, if the Superpave Binder Specification (AASHTO MP1) is truly performance based, there should be no concern about its applicability to all asphalt binders, is a valid argument.

There were, however, two problems that raised concerns about applicability of PG specification to all asphalt binders. The first was that the majority of the testing during SHRP was done on unmodified asphalts of certain PG-grades that did not cover the extreme grades required by the new specifications. A review of the asphalts included in the SHRP MRL indicates that they range between a PG 64-28 and a PG 46-34 with one PG70-22. This range of grades does not cover extreme grades that are being specified for high volume traffic in warm regions and grades being considered in many cold regions. The second problem was the fact that these extreme grades, such as 76-22, 82-22, 64-34, and 58-40 did not exist at the time SHRP research was active.

The concerns about the Superpave binder specification applicability to all asphalt binders resulted in the initiation of the NCHRP 9-10 project, "Superpave Protocols for Modified Asphalt Binders." The first phase of the project included a survey of users and producers of modified binders to identify the types of asphalt additives most commonly used in practice, to summarize concerns about use of Superpave protocols for modified asphalts, and to define the current and future needs for modified asphalts. It also included a comprehensive literature review to evaluate the research done to evaluate modified binders using the Superpave protocols. The first phase resulted in recommendation for classifying asphalt binders into simple and complex binders. Based on this classification, it is recommended that the Superpave binder specification be used for asphalts that exhibit simple rheological behavior. The first phase has also resulted in the recommendation for the addition of new or revised testing procedures to characterize specific properties that are important for asphalts modified with additives. These procedures include modification of the RTFOT procedure, the Particulate Additive Test (PAT) and the Laboratory Asphalt Stability Test (LAST). The section covers the details of the deficiencies in the existing SHRP PG grading protocols and the recommended modifications.

The Superpave Binder Grading Assumptions

The Superpave binder specification contains criteria based on assumptions that were made to simplify the testing required and evaluate characteristics that are most critical to pavement performance. These assumptions although were validated for neat asphalts, may not be valid for asphalts modified with different additives.

Based on detailed review of the SHRP Project A-002A report (Anderson et al. 1994, Anderson & Bahia 1995, Anderson and Kennedy 1993) and other recent published literature (Isacsson & Lu 1995, Claudy et al. 1997, Isacsson & Lu 1996, Brule & Maze 1995, Bahia 1995, Collins and Bouldin 1991, Lauzier & Masson 1993), the following assumptions are found to be the most important that are related to the behavior of modified binders:

1. No strain/stress dependency of rheological response (wide linear range).
2. No shear rate dependency of viscosity (wide Newtonian range).
3. Testing at one loading rate is sufficient (similar loading rate dependency).
4. Binders are homogeneous and isotropic (no sample geometry or particulate additives effects).
5. Similar time-temperature equivalency for all binders (one shift is used).
6. Binders are not thixotropic (no effect of mechanical working).
7. Stability of asphalts is affected mainly by oxidation.

The essence of the above assumptions is that asphalt binders are simple systems that can be characterized using linear visco-elasticity and simple geometry within which stress and strain fields are simple to calculate. Except for number 7, the assumptions in the specification are related to rheological and thermo-rheological behavior. Assumption number 7 may not be very important for neat asphalt, but it is critical for asphalts modified with additives.

Strain/Stress Dependency

The assumption that asphalt binders perform in the linear visco-elastic region has resulted in great simplification of the testing requirements. This assumption appeared to be reasonable for two reasons: (1) Most asphalts tested during the SHRP program showed a relatively wide linear range. (2) Asphalt pavement structural design should be selected such that materials will be exposed to small stresses and strains. Therefore, the specification testing was designed to measure behavior within the linear range expecting that asphalts in the field will be mainly performing within this region.

Both concepts can be challenged, particularly for modified asphalts. The wide range of linear behavior is not observed for all modified asphalts. It is expected that, as more additives are used at higher concentrations, modified asphalts will show a narrower linear range, which violate the specification assumption. In addition, even unmodified asphalts of extreme grades are showing narrower linear ranges, particularly at intermediate temperatures at which fatigue cracking is dominant. Figure 12 shows the relation between temperature and the linear visco-elastic limit on the strain scale based on a drop in G* to 90 percent of the initial value (LVE90). The data shown is for a typical PG 58-28 asphalt before and after mixing with rigid fillers of fine, medium, and coarse mono-size at 33 percent by volume. The figure depicts how such additives can significantly reduce the linear range limits. It also shows that the linear range is highly sensitive to temperature and can be as small as few percent strain at intermediate temperatures.

With regard to the second reason, it is important to consider the conditions of asphalt binders within a typical mixture with regard to the film thickness because it is one of the important issues related to the actual strain in the asphalt binder. It should be recognized that there is a wide distribution of film thicknesses in a typical mixture that depends on the asphalt content and the aggregate gradation. It is, therefore, expected that some of the asphalt binder will be performing well in the non-linear range. If an equal distribution of film thicknesses is assumed, the average film thickness for an asphalt content of 4.5 percent to 5 percent by weight will result in a film of 5 to 20 microns. It will take a 0.5 to 1 micron deformation in shear to cause 10 percent strain. If binders vary significantly in their non-linear behavior then the current specification falls short of providing performance-based evaluation of the binders. This simple analysis points out the need for considering non-linear behavior of asphalts.

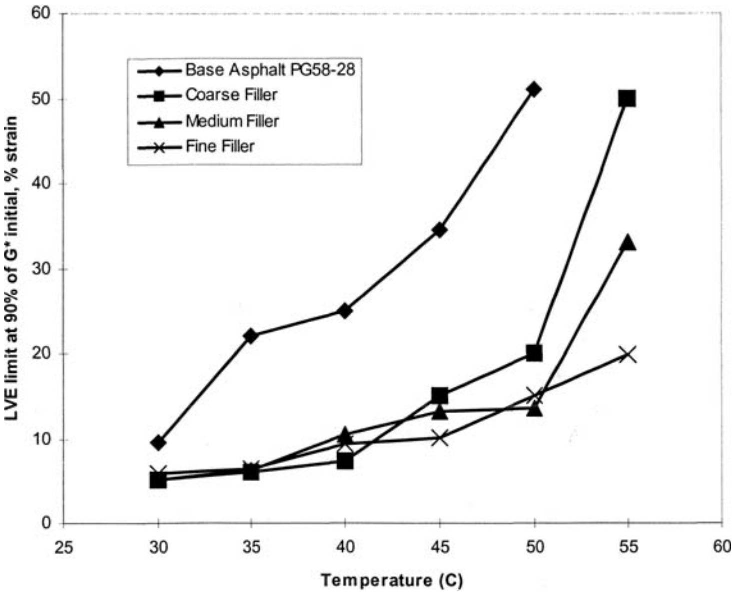


Figure 12. Relationship between the Linear Viscoelastic Limit and Temperature

Newtonian Behavior at High Temperatures

In the Superpave binder specification, the rotational viscosity is measured at 135C at a recommended rate of 20 rpm. This shear rate was selected because (a) most unmodified asphalts show Newtonian behavior (viscosity independent of shear rate) at this rate, and (b) it simulates the shear rates during the pumping and handling operations in refineries and asphalt plants (Anderson et al. 1994). The behavior of some modified asphalts is expected to be shear dependent (non-Newtonian) within the range of 20 rpm. In addition, some modifiers, such as crumb rubber are known to result in elastic behavior at mixing and compaction temperature. In such cases a decrease in density is observed after compaction due to residual elastic behavior (Bahia et al. 1997). Although viscosity at construction and production temperatures is not a performance-related parameter, it can have significant effect on the quality of the compacted mixture and its performance under traffic. It is, therefore, necessary to measure the non-Newtonian behavior and the elastic behavior for binders to define optimum conditions for production of asphalt mixtures.

Non- Thixotropic Behavior

The effect of repeated loading on $|G^*|$ and phase angle (δ) is a response not considered in the Superpave specifications. It can be significant in the case of some modified asphalt systems because certain additives can produce a thixotropic network structure that can be altered or destroyed by repeated shearing (mechanical working). In some cases, the affected structure can regain its original condition when the material is left to rest. Certain asphalts modified with Tall oil and gel-like compounds can show significant changes due to mechanical working. As shown in Figure 13, at strain levels that are common for typical pavement structures, some modified asphalts can show significant reductions in G^* due to mechanical working. In the field of asphalt emulsions, technology is available to produce such thixotropic materials, such as high-float emulsions. It is reasonable to assume that this same technology will find its way in the production of paving grade asphalts.

The Loading Rate Dependency (Under Cyclic Loading)

The Superpave binder testing is conducted at selected loading rates assumed to be typical of open highway traffic and typical cooling cycles. It is well recognized that traffic does not move at one speed. It is also known that thermal cooling cycles can vary significantly in their cooling and warming rates. The dependence of rheological response on loading rates is material specific, both for modified and unmodified asphalts.

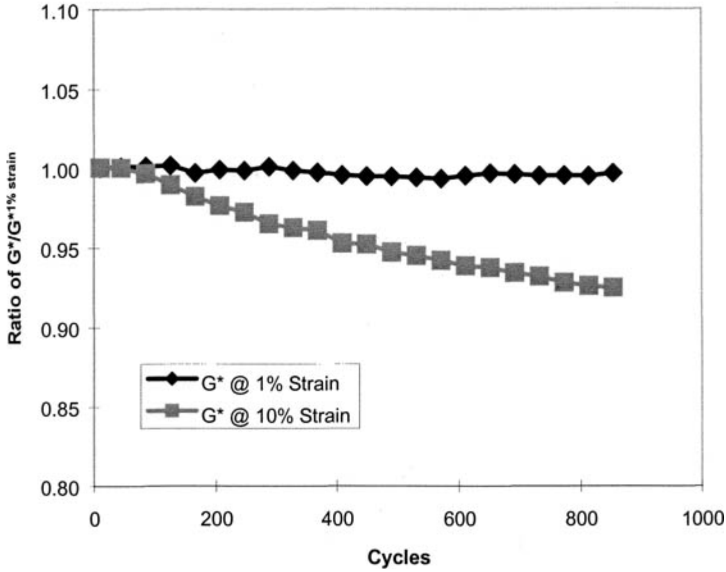


Figure 13. Thixotropic Behavior of Modified Binder at 10 percent Strain

Both the rate of 10 rad/sec used for cyclic testing and the 60 seconds used for creep testing are based on simplifications of asphalt behavior. Figure 14 shows that the relationship between effect of changing the loading rate and effect of changing temperature on G* for typical asphalts with rheological behavior within the range of paving grade asphalts. The R^2 values are 0.17 for the shift from 5 to 10 rad/s and 0.30 for the shift from 1 to 10 rad/s. This clearly indicates that there no strong correlation between the effect of frequency shift and the temperature shift. In other words, the practice of changing the grade to compensate for traffic speed cannot be justified. The polymer-modified asphalts appear to be significantly less affected by a change in frequency than the unmodified asphalts. Also, the effect on G* of changing temperature by 6°C is highly variable, even for the unmodified asphalts. For the asphalts examined, which include 32 sources tested during the SHRP program, the change ranged from 1.7 to 2.6 fold. This result is expected, since temperature susceptibility is highly asphalt source specific. It appears that the simplification used in the specification is not acceptable.

The Time - Temperature Equivalency

The glass transition behavior and the uniqueness of the time-temperature equivalency of asphalts are not fully considered in the specifications. The current specification requires testing at the minimum pavement temperature plus 10 °C. This simplification was based on testing of unmodified asphalts (Anderson et al.1993). There are several recent studies that question the thermo-rheological simplicity of asphalt binders, particularly modified binders (Bahia & Hanson 1996). It is, therefore, important to have direct estimate of the time-temperature equivalency factors for the binders based on actual testing rather than simplified assumptions.

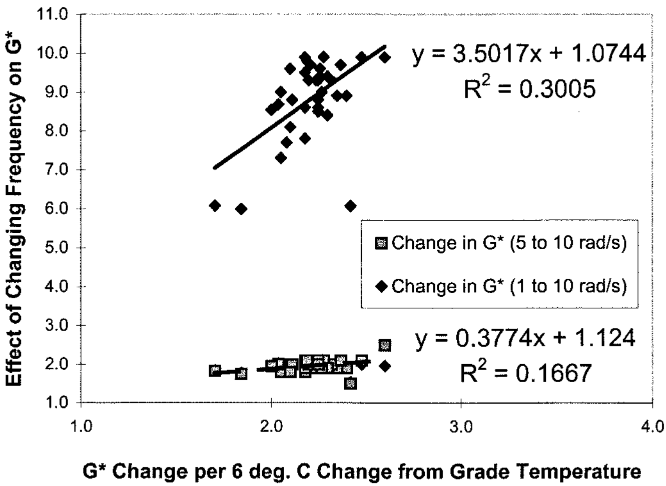


Figure 14. Relationship Between 6C Temperature Change and Loading Rate

Homogeneous and Isotropic Properties

The Superpave binder testing is done using selected geometries for different temperatures and response types. These geometries were selected to give stress fields that can be easily estimated and thus used in calculating the material response. The concept is based on the assumption that binders are homogeneous materials that exhibit isotropic behavior. Therefore testing in one mode of loading, using a single geometry, can give a comprehensive evaluation of material under different geometric and loading conditions. For some modified asphalts, because of the nature of additive used, the response can be dependent on sample geometry because of anisotropy of additive itself, such as fibrous materials with high aspect ratio, and the size of particulate relative to testing geometry. It is known that as the ratio of particle size to sample size increases, and/or as the volume concentration of the additive increases, the geometry of the sample can interfere with the measurement. The size and the volume concentration of particulate additives are not fully controlled in the current specification. The only existing limitation is particulates should not exceed 250 microns, which was selected arbitrarily (AASHTO TP5). Size and volume concentrations are both required in limiting the amount of additive that can result in interference with measurements. The concept is depicted in Figure 15, which shows that, at each volume concentration, there is a size range which will result in a non-linear behavior (strain dependency in this case) of a neat asphalt. The limits will also depend on testing geometry. A thinner film in testing will be more sensitive to size. It is, however, expected that volume concentration of additives in asphalt do not reach levels where particle to particle interaction or particle to testing fixture interaction will occur. It is, therefore, necessary that the presence of solid additives be detected and the nature of the additive be checked. A new test, called the Particulate Additive Test (PAT) is described in this article to address this need.

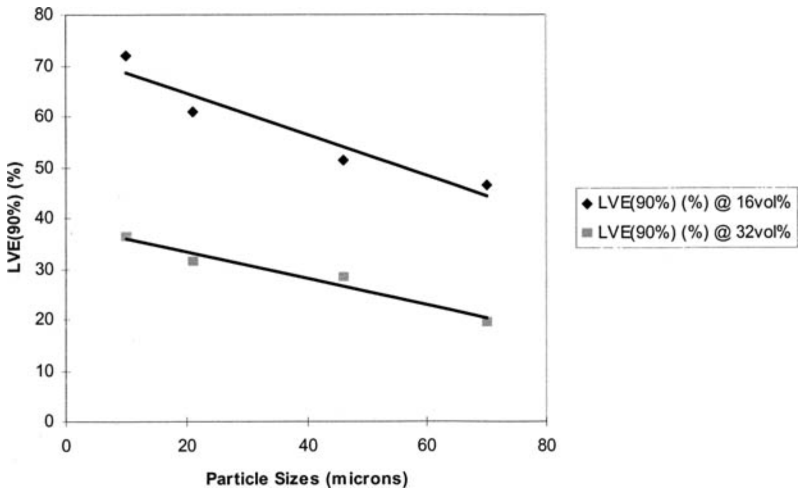


Figure 15. Effect of Size and Volume Concentration on the Linear Viscoelastic Limit

The Stability of Modified Asphalts

The current Superpave binder specification considers oxidation as the main mechanism by which asphalt binder properties can change in service. Modified asphalt binders however can undergo changes due to factors other than oxidation. It is recognized that modified binders are multiphase systems in which the modifiers are dispersed into the asphalt cement phase (Isacsson & Lu 1995, Brule et al. 1986, Berker et al. 1991, Brion et al. 1988, Brule et al. 1986, Chavenot et al. 1992). This dispersion is generally accompanied by a degree of incompatibility that is affected by various physical, thermal, and chemical factors. Excessive incompatibilities can negatively affect the performance of these binders as gross separation occurs during storage and handling of the binder, production and transportation of asphalt mixtures, and during the construction of pavement layers. There are four separation mechanisms (physical, thermal, chemical, and oxidation) that need to be considered (Isacsson & Lu 1997, Anderton & Lewandowski 1993, Bahia et al.1998). In the current Superpave system, there are protocols for characterizing the oxidative stability (PAV and RTFO). There are also proposed protocols for measuring physical separation (Cigar tube test). There are, however, no provisions to separate these effects and to take into account the different physical, thermal, and chemical treatments expected in the field.

NEW CLASSIFICATION OF ASPHALT BINDERS

Two main conclusions can be drawn from the review of the Superpave binder test protocols and specification and the existing knowledge of modified binders:

1. The existing protocols cannot be used to fully characterize all asphalt binders modified with different additives. The main reason is that they are based on simplifying assumptions that cannot be reliably extended to modified binders.

2. Some additives can result in binders that are too complex to be evaluated by any binder-only protocols. Such additives will result in anisotropy or interference with testing geometry such that only actual replication of films that will exist in mixtures will allow reliable estimation of their role in pavement performance.

To apply the current Superpave binder protocols for modified binders these two conditions have to be satisfied. In other words, the modified binders have to be "simple" rheological systems. Based on this concept, asphalt binders should be classified into simple binders and complex binders, as follows.

Simple Binders: Asphalt binders with simple behavior that do not violate the assumptions, which the PG-grading system is based upon; these assumptions include

1. Wide linear range (independence of strain),
2. Non-thixotropy (independence of mechanical working effects)
3. Isotropy and independence of sample geometry (no additives that result in geometric effects)

Complex Binders: Asphalt binders that cannot be classified as simple binders because their behavior violates one or more of the PG-grading system assumptions.

This new classification is based on the hypothesis that the role of simple binders in mixture and pavement performance can be estimated using the existing (or slightly revised) Superpave binder protocols, regardless of their constituents or the method of production. The role of complex binders in mixture and pavement performance, on the other hand, cannot be estimated using binder testing. Mixture testing will have to be used. An asphalt binder can be classified as a complex binder because of the physical characteristics of the modifier or because of the nature of the effect of the modifier. Binders modified with particulate matter can be complex because of their dependency on sample geometry. Other binders can be complex because they are thixotropic or strain dependent.

REQUIRED REVISIONS TO CHARACTERIZE SIMPLE BINDERS

Although simple binders satisfy the basic assumptions on which the specification is based, there are few assumptions that are not valid and, therefore, testing protocols need to be more comprehensive.

- Tests to qualify the binder system as a simple binder are needed. These tests will include strain dependency, thixotropy, and nature and volume concentration of additives. If a binder is simple, testing can continue to find its PG-grade based on more rigorous testing.
- Time-Temperature Equivalency: The simplification of similar time-temperature equivalency factors is not appropriate and should be replaced by direct measurements. To grade an asphalt binder, testing at two temperatures is required. Instead of conducting the single point measurement recommended currently, a frequency sweep should be conducted at each of the two temperatures. Using simple statistical routines, computers used in conducting the testing should allow accurate calculation of the time-temperature shift factor and the response required at the selected pavement design temperature and the selected traffic speed.
- Low-Temperature Properties: The criteria of S(60) and m(60) measured at 10C shift should be changed such that the time-temperature equivalency calculated for each asphalt are used to estimate the S(7200) at minimum pavement design temperature. This will reduce error caused by equal shift factors assumed for all asphalt binders (Anderson & Kennedy 1993).
- Direct tension test should be conducted at realistic rates that represent the actual cooling rate cycles measured in the field for different regions. The SHRP weather database can be used to select typical cooling cycles for the region asphalt is intended for. As different additives are used, the similarity of effect of loading rate assumed in the specification may not be a valid assumption.
- Elastic behavior at mixing and compaction temperatures needs to be measured. Viscosity by itself is not sufficient for evaluating workability at high temperatures. The current test should be modified to put equal emphasis on workability of the binder and address possible elastic behavior.
- New tests for measuring thermal stability of asphalts that include conditions similar to field conditions are required. These conditions

should include temperature, mechanical agitation, and time of conditioning.

- Modification of the current RTFO procedure to allow aging asphalts with high viscosity is needed.
- Modification of the PAV procedure to isolate possible reaction/degradation effects from pure oxidation is necessary. In addition, long-term stability should be evaluated separately from oxidative effects

The above list of revisions require more elaborate use of the currently collected data, additional testing with the existing equipment, and new test methods for stability of asphalt binders.

NEW TESTS AND NEW EQUIPMENT NEEDED

To qualify asphalt as a simple binder, two initial tests will be required: (a) strain sweep, and (b) time sweep. Each should be done at high and intermediate temperatures. Both these tests are easily performed using the current version of the Dynamic Shear Rheometer (DSR). A change by more than 10 percent for either test will indicate a complex behavior.

If the binder passes both tests, an evaluation of the nature of additive is required. A new test called the Particulate Additive Test (PAT) is proposed to separate the additive and evaluate its nature. If the binder does not contain more than 2 percent by volume of a particulate additive, and passes the strain and time sweep tests, it can be graded as a simple binder and can be graded according to the PG system.

The current Superpave-testing protocols do not cover stability under non-oxidative conditions. The stability under non-oxidative conditions requires a new test method. The Laboratory Asphalt Stability Test (LAST) is proposed to measure potential for phase separation and thermal degradation of asphalts. The LAST can be used to simulate the conditions of hot storage with and without agitation under minimal exposure to oxidation. It is proposed that potential rate factors for degradation (K_d) and for separation (K_s) be calculated from the results of this test used to evaluate the stability of asphalts. In addition to these new tests, the RTFO test needs modification to make it more effective in handling the highly viscous polymer modified asphalts. The following sections give a brief overview of the PAT, LAST, and the modification of the RTFO test. A detailed description of the background and the initial data collected is available in another publication by the author (Claudy et al. 1997). The following sections give brief overview of these methods.

The Particulate Additive Test (PAT)

One of the alternatives to using microscopy to determine the nature of the asphalt additives is separation of the additive from asphalt. Using separation, the general type of the additive and its characteristics can be determined. The concept behind the PAT is to pass a diluted solution of the asphalt binder through a sieve to separate particulate additives from the base asphalt. Particulate additives can result in potential separation, or in interference with test sample geometry. In the current standard test methods for Superpave binder specification, the particulate size is limited to 250 microns, selected arbitrarily as ¼ of the minimum testing sample dimension. The PAT test separates material larger than 75 microns using a #200 (0.075-mm) mesh. This size has been selected because it is commonly agreed that particulate larger than this size is considered part of the mineral aggregates in the asphalt mixture.

The schematic of the test set up is shown in Figure 16. The test starts by heating the modified asphalt binder to 135C, until it becomes soft enough to pour. Approximately 10 ml of sample is transferred into a 125 ml Erlenmeyer flask. While hot, the sample is diluted using 100 ml of a solvent in small portions with continuous agitation until all lumps disappear and no undissolved sample adheres to the container. A bronze 50-mm diameter # 200 sieve disk is placed in the vacuum filtering apparatus, and the vacuum filtration is started. The container is washed with small amounts of solvent to facilitate the filtering. The filtration is continued until the filtrate is substantially colorless, then suction to remove the remaining distillate is applied. The material retained on the filtering sieve is transferred to a centrifuge tube and the volume is measured partially filled with the solvent. The centrifuge tube is placed in a centrifuge for 30 min at approximately 3000 rpm. At the end of the centrifugation time, the volume of material at the bottom of the tube is measured to the nearest 0.01 ml. Using the final volume of particulate and the initial volume of sample, the percentage of compacted volume of the particulate retained on #200 sieve by volume of the asphalt can be calculated. The conditions used for the protocol have been selected based on several experiments.

Selection of Test Conditions

A sample size of 10 ml was selected for convenience taking into account an approximate ratio of 10:1, solvent to asphalt, for complete dissolution of asphalt The tentative limit of two percent particulate additive by volume will be used in this test. Thus the expected volume retained is 0.2 ml.

A filtration temperature of 90°C was selected. A pressure of 200mm Hg was selected for the filtering step because it is an obtainable pressure. This is a very obtainable pressure, low enough for the purpose of the test, and for the membrane filter pore size used. The time required to separate solvent and particulate additives in the centrifuge has been established at 20 minutes to ensure complete packing of particulate additive in the bottom of the centrifuge tubes.

Selection of Solvents and Trials with Different solvents

Four different solvents were tested: Petroleum Distillate, n-Heptane, n-Octane and Toluene. Different results from this test procedure can be achieved depending on the solvent used. Testing using the different solvents have resulted in selecting n-Octane and Toluene as the selected solvents. N-Octane was considered as an alternative because of its zero aromaticity, its suitability for dissolving asphalt, and its relatively high boiling point. The tests conducted, however, have shown that it can separate polymers that are not particulate in nature, which can give misleading results. For several of the tested modified asphalts, some sort of a polymer-asphalt melt is retained on the filter that cannot be washed through the sieve with the solvent. There is also a concern that the n-octane will precipitate some of the asphaltenes with the additive. It is found that although the n-octane is a useful solvent to separate the additive from the asphalt, it is necessary to further evaluate the material retained on the sieve to determine whether it's particulate or not.

Toluene was used because its solubility parameter is considered very similar to asphalts' solubility. The tests conducted showed that it is a very powerful solvent for asphalt, and the filtration step is completed very rapidly. The concern with this solvent is that it can dissolve some of the polymers, but not all depending upon the solubility and crystallinity of the polymer. This may result in dissolving some of the polymers that are particulate in nature and, therefore, give misleading results.

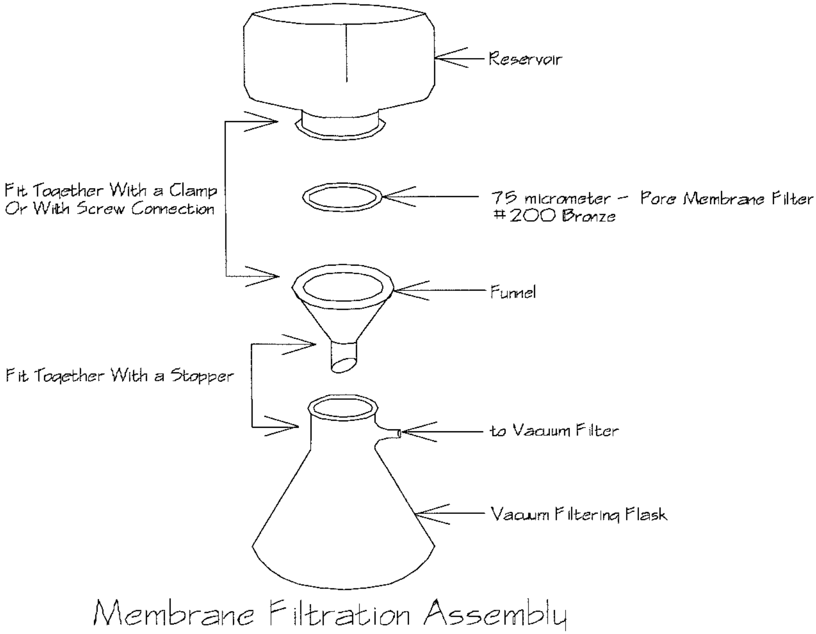


Figure 16. Schematic of the PAT test

Typical Results Collected

Table 5 gives results collected using the PAT for a number of PG 82 and PG 58 grades modified with different additives. The results are shown for one or more of the solvents used. The results support the statements made about the n-octane and the toluene. It appears that n-octane can be used to detect the existence of any additive in the asphalts. The material retained on the sieve after treatment with n-octane cane is further treated with toluene to determine its solubility.

Table 5. Results of Testing Selected Modified Asphalts Using the PAT Test

Filler	Petroleum Distillate	n-Heptane	n-Octane	Toluene
	% retained by Volume of Total Binder			
Hydrated Lime	3.5	3.0	13 (PG70 + 16%HL)	7.0 (PG70 + 16%HL)
SBS	2.1	16.0	13.0	0
EVA	6.0	13.0	2.5	0
Crumb Rubber	15	15.0	14.0	14.0
Silica Quartz	1.5	1.0	0.8	0.8
Gilsonite	1.0	2.0	1.5	1.0
SBR (LMW) (PG58)	-	-	0	0
SB Diblock (PG70)	-	-	17.0	0
PE Stabilized (PG52)	-	-	0	0
PE Unst.(PG58)	-	-	0	0
PE Unst. (PG76)	-	-	0	0
PE Unst. (PG82)	-	-	17.0	2.5
PE Stabilized (PG82)	-	-	15.0	0
Oxidized (PG58)	-	-	0	0
Oxidized (PG64))	-	-	2.5	0
SBS Linear (PG64)	-	-	25.0	0
SBR HMW (PG58)	-	-	0	0
PG58 Neat	0	0	0	0
PG70 Neat	0	0	0	0

The Modified RTFOT (RTFOTM)

One of the main problems with the RTFO procedure for modified binders is that these asphalts, because of their high viscosity, will not roll inside the glass bottles during the test. Also, some of these asphalts are rolling out of the bottles. To solve this problem, two modifications were considered: (a) Using a number of steel spheres to create shearing forces to force the spreading of thin films, and (b) using a steel rod to induce the same action. Initial evaluations indicated that the steel rod is more practical, and it easier to use and clean after aging. Several length and diameter combinations were tried with asphalts that varied in their viscosity at the testing temperature of 163 C between 0.5 and 3.0 pa-s. The optimum conditions were achieved by using the 127 mm long by 6.35 mm in diameter steel rods (Isacsson & Lu 1996).

The criteria for accepting the use of the steel rods in the RTFO included two items. First, the rods should not have major effects on aging of neat asphalts, otherwise the relation between the test results and the field aging, which is accepted for the standard test without the rods, can be questioned. And second, the aging with the rods should not interfere with the stiffening effect of inert additives.

To evaluate the first criterion, neat asphalts were aged with and without rods and the changes due to the aging were evaluated using the dynamic shear rheometer.

To evaluate the second criterion, Ottawa sand (considered to be inert filler) was mixed with an asphalt and aged in the RTFO with rods. The same amount of the sand was mixed after aging the neat asphalt in the RTFO with rods. The stiffening effect of the sand was evaluated with premixing and with post-mixing to determine the interference of the RTFO aging.

Samples of the results from both experiments are listed in Table 6. It can be seen that the effect on the aging of the neat asphalt is minimal. The effect on the filled asphalts, however, is significant and consistent. The stiffening of the asphalts by the filler (OS) appears to be the same with premixing and post mixing when the rods are used, which supports the hypothesis that using the rods is not interfering with the stiffening by filler.

Table 6. Evaluating the Effect of Metal Rods in the RTFO on Neat and Filled Asphalts

	$G^*/\sin\delta$ RTFOT + Rods	$G^*/\sin\delta$ RTFOT w/o Rods	Diff. %
PG58-28 (Neat)	2784	2872	-3.06
PG64-22 (Neat)	3711	3741	-0.80
PG70-22 (Neat)	3326	3200	+3.94
Premixing			
PG 70 + 15% OS	4133	3270	+26
	4173 (2nd run)	3363 (2nd run)	+24
PG 64+ 20 % SQ	4508	3578	+25.9
	4617 (2nd run)	3590 (2nd run)	+28.6
PG58+ 15%CR	3494	3081	+13.5
	3555 (2nd run)	2961(2nd run)	+20.1
Post Mixing			
PG 70+15% OS	4107	3823	+7.4
	4142 (2nd run)	3856 (2nd run)	+7.4

Several modified asphalts were tested in the modified RTFO. As shown in Table 7 the results show the significant effect of using the rods which confirms the need to generate the shearing forces to spread the films of these asphalts and cause the required aging. The apparent reduction in aging of modified asphalts compared to the neat asphalts have been reported before and it appears from the results of this study that a main contributor to this behavior is the reduced exposure of the highly viscous asphalts to oxidation in the standard RTFO procedure.

Table 7. Results of RTFO Aging of Typical Modified Asphalts with and without Rods

	G* /Sinδ RTFOT w/ Rods	G* /Sinδ RTFOT w/o Rods	% Diff.
PG58 + 15 % CR	3494 3555 (2nd run)	3081 2961	+13.5 +20.1
PG64 + 20 % SQ	4508 4617 (2nd run)	3578 3590	+25.9 +28.6
PG70 + 15 % OS	4133 4173 (2nd run)	3270 3363	+26.4 +24.0
PG70 + 9 % Gilsonite	2490	2324	+7.14
PG70 + 16 % Hydrated Lime	2675 2649 (2nd run)	2204 2217	+21.4 +19.5

The results indicate that the steel rods in the RTFOT can be used to alleviate the problem reported for aging modified binders in RTFO bottles. For all the tests conducted, it was observed that the rods inside of the bottles uniformly spread the asphalt binder. Creeping of material outside the bottles was observed for few of the highly modified asphalts. Titling the oven slightly (2 degrees) to keep these asphalts from rolling out of the bottle solved the problem.

The Laboratory Asphalt Stability Test (LAST)

The general requirements for a new test to evaluate the storage stability of modified asphalts were selected based on the review of research done in the past, and based on evaluation of typical storage tanks and conditions used to store such asphalts in the field. From previous research, it was clear that a new test should allow for evaluating the following factors:

1. Effect of extended storage at high temperatures in the range of 160 to 180° C.
2. Effect of mechanical agitation of the modified binders.
3. Performance related properties measured at multiple temperatures and frequencies.

From the review of field practices and of the design of storage tanks typically used in the field, it was concluded that, in almost all cases, these asphalts are stored with some sort of continuous agitation to maintain uniform temperature and to maintain homogeneity of the material. Thermal history should not include a freezing step, because it does not simulate field conditions. Two basic designs of tanks (horizontal and vertical) are used in the field. The vertical tanks are recommended to have double propellers to induce enough agitation and maintain uniformity of product. The general consensus is horizontal tanks are less efficient in mixing than the more recently designed vertical tanks.

Using the information gathered, it was decided to scale down a typical design of a vertical storage tank as manufactured by one of the main suppliers in the United States. The selected design is shown in Figure 17. The main design features include an internal heating element controlled by an electronic temperature-control feed back system to maintain isothermal conditions, and a constant speed, double propeller agitator centered in the middle of the cylindrical container. The dimensions are such that a sample of 400 ml is used and the sampling is done periodically using a pipette from the top and bottom of the container without stopping the conditioning.

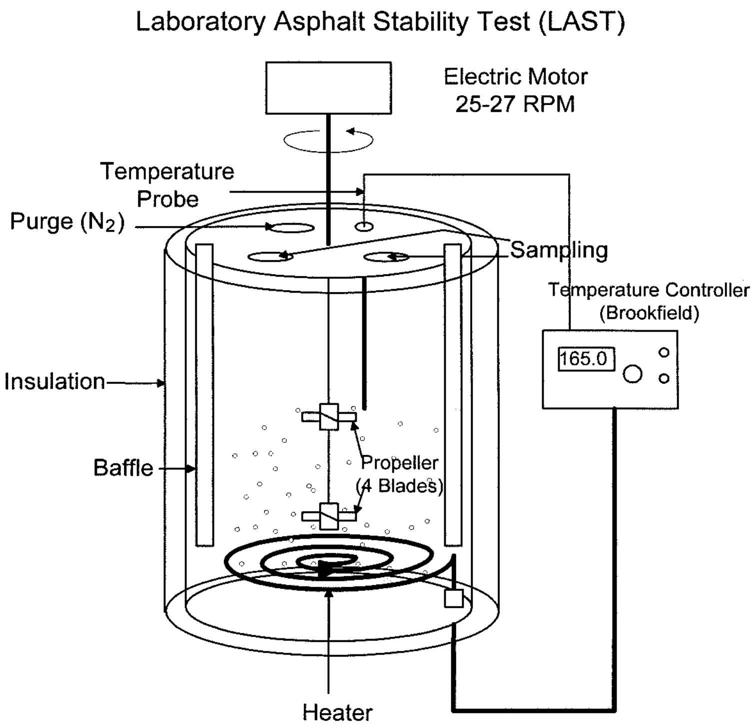


Figure 17. Schematic of the LAST Apparatus

Results Collected Using the LAST

To evaluate the LAST design, several experiments were performed to study the effects of the different controlled factors of the design. They are as follows:

- The source of heat: internal and external
- The time of conditioning: 1, 3, 6, 12, 24, and 48 hours
- Agitation: none, and slow agitation
- Testing frequency: 0.15 to 30 Hz

Testing was performed on one modified asphalt that is known to separate in the Cigar Tube Test (CTT). Figures 18 and 19 show DSR results at high performance temperature for the LAST of EVA modified PG76-22 with internal heating (with and without agitation). The figures also include the results from the CTT after 48 hour of conditioning. The figures show the ratio of G* and δ of a sample taken from the bottom 1/3rd to the G* and δ of the sample taken from the top 1/3rd of the container. As shown the results for the CTT are highly dependent on the frequency, and the LAST results are very similar with and without agitation. These results lead to the need to test the use of the external source of heating in the LAST. It appears that the internal heating by itself has generated significant thermal agitation that is keeping the additive dispersed uniformly in the asphalt. The CTT is a complete static test and the thermal agitation is not happening because the heating is done in a convection oven, which uniformly heats the sample from the outside.

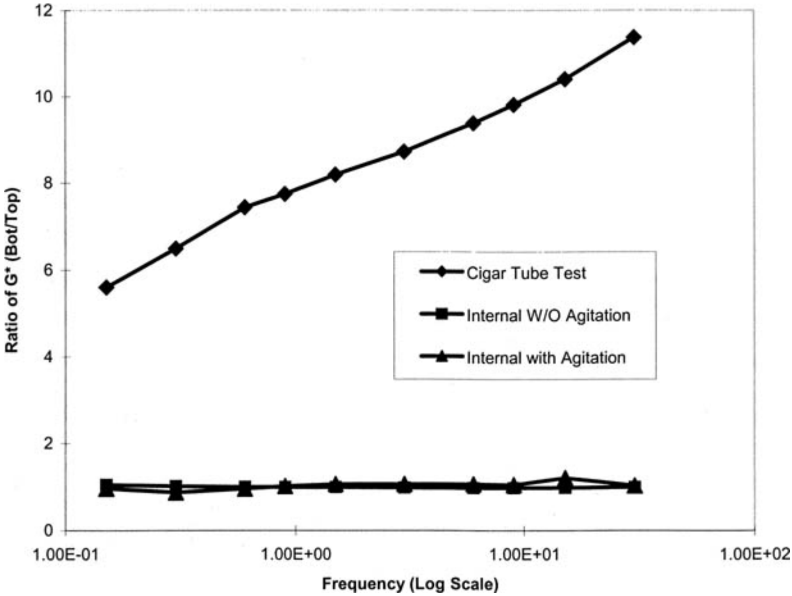


Figure 18. Ratio of G* for an EVA Modified PG76-22 Asphalt

To simulate the static conditions, the LAST container was placed in a convection oven and a similar test was done. Figure 20 shows the frequency sweep at high performance temperature for the LAST using external heat, without agitation after 24 hours. The results prove the hypothesis that with internal heat, the thermal agitation is the cause of the difference between the LAST and the Cigar Tube test. It appears that the LAST, with external heating and no agitation, is the most representative condition for evaluating the potential of separation under static conditions.

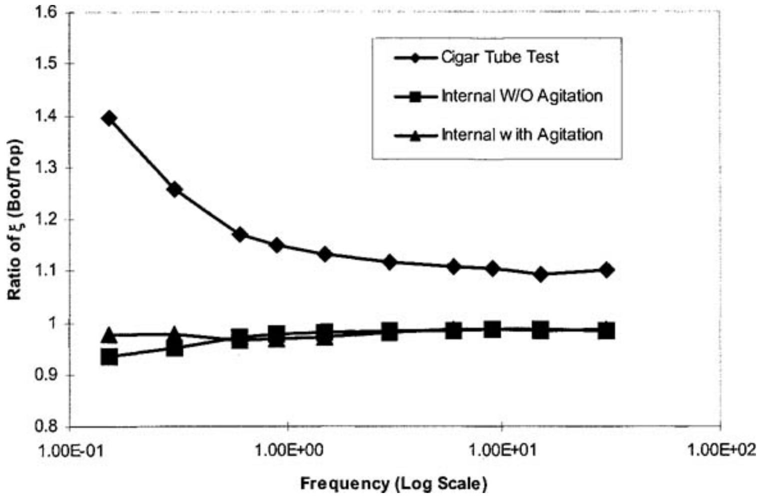


Figure 19. Ratio of δ for an EVA Modified PG76-22 Asphalt

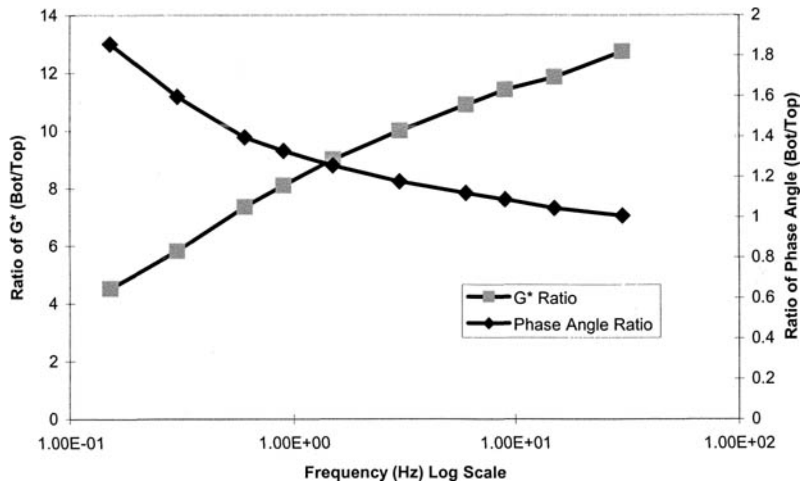


Figure 20. Ratio (Bot/Top) vs. Frequency of external heating w/o agitation (24hrs)

Figure 20 also shows that separation is resulting in different trends for G* compared to the phase angle. The G* ratio (bottom to top) increases with frequency continuously while the phase angle ratio (bottom to top) decreases continuously.

Effect of Conditioning Time

The issue of time in the storage stability test has not been evaluated adequately in previous studies. Most proposed test procedures use one time sampling after forty eight to seventy two hours based on the potential for separation. Presumably this is done with the expectation that material will be stored for similar periods of time in the field. The LAST protocol allows sampling periodically from the container and thus allows measuring the change with time. Such results allow measuring a rate factor for the separation potential and a rate factor for the thermal degradation. Figures 21 and 22 show typical results from the LAST separation and degradation/reaction, respectively. It is shown that, for this asphalt, the major part of the separation is occurring within the first 24 hours after which the material is stable. It is believed that such a test can be used to plan for the desired storage conditions in the field and to calculate the minimum grade properties that such a binder can degrade to in case of separation.

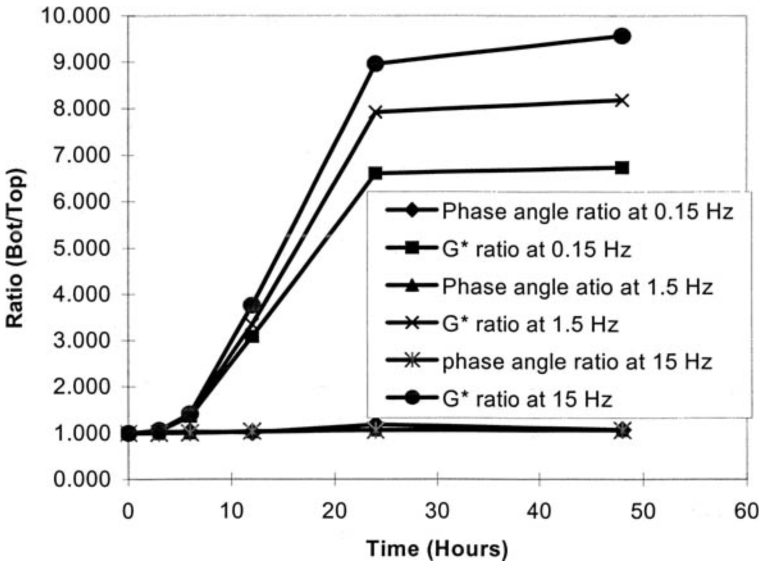


Figure 21. LAST Separation Data (EVA modified PG76-22)

The conditioning curve (Figure 22) shows that this binder is not degrading as evident by no reduction in the G* ratio. Although it is showing separation (Figure 21), the average change between top and bottom is similar to the initial value. The data also show that this binder exhibits hardening after twenty-four hours. This marginal increase in the G* ratio can be attributed to limited oxidation in the test container, or to the continued reaction between the additive and the asphalt with time, or to both.

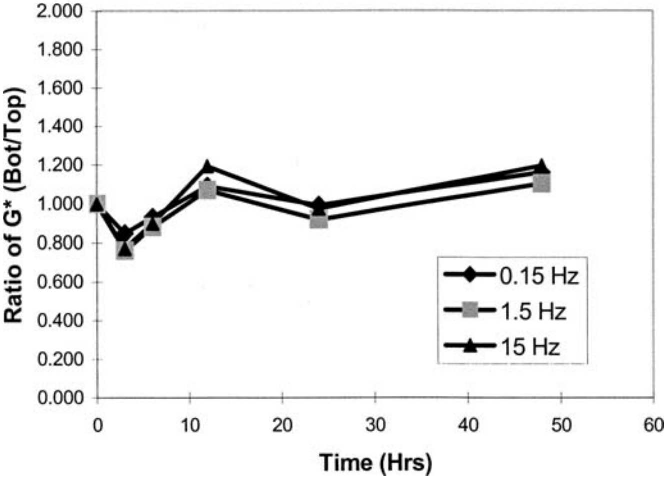


Figure 22. LAST Degradation Data (EVA modified PG76-22)

SUMMARY OF FINDINGS

Based on survey of fifty seven State Highway Agencies, and based on review and analysis of concepts introduced during the SHRP asphalt research program, the importance of modified binders and the changes needed to apply the Superpave binder specification to all asphalt binders (modified and unmodified), was evaluated. The following points summarize the findings of this evaluation and the new proposed tests.

1. The vast majority of the State Highway Agencies (SHA) plan on using the same or more modified binders to mitigate rutting, thermal cracking, fatigue, and other problems observed in pavements.
2. Asphalt modifiers vary significantly in chemical composition, physical nature, and effects they impart on asphalts. Out of 57 types of modifiers, the SHA identified the 18 types most commonly used. Among these types, the polymeric elastomeric modifiers are among the most widely used for paving applications.
3. The main concerns SHAs have regarding the Superpave binder specifications and the use of modified asphalts is related to storage stability and incompatibility during construction and production of HMA. The

applicability of short- and long- term aging procedures (RTFO and PAV) for modified binders, and the mixing and compaction temperatures for mixture design are also important concerns.

4. The Superpave binder specification is based on simplification assumptions that are not valid to all asphalt binders particularly modified asphalts.
5. To expand the applicability of the Superpave specification to more binders, binders need to be classified as simple and complex binders. The concept of performance based binder specification can be applied only to binders that exhibit simple rheological behavior.
6. Simple binders are homogeneous, isotropic, and non-thixotropic. They should exhibit wide linear visco-elastic range at the range of application temperatures.
7. Complex binders are highly strain dependent, thixotropic, non-homogeneous, non-isotropic, or include high volume of particulate additive. The role of complex binders in pavement performance cannot be estimated based only on binder testing because geometric and loading conditions of binders in mixtures cannot be simulated in a simple binder test.
8. Testing for the PG-grading system needs to be expanded to include protocols for estimating time-temperature shift factors, loading frequency dependency, potential for phase separation and degradation. These expansions are necessary to cover special characteristics of modified binders that exhibit simple rheological behavior. It is expected that the vast majority of modified asphalts is simple rheological systems and will qualify under a slightly revised Superpave binder grading system.
9. A new test called the Particulate Additive Test (PAT) is introduced to separate additives from asphalts and measure their effective volume. The test can also be used to measure the solubility of these additives in asphalts based on solubility of solvents selected.
10. A new test system, called the Laboratory Asphalt Stability Test (LAST), is introduced to measure the potential for separation and the potential for degradation of additives in asphalt during high temperature storage. The test allows better simulation of field conditions and allows evaluation of this phenomenon as a function of time, agitation, and source of heat. The test has been compared to the Cigar Tube Test and the advantages of the new test are discussed.
11. A modification of the RTFO procedure is introduced to solve the problem of highly viscous asphalts. A comparison of aging several modified asphalts with and without the modification of the RTFO is presented to show that the modification has a significant effect on the aging.

Section 4: The Future of Testing Asphalt Binders – Damage Resistance Characterization

To understand the causes of the distresses and study their effects on the pavement performance has been the goal of asphalt researchers attentions for several decades [AASHTO 1996, Kim et al. 1997, Kaufmann et al. 1972, Dijk 1975]. Among these distresses, rutting and fatigue, which are recognized as mainly associated with the increasing traffic volume, have led to an apparent reduction in the long-term performance of flexible pavement. To predict these damages efficiently, many research projects have been designed to evaluate the effects of different factors on the performance of the asphalt pavement [Dijk & Visser 1977, Hopman & Pronk 1990, Pronk 1995, SHRP-NRC 1994, Carpenter & Jansen 1996, Carpenter & Ghuzlan 2000]. Although it is recognized that these distresses are mainly caused by the deformation and/or damage within the asphalt binders, very few studies have used binder testing to evaluate damage behaviors of binders under simulated testing conditions [Findakly et al. 1974]. While it is recognized that mixture factors and pavement structure factors can have important effects, efforts to understand damage behaviors are very limited.

In the existing specification, the parameter $G^*\sin\delta$ is used to rate the binder contribution to fatigue damage resistance, while $G^*/\sin\delta$ is used to evaluate the rutting damage resistance [Bahia et al. 1994]. Both parameters were selected based on the dissipated energy concept as applied to linear visco-elastic range [Anderson & Bahia 1995]. However, there is a significant lack of information about the role of binder composition or rheological properties of binders in damage progression under cyclic loading. Furthermore, because the current parameters were derived from the results of mostly unmodified asphalts, the validity of these parameters for the modified asphalts is still in doubt.

To evaluate the parameters in the current specification and measure the damage behavior of the modified asphalts, mixture rutting and fatigue tests are conducted and the correlations between mixture test results and the binder properties based on the specification are discussed in the following section.

Evaluation of current Parameters for Binder Damage Behaviors

Materials

Two sources (gravel and crushed limestone) and two gradations (12.5 mm coarse and 12.5 mm fine) of aggregates were used in this study. One asphalt binder content was used throughout the testing. Nine modified asphalts were included the study. All of these asphalts were modified from the Boscan base asphalts. These asphalts were divided into three groups, as shown in Table 8.

Table 8. Binder Used in the Testing

Grade	PG 82 + X – X	PG X + 23 - X	PG X + X - 40
Modifier	PG 82-22 SBS Radial	PG 82-22 SBR	PG 58-40 SB
	PG 82-22 PE Stabilized	PG 76-22 Ethylene Terpoly	PG 58-40 SBS Linear
	PG 82-22 Steam Distilled	PG 76-22 Oxidized (SR)	PG 52-40 Oxidized (BB)

Five asphalts were modified with elastomers: ethylene-propylene diene terpolymer (Ethylene terpoly), styrene- butadiene-styrene (SBS) radial, styrene-butadiene diblock (SB), styrene- butadiene-styrene (SBS) linear and styrene-butadiene rubber (SBR). One asphalt was modified using stabilized polyethylene (PE) and three oxidized asphalts were also included in this study. The oxidized asphalts were produced by steam distilled, oxidized by back blending (BB) and oxidized by straight run (SR). All of the asphalts were aged using a rolling thin film oven (RTFO) prior to conducting the binder tests.

Mixture Experimental Protocol and Conditions

Rutting

To measure the effect of modified asphalts on the rutting behavior of asphalt mixtures the Repeated Shear Constant Height (RSCH) test was used in this study. The initial testing in this study indicated that at the selected testing temperature (58 C) the level of shear stress used (68 kPa) couldn't result in a significant permanent strain. The data collected for several mixtures were not useful because the accumulated strains were very low and not useful to directly compare mixtures. It was therefore decided to increase the stress to a higher level or increase the temperature of the test, or do both to ensure a higher level of permanent strain that simulates typical rutting in the field within reasonable number of cycles. After several trials, the testing conditions were changed to include a stress level of 204 kPa, which is three times the recommended stress level, and to change the testing temperature to the high PG grade temperature less 12° C.

Fatigue

The flexural beam fatigue test was used as the measurement of mixture fatigue behavior. The test is terminated when the stiffness decreases to 50 percent of the initial stiffness (N50). The output of the test is a curve of flexural stiffness as a function of load repetitions. Triplicate specimens are commonly tested using the Flexural Beam Fatigue test for each mixture. All of the tests were conducted under strain-controlled conditions according to the AASHTO protocols. The strain amplitude of 400 micro-strains was used in all the testing. The temperatures at which the fatigue testing was done were the intermediate grade temperatures of the binders.

Analysis of Results

Rutting

In order to evaluate the effect of binder modification on the results of the RSCH test results, certain mixture behavior indicators are needed. These indicators are commonly derived from the typical power law model recommended for representing rutting by the SHRP program.

The model, defined in equation (1), includes an initial strain factor ($\varepsilon_{p(1)}$) and a slope (S) factor.

$$\text{Log } \varepsilon_p = \log \varepsilon_{p(1)} + S \log N \qquad (1)$$

where "ε_p" is the total accumulated permanent strain and "N" is the number of cycles.

It is believed that the initial permanent strain can be affected by many mixture factors that are not related to the binders and this effect is carried to the total strain that is commonly used in comparing mixtures. Based on this, the value of the logarithmic slope (S) is selected as the more representative parameter that needs to be considered for studying the role of the binders in permanent deformation of the mixtures.

Figure 23 shows the correlation between the average mixture rate of accumulated strain (S) and the parameter G*/sinδ measured at 10 rad/s. As can be seen, there is hardly any reasonable correlation. Correlations with mixture rutting of individual aggregate blends showed similar lack of correlations. These results made it necessary to explain the reason of this lack of correlations and search for a better indicator of binders' contribution to mixtures' rutting performance.

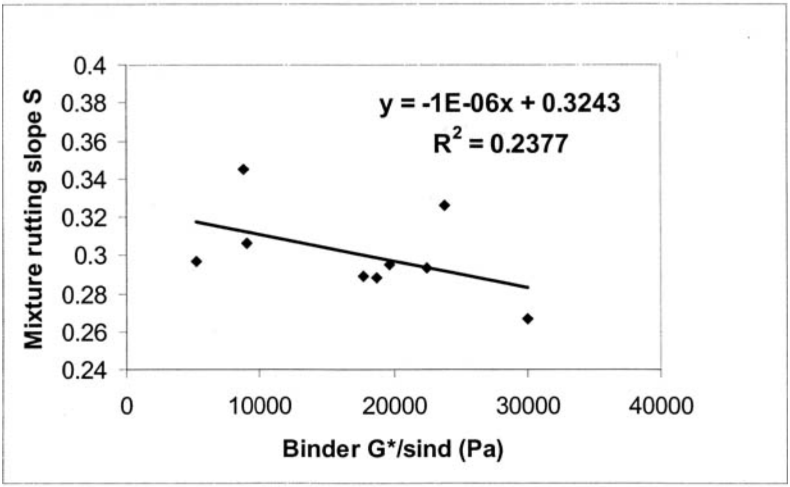


Figure 23: Correlation between average rate of accumulation of strain (S) and G*/sinδ

Fatigue

To look at the overall relations, the fatigue lives of mixtures (N50) were averaged across the aggregate types and correlated with the binder G*sinδ values as shown in Figure 24.

The same lack of correlation is observed, as the R-squared value is lower than 20 percent. Similar values of correlation were observed for the individual aggregates. These results show clearly that the G*sinδ of the binders is a poor indictor of the fatigue life of the mixtures (N50), as defined by the number of cycles at which the initial mixture modulus reduces by 50 percent. This finding started the process of looking for a better indicator of role of binder in fatigue damage and resulted in putting more emphasizes on the development of a better binder test for the evaluation of binder fatigue behavior, as will be discussed next.

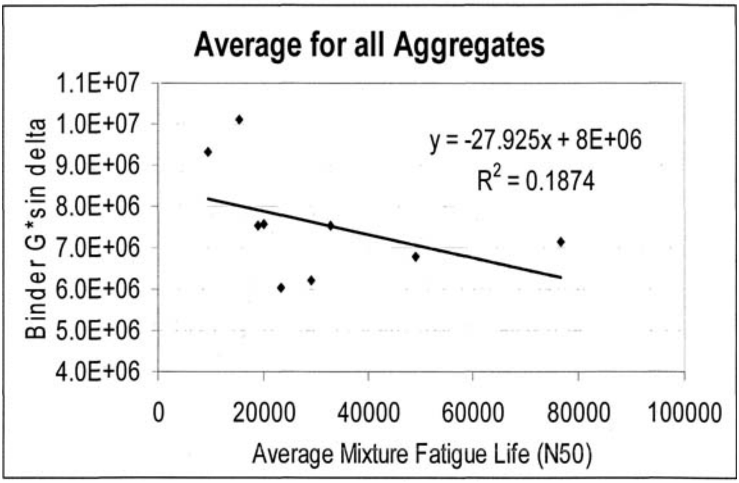


Figure 24: Correlation between G*sinδ and mixture fatigue life measured at 10Hz

Development of New Tests for Binder Damage Behaviors

Rutting Test

Several protocols were tried to select a test procedure and a rheological parameter that could be used as a more effective indicator of the role of binders in mixture rutting than the parameter G*/sin δ. The selection process was based on two main hypotheses:

1. The strains in the binder domains are significantly larger than the strain at which the binders are being tested in the DSR.
2. The cyclic loading with complete reversal in strain or stress is not appropriate for rating contribution of binders to rutting resistance caused by cyclic irreversible loading (also called non-steady-state cyclic deformation or more simply repeated creep).

The first hypothesis is based on the data collected in a previous paper by the authors [Bahia et al. 1999], which indicated that modified binders vary significantly in their strain dependency. It is also based on the finding that mixture rheological behavior was found highly sensitive to strain level [Kwok et al. 1998]. The second hypothesis was based on the concept of the RSCH and the review of literature related to the concept of energy dissipation.

To test these hypotheses different testing protocols were used to find a relationship with mixture rutting performance. The protocols included strain sweeps, stress sweeps, time sweeps at constant strain and time sweeps at constant stress. In addition, a repeated creep test was developed and conducted to measure permanent strain behavior of binders.

Figure 25 depicts an example of the strain sweep testing conducted for the three PG 82 binders used in the mixture testing at the same temperature (70°C). (1B02: SBS radial; 1B09A: PE Stabilized; 1B15 Steam distilled). The data indicates that at this temperature the binders are sensitive to strain only at very high strain levels that exceed the reasonable range of approximately 50%. The evaluation of the results of strain sweeps led to the conclusion that the strain sensitivity could not be used to explain the ranking observed in the mixture rutting tests.

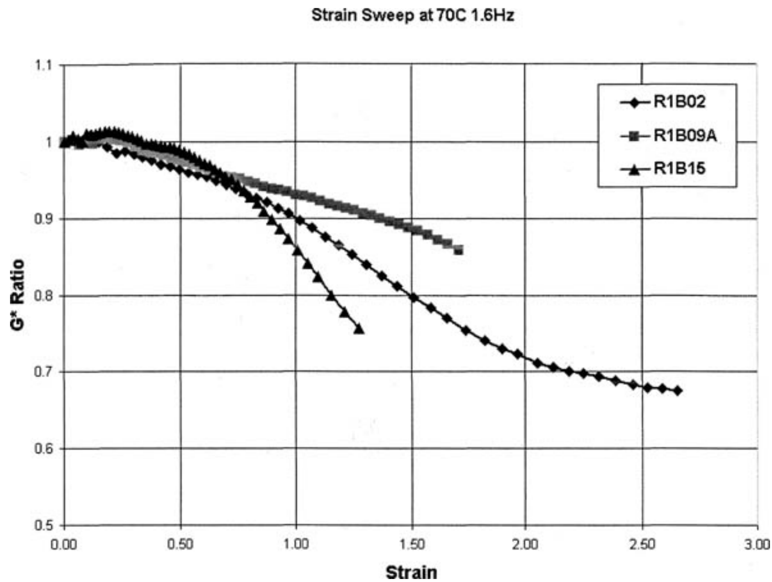


Figure 25: Strain sweeps for the three PG 82 binders at 70C

Figure 26 shows the time sweeps (repeated cyclic testing) under controlled strain conditions. It can be seen that the oxidized binder (1b15) shows a rapid reduction of the G* value which is an indicator of poor contribution to rutting resistance. It is however not clear which of the other binders offer a better contribution to rutting resistance. In addition, the ranking of the elastomeric binder (1b02) and the plastomeric binder (1b09a) changes depending on the mode of loading.

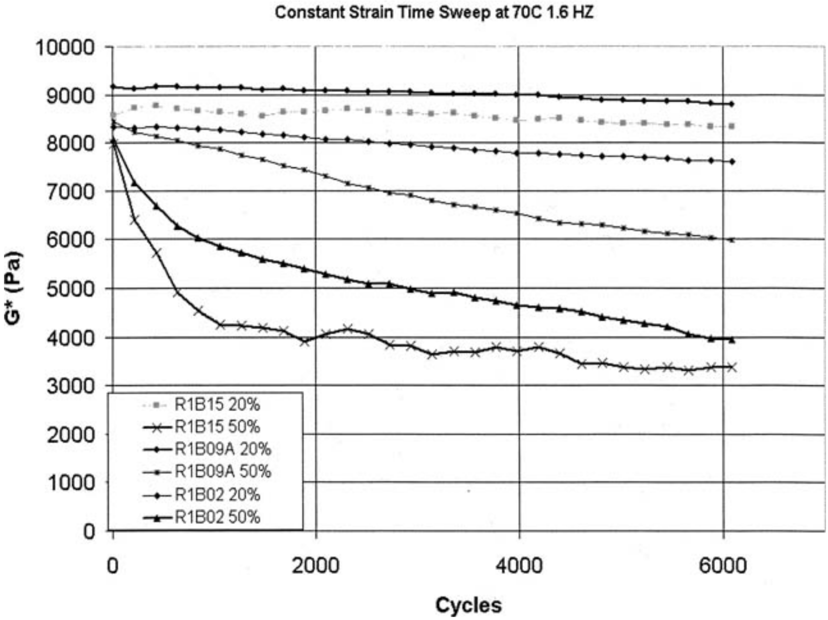


Figure 26: Time sweeps at constant strain for the three PG 82 binders at 70°C

The analysis of the results led to believe that these strain sweeps and time sweeps are not promising. None of the cyclic-reversible tests showed a clear potential to differentiate between binders and to relate strongly to mixture performance. As a result of this finding a detailed review of the dissipated energy concept and the derivation of the binder parameters was initiated. This review indicated that although the cyclic reversible loading could be used to estimate the total energy dissipated during a loading cycle for visco-elastic materials that combines permanent deformation and delayed elasticity, this type of test does not allow the separation between these two different types of dissipated energy. As shown in Figure 27, during the cyclic reversible loading only the total energy dissipated is possible to estimate. The rutting mechanism, as described by many research efforts and measured in the field does not include reversible loading required to bring the pavement material to zero deformation. As shown in Figure 28, rutting is in fact a repeated creep mechanism with sinusoidal loading pulses. In this case the pavement layer is not forced back to zero deflection but would recover some deformation due to elastic stored energy in the material of the layers. Under this type of loading, the energy is dissipated in damping (also called visco-elasticity or hyesteresis) and in permanent flow. The damping energy is mostly recoverable but requires time to be effectively utilized. The energy related to the permanent flow, however, is lost and thus called permanent. The permanent part of the dissipated energy is believed to be the main contributor to the rutting behavior of asphalt mixtures and pavements. The main problem with the reversible cyclic loading used today is the inability to distinguish between these two mechanisms that result in energy dissipation. Based on this analysis the focus of this study shifted towards developing a creep and recovery testing procedure to better simulate the contribution of binders to rutting resistance based on a more fundamental understanding of rutting behavior.

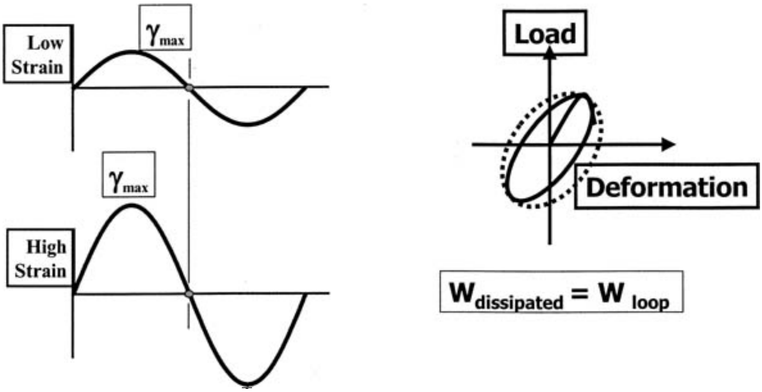


Figure 27: Current Concept Used in Deriving the G*/sind for the Binder Specification (MP1)

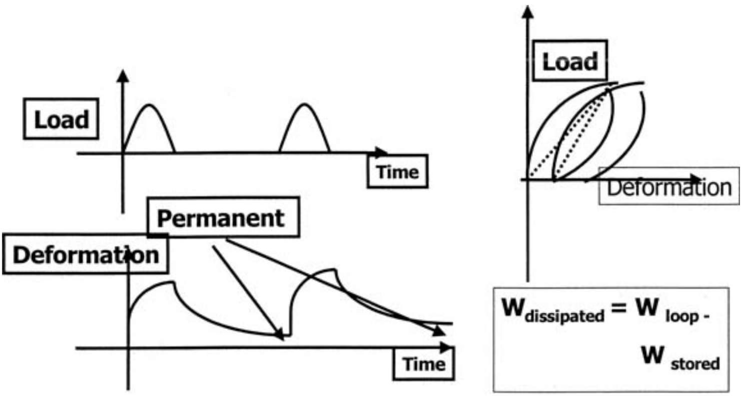


Figure 28: An Improved Application of the Dissipated Energy Concept to Derive a Fundamental Rutting Parameter for Binders

Realizing that the cyclic reversible tests that have been tried in the project do not offer a good indicator, and recognizing that there is a fundamental problem with estimating energy dissipation during repeated creep from the cyclic reversible loading, it was decided that a new approach that utilizes repeated creep testing is required. Evaluation of the DSR software and capabilities indicated that a repeated creep test could be conducted on binders using same geometry and temperature range. The testing was therefore started to develop a repeated creep test using the DSR.

Figure 29 depicts the results of the repeated creep testing of 3 binders of the PG 82 grade at selected loading conditions of 1 second loading and 9 seconds unloading. The results show clear distinction between the accumulated permanent strains of the binders which could not be detected using the G*/sin δ. It can be observed that the elastomeric binder (SBS) is offering significantly higher recovery during the creep testing and thus results in less accumulated deformation compared to the other binders.

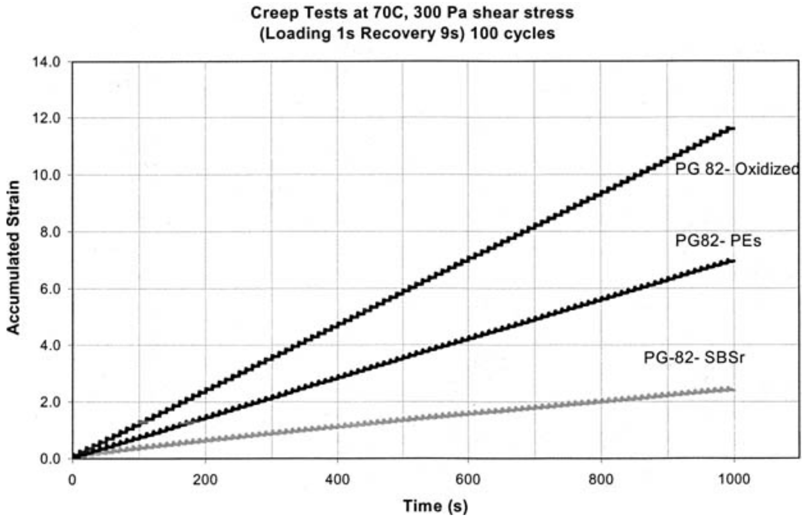


Figure 29: Results of the accumulated strain under repeated creep testing for 3 PG 82 binders at selected conditions of 1-second loading and 10 seconds recovery

The results of the creep and recovery and the G* and phase angle are listed in table 9. As shown, the accumulated strain after 100 cycles at 70 C is only 20 percent of the accumulated strain of the oxidized binder of the same grade. The G*/sinδ of the oxidized binder at the testing temperature is 15900 Pa which is higher than the elastomeric binder that has a G*/sinδ value of 13000 Pa. This inversion of ranking is very critical and can be well explained by the ability of the elastomeric binder to recover under the testing conditions. The recovery, however, is not being captured by the G*/sinδ due to the fact that the parameter cannot distinguish between total energy dissipated and the energy dissipated in permanent flow. The results from the creep and recovery test can also be explained by what is known about the molecular nature for these materials.

Table 9: Comparison of the creep and recovery indicators with the values of G* and sinδ measured with the DSR at small strains

	SBS Modified PG 82-22 (R1B02)	PEs Modified PG 82-22 (R1B09A)	Oxidized PG 82-22 (R1B15)
ε_{total}	2.389	6.948	11.599
G* at 300 Pa	10989	11379	15272
δ at 300 Pa	56.2	60.3	73.9
sinδ	0.831	0.869	0.961
G*/sinδ	13224	13100	15895
$\varepsilon_L/\varepsilon_p$ at 1 cycle	2.787	1.656	1.169
$\varepsilon_L/\varepsilon_p$ at 100 cycle	5.571	1.764	1.179

To evaluate the effectiveness of using the creep and recovery binder test, 9 binders of various grades were aged in the RTFO and tested at conditions that matches the temperatures and loading time conditions at which the RSCH testing was conducted. The rates of accumulation of permanent strain of the mixtures were plotted versus the rate from the new binder test, as shown in Figure 30.

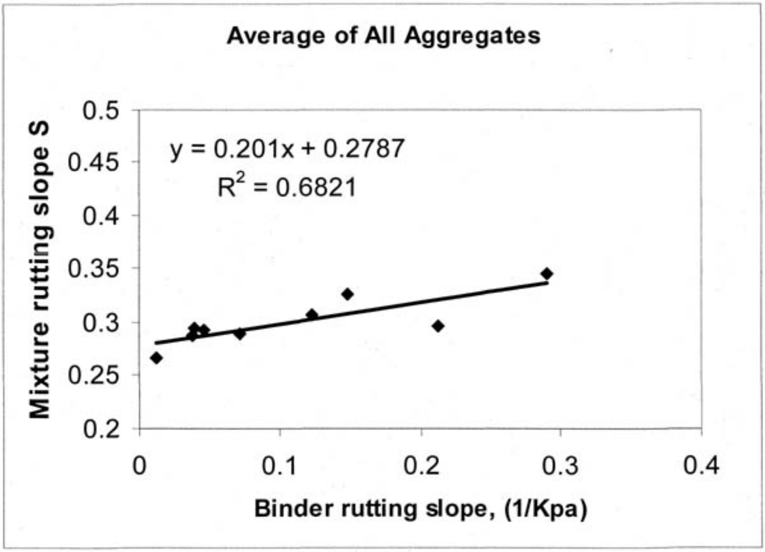


Figure 30: The correlation between binder rutting parameter and the average mixture rutting parameter.

The correlation for the average values for all mixtures has improved significantly from 23 percent to approximately 68 percent. It appears that the new approach based on the creep and recovery testing is very promising. The correlations with the individual aggregate blends varied between higher than the average value to very low correlations for the gravel aggregates. The poor correlations for certain binders were expected since aggregates play a main role in the rutting performance. An analytical approach was used to isolate the effect of binders from the effect of aggregates based on modeling the relationship between mixture and binder accumulation of permanent strain. This type of analysis enabled the separation of the main effect of binders which showed correlations of the mixture and binder rutting behavior in the range of 80 to 90 percent [Bahia et al. 2000].

Binder Fatigue Test

As discussed earlier, it is found that the parameter G*sinδ is not well related to the accumulation of fatigue damage of mixtures as measured in a beam fatigue test, under strain controlled conditions. It is believed that the main reason is that the parameter G*sinδ is measured in the linear visco-elastic range using small strains. There is a fundamental problem with this approach because it is unlikely to be useful in representing the effect of repeated cyclic loading and the changes in binder properties with accumulation of damage. The fatigue damage behavior was the subject of a previous publication that showed the effect of modification on the non-linear behavior and specifically the damage behavior [Bahia et al. 1999].

The effort to develop a new test was focused on simulating the fatigue phenomenon in a binder-only fatigue test. The DSR was used to conduct what is called a time sweep test. The test provides a simple method of applying repeated cycling of stress or strain loading at selected temperatures and loading frequency. The initial data collected were very promising and showed that the time sweeps are effective in measuring binder damage behavior under repeated loading in shear [Bahia et al. 1999].

To understand the test results and to establish the best testing conditions that could lead to effective characterization of binder fatigue behavior, all 9 binders used in the production of mixtures were tested at conditions that match the mixture beam fatigue conditions. The binders were aged in the RTFO to simulate the effect of mixing and compaction and the testing was conducted at 10 Hz at temperatures as close as possible to the mixture beam fatigue temperatures. The testing of binders was conducted in strain-controlled mode, and to match the mixture strain level, an estimated strain of 3 percent was used for all binders. Figure 31 shows the results of the binder testing and indicates that although the initial G* values are similar; these binders show significantly different fatigue behavior. Some of these binders did not reach 50 percent of the initial G* even after applying close to 1,000,000 cycles while others reached this level of G* after only 10,000 cycles. In all these tests the maximum strain at edge of plate was kept constant at 3 percent.

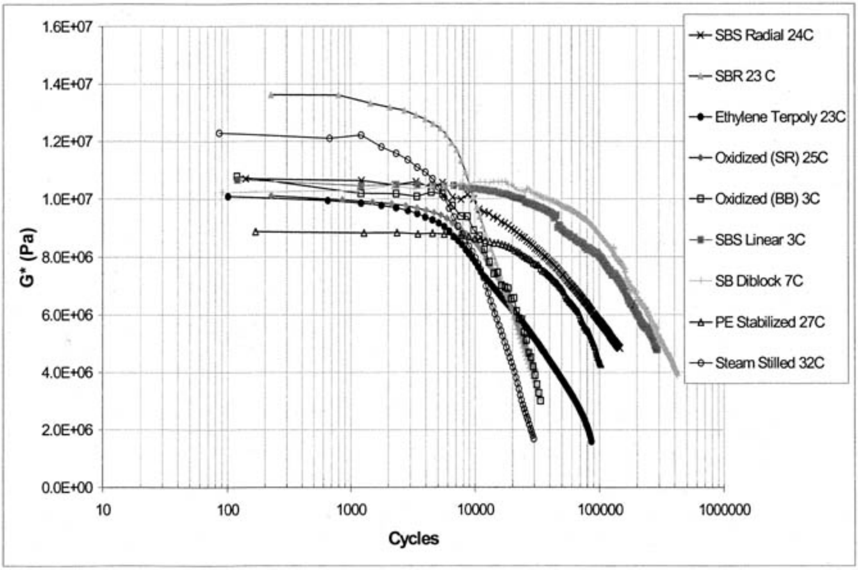


Figure 31: Binder fatigue results at 10 Hz, 3 percent strain, and the temperatures selected for the mixture beam fatigue testing (binders were RTFO aged)

To see if the binder fatigue life measured in the strain-controlled binder test has any relationship to the mixture strain-controlled fatigue life, Figure 32 was prepared to show the relationship between the average mixture performance and the binder fatigue life as determined by the number of cycles to 50 percent G* value. As shown in the figure, there is a high correlation (R-squared = 84 percent) for the nine binders that showed binder fatigue failure. This result is very encouraging and indicates that the newly developed binder fatigue test is promising and could be a better indicator of fatigue damage of mixtures.

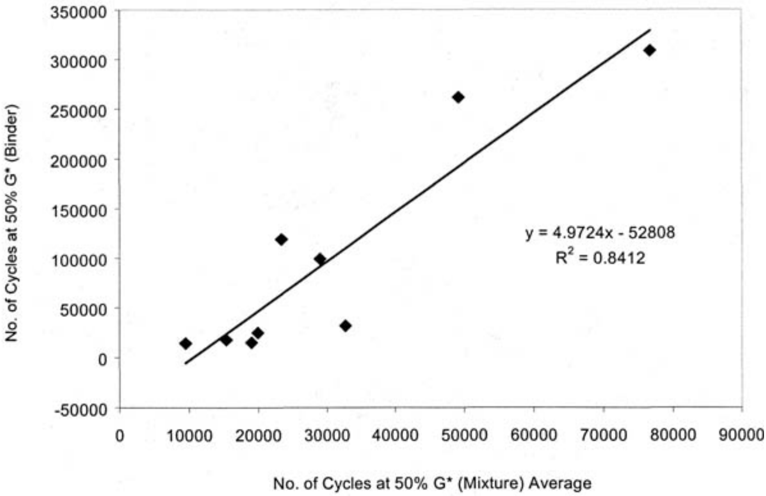


Figure 32: Correlation between binder fatigue life and average mixtures fatigue life measured at the same temperature and frequency

Selection of New Damage Behavior Parameters

Binder Rutting Parameter

Based on the above analyses of the repeated creep tests, it was determined that a creep and recovery test would significantly improve the estimation of resistance to accumulation of permanent strain of binders and their contribution to resistance of mixture rutting. To derive a new parameter for rutting resistance, a basic approach based on visco-elasticity was followed. The approach is based on the well-known power law model that represents the relationship between secondary creep rate as a function of number of cycles of loading (Equation (1)).

Although this concept has been used extensively for asphalt mixtures, it has not been used for binders before. The data collected in this study indicate that modified asphalt binders can be evaluated using the same concept. The binder data, however, indicated that the rate of secondary creep of binders is a simple direct function of the number of cycles and thus the logarithmic transformation is not needed. The following model proved to be reliable:

$$\varepsilon_a = I + S \times N \qquad (2)$$

where, ε_a = accumulated permanent strain

I = intercept with permanent strain axis (arithmetic strain value, not log value)

N = number of load applications; and

S = slope of the linear portion of the logarithmic relation.

It is rather difficult to use the parameter of creep rate "S" as a specification parameter because it is an experimental parameter that is affected by a few testing attributes such as stress, loading time, and number of cycles. A better choice for a specification parameter is to use rheological models that combine fundamental behaviors to understand the performance of the material. Although several models have been used to describe the behavior of asphalt binders [Rosen 1982, Aklonis & MacKnight 1983], the "four-parameter" (Burgers) model, shown in Figure 33, was shown to offer a good representation of the binder behavior [Dekker 1986].

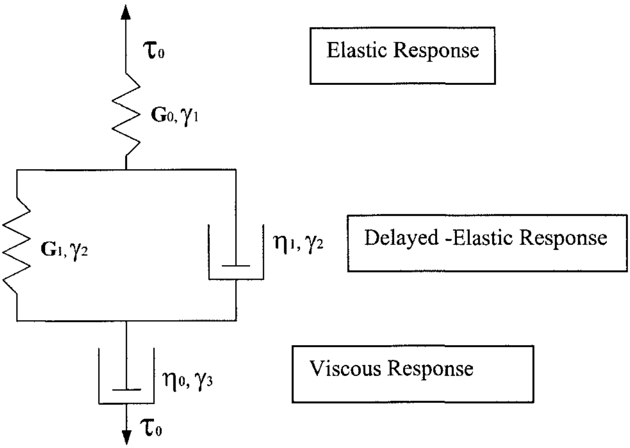


Figure 33: Four-Element (Burgers) Model and its Response

This model is a combination of a Maxwell model and a Voigt model. The total shear strain versus time is expressed as follows:

$$\gamma(t) = \gamma_1 + \gamma_2 + \gamma_3 = \frac{\tau_0}{G_0} + \frac{\tau_0}{G_1}(1 - e^{-t/\tau}) + \frac{\tau_0}{\eta_0} t \qquad (3)$$

By normalizing the strain to the stress applied, the following equation representing the creep compliance, J(t), in terms of its elastic component (J_e), the delayed-elastic (J_{de}), and the viscous component (J_v) could be defined:

$$J(t) = J_e + J_{de} + J_v \qquad (4)$$

The viscous component is inversely proportional to the viscosity (η) and directly proportional to stress and time of loading. Based on this separation of the creep response, the compliance could be used as an indicator of the contribution of binders to rutting resistance. Instead of using the compliance (Jv), which has a strange unit of 1/Pa, and to be compatible with the concept of stiffness introduced during SHRP, the inverse of the compliance (Gv) could be used. "Gv " is defined as the viscous component of the creep stiffness. The creep and recovery response measured with the DSR could be used to estimate the Gv value and the accumulated permanent strain for any selected combination of loading and unloading times.

This finding implies that the accumulated permanent deformation is a function of viscosity, load and loading time.

$$\gamma_1 = f(\eta, \tau, t) \tag{5}$$

$$S = f(\eta, \tau, t) \tag{6}$$

By selecting the appropriate testing stress (τ) and the appropriate time of loading (t) the viscous component of the stiffness Gv could be directly related to the rate of accumulation of permanent deformation "S" and thus used as a fundamental indicator of rutting resistance of asphalt binders.

Effect of Loading Time

It is assumed that, within the possible range of loading times in the field, the accumulation of viscous flow is linear and is proportional to the loading time. In other words, the effect of 10 cycles of 0.1 sec loading time is equivalent to the effect of 2 cycles of 0.5 sec loading time. To check this assumption testing was conducted at 0.1-0.9, 0.5-5.0, and 1.0-9.0 loading cycles. An example of the results adjusted for a total time of 1000 sec are shown in Figure 34.

It is observed that there are significant differences and that the shorter the loading time, the less is the rate of accumulation of permanent strain. The possible explanation of this observation is that the viscosity is a decreasing function of the loading time. The other plausible explanation is that the testing rheometer cannot apply a perfect square wave in the short time of 0.1 sec loading nor the 0.5 loading time.

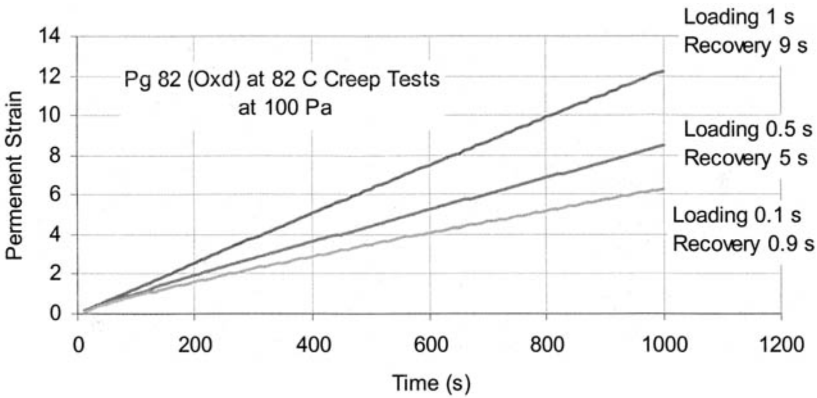


Figure 34: Sample of data collected to study effect of loading time and cycle time on measurements of binder rutting for a modified binder

The total accumulated deformation will therefore be larger for the longer loading time because the rheometer can actually reach the target loading. More testing was conducted to determine the effect of loading time by using longer loading cycles during which the effect of reaching the required load is not an issue. Loadings of 1.0-9.0 sec, 2.0-20.0 sec, and 3.0-30.0 sec were used. Figure 35 depicts the effect of loading time during the repeated creep test on the normalized viscous component of response (Gv).

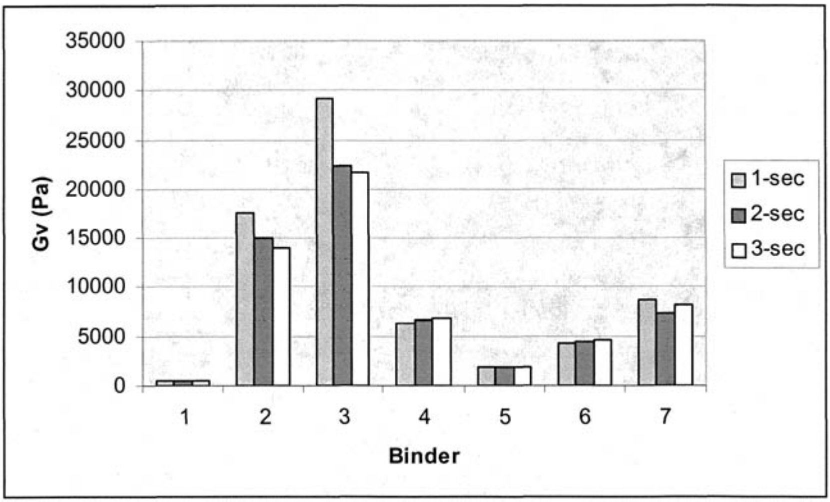


Figure 35: Effect of loading time on the normalized viscous component of the response for a modified asphalt of PG 82 at 70 C

As shown in the figure, it appears that the change in normalized response becomes relatively small after the 1.0 sec loading time. This was not the case when the loading of 1.0-9.0 sec was compared to the loading of 0.1-0.9 sec, and to the loading of 0.5- 4.5 sec. Based on the limited data collected, the loading time of 1.0-9.0 appears to be the most practical and appropriate for test.

Binder Fatigue Parameter

Although there are different loading modes that could be used in fatigue testing, a reliable indicator of fatigue failure should be independent of the loading mode. It should provide a consistent indication of the level of damage and progression of damage in the material in terms of changes in mechanical behavior under any loading conditions.

The most commonly used definition of fatigue failure in asphalt mixtures is a decrease in the initial stiffness by 50 percent, as was indicated in the previous sections. This arbitrary definition, however, does not allow evaluation of the distinctly different mechanism by which a material would respond to the energy input during a loading history for the different loading modes. Researchers have therefore focused on using the concept of dissipated energy to explain fatigue behavior of asphalt mixtures. For many decades researchers have used the loss modulus as an indicator of fatigue resistance because of the relationship between this modulus (G*sinδ) and the energy dissipated per cycle. The success of this approach has been questioned, however, in many studies because this parameter tends to give different results at different loading conditions. Recent advancements in fatigue research have indicated that a better indicator of fatigue is the rate of change of dissipated (distortion) energy per load cycle.

There are several approaches to present the criterion of fatigue based on the rate of change in the dissipated energy. The most promising approaches are presented by Carpenter and co-workers [Carpenter & Ghuzlan 2000], and by Pronk and co-workers [Pronk 1995].

Rate of change of dissipated energy

Ghuzlan and Carpenter defined the ratio of dissipated energy as:

$$\frac{\Delta DE}{DE} = \frac{W_i - W_{i+1}}{W_i} \tag{7}$$

where W_i = the total dissipated energy at cycle i;

W_{i+1} = the total dissipated energy at cycle i+1.

Plotting the values of this ratio versus loading cycles gives a curve that can be used to determine the fatigue life (N_p) by identifying the sudden change in the rate. The problem with this approach is that the data points, especially for the constant stress tests, are scattered widely, which makes it difficult to determine an accurate N_p value.

Applying this concept to binder data, it is observed that the approach is useful for the stress-controlled testing, but is not useful for the strain controlled testing. As shown in Figure 36 the strain controlled measurement result in scattered points, which makes it almost impossible to clearly define the N_p value.

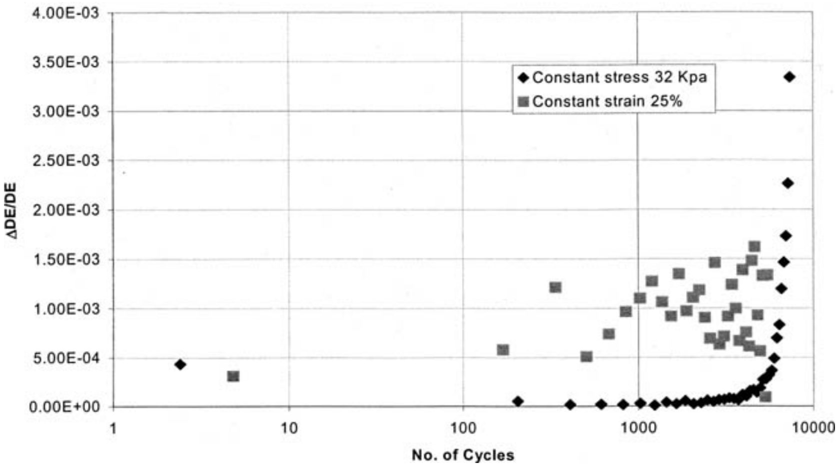


Figure 36: Application of rate of change of energy dissipation concept to binder data

Conceptually, it is difficult to apply this approach for most binders because of the nature of the strain-controlled test. Since the test is strain controlled, the rate of energy dissipation will stabilize when the damage starts accumulating because the material will soften due to damage resulting in reduction in stress required to cause same strain. The test can therefore take a long time to result in transition of material from the crack initiation stage to the propagation stage.

Cumulative Dissipated Energy Ratio

Pronk (1995) defined the dissipated energy ratio as follows:

$$\text{Dissipated Energy Ratio} = \frac{\sum_{i=1}^{n} W_i}{W_n} \qquad (8)$$

where $\sum_{i=1}^{n} W_i$ = the total sum of the dissipated energy up to cycle n

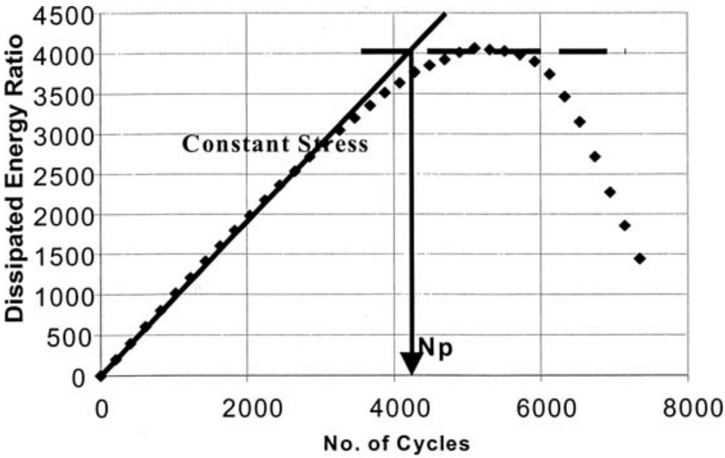
W_n = the dissipate energy at cycle n.

Figure 37 show examples of applying this concept to binder data. The results shown indicate that binders can be evaluated effectively using this method. The curves of the binder data are similar to the mixture data published earlier by other researchers, which is very promising. This also implies that the main factor in the fatigue behavior of mixtures could be well related to the fatigue damage in the binder.

It is also observed that the slope of the relationship between the energy ratio and the number of cycles to failure is equal to 1.0 when the material is not undergoing fatigue damage as indicted by the constancy of the dissipated energy per cycle. This actually can be derived from equation (8) by assuming that W_i is constant and is equal to W_n:

$$\sum_{i=1}^{n} W_i = N.W_n \qquad (9)$$

Based on this derivation, an understanding of the fatigue curve could be introduced. It is assumed that the first portion represents the stage during which the energy per cycle is dissipated in viscoelastic damping with negligible damage. In the next stage, cracking initiation is consuming an additional amount of energy beyond the viscoelastic damping. In the third stage, crack propagation starts and a noticeable increase in dissipated energy per cycle is observed. This is assumed to be the most critical stage during which damage per cycle is so high that healing and recovery from damage are difficult to occur.



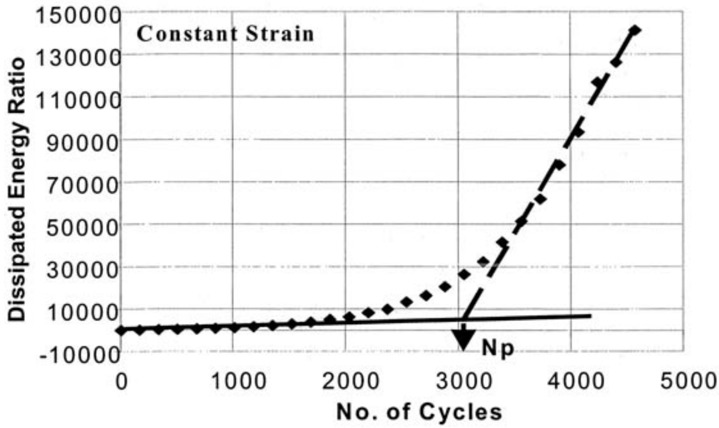


Figure 37: Using the dissipated energy ratio to analyze fatigue data

Validation of Binder Fatigue Test

Being a new test, many questions were raised regarding what is being measured by this test. The main concern was whether what we are measuring is a classic fatigue phenomenon or is it a different damage phenomenon that is caused by the boundary condition and the non-uniform loading model. While it is very difficult to have a direct answer to this question, an indirect approach was taken to produce proof that the measurements relate to classic fatigue. One of the well-recognized facts about fatigue of asphalt mixtures and other materials is the power function relationship between stress or strain level and the fatigue life. An experiment was conducted to increase the stress by logarithmic intervals and measure fatigue life as defined by the Np factor. Figure 38 depicts the measurements at six stress levels. The results show very classic and uniform behavior that tends to show the classic fatigue behavior known for such semi-solid materials.

All fatigue curves show an initial linear portion that is indicative of minimum damage and constant dissipation in visco-elastic damping. This initial phase is followed by the gradual change in energy ratio as the damage accumulation in the binder takes place. It is clear that the on-set of this phase is very sensitive to the stress level since as the crack initiation starts the effective stress bearing area of the sample decreases and the crack propagates rapidly in proportion to the effective radius of the un-cracked sample.

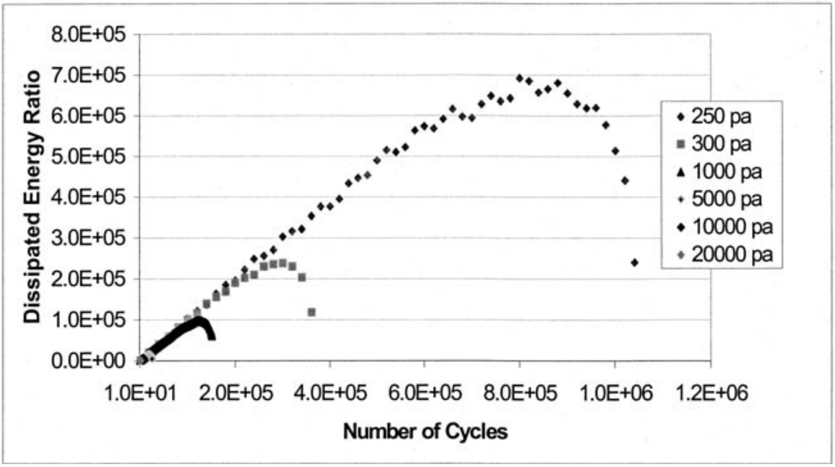


Figure 38: Fatigue behavior of one binder using different stress condition

Validation of Independence of Loading Model

In the process of development of the binder fatigue test, one of the objectives was to develop a test to evaluate the fatigue resistance of binders independent of the mode of loading or the pavement type. The dissipated energy rate approach was selected because it is believed to be independent of the loading mode. To verify this, testing was conducted in a stress-controlled rheometer and a strain-controlled rheometer at selected levels of stress and strain. The data collected was used to estimate the dissipated energy ratio determine Np values for these stresses and strains. The results were plotted in classic fatigue plots showing the cycles to failure as a function of energy input per cycle, as shown in Figure 39.

The energy input was based on the first few cycles. The number of cycles to failure for the stress-controlled testing was defined by Np, as determined from the peak of the energy ratio versus cycles. For the strain-controlled testing the number of cycles to failure was defined by the deviation of 20 percent in energy from the linearity observed between the energy ratio and the number of cycles. This was done because of the difficulty of defining a sharp change in the strain-controlled testing.

The results shown in Figure 39 are very promising and indicate that the fatigue behavior follows a common relationship for both modes of loading. This is significant evidence that the test and the analysis approach can provide results that are independent of the loading conditions. It is also a string evidence that what is being measured is classic fatigue that is not affected by geometry or testing equipment.

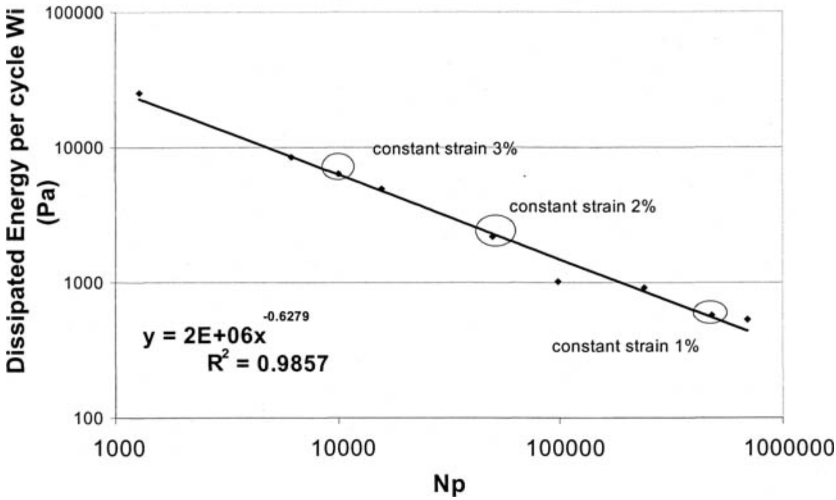


Figure 39: Fatigue behavior of one binder using different loading condition

These results also indicated that the stress-controlled testing is the test of choice because it can give a clear definitive measure of the parameter Np. Np is not an arbitrary selected parameter because it is related to the fatigue damage stage at which the material undergoes the transition from crack initiation to the crack propagation.

Summary of Findings

Based on the results and analysis of the rutting and fatigue studies, the following points provide a summary of the findings:

1. There are critical questions about the validity of the binder parameter G*/sinδ. The correlation between the mixture rutting indicators and G*/sinδ is very poor. The parameter is derived from testing that does not provide a good representation of traffic loading in the field. The parameter could not be found useful in describing the accumulation of permanent flow, which is important in rutting evaluation.
2. Repeated creep testing of binders is introduced as a better method for estimating binder resistance to permanent strain accumulation. The viscous component of the creep stiffness (Gv) is found to be a good indicator of the

1. rate of permanent strain accumulation for binders. It is proposed as a better specification parameter.
2. Compared to the current binder protocol, the repeated creep test protocol for measuring binder accumulated permanent strain represents improvements in the theoretical and practical concepts for better rating of binder properties as related to rutting of pavements.
3. The lack of correlation between the mixture fatigue life and current binder fatigue resistance indicator, $G^*\sin\delta$ at the intermediate temperatures, indicates that a new test for the binder fatigue resistance is needed to determine the relation between mixture fatigue life and binder rheological properties.
4. Time sweep test is introduced as a promising binder-only fatigue test to evaluate the fatigue resistance of the binder. The test can be conducted using the current DSR within a relatively short testing time.
5. The dissipated energy ratio approach is employed as the method to determine the fatigue life of binders, because of the independent nature of this approach to the loading modes. Although the geometry has certain effects on the results, by selecting the proper test conditions, this approach can give reliable results that are found to correlate well with mixture performance.

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Disclaimer

The opinions and conclusions expressed or implied in the report are those of the research agency. They are not necessarily those of the Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or of the individual states participating in the National Cooperative Highway Research Program.

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Asphalt QC/QA and Contractor-Performed Quality Control

by

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ABSTRACT

An effective program of quality control /quality assurance (QC/QA) is the key to a successful asphalt construction project. There is a national trend in state departments of transportation (DOTs) toward allowing contractor-performed quality control (CPQC). At the same time, there is a concern in many DOTs with using the contractor-reported data for acceptance and payment purposes. In order to learn from experience of others, a national survey was conducted. Additionally, a large number of asphalt projects in Kentucky were examined. The statistical analyses showed that, for the most part, there is not a significant difference between the contractor-performed acceptance data and side-by-side highway agency-performed verification data. Obviously, this is a very encouraging finding, and it is expected to enhance the level of trust between the contractors and highway agencies.

KEYWORDS

Quality Control, Quality Assurance, Contractor Testing, DOT Testing, Asphalt Quality

Introduction

Since 1970's the Federal Highway Administration (FHWA) has been encouraging the use of quality control / quality assurance (QC/QA) specifications, which were intended to be statistically based (FHWA, 1973). In response, state transportation agencies have shown varying degrees of success in implementation of quality assurance specifications. Many states are in the process of developing their own QC/QA specifications.

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The National Quality Initiative (NQI) was formed as a partnership between industry stakeholders such as: officials from the FHWA, American Association of State Highway and Transportation Officials (AASHTO), material suppliers, and contractors. This group met to discuss the need to continually improve the quality of the design, construction, and maintenance of the nation's system of highways. The NQI steering committee developed an initial long-range plan to move into some of the more pervasive quality issues in the highway industry. This long-range plan was intended to be a flexible document that was supposed to be modified as necessary. The initial plan was conceived to provide a long-term commitment to continuous improvement rather than a short-term program. Some of the overall objectives of this long-range plan included:

- Considering international applications and technology for possible use.

- Building regional and national consensus on issues in this country that may enhance cost, quality, and performance of U.S. highway system. This included such issues as specifications, design and design assumptions, training and certification requirements, laboratory quality control requirements and accreditation.

- Improving the technology and technology sharing through research, training, incentives, demonstration, and use of information-sharing techniques.

- Heightening the awareness for quality and encouraging the use of quality improvement techniques, partnering, and state-of-the-art planning, design, construction, and maintenance techniques in the highway industry.

- Providing a follow-up mechanism for Transportation Circular 386 on "Innovative Contracting Practices" (TRB 1991 and, Tuggle 1994) to explore new ways of contracting and providing increased quality and quality incentives in the highway industry.

The New Jersey DOT was the first state agency to implement a statistical performance related specifications. Mr. Richard M. Weed was responsible for the original development of the New Jersey QC/QA Specifications. In 1989, Mr. Weed also initiated the development of a software package. This package enables the user to analyze both pass/fail and pay adjustments. It can also construct operating characteristic curves, plot control charts, and perform statistical comparisons (Weed 1995, 1996, and 1999).

In the year 2000, the FHWA released the following publication: "Latest Guidelines for Developing Performance-Related Specifications for PCC Pavements" on a CD-ROM to predict the future maintenance, rehabilitation, and other life-cycle costs of PCC pavements (FHWA-RD-00-131, August 2000). This instructive CD contained a four-volume report detailing guidelines for implementing performance-related specifications (PRS), as well as the 2.0 version of the PaveSpec software. The Indiana DOT used the PaveSpec software in connection with their efforts to develop performance related specifications for I-465 in Indianapolis.

Specification Types in Construction Industry

Generally, there are four types of specifications recognized in the construction industry (Burati and Hughes 1993, Chamberlin 1995). These are commonly known as:

- Proprietary Product Specifications
- Method Specifications
- End-result Specifications
- Performance Related Specifications

Proprietary Product Specifications

This type of specification refers to some specific product or its equivalent in its clauses. Therefore, it limits the competition and often results in a cost increase. Since buyer has to accept the product as a "black box", the buyer's risk is much higher than the other three types of specifications.

Method Specifications

This type of specification outlines a specific materials selection and construction operation process to be followed by the contractor in providing a product. Since there is no specific product specified, this type of specification allows competition among various suppliers and contractors. But, because the buyer sets the requirements for materials and methods, the owner has to bear the responsibility of the performance.

End-result Specifications

The final characteristics of the product are stipulated in the end-result specification and the contractor is given considerable freedom in achieving those characteristics. It may specify, a limit or a range for any given material and/or construction characteristic. The risk for the contractor or agency depends on how the acceptance limits and processes are specified.

Performance Related Specifications (PRS)

This type of specification holds the contractor responsible for the finished product's performance. Thus, the contractor assumes considerable risk for the performance of the finished product. This type of specification is often used in conjunction with some type warranty. The challenge here is to use "true" performance indicators, which may not be available for all materials and processes.

DOT Experiences

Most of the U.S. highway agencies are in various stages of adopting end-result specifications plus QC/QA quality management schemes. It is important to note that materials do not always completely conform to specifications. Therefore, specifications must be designed to reward good quality, and penalize poor quality. The FHWA report reveals that many states are actively implementing QC/QA concepts into their specifications (FHWA, 2000). Table 1 presents a summary of the FHWA survey results.

Table 1 – Survey of State DOTs (Courtesy of FHWA, 2000)

State DOT	**With Formal QC/QA System**	**Without Formal QC/QA**	**QC/QA in Development**
Alabama		X	
Alaska		X	
Arizona	X		
Arkansas	X		
California		X	
Colorado	X		
Connecticut		X	
Delaware		X	
District of Columbia			X
Florida			X
Georgia	X		
Hawaii			X
Idaho		X	
Illinois	X		
Indiana	X		
Iowa			X
Kansas	X		
Kentucky	X		
Louisiana	X		
Maine		X	
Maryland	X		
Massachusetts		X	
Michigan	X		
Minnesota	X		

Table 1 – Survey of State DOTs (Courtesy of FHWA, 2000) (cont'd)

State DOT	With Formal QC/QA System	Without Formal QC/QA	QC/QA in Development
Mississippi		X	
Missouri		X	
Montana	X		
Nebraska			X
Nevada			X
New Hampshire		X	
New Jersey	X		
New Mexico			X
New York			X
North Carolina		X	
North Dakota		X	
Ohio		X	
Oklahoma			X
Oregon	X		
Pennsylvania	X		
Puerto Rico	X		
Rhoda Island		X	
South Carolina			X
South Dakota			X
Tennessee	X		
Texas			X
Utah			X
Vermont		X	
Virginia		X	
Washington			X
West Virginia	X		
Wisconsin	X		
Wyoming	X		

QC/QA and PRS Concepts

Adjustable Payment Schedules are an integral part of a well-written specification. It can be justified that withholding a portion of the contracted price is related to the estimated loss of service life and performance. Based upon the work of Mr. Richard

M. Weed, one of the pioneer researchers in this area, the relationships between performance and deviation from specified quality were proposed (FHWA 1998). Using this methodology, the measured acceptance quality characteristics (AQC), which may include: hot mix asphalt (HMA) density, asphalt content, lift thickness, initial smoothness, etc., are related to pavement performance through mathematical relationships. Performance is defined by key distress types, and smoothness may be related to the future maintenance, rehabilitation, and user costs of the highway.

If the economic impact of varying quality can be quantified, the results can be used to adjust the price of the finished product by giving either a penalty or a bonus. The penalty should not be more than the present worth of the estimated additional cost associated with deficient quality. On the other hand, the bonus must be estimated on the basis of how much performance potential is enhanced by exceeding the minimum measures of quality. Establishing a link between measured AQC's and future life-cycle costs (LCC's) by a mathematical formula is an on-going area of research.

History of Statistical Specifications

The history of quality control is as old as the manufacturing or construction industry itself. During the Middle Ages, quality control was addressed to a large extent by the long periods of training required by the guilds. The concept of specialization of labor was introduced during the Industrial Revolution. As a result, a single worker no longer made the entire product, only a portion. This change brought about a decline in workmanship and caused the quality to suffer. Therefore, it became necessary to inspect the finished product. In 1924, Walter A. Shewhart of Bell Telephone Laboratories developed a statistical chart for the control of product variables (Grant, 1988). Later in the same decade, (H.F. Dodge and H.G. Romig 1959), both of Bell Telephone Laboratories, advocated an acceptance sampling as a substitute for 100% inspection. Thereafter, the value of statistical quality control became apparent in large scale. The American Society for Quality was formed in 1946 (Besterfield 1998).

Sampling is a method for checking the quality of a part as an evidence of the quality of the whole. Thus, characteristics of the sample of a lot are usually assumed to be indicative of the entire lot, Figure 1. Therefore, sampling plans are used as a statistical tool to decide which lots of the product to accept or which lots to reject. Ideally, a sampling plan should reject all bad lots while accepting all good lots. However, because acceptance/rejection decisions are made based upon a sample of the lot and not the entire lot, there is always a risk of not catching a bad lot. Quantification of this risk will be described later in this document.

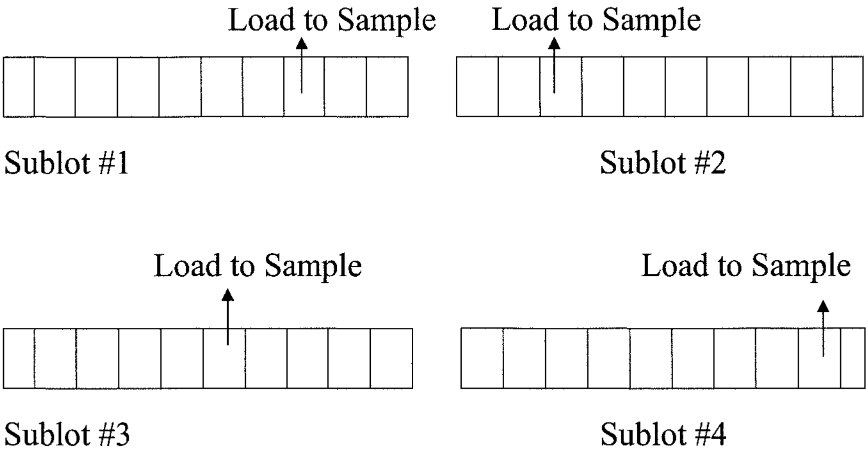


Figure 1 - Stratified Random Sampling (For example: One Lot = 4 Sublots, One Sublot = 10 Truck Loads). Arrows: Load to Sample.

Statistical tools such as the histogram, control charts, and operating characteristic curves organize sample information into a format which is simple to understand. Random sampling is the key to a valid statistically-based QC/QA. A statistical sampling plan assesses compliance with the specifications in a manner which allows for natural variability. Calibration of field testing equipment and batch plants and training of all personnel (the Contractor's and the Department's representatives) are of great importance. A Contractor's incentive to provide competent field personnel becomes apparent when pay factors are based upon Contractor-performed test data.

A basic requirement for most of statistical tools is that samples are taken from a normally distributed population. But well-defined normal distributions become evident only after a relatively large amount of data have been collected. Normality may not be readily apparent until the entire project is evaluated using techniques such as histogram, skewness and kurtosis, probability plots, and chi-square test. The minimum recommended sample size for each technique to get a reasonable representation of normality is given in Table 2 (Besterfield, 1998).

Table 2. The Minimum Recommended Sample Size for Normality

Technique	Minimum recommended sample size
Histogram	50
Skewness and Kurtosis	100
Probability Plots	30
Chi-Square Goodness of Fit Test	125

Control Charts for Process Control

One could say that the variability is a law of nature; no two natural items in any category are the same (Besterfield, 1998). As long as these sources of variation fluctuate in a random manner, a stable pattern of many random causes develops. Those causes of variation that are large in magnitude, and therefore readily identified, are classified as assignable causes. When only chance causes are present in the process, the process is considered to be in a state of statistical control. However, when an assignable cause of variation is also present, the variation will be excessive, and the process is classified as out of control, or beyond the expected natural variation (Samson 1970, Grant 1988, Duncan and Acheson 1952).

In order to track the status of variations in quality, control charts are used. The control chart method is a means of depicting variations that occur around an average and within a range (Besterfield, 1998). It is a graphical record of a particular measure of quality. It shows whether or not the process is in a stable state.

If the samples are taken from a normally distributed population, variations among sampling pool may be expected to occur within plus or minus three standard deviations ($\pm 3\sigma$) from the average. This range covers 99.73% of all data. Thus, the central lines and trial control limits for the $\overline{X}$ chart and R chart are obtained as follows:

The central line of the $\overline{X}$ chart; $\overline{\overline{X}} = \dfrac{\sum_{i=1}^{g} \overline{X}_i}{g}$ *(1)*

The central line of the R chart; $\overline{R} = \dfrac{\sum_{i=1}^{g} R_i}{g}$ *(2)*

The Upper Control Limit of $\overline{X}$ chart, $UCL_{\overline{X}} = \overline{\overline{X}} + 3\sigma_{\overline{X}}$ (3)

The Lower Control Limit of $\overline{X}$ chart, $LCL_{\overline{X}} = \overline{\overline{X}} - 3\sigma_{\overline{X}}$ (4)

The Upper Control Limit of R chart, $UCL_R = \overline{R} + 3\sigma_R$ (5)

The Lower Control Limit of R chart, $LCL_R = \overline{R} - 3\sigma_R$ (6)

Where

$\overline{\overline{X}}$ = average of lot averages (i.e. average of $\overline{X}_i$)

$\overline{X}_i$ = average of i^{th} lot

g = number of lots

$\overline{R}$ = average of lot ranges (i.e. average of R_i)

R_i = range of the i^{th} lot (i.e. the difference between the highest and the lowest observed values)

$\sigma_{\overline{X}}$ = the population standard deviation of the lot averages

σ_R = the population standard deviation of the range

The calculations are often simplified by using the product of the range ($\overline{R}$) and a factor (A_2) to replace the three standard deviation term ($A_2\overline{R} = 3\sigma_{\overline{X}}$) in the formulas for the $\overline{X}$ chart. For the R chart, the range $\overline{R}$ is used to estimate the standard deviation of the range (σ_R). Therefore, the derived formulas are:

$UCL_{\overline{X}} = \overline{\overline{X}} + A_2\overline{R}$ (7)

$LCL_{\overline{X}} = \overline{\overline{X}} - A_2\overline{R}$ (8)

$UCL_R = D_4\overline{R}$ (9)

$LCL_R = D_3\overline{R}$ (10)

Where A_2, D_3, and D_4 are factors that vary with the sample size and are found in most statistical tables.

In addition to $\overline{X}$ and R control charts, the Sample Standard Deviation control Charts, Moving Average and Moving Range Charts, Median and Range Charts, X Charts can also be used to monitor a process. From the material producer's point of view, he/she must pay close attention to the control limits in addition to the specification limits. These control limits should not only be within the specification limits, but also the centerline must be close to the target value (typically, the mid-point in the specification range). Finally, if the DOT QC/QA specifications do not specifically require preparation of control charts, it is strongly recommended to monitor the progress of the QC/QA projects using some type of a control chart.

QC/QA and Verification Testing

A QC/QA program is a comprehensive system for overseeing quality-related activities in an integrated fashion. As agency personnel resources are reduced, more reliance is placed on the contractor-performed testing. Under this regime, state agencies perform a supervisory role, and conduct a limited number of tests to verify the contractor reported data. For example, the Kentucky Transportation Cabinet (KYTC) uses the contractor quality control test results for acceptance purposes if the contractor data are verifiable by the DOT.

Trust in the contractor-reported data remains a major concern; hence, the DOTs prefer to perform their own testing for acceptance and payment purposes. However, this may not be possible if DOT resources happen to be inadequate. The frequency of the DOT verification testing is often a compromise between the available agency resources versus the risk of accepting poor materials and/or poor construction. The rate of agency side-by-side verification testing as compared to the acceptance testing ranges from 10% to 33% (25% for Kentucky, see Table 3). A higher frequency of DOT verification testing may give the agency more confidence with using the contractor's data for acceptance, but it also comes at a higher cost to the agency. State DOTs use various methods to judge the consistency between their own data and the contractor-generated data (Table 3). Some DOTs set a tolerance limit between the contractors-reported data and the DOT-reported test result on split, or paired samples. If the difference exceeds the tolerance limit, dispute resolution clauses are invoked (example: KYTC 2000, 2001). A more sophisticated approach involves comparing the statistical characteristics of the two sets of data (Govindaraju 1999, Duncan 1986, Montgomery 1985). If the two sets of data demonstrate similar statistical parameters (i.e. similar means and variances), the contractor-performed data may be accepted with a high degree of confidence. Finally, if the contractor data are shown to be reliable and statistically similar to the DOT data, using the contractor-reported data for payment purposes can reduce duplication and improve DOT operations efficiency.

Table 3. Some Examples of Asphalt Verification Methods and Frequencies

DOT	Verification Frequency (HMA)	Verification Method
Alabama	1 per lot	compare the difference
Arkansas	1 per lot	compare the difference
California	not less than 10 percent of the minimum quality control sampling and testing frequency required of the Contractor	t-test and F-test
Connecticut	minimum 1 lot/project	t-test and F-test
Illinois	>=10% for gradation; >= 20% for asphalt content, bulk specific gravity, maximum specific gravity, and field density	Split Sample, compare the difference
Kansas	1 per lot	t-test and F-test
Kentucky	1 per lot (four sublots per lot)	compare the difference
Michigan	a minimum of one set per grade of concrete daily, 33% of contractor's test for HMA	compare the difference
Missouri	1 per day	compare the difference
Nebraska	1 per lot	compare the difference
New Mexico	1 per lot	t-test and F-test
North Dakota	1 per lot	compare the difference

Statistical Quality Acceptance Procedures with Risk Analysis

The following terms used in this part are defined for clarification:

The *specification limit* is a specified value for a certain material characteristic, for example, the asphalt content or the air void, for which laboratory and field experiments show a link with performance.

The *acceptable quality level* (AQL) is a percent defective below which the product should mostly be accepted, while the *rejectable quality level* (RQL) is a percent defective above which the product should mostly be rejected. Both AQL and RQL are expressed in terms of percentage of the poor material. Here the "poor" means it violates the specifications limits.

The probability of non-acceptance of a lot that has a defect level equal to or below the AQL is called the Producer's Risk (here we call it the contractor's risk). The probability of acceptance of a lot with a defect level equal to or higher than the RQL is called the Consumer's Risk (here we call it the DOT's risk). This is demonstrated in Figure 2.

It should be noted that the *Acceptable quality level* (AQL) *or rejectable quality level* (RQL) are not used directly to accept or reject materials, but are selected by the DOT to calculate the number of acceptance sample required and a critical value for acceptance, which can be a single number of percent defective or percent within limits.

		Decisions Based on Sampling	
		Accept the Lot	**Reject the Lot**
True Lot Quality	**Less than AQL**	Right Decision	Contractor's Risk Type I Error
	More than RQL	DOT's Risk Type II Error	Right Decision

Figure 2 - The Contractor's risk and the DOT's risk.

Types of Acceptance Sampling Plans

There are two types of acceptance sampling plans: the attribute acceptance plan and the variable acceptance plan. The attribute acceptance plan only grades the material as "conforming" and "nonconforming", without looking any furhter at the quantitative measurements. Most highway materials, however, are evaluated using the variable acceptance plan because it requires a smaller sample size and yields good performance. Therefore, the analysis presented in this document was concentrated on the variable acceptance plan. The approaches usually used in the variable acceptance plan are the average method ($\bar{x}$ method), k method, and m method. The following sections are designed to provide an over view of these three methods.

The Average Method ($\bar{x}$ Method)

For the materials to be accepted, the average value of the acceptance sampling data must satisfy a certain value when there is only one single specification limit, or within a certain range when there is double specification limits. For example, if there is only a lower specification limit L (such as minimum indirect tensile strength), the procedure for the average method is to:

1. Take a random sample of size n and find the average $\overline{X}$.
2. Using $A = L + k\sigma$ (11)

One would accept the lot if $\overline{X} \geq A$, otherwise one would reject it.

Where A is called a quality level parameter. L is the lower specification limit and σ is the population standard deviation of the material. The term k is a parameter that works in combination with σ in a manner similar to a safety factor.

In the case of an upper specification limit, A is set as $U - k\sigma$ and the acceptance criterion is reversed as $\overline{X} \leq A$. In the case of double specification limits, A should be within the two end points of an acceptance interval: $L + k\sigma$ and $U - k\sigma$.

It must be noted that the material usually should not be accepted when the average value of acceptance samples falls exactly either on the upper or lower specification limits. The reason is that even if the average value meets the specification limit, statistically there would be fifty percent of the data which would fall outside of the acceptable limits. Thus, the tolerance quality limits, the quality level "*A*", are set in such a way to address this issue.

This method requires a previously known (or estimated) standard deviation σ and a predetermined number of measurement numbering n and a critical value of k. The procedure for deducting n and k, which can be estimated by using the DOT's risk, the contractor's risk, AQL, and RQL, will be addressed later. In practice, many DOTs just specify "$L + k\sigma$" or "$L - k\sigma$" as a single number and assume the standard deviation is the same for all contractors. The disadvantage of this practice is that materials with large variation are paid the same as those with relatively small variations, if their means are the same.

The k-Method

The k-method is basically the same as the $\bar{x}$ method. The only difference is that for highway materials, the average method is thought to have a fixed standard deviation. Under the k-method, the term k is a critical value in a normal curve that corresponds to a specified proportion m. If there is a lower specification limit, the procedure for the k-method is to:

1. Estimate $Z = (\overline{X} - L)/\sigma$ (12a)

or sample standard deviation (S) is used when the population standard deviation (σ is not known:

$$Z = (\overline{X} - L) / S \tag{12b}$$

2. Accept the lot if $Z > k$, otherwise, reject it.

In the case of an upper specification limit, Z is computed as follows:

$$Z = (\overline{X} - L) / \sigma \tag{13a}$$

or sample standard deviation (S) is used when the population standard deviation (σ) is not known:

$$Z = (\overline{X} - L) / S \tag{13b}$$

When the population standard deviation is unknown.

The acceptance criterion remains the same as $Z > k$. The parameters that need to be determined are the number of required acceptance sample n and the critical value k.

The m-Method

Instead of using the Z term (as described earlier) to estimate the percent of nonconformance, the m-method uses an unbiased estimation as follows.

$$Q_L = \frac{\overline{X} - L}{s}\sqrt{\frac{n}{n-1}} \qquad (14)$$

The term Q_L is a normal deviation which is used to get the estimation of percent defectives p' (Duncan, 1986). In case of a lower specification limit, the following relationship is used as an estimate:

$$\hat{p}_L = \int_{Q_L}^{\infty} \frac{1}{\sqrt{2\pi}} e^{-0.5t^2}\, dt \qquad (15)$$

Again, Q_L is the minimum variance unbiased estimate of p'. The estimate $\hat{p}_L$ is compared with the maximum allowable percent defectives m, and the lot is accepted if $\hat{p}_L \leq m$.

In case of an upper specification limit, the following standard normal deviation is used and the acceptance criterion remains the same. These relationships are:

$$Q_u = \frac{U - \overline{X}}{S}\sqrt{\frac{n}{n-1}} \qquad (16a)$$

or

$$Z_M = k\sqrt{\frac{n}{n-1}} \qquad (16b)$$

According to the Department of Defense standard for sampling inspection of variables, the k-method is called procedure 1 and the m-method is called procedure 2. Because the average method is equivalent to the k-method when the standard deviation is known, the following discussions expand upon the k-method and the m-method.

Determining Sample Sizes and Risk Analysis

The primary task in designing a statistical sampling plan is to find the sample size n and the acceptance criterion – the k method or the m method – that will yield the characteristics such as acceptance quality level, reject quality level, DOT' risk, and contractor's risk (Duncan, 1986). On the other hand, given n, k or m, one can evaluate the contractor's risk and the DOT's risk by back calculation.

The procedures for obtaining these numbers are different, and they depend upon the following circumstances.

- Standard deviation known, a single specification limit
- Standard deviation known, double specification limits
- Standard deviation unknown, a single specification limit
- Standard deviation unknown, double specification limits

Standard Deviation Known, Single Specification Limits Sampling Plan

In this situation, the samples are assumed to be normally distributed with a known σ from the historical data, and a lower specification limit of L or an upper specification limit of U. For example, this lower specification limit can be the HMA percent density. The first step in making an acceptance plan for the DOT is to determine an Acceptable Quality Level (AQL, p_1) and the Reject able Quality Level (RQL, p_2) represented in terms of percent defectives (p parameters), as well as the contractor's risk α and the DOT's risk β. This process yields the information on the number of samples denoted by n and critical value denoted by k. It is the critical value that will be eventually used to make acceptance or rejection decision by engineers. The equations for calculating the number of samples and the critical value are as follows.

$$n = \left(\frac{Z_\alpha + Z_\beta}{Z_{p1} - Z_{p2}} \right)^2 \qquad (17)$$

$$k = \frac{Z_{p2} Z_\alpha + Z_{p1} Z_\beta}{Z_\alpha + Z_\beta} \qquad (18)$$

where Z_ε (the ε subscripts α, β, p_1, or p_2) is the standard normal Z score with (upper) tail area of ε (K. Govindaraju, 2000).

For example, suppose one knows the HMA field density of a lot, and one decides to use AQL = 10%, α = 5%, RQL = 25%, and β = 5%. The variables plan parameters are found to be as follows:

$$k = \left(\frac{0.675 \times 1.6449 + 1.282 \times 1.6449}{1.6449 + 1.6449} \right) = 0.98 \qquad (19)$$

$$n = \left(\frac{1.6449 + 1.6449}{1.282 - 0.675} \right)^2 = 29.4 = 30\,. \qquad (20)$$

For this sampling plan, one needs to take 30 samples. Obviously, this is too many for a lot, and thus it is not feasible for highway applications. Let us change the AQL, α, RQL and β to be: AQL = 5%, α = 10%, RQL = 25% and β = 10%. The plan parameters are found to be as follows:

$$k = \left(\frac{0.675 \times 1.17 + 1.555 \times 1.17}{1.17 + 1.17} \right) = 1.115 \tag{21}$$

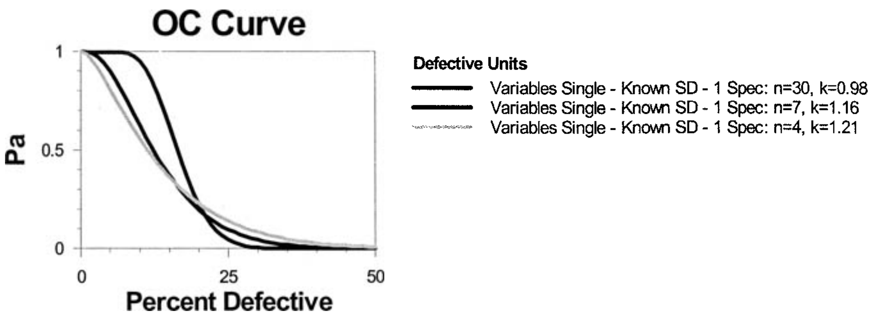
$$n = \left(\frac{1.282 + 1.282}{1.6449 - 0.675} \right)^2 = 7.07 = 7\,. \tag{22}$$

Now, let us further change the AQL, α, RQL and β to be: AQL = 4%, α = 15%, RQL = 25% and β = 15%, the plan parameters are found to be:

$$k = \left(\frac{0.675 \times 1.037 + 1.7505 \times 1.037}{1.037 + 1.037} \right) = 1.21 \tag{23}$$

$$n = \left(\frac{1.037 + 1.037}{1.7505 - 0.675} \right)^2 = 3.7 = 4\,. \tag{24}$$

Table 4 lists some possible combinations of n, k, α, and β, calculated from the above equations, where *m* is the maximum allowable percent defective. As one can see in Table 4, the discriminating power of the sampling plan will deteriorate as one decrease the number of samples. For the sample size of four, if one decides to accept 4% or less defective products, then the product will have a 15% of the time chance of being rejected. The Operating Characteristic (OC) Curves can reveal the discriminating power of different acceptance plans (Figure 3).



*Pa: probability of acceptance

Figure 3 - The Operating Characteristic Curves of the Three Sampling Plans

Table 4 - Possible Combinations of n, k, m, The Contractor's Risk and The DOT's Risk (when σ is known)

Sample Size	Critical Value	AQL	RQL	Contractor's Risk (α)	DOT's Risk (β)
n=30	k=0.98 m=16%	10%	25%	5%	5%
n=8	k=1.12 m=12%	6%	25%	10%	10%
n=7	k=1.12 m=12%	6%	25%	12%	12%
n=6	k=1.16 m=10%	5%	25%	15%	15%
n=5	k=1.12 m=11%	5%	25%	15%	16%
n=4	k=1.21 m=8%	4%	25%	15%	15%

The discussion presented above is based upon the *k*-method. In the *m*-method, the *k* parameter is replaced by a maximum allowable percentage defective number denoted by *m*, which is the area under the normal curve beyond $k\sqrt{\frac{n}{n-1}}$. In the last example, one can get $n = 4$ and $k = 1.21$, the maximum percent defective denoted by *m* will be the proportion of the area under the normal curve beyond $1.21 \times \sqrt{\frac{4}{3}} = 1.397$, which equals to 8.1%. Other values of *m* that correspond to a different sample size *n*, and critical value *k* are reported in the Table 4 as well.

After deciding on the m-parameter, one can determine whether to accept a lot or not by the following criterion: taking a random sample of size 4 for each lot, then computing the $Q_L = \frac{\overline{X} - L}{\sigma'}\sqrt{\frac{4}{3}}$ and using this as a normal deviation, obtaining the area (p') in excess of Q_L in a standard normal distribution table. If $p' \leq 8.1\%$, one would accept the lot; otherwise one would reject it.

When this is only an upper specification limit, the acceptance procedure can be conducted in a similar manner.

Standard Deviation Known and Double Sampling Plan

In the case of double specification limits (with both an upper and a lower specification limit), the evaluation of the acceptance sampling plan is more complicated. One needs to review the following situations separately, and check whether

1. The upper and lower limits are close;
2. The upper and lower limits are widely apart;
3. The upper and lower limits are relatively close.

Upper and Lower Limits Are Close

When the material characteristic is normally distributed and σ' is known, the first step is to note whether the area under a standard normal curve beyond the point $z = \pm\frac{U-L}{2\sigma'}$ is greater than an acceptable percent defective (Duncan, 1986). If this is true, the acceptance samples will always be rejected. Because even when the average of the acceptance samples falls equally between the upper and the lower specification limit (the best possible scenario), the percent defective will be larger than would be acceptable. The lesson to be learned here is that if the DOT made the specification limits too tight, the contractor's material would be under the risk of being rejected all the time.

Upper and Lower Limits Are Widely Apart

If the upper and the lower specifications limits are widely spread, i.e., $\frac{U-L}{2} \geq 3\sigma'$, two sampling plans can be used, one for application at the lower specification limit, the other for application at the upper specification limit (Duncan, 1986).

If one is going to use the *k*-method, then one can accept a sampled lot if $\frac{\overline{X}-L}{\sigma'} \geq k$ and $\frac{U-\overline{X}}{\sigma'} \geq k$, otherwise the lot must be rejected (Duncan, 1986). If one is going to use the m-method, one needs to first calculate the maximum allowable defective proportion using $m = k\sqrt{\frac{n}{n-1}}$. Then one needs to compute $Q_L = \frac{\overline{X}-L}{\sigma'}\sqrt{\frac{n}{n-1}}$ or $Q_U = \frac{U-\overline{X}}{\sigma'}\sqrt{\frac{n}{n-1}}$, and find the percent defectives (p_L' or p_U') corresponding to Q_L or Q_U. If either p_L' or p_U' exceeds the maximum allowable percent defective *m*, one would reject the lot.

Upper and Lower Limits Are Relatively Close

When the upper and the lower specification limits are not widely apart, the procedure to get *n, k,* and *m* will be different. The sample size *n* and the maximum allowable percent defective *m* will be influenced by the upper and lower specification limits. The computation of these parameters should be performed on a case-by-case basis. This analysis assumes a historically known population standard deviation.

Standard Deviation Unknown and Single Specification Limit

In the case of having no previous knowledge about the standard deviation of a material characteristic, the DOT has to estimate it using the sample standard deviation S. For a given AQL (p_1), RQL (p_2), the contractor's risk (α), and the DOT's risk (β), the Equation 18, as described earlier, can be used to calculate the k-parameter for calculating n and k become as follows:

$$k = \frac{Z_{p2}Z_{\alpha} + Z_{p1}Z_{\beta}}{Z_{\alpha} + Z_{\beta}} \tag{18}$$

However, the n-parameter is adjusted in this case and equation 19 is modified to take on the following form

$$n = \left(\frac{Z_{\alpha} + Z_{\beta}}{Z_{p1} - Z_{p2}}\right)^2 \left(1 + \frac{k^2}{2}\right) \quad (\sigma \text{ is not known}) \tag{25}$$

where the Zs are the standard normal Z score with (upper) tail area corresponding to p_1, p_2, α, and β(E. G. Schilling, 1982, A. J. Duncan, 1986, K. Govindaraju, 2000) .

It is important to note that in comparison with the case when the standard deviation is known, the sample size is multiplied by the following factor: $1 + \frac{k^2}{2}$. Therefore, one can see that a larger sample is required to compensate for the uncertainty of material variability in order to get the same discriminating power. For example, when AQL = 4 %, α = 15%, RQL = 25% and β = 15%, the k and n will become:

$$k = \left(\frac{0.675 \times 1.037 + 1.7505 \times 1.037}{1.037 + 1.037}\right) = 1.21 \tag{26}$$

$$n = \left(\frac{1.037 + 1.037}{0.675 - 1.7505}\right)^2 \left(1 + \frac{1.21^2}{2}\right) = 6.4 = 6 \tag{27}$$

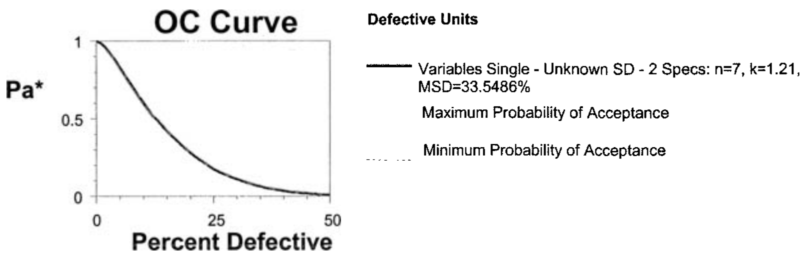
The k acceptance method remains the same as the standard deviation known case, but the m-method is different. The m-method becomes a case similar to the widely used Percent Within Limits (PWL) acceptance method in the highway construction industry. The difference is that m is the percent outside the limit while the PWL is the percent within limit (PWL = 100% - m%).

Using the m-method, one needs to estimate a proportion of non-conforming ($\hat{p}$) from either $Z_L = \frac{\overline{X} - L}{S}$ or $Z_U = \frac{U - \overline{X}}{S}$ relationships. If $\hat{p} < m$, one would accept the lot; otherwise one would reject it.

When the standard deviation is unknown, the estimation of $\hat{p}$ and m is rather complicated. The minimum-variance method of Lieberman and Resnikoff requires special tables and a special procedure for determining p and m (Duncan, 1986). A chart developed by A.J. Duncan can be used to find the value of m by inputting the previous calculated n and k (page 281 of A.J. Duncan, 1986).

Standard Deviation Unknown and Double Specification Limits

Many acceptance sampling plans for highway materials are based upon double specification limits. When the population standard deviation from historical data is unknown, one can no longer find a one to one correspondence between a finite number of normal z's and a given fraction of non-conforming (Duncan, 1986). In other words, if the AQL (p_1), RQL (p_2), producer's risk (α) and consumer's risk (β) are given, there will be a band of OC curves. For example, for AQL = 96%, α = 15%, RQL = 25% and β = 15%, one can calculate that the k and n will be 1.21 and 7, respectively. The very narrow band of OC curves is shown in Figure 4. The k-method and m-method will be applicable to double specifications. The following sections describe this further.



*Pa: probability of acceptance

Figure 4 – A very narrow band of Operating Characteristic Curves for an Acceptance Sampling Plan (σ' is not known, Double Specification Limits). A corrected *k*-method and *m*-method are recommended in this situation (Duncan, 1986).

1. The Corrected *k*-Method

The criteria for acceptance under the corrected k method should be:

$$\frac{\overline{X} - L}{S} \geq k \tag{28}$$

$$\frac{U - \overline{X}}{S} \geq k \text{ , and} \tag{29}$$

$$s \leq \text{MSD} \tag{30}$$

where the MSD stands for Maximum Standard Deviation.

2. The Corrected *m*-Method

The corrected *m*-method for the double specification limit, with unknown standard deviation is almost like the single specification limit. The difference is that the percent defective becomes the combination of percent defectives with respect to both the upper as well as the lower specifications limits. The $\hat{p}_L$ can be estimated by using $Z_L = \dfrac{\overline{X} - L}{S}$, and $\hat{p}_U$ by using $Z_U = \dfrac{U - \overline{X}}{S}$. One would accept a sampled lot if $\hat{p}_L + \hat{p}_U \le m$, where the *m*-parameter is the same *m*-parameter that would be derived for a single-limit plan.

For example, suppose the DOT is treating the air void as an acceptance material characteristic for the Hot Mixed Asphalt (HMA), and the specification limit requires air void between 3% and 5% according to the Superpave N_{des} (Number of Design compaction gyration).

Eight tests (a combination of two lots) are performed and the test results are 4.3% , 4.7%, 3.7%, 3.8%, 3.3%, 3% , 5.2%, 4.1%, which yield the average value of 4.01% and the standard deviation of 0.72%.

Using the statistical charts developed by Duncan (A.J. Duncan 1986), one can get the maximum allowable percent defective of $m = 14\%$.

Then one can calculate the quality level and find the estimated percent defective,

$$Z_U = \frac{U - \overline{X}}{S} = \frac{5 - 4.0125}{0.722} = 1.3677, \text{The corresponding } \hat{p}_U = 7.73\%. \quad (31)$$

$$Z_L = \frac{\overline{X} - L}{S} = \frac{4.0125 - 3}{0.722} = 1.4024, \text{The corresponding } \hat{p}_L = 7.19\%. \quad (32)$$

Total percent defectives: 7.73% + 7.19% = 14.92% > m=14%.

Because the total percent defective is larger than the allowed maximum percent defective, one should reject this lot. However, if only the average of the air voids is considered, one may accept and perhaps even give the contractor a bonus because the average of 4.01% is at the mid point and well within the specifications range. This may appear to be counter intuitive; however, it must be noted that QC/QA methods are designed to account for the average as well as variability around the average.

Sampling Verification Techniques

Samples taken for the acceptance testing and verification testing come from the same population. Therefore, they should have the same distribution or statistical parameters if the testing equipment, testing methods, and recording employed by the Contractors and the DOT are the same. Two parameters are used to test this equality, namely: mean and variance. Depending on the verification sampling methods, one can treat the verification samples as dependent or independent from the acceptance testing samples. Therefore, the following combinations should be considered.

- Independent Sample; Test for Equality of Means
- Independent Sample; Test for Equality of Variances
- Dependent Sample; Test for Equality of Means
- Dependent Sample; Test for Equality of Variances

Independent Sample, Test for Equality of Means

In small data bases, such as HMA sample lot data, one needs to use a two-sample t-test to check if a difference exits between the acceptance test data and the verification test data. If the variances of the two sets of data are the same, one needs to test the following hypothesis:

- Null Hypothesis: the mean values of the verification data and the acceptance data are equal.
- Alternative Hypothesis: the mean values of the verification data and the acceptance data are not equal.

The following procedure can be applied to this test:

At first, one can compute a pooled estimate of the variance from the two independent samples:

$$s_p^2 = \frac{[(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2]}{n_1 + n_2 - 2} \qquad (33)$$

Then one can compute a t-statistic as follows

$$t = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{s_p^2}{n_1} + \frac{s_p^2}{n_2}}} \qquad (34)$$

Where $\overline{X}_1$ and $\overline{X}_2$ are the mean of the acceptance test results and the mean of the verification test results, respectively.

Finally, one needs to look for a t-value, $t_{\alpha(2),v}$, in a standard table where α is the significance level that one would like to use and v=$n_1 + n_2 - 2$, and compare the t-statistic with this $t_{\alpha(2),v}$. If $|t| \geq t_{\alpha(2),v}$, one can conclude that the verification data and that of the acceptance data are different.

Like all the other statistical test methods, this test requires some assumptions. The assumption for this test is that both acceptance and verification data come at random from normal populations with equal variances. When the variances are unequal, one can use an alternate procedure that is attributed to Smith (1936) and also know as "Welch's approximate t-statistic". The t-statistic is as follows:

$$t' = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \tag{35}$$

And the critical value is the Student's t with degrees of freedom of as follows

$$\nu = \frac{\left(\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}\right)^2}{\frac{\left(\frac{s_1^2}{n_1}\right)^2}{n_1 - 1} + \frac{\left(\frac{s_2^2}{n_2}\right)^2}{n_2 - 1}} \tag{36}$$

The procedure is more complicated when the variances of the two samples show a significant difference.

Independent Sample, Test for Equality of Variances

One of the purposes of the quality control is to reduce the variability of the materials and construction. So there should be a way to compare the variances of the acceptance data reported by the contractor with that of verification data reported by the DOT. The null hypothesis for this test is that the variance of the contractor's data is the same as the DOT's data. The procedure is called the variance ratio test, for which one would calculate the following (Zar, J. H., 1996)

$$F = \frac{s_1^2}{s_2^2} \tag{37}$$

or

$$F = \frac{s_2^2}{s_1^2} \tag{38}$$

One would choose whichever is that larger.
Then one can find the critical value F' in a standard table that corresponds to a certain significance level and degree of freedom. If F>F', the null hypothesis is rejected and one can conclude that the pair of variances are different. For example, this procedure was applied to Kentucky data, and the results are presented in Table 5.

Table 5. Paired t-Test Comparisons Between KYTC and Contractor Data

	Significant Difference at 5% Error Rate	
KY Testing Category	**Mean (p-value)**	**Standard Deviation (p-value)**
HMA-Air	NO (p-value = 0.462)	YES (p-value <0.0001)
HMA-Asphalt Content	NO (p-value = 0.851)	YES (p-value <0.0001)
HMA-VMA	NO (p-value = 0.083)	NO (p-value = 0.854)

Legend
HMA: Hot Mix Asphalt
VMA: Voids in the Mineral Aggregate

It is important to note that the variance ratio test is severely and adversely affected by sampling non-normal populations (Markowski and Markowski, 1990, p139). Typically, the DOT materials data bases do not always conform to a normal distribution. Therefore, one must be very careful when using this method.

Dependent Sample, Test for Equality of Means

The verification samples used currently are not totally independent. In the QC/QA specifications, often it is required that the verification test samples should be taken at the same place and the same time along with one of the contractor's acceptance test samples per lot. Therefore, the verification test is closely related to the contractor-performed acceptance tests. Although they are not split samples, they are paired samples from the statistical point of view.

The paired-sample t-test does not require the assumption of normality and equality of variances as it was necessary with the two-sample t-test. However, it does assume that the differences within each pair, d_j, come from a normally distributed population of differences (Zar, J.H., 1996). The equation for the paired-sample t-test is as follows:

$$t = \frac{\overline{D}}{s_D / \sqrt{n}} \qquad (39)$$

where

$$s_D = \sqrt{\frac{\sum\left(Di - \overline{D}\right)^2}{n-1}} \qquad (40)$$

and D_i is the difference of each pair of samples and $\overline{D}$ is the average of the differences.

Similarly, one can look for a t-value with a significance level α in a standard table, using df = n-1. If $|t| \geq t_{\alpha(2),n-1}$, the mean of the verification data and that of the acceptance data are different.

Dependent Sample, Test for Equality of Variances

The equation for testing the difference between variances of two correlated samples is complicated. A t-statistics can be computed using the following equation (Zar, J.H., 1996) as follows:

$$t = \frac{(F-1)\sqrt{n-2}}{2\sqrt{F(1-r^2)}} \qquad (41)$$

where F is variance ratio as described earlier, the term n is the sample size common to both samples, and r is the correlation coefficient. The degrees of freedom associated with this t-test are $n-2$ (Zar, J. H., 1996).

Dispute Resolution

1- Avoidance of Disputes: Both the DOT and the Contractor should do all in their capacity in order to avoid disputes. The following dispute avoidance steps ensure that all data are reliable, unbiased, and truly representative of the product quality.

- Ensuring personnel and laboratory facilities meet the specified certification requirements.
- Ensuring that all samples are obtained in accordance with the proper DOT specifications.
- Ensuring good communications of test results between parties occurs within the specified time limits
- Discussing all questions regarding the specifications, or sampling and testing procedures during the pre-construction, pre-paving, or any similar type of meeting to clarify any confusion.
- Resolving disputes at the lowest appropriate level of authority.

2- Levels of Dispute Resolutions: When the contractor's acceptance test results and the DOT's verification test results are not within the specified tolerances, and a dispute is therefore unavoidable, the following levels are the levels to resolve the dispute:

- **Project Level Dispute Resolution:** Both the Engineer and the contractor should attempt to determine the reason for the discrepancy at the project level by having testing personnel review previous tests and other possible contributing factors.
- **Materials Central Laboratory (MCL) Level:** The MCL should conduct further investigation on reviewing test data, checking both the engineer's and the contractor's calculations, inspecting of the instruments etc.
- **Third Party Resolution Level:** if the dispute is not resolved at the project or the DOT's central material laboratory level the department and the contractor should use a mutually agreed upon independent laboratory. The results of this independent laboratory should be final and binding.

Conclusions and Recommendations

State DOTs are increasingly transferring the responsibility for quality control to the contractors and the state agencies only performing quality assurance checks. The survey discovered that most DOTs are implementing contractor-performed quality control (CPQC) on hot mix asphalt (HMA) projects. The survey revealed that most DOTs are concerned about accepting the contractor-reported data for payment purposes. The study reveled that, at least in Kentucky, the majority of the contractor-reported data and side-by-side DOT verification data are similar. This means that there is not a significant difference between the contractor-reported data and the DOT-reported data. Furthermore, in some cases the contractor-reported data showed less variability, which suggests that contractor testing seems to be very accurate. Obviously, this is an encouraging finding and it is hoped to alleviate the trust issue. Contractor-performed quality control is the natural choice as more state agencies move toward a system of end-result type of specifications. This paper demonstrated that we can move beyond the issue of trust and focus our energies on improving quality. Obviously, future research is needed to address various issues surrounding the complete privatization of QC/QA activities. Additionally, there is the option of independent testing conducted by a third party. This entails hiring an independent testing consultant by the DOT and/or the contractor to perform QC/QA testing, which may provide an alternate solution, although, as noted earlier, further investigation is warranted.

The transition into a percent within limits (PWL) type of specifications may encounter opposition from the contracting industry. The contractors will have some difficulty with a specification that on one hand shows their data to be within the acceptable range, while on the other hand it exacts a penalty for the variability of their data. To address this issue, it might be a good idea for the DOT to somewhat temper their PWL-based pay factors at least for a transitionally period of time.

The assumption of normality is the basis for the PWL-based specifications. It is important to note that not all asphalt construction data fit the normal distribution pattern at all times. This is yet another reason for the DOT to temper their PWL-based pay factors.

Current FHWA policies require that contractor testing be witnessed by the DOT personnel if the contractor data are to be used for pay determination purposes. Additionally, Independent Assurance testing must be conducted by the DOT personnel or their contracted independent testing laboratory.

Finally, the key to a successful specification is its relationship to performance. All parties are better served when decisions with regard to acceptance, rejection, and payment are made based upon performance-based parameters. There would be less controversy surrounding pay factors if they are related to performance in a quantifiable manner.

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