274

National Cooperative Highway Research Program

NCHRP Synthesis 274

Methods to Achieve Rut-Resistant Durable Pavements

A Synthesis of Highway Practice

Transportation Research Board
National Research Council

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Synthesis of Highway Practice 274

Methods to Achieve Rut-Resistant Durable Pavements

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Subject Areas
Pavement Design,
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Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis report will be of interest to state, local, and federal agency pavement materials, design, and construction engineers, as well as pavement research engineers and scientists. Those with supervisory oversight for pavement programs will also find it of interest. It describes the current practice for methods to achieve rut-resistant durable pavements. The synthesis documents current experience with permanent deformation of asphalt pavements and identifies methods to improve performance. Information for the synthesis was collected by surveying U.S. and Canadian transportation agencies and by conducting a literature search using domestic and international sources.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board describes the extent of the rutting problem on the National Highway System, pavement mixture design issues, and the design of rut-resistant mixtures. In addition, alternate mixture types, including stone matrix asphalt and porous asphalt, are discussed, as well as international approaches to mixture design. Finally, the construction of rut-resistant mixtures, including the role of quality control and quality assurance methods, are discussed. A summary of permanent deformation causes and solutions is included in the Appendix.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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This study was managed by Stephen F. Maher, P.E., Senior Program Officer, who worked with the consultant, the Topic Panel, and the Project 20-5 Committee in the development and review of the report. Linda S. Mason was responsible for editing and production.

Crawford F. Jencks, Manager, National Cooperative Highway Research Program, assisted the NCHRP 20-5 staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance are appreciated.

METHODS TO ACHIEVE RUT-RESISTANT DURABLE PAVEMENTS

SUMMARY

Permanent deformation has been a concern of asphalt pavement engineers since early use of asphalt, and by the late 1970s and 1980s, it was acknowledged as a common problem. Significant changes have occurred in the last 10 years to both mix design criteria and quality acceptance procedures, in part because of rutting problems. This synthesis documents current experience with permanent deformation of asphalt pavements and identifies methods to improve performance.

A large number of research reports have been written on the subject of rutting. A literature search found 1,960 citations with rutting as a keyword from 1988 to 1996. Eleven hundred abstracts were selected, from which 300 reports were read. From this body of information 154 references were selected for the synthesis.

Both Marshall and Hveem methods of mix design were developed during the World War II era. During the period from the 1950s to the 1970s, Marshall stability criteria were increased and voids in the mineral aggregate (VMA) was added. Both methods remain for the most part unchanged. During 1984, in the face of widespread rutting problems, the Western Association of State Highway and Transportation Officials published a report recommending changes to improve resistance to rutting. Based on the WASHTO report the Federal Highway Administration issued Technical Advisory 5040.227 in 1989 recommending mix design standards.

During the last decade mix design technology has changed more rapidly. In the late 1980s, at the same time the FHWA advisory was issued, the National Cooperative Highway Research Program Project 9-5 developed a new mix design method called AAMAS (asphalt aggregate mixture analysis system). In turn, AAMAS became the starting point for the Strategic Highway Research Program, which developed another mix design method, Superpave. Superpave is currently undergoing evaluation and many agencies are actively implementing the new method.

Rutting is often the highest visibility distress but it is not the most prevalent distress. The greatest demand for rehabilitation funding comes from fatigue-cracked pavements. According to the survey done as part of this synthesis, more than one-third of rehabilitation funds, 38 percent, is spent correcting fatigue cracking. Despite perceptions, rutting accounts for only 17 percent of spending and moisture damage accounts for another 11 percent. The other 34 percent of spending was not specifically identified, although low-temperature cracking was mentioned by some respondents.

The current situation can be compared to published data from 1987. When viewed as a percentage of rehabilitation budgets, more funds are being spent on rutted pavements than in 1987. Part of the reason for increased proportion of funds is reduced tolerance to rutting than in 1987. The survey indicates that the average age of pavements that are rehabilitated because of rutting is 11 years. The average age of all asphalt pavements evaluated is greater than 11 years, indicating that pavements that fail in rutting are failing before reaching the average age.

An LTPP (long-term pavement performance) study of 453 pavement sections was done as part of SHRP. Most sections (> 50 percent) between the ages of 10 and 15 years accumulated ruts at a rate of less than one millimeter per year. Only seven percent accumulate ruts at a more accelerated rate of greater than two millimeters per year and most of those are asphalt over granular type structures, suggesting that subsidence of lower layers is contributing to rutting. Asphalt overlays of concrete pavements, which are often considered to be the most severe situation for rutting, have only two percent of sections that rutted more than two millimeters per year.

Studies indicate that the asphalt binder property is linked to rut resistance. The Superpave asphalt binder specification specifies minimum asphalt binder stiffness at high temperature to control the binder's contribution to rutting.

Numerous studies indicate the benefit of adding a modifier to the asphalt. Many different materials are used to modify asphalt. The modifier role is to stiffen the binder, particularly at high temperatures when the base asphalt is becoming soft. Hard asphalt binders will perform as well or better than modified binders will if the high temperature stiffness is high enough. However, hard, unmodified binders sacrifice low-temperature properties to achieve high-temperature stiffness. Most modified asphalt binders are able to maintain reasonable low-temperature properties without sacrificing high-temperature properties.

Aggregates have a large influence on permanent deformation properties. Agencies have recognized the desirability of coarse aggregate with crushed faces and have been increasing crush count specifications. Natural sands have been identified as a cause of rutting. Prior to Superpave, no tests were being routinely used to measure the strength of sand. Inter-particle friction as measured by the fine aggregate angularity test was the most influential property controlling rutting. Other studies show that coarse aggregate fractured faces and fine aggregate angularity when considered together are linked to rutting.

Superpave promised a performance-based test or tests that could be used to identify rutting resistance in the laboratory. These tests are not yet ready for implementation. Other empirical tests including rut testers are being evaluated. Rut testers may offer some promise. Georgia has developed and uses a rut tester. Colorado has evaluated the Hamburg rut tester and adapted it to Colorado's climate. Disturbingly, rut testers indicated sections in the WesTrack experiment to be acceptable when the actual performance ranged from acceptable to very poor. This may have resulted from the parameters used for pavement slab thickness.

Two alternate types of mixture with high resistance to rutting are used in North America. Stone matrix asphalt is composed of a coarse aggregate skeleton bound together with mastic containing a high filler content and fine aggregate. The mastic, which is stiffened by the filler, is not intended to carry any load. It locks the coarse aggregate into place. Open-graded mixtures, also known as porous mixes, are used by some agencies. The mixtures, used for safety and comfort as well as rut resistance, remove water from the pavement surface, decreasing the risk of hydroplaning and increasing driver visibility by reducing vehicle spray.

Internationally, most countries use the Marshall method of mix design. France, however, developed a method in the 1970s that produces mixtures with high rut resistance. French mixtures are designed with high stone content, stiff asphalt binders and high filler content. Portions of French mix philosophy are contained in Superpave but the French method is not currently used in North America. The province of Quebec uses a hybrid of Superpave and French mix design technology.

Construction methods affect asphalt pavement rut resistance. Historically, asphalt content and gradation were used to control mixture production. Volumetric properties were either not measured or were just monitored. Usually, mixtures produced at the design asphalt content and gradation have one to two percent less air voids than the laboratory design. Since air voids of laboratory compacted plant mix control rutting, agencies are shifting from asphalt content and gradation to volumetric properties for mixture acceptance at the hot-mix plant. If air voids drop below three percent there is risk of rutting.

Density as compacted on the road also affects rutting but less so than air voids of the laboratory compacted plant mix. Rutting increases slightly as inplace air voids increase. The greatest detriment of high inplace air voids is reduced durability caused by early aging of the asphalt binder or damage from moisture infiltration.

Innovative contracting is being investigated. Pavement warranties are being used successfully on a trial basis. A 5-year term has been the most common length of warranty. Threshold values are set such that a 15-year life is expected. If performance fails to meet the warranty, the pavement is corrected.

CHAPTER ONE

INTRODUCTION

SYNTHESIS OBJECTIVES

Premature rutting of asphalt pavements remains a concern in North America. In the last decade several state and federal agencies have performed research projects and developed specifications to address rutting. In 1989 an American Association of State Highway and Transportation Officials (AASHTO) Joint Task Force on Rutting developed criteria and recommendations for rut-resistant pavements. Today, the Superpave system is being implemented by states to improve asphalt pavement performance.

Other activities to develop rut-resistant pavements are occurring. Stone matrix asphalt (SMA), another rut-resistant mixture, was introduced to North America around 1990 and is being used by several agencies. The National Cooperative Highway Research Program (NCHRP) recently completed a study (project 4–18) of large stone asphalt mixtures that promise increased rut resistance.

Several states have recently made significant changes in their specifications to minimize rutting. In the past, some specification changes to address rutting have caused other pavement performance problems, such as moisture damage, that need to be recognized and avoided. Any changes to specifications should be balanced to ensure that permanent deformation problems are not traded for durability problems.

The objectives of this synthesis study are to

1. Evaluate the severity of permanent deformation and moisture damage on highways. Permanent deformation is viewed as the major distress encountered on the highway network. The first step toward suggested means of improving performance is to quantify the current situation.
2. Identify causes of rutting. Rutting can be caused by more than one condition. A preliminary step toward increasing rut resistance is to identify causes of rutting.
3. Identify solutions for rutting. Solutions for rutting may be identified at several points during the design and construction of pavements. Using knowledge gained about the severity of rutting and the sources of the distress, a set of balanced proposals can be made to offer solutions.

This synthesis of current practice and recent research findings is intended to aid owner agencies and the construction industry in developing high-performance rut-resistant durable pavements.

ORGANIZATION OF SYNTHESIS

This synthesis is organized according to the steps used to construct asphalt pavements. The first chapter considers the size of the rutting problem on the main highways (the National Highway System) today and compares the findings to a similar survey done in 1987. The second chapter discusses mixture design issues including the mechanisms of pavement rutting. The third chapter discusses the design of rut-resistant mixtures including properties of the constituent materials and performance indicator tests including rut testers. Alternate mixture types including stone matrix asphalt and porous asphalt are also discussed, as are international approaches to mix design. The fourth chapter discusses the construction of rut-resistant mixtures including the role of quality control and quality assurance methods. Innovative contracting methods such as pavement warranties are also discussed.

BACKGROUND

Permanent deformation of asphalt mixture that leads to channels in the wheelpaths has been a concern of pavement engineers since asphalt pavements were originally used. The Marshall mix design method was developed partially in response to permanent deformation in airfields used by bombers in World War II. In the late 1970s and throughout the 1980s, permanent deformation occurred in many pavements in North America. For years asphalt mixture design and acceptance methods had remained relatively unchanged and increasing truck volumes exceeded the ability of marginal mixtures to resist rutting. In response, significant changes have occurred in the last 10 years, in part because of rutting problems (*1*).

In the last 10 years several changes were made to the Marshall method of mix design. However, in the same time period, two new methods of mix design were developed. NCHRP developed the Asphalt Aggregate Mixture Analysis System in 1989 and in 1993 SHRP developed the Superpave mixture design method.

Monitoring of mixtures during construction has also changed. Mixtures are being accepted based on air voids and asphalt content. Aggregate gradation is becoming a quality control parameter only.

Several states have made significant changes in their specifications to minimize rutting. Advances against permanent deformation have occurred since the 1980s, yet a level of concern remains. This synthesis will aid owner agencies and the construction industry to develop pavements that are resistant to permanent deformation and at the same time remain durable.

DEFINING THE PROBLEM

Permanent Deformation

Rutting of asphalt is a concern dating back to the earliest use of asphalt pavements. Today, concern remains despite recent advances in mix design technology and changes in construction specifications. For perspective, a comparison of the current situation is made to a 1987 survey (*2*).

In 1987, AASHTO prepared a report on the status of rutting based on questionnaire responses from 48 states and four Canadian provinces. The study focused on permanent deformation only and considers the entire highway network maintained by the agency.

The 1997 data is based on the questionnaire for this synthesis. It focuses only on the high-volume roads that are part of the National Highway System. Forty-one states and six Canadian provinces responded to this questionnaire.

In 1997, 16 percent of respondents believe rutting is not a problem, the same as the 1987 survey. However, 22 percent of the 1987 respondents believe rutting was a major problem and only 9 percent of the 1997 respondents feel the same way. Therefore rutting is not believed to be as large a problem as it was 10 years ago.

Another indication of the size of the problem can be obtained by considering the percentage of each year's rehabilitation budget that is spent on rutted pavements. Figure 1 shows a comparison of rehabilitation spending for rutted pavements in 1987 and 1997. In 1987, 68 percent of the respondents spent less than 10 percent of their entire rehabilitation budget on rutted pavements and 22 percent were spending between 10 and 30 percent. In 1997, 51 percent of the agencies were spending less than 10 percent of their NHS rehabilitation budget on rutted pavements and 32 percent were spending between 10 and 30 percent of the budget. The surveys indicate a greater percentage of rehabilitation cost is spent for rutted pavement. The comparison must be qualified, however, because the 1997 data is for NHS roads only. These roads carry a greater share of traffic and should have more rutting problems than the entire network.

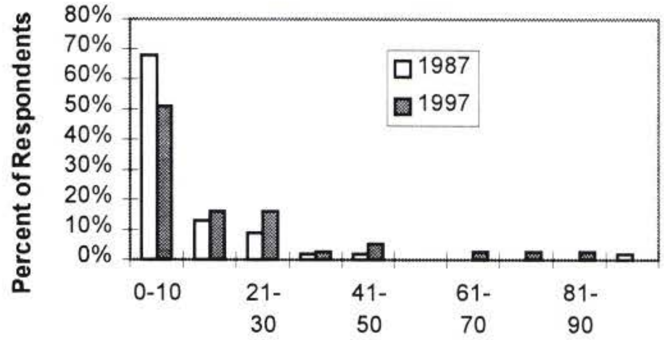

FIGURE 1 Comparison of annual rehabilitation budget spending for rutted pavements, 1987 and 1997.

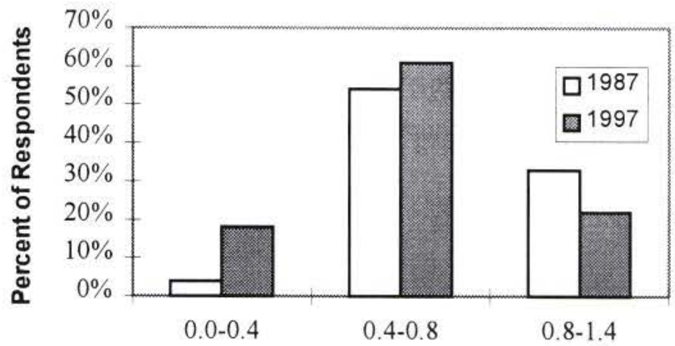

FIGURE 2 Depth of rut that triggers rehabilitation.

The depth of rut that triggers rehabilitation changed from 1987 to 1997, as shown in Figure 2. In 1987 only 4 percent of agencies would trigger rehabilitation for rut depths of less than 0.4 inches. In 1997, 18 percent of the agencies indicated that rehabilitation would be triggered with rut depths of less than 0.4 inches. In 1987, 33 percent of the agencies would tolerate a rut depth of more than 0.8 inches before triggering rehabilitation, compared with 22 percent in 1997.

In the 1997 survey, the age of pavements rehabilitated because of permanent deformation was collected. Results are shown in Figure 3. Less than one percent of pavements are reported to fail in permanent deformation within the first year. Fourteen percent fail before the age of 5 years. Many pavements, 42 percent, fail at ages between 6 and 10 years. The average age of pavements experiencing permanent deformation is 11 years.

In summary, a comparison of 1997 data with 1987 data shows that fewer agencies perceive rutting to be a major problem. Yet at the same time, agencies have reduced

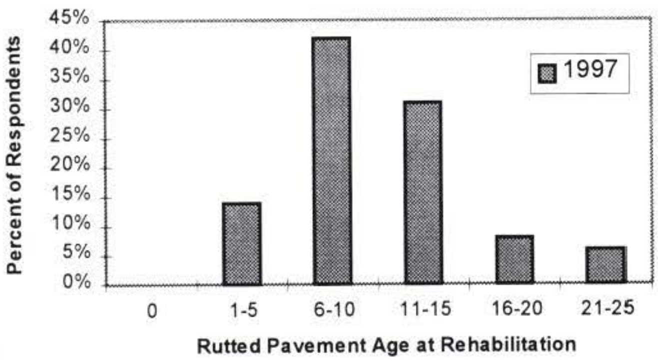

FIGURE 3 Age of pavements rehabilitated for permanent deformation.

their tolerance to rutting, react sooner to rehabilitate rutted pavements and have increased the amount of resources spent on rutted pavements as a percent of rehabilitation budgets.

Moisture Damage

Moisture damage refers to the loss of adhesion between the asphalt and aggregate induced by the presence of water. Moisture damage weakens the asphalt mixture leading to rutting, cracking, and, in extreme cases, disintegration of the pavement. Based on the agency questionnaire results, the cost of moisture damage to pavements as a percentage of their rehabilitation budget is shown in Figure 4. Many agencies are not reporting pavement failure caused by moisture damage. Forty-four percent of the agencies report no expenditure for moisture-damaged pavements and 29 percent report that less than 10 percent of the rehabilitation budget is required. Only 12 percent of agencies report a significant amount, 10 to 20 percent of the budget, is spent to repair moisture-damaged pavement. In general, moisture damage does not demand as great a percentage of the budget for rehabilitation as permanent deformation does.

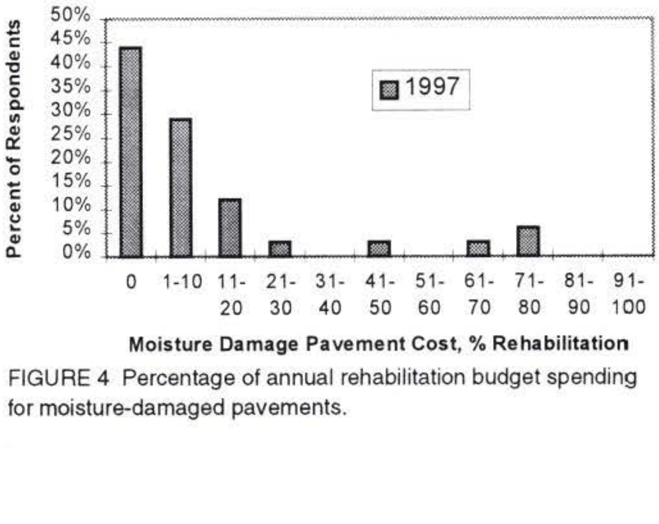

FIGURE 4 Percentage of annual rehabilitation budget spending for moisture-damaged pavements.

Fatigue Cracking

Repeated traffic loads induce fatigue cracking. Inadequate pavement thickness allows the asphalt layer to flex excessively and generate large tensile stresses that cause early cracking. Premature aging of the asphalt layer, usually a result of low asphalt content or air permeable mixture, makes the mixture intolerant of deflection and leads to premature fatigue cracking.

Agency reports of rehabilitation funding spent for fatigue-cracked pavements are summarized in Figure 5. Fatigue cracking is quite predominant. Only 6 percent of the agencies reported that fatigue cracking is not present. A large number of agencies report fatigue cracking as the predominant distress encountered. Thirty-nine percent of the agencies report that more than 50 percent of their rehabilitation budget is required for fatigue-cracked pavements.

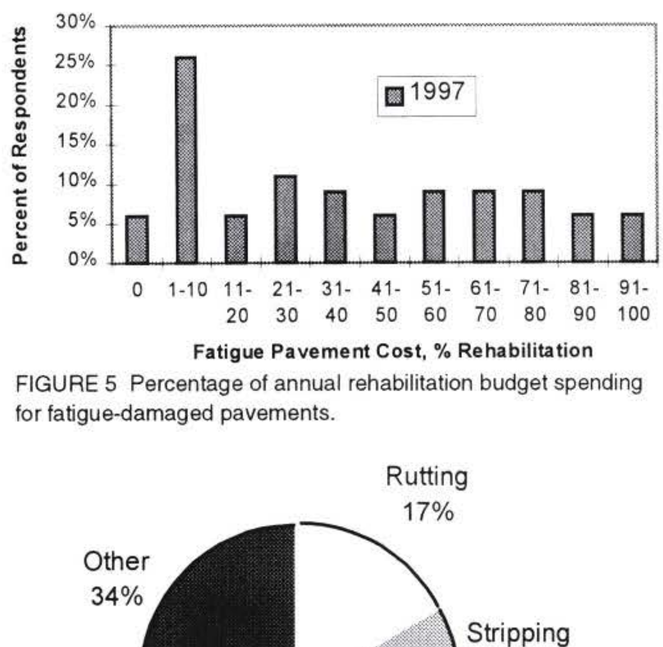

FIGURE 5 Percentage of annual rehabilitation budget spending for fatigue-damaged pavements.

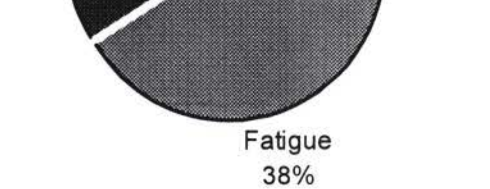

FIGURE 6 Distribution of rehabilitation budget for different distresses.

Distress Summary

A summary of asphalt pavement rehabilitation funds spent for different distresses is shown in Figure 6. The questionnaire focused on permanent deformation, stripping, and

TABLE 1

RUT DEPTHS OF GENERAL PAVEMENT STUDY ASPHALT PAVEMENTS BETWEEN AGES 10 AND 15 YEARS AT TIME OF FIRST MEASUREMENT *(3)*

	Rut Depth (mm)		
Pavement Type	Mean	Minimum	Maximum
Full depth asphalt pavement	8	3	25
Hot mix asphalt on cement stabilized base	5	2	10
Hot mix asphalt on granular	8	2	21
Hot mix asphalt overlay of existing hot mix asphalt	9	4	18
Hot mix asphalt overlay of portland cement concrete	8	3	16
Rut data for AC/PCC for 5- to 10-year old pavements	8	3	16

TABLE 2

RATE OF PERMANENT DEFORMATION DEVELOPMENT OF ASPHALT PAVEMENTS *(3)*

	Percent					
Rutting Rate	FDAC[1]	AC/Granular[2]	AC/PCTB[3]	AC/AC[4]	AC/PCC[5]	Combined
Less than 1 mm/year	75	57	53	49	54	57
1 to 2 mm/year	4	12	9	13	15	11
More than 2 mm/year	0	13	2	4	2	7
Decrease[6]	15	8	13	16	4	11
Increase and decrease[7]	6	10	23	18	25	14
Number of Sections	48	208	75	94	28	453

Pavement types: [1]Full depth asphalt pavement; [2]Hot mix asphalt on granular; [3]Hot mix asphalt on cement stabilized base; [4]Hot mix asphalt overlay of existing hot mix asphalt; [5]Hot mix asphalt overlay of portland cement concrete; [6]Decrease defined as a pavement that showed a distinct decrease in rut depth; and [7]Increase and decrease defined as a pavement that show both an increase and decrease in measured rutting.

fatigue cracking. These three distresses are the main objective of this synthesis. The category of "Other" did not specifically identify distresses. Several states indicated low-temperature cracking as a major cause of asphalt pavement rehabilitation.

Thirty-seven agencies were able to provide estimates of the number of miles rehabilitated because of rutting, fatigue cracking, and moisture damage. According to the survey results, rutting does not receive the largest share of rehabilitation spending. Fatigue cracking received the greatest portion of the rehabilitation budget.

Comparison to General Pavement Studies

The LTPP study has monitored many existing in-service pavement sections since 1989 as part of the general pavement studies (GPS) experiment. In 1996, an evaluation of permanent deformation in GPS sections indicated that the rate of permanent deformation development is less severe than had been anticipated (*3*).

Descriptive statistics are shown in Table 1 for pavement between 10 and 15 years old at the time of first measurement. The values shown include mean, minimum, and maximum rut depths. Mean rut depth for all pavements, regardless of the underlying layer type, is approximately 8 mm. The data show that the range of rut depths is similar for all pavement types, indicating the behavior is the same regardless of the underlying layer type. Rut depths generally range from 3 to 20 mm. In each data set, there is considerable scatter.

Table 2 shows the rate of change of rutting as observed by comparing condition surveys at different points in time. A nominal rate of rutting was selected as less than 1-mm rut depth increase per year. Moderate was selected as between 1 and 2 mm per year and high is more than 2 mm per year. Most of the sections, 56 percent of the entire group, are rutting at a rate of less than 1 mm per year. Only 7 percent of the sections are rutting at a rate of more than 2 mm per year.

Some section measurements actually had decreased rut depth in subsequent condition surveys. No opinion is given whether the decrease is real or caused by measurement error. Sections that showed a decrease in rut depth and sections that showed both an increase and a decrease at different times are listed separately.

The data show that pavements exhibiting a high rate of rut depth development are not very common. Thirteen percent of asphalt pavements built over granular base experience more than 2 mm per year, the highest category of rutting rate. Thick asphalt pavements, either full-depth

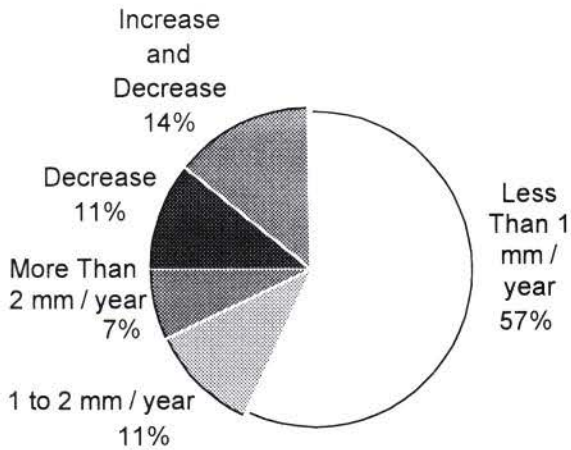

FIGURE 7 Distribution of pavement rutting rate for general pavement studies.

asphalt concrete or overlays of asphalt pavements, have small percentages of pavements with high rates of rutting. In addition, asphalt overlays of portland cement concrete or asphalt pavements on portland cement treated base have low rutting rates. Figure 7 shows the percentage of sections showing different rates of rutting for the entire sample of 453 sections.

QUESTIONNAIRE

To identify current practices a questionnaire was developed and distributed to the DOTs in 50 states, 10 Canadian provinces, and two Canadian territories to obtain information on

- The amounts of premature rutting, fatigue cracking, and stripping,
- The age of prematurely rutted pavements,
- Asphalt and aggregate properties used in asphalt mixtures,
- Mix design practices and criteria,
- Quality assurance items and criteria,
- Density measurement, and
- Payment factors.

Table 3 summarizes the responses to the questionnaire.

TABLE 3
SUMMARY OF QUESTIONNAIRE RESPONSES FOR SYNTHESIS

Departments of Transportation	Number Sent	Number Returned	Percent Responding
U.S. States	50	41	82
Canadian Provinces	10	5	50
Canadian Territories	2	0	0
Total	62	46	74

CHAPTER TWO

MIXTURE DESIGN ISSUES

HISTORICAL CONTEXT

Premature rutting of asphalt pavement has been a concern since the introduction of asphalt as a road building material. Early approaches to mix design were based on experience with asphalt content based on gradation. Later mix design methods began to develop. The Hubbard-Field method began in the early 1920s and was followed some 20 years later by the Marshall method.

The Hubbard-Field method of mixture design was one of the first methods to evaluate air voids and voids in the mineral aggregate (*4*). Specimens were compacted with a drop hammer in a 4-inch mold, a method borrowed from soil evaluation. The Hubbard-Field method used a stability test to measure resistance to deformation. The 4-inch specimen was extruded at 60°C through a ring slightly smaller than the specimen diameter. The force required to extrude the specimen was called the Hubbard-Field Stability. The Hubbard-Field method of mixture design worked acceptably well for fine-graded, small aggregate mixtures but less so for larger sized or coarsely graded mixtures.

Following World War II, the Marshall method of mixture design was adapted for highway use. The method, originally developed in the 1930s, used a similar approach to Hubbard-Field. Impact compaction was used but the hammer face completely covered the specimen. Volumetric properties, predominantly air voids and voids filled with asphalt, were calculated. A stability test loaded the specimen diametrically using two collars that were almost semi-circular. The maximum sustained load at 60°C is referred to as Marshall stability. In 1962, voids in the mineral aggregate (VMA) was added to the specification to address durability concerns caused by low asphalt content (*5*). Except for changes in specification values, the method has remained virtually unchanged since the late 1940s (*6*).

The Hveem method of mix design was developed about the same time as the Marshall design. A kneading compactor was developed that used a tamping foot to compact the mixture. The mixture is evaluated using Hveem stability, a pseudo-triaxial test. Hveem mix design gained acceptance predominantly in the western states. Most states adopted Marshall mix design.

The years following World War II witnessed strong growth in the national economy that required increased movement of goods by highway. In 1956, the Interstate Defense Highway System was passed, creating an explosive increase in new highway construction that lasted until the early 1970s. The AASHO road test, which occurred in 1959 and 1960, was a full-scale structural design experiment in which loaded trucks were applied to newly constructed pavements. Marshall designed mixtures were subjected to accelerated loading providing confirmation that the mix design method was suitable for trucks then current.

In the years that followed, asphalt pavement rutting occurred but not on a wide-scale basis. Deficiencies in materials or construction were often identified as the cause. However, by the late 1970s and particularly in the early 1980s, rutting became a more common problem in North America. Investigative studies often found materials and construction were not the cause. Attention began to be focused on the mix design method.

The Western Association of State Highway and Transportation Officials (WASHTO) issued a report in 1984 recommending changes to material requirements and mix design specifications to improve resistance to rutting (*7*). In 1989, the Federal Highway Administration issued Technical Advisory 5040.227 recommending standards based on the WASHTO report (*8*).

The need for changes in asphalt mixture design was recognized. In 1987, NCHRP began the Asphalt Aggregate Mixture Analysis System (AAMAS) Project 9-5, a 2-year project to develop a new mix design method including a new laboratory compaction method, evaluation of volumetric properties, and development of tests to identify rutting, fatigue cracking, and low-temperature cracking resistance (*9*).

The Strategic Highway Research Program (SHRP) also started in 1987. The NCHRP AAMAS was intended to provide a starting point for a SHRP mix design method. When completed in 1993, the new method, Superpave, contained a new volumetric mix design method complete with performance-based prediction tests and models for permanent deformation, fatigue cracking and low-temperature cracking. The performance prediction models were judged not to be ready for implementation and are currently undergoing additional development. The performance models will not be completed until some time after the year 2000. Superpave volumetric mixture design is currently being implemented by several states and is under evaluation by several others.

The importance of construction quality control has been emphasized in the last 10 years. The FHWA has been demonstrating and encouraging the use of mixture volumetric properties as the basis for control and acceptance (*10*). A link between volumetric properties of the hot mix and performance is being recognized. As a result, quality acceptance is shifting from gradation to volumetric properties.

TYPES OF RUTTING

Rutting, often referred to as permanent deformation, is visible as a depressed channel in the wheelpaths of a roadway. Rutting caused by densification of material in the wheelpaths is minor, potentially contributing to less than 5 mm rut depth. Other causes of rutting include subsidence of the pavement over yielding lower layers; loss of surface material within the wheelpath; and plastic shear deformations within the asphalt layer (*11*).

Weak and yielding layers below the pavement structure subside from traffic loading. If the asphalt layer is sufficiently flexible, it will deform to match to the lower layers of the pavement. Subsidence ruts tend to be fairly wide, 750 to 1000 mm wide, with a shallow sloping saucer shape cross section and no cracking. A trench cut through the deformed pavement will indicate that the pavement thickness has remained constant and that the lower layers, either unbound granular materials or subgrade, have deformed. If the pavement structure is too stiff to deform, fatigue cracking will occur across the entire width of the wheelpaths. Sometimes the cross section could resemble a punching type failure with the broken pieces pushed downwards. Such ruts tend to have sharp slope with broken edges on both sides of the wheelpath.

The second type of rutting may be caused by loss of material from the wheelpaths. Material loss may be caused by wear of the aggregate particles if conditions are abrasive or the aggregates are soft. Rutting caused by studded tires is an example of abrasive conditions. These ruts are continuous with exposed aggregate showing in the wheelpath. The surface is very noisy, caused by knobs of larger, more resistant aggregate particles that stick up above the surrounding mixture.

Rutting from loss of material in the wheelpath may also be caused by raveling—dislodgment of individual particles under the action of tires. Raveling may be a result of low compaction, low asphalt content, or excessive aging of the asphalt binder. The net result is loss of adhesion. Rolling tires dislodge particles and a rut is formed. Ruts caused by raveling tend to be dry and ragged looking. They tend to be very non-uniform. In one location the mixture may ravel through the entire layer thickness while a more resistant mixture nearby will be nearly intact. As a result potholes will sometimes form and the ride is very rough.

The third type of rutting is caused by shear deformation within the asphalt mixture. Material is displaced laterally along shear planes within the mixture. The rut is formed by depression of the loaded area in the wheelpath and by ridges of mixture, which are formed along both edges of the wheelpath. In the wheelpath, the surface is usually smooth and asphalt rich. The bottom of the rut may be smooth and saucer shaped. Often, one or more small ridges form at the bottom of the rut from the space between dual tires. Ruts formed by shear deformation tend to change gradually along the road, therefore the pavement continues to have a good ride despite being rutted.

Shear deformation is caused by a lack of resistance to shear loads generated in the mixture by application of a vertical load on the surface. Often the lack of shear resistance can be caused by an imbalance in the amount of asphalt in the mixture. Shear weakness can also occur from moisture damage within the mixture or a weak aggregate skeleton.

Rutting caused by shear deformation is the main focus of this synthesis. Some consideration will also be made of rutting from raveling and stripping, both of which are caused by poor mixture durability.

RESISTANCE TO RUTTING

This section addresses the mechanism that provides resistance to permanent deformation. Hot-mix asphalt is a composite material composed of hard linear-elastic aggregate particles and visco-elastic, visco-plastic asphalt binder. The aggregate skeleton is best suited to carry the traffic loads. Ideally it should be capable of supporting loads applied to the mixture. The asphalt binder is not well-suited to carrying load since it will flow with applied load and time.

The role of the asphalt binder is to act as glue and keep the aggregate skeleton together. Without asphalt binder the aggregate skeleton will not remain in place when subjected to traffic. The asphalt binder must be strong enough to prevent particles from being dislodged by rolling tires. The binder must also be strong enough to resist shear loads generated at the aggregate contact points that exceed friction between the aggregate particles. If the binder cannot hold the particles in place, the rocks will move and the skeleton will collapse, that is, the skeleton will compact to a denser configuration.

Linear Elastic Behavior

If a load is placed on a steel beam, the beam will deflect. When the load is removed, the beam will return to its original position. If the load is doubled, the beam will deflect twice as much and, assuming that the load is not large enough to cause failure, the beam will return back to its original shape after the load is removed.

The reaction of a material to load is controlled by engineering properties. Steel for example, can be described using linear elastic behavior. If the steel modulus is known, stresses and deflections from loads can be calculated in steel structures. At low temperature or under fast loading conditions when the asphalt mixture is very stiff, it will have linear elastic behavior. At warmer temperatures or slower loading rates the behavior cannot be described by simple linear elastic behavior. According to SHRP researchers, asphalt mixture must be described using non-linear elastic and visco-elastic behavior (*12*).

Non-Linear Elastic Behavior

Asphalt mixture has a non-linear elastic response as a component of its behavior. If load is placed on an asphalt mixture, it will deflect. If the pavement is completely non-linear elastic the mixture returns to the original position after the load is removed. If the load is doubled, so long as it is not enough to cause failure, the mixture will deflect more but the deflection will not double. Under load, the aggregate skeleton is compressed together. Additional particles come into play and the mixture becomes stronger. When additional load is applied the additional deflection will be less than deflection from the original load because it is now stiffer, again assuming that so much load had not been placed as to cause failure.

Viscous Behavior

Viscous behavior in the mixture comes from the asphalt binder (*13*). At a constant temperature, stiffness will depend on the rate of loading. The faster a load is applied, the stiffer the mixture will react. The slower a load is applied, the softer the mixture will react.

Viscous materials change stiffness depending on how fast the load is applied. Although usually considered a liquid, water is an example of a viscous fluid. If a swimmer falls into the water from the side of a swimming pool, the water has very little stiffness and moves out of the way of the oncoming swimmer. If the same swimmer jumps from a 10-meter diving platform and does a "belly flop" the water reacts very stiffly. The diver is travelling approximately 70 km per hour and at that loading rate the water is very stiff indeed.

So viscous materials change stiffness with a change in loading rate. In addition, there is a delay from the time a load is applied until deformation occurs. This lag is called the phase angle.

To illustrate the difference between elastic and viscous behavior, consider Figure 8, which shows a cyclic applied load and the resulting deformation. In a purely elastic material, load and deformation are in phase, that is, maximum deformation occurs at the same time as maximum load. The phase angle between load and deformation is zero.

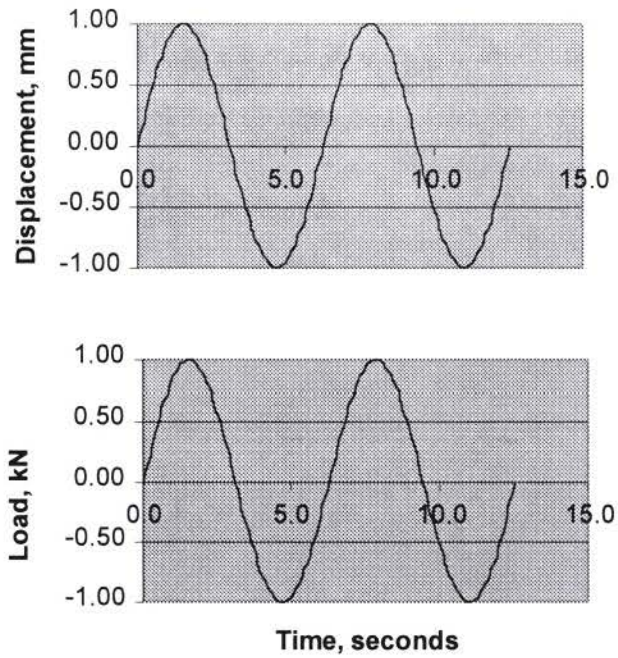

FIGURE 8 Deformation and load of a linear elastic material.

In a purely viscous material, maximum load occurs when deformation is changing most rapidly. If a sinusoidal load is applied, as shown in Figure 9, deformation is changing fastest when displacement is zero. Load is at maximum when the deformation is zero. The phase angle between load and deformation is 90 degrees.

As an example of phase angle, water will again illustrate the point. Consider a rower moving an oar back and forth in the water. Assume the boat is tied to the dock and cannot move forward or backward. When the oar tip is at the back of the boat it is furthest away. It is stopped and turning around. The force on the oar when it is stopped and just sitting in the water is zero. That is, when deformation is maximum, the load is zero. Now the boater pulls the oar back and the tip starts to move forward. As the oar moves past the boater toward the front of the boat it is travelling at maximum speed. The force on the paddle

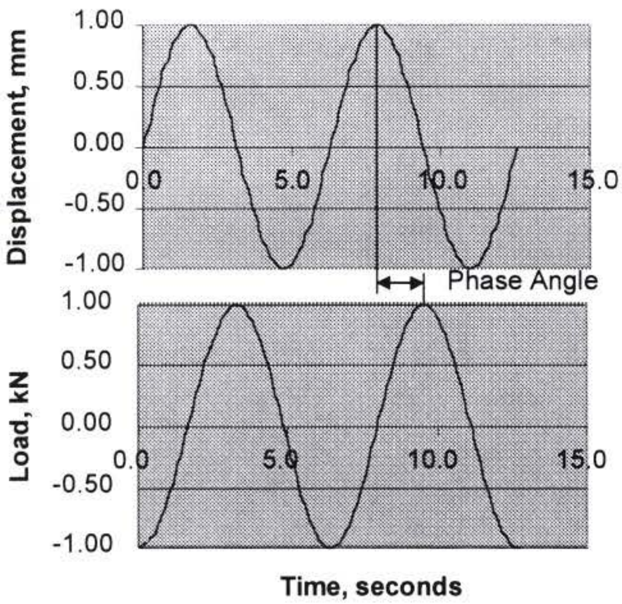

FIGURE 9 Deformation and load of a visco-elastic material.

from the water is also at a maximum. So when the deformation is zero, that is when the paddle is beside the boater, the force is at a maximum. It is at a maximum because it is travelling at its fastest speed. As the oar continues to move to the front of the boat its speed slows and returns to zero. The force also returns to zero.

Asphalt binder is a visco-elastic material, neither totally viscous nor completely elastic. Asphalt behavior is partly viscous and partly elastic.

The long arrow in Figure 10 represents asphalt binder shear stiffness, G*, also known as complex modulus. The horizontal arrow represents the elastic component of stiffness; the vertical arrow represents the viscous component of stiffness. The angle between the elastic stiffness and the complex modulus is known as the phase angle.

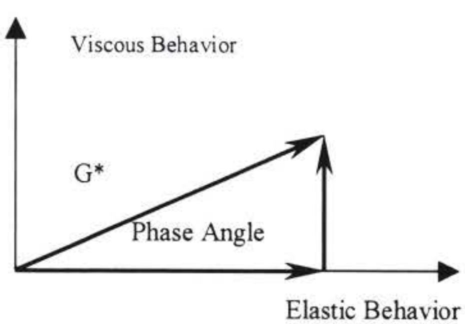

FIGURE 10 Complex stiffness of asphalt binder and phase angle.

At high temperatures, 150°C, asphalt binder will be almost completely viscous with very little elastic behavior. At cold temperatures, say -40°C, asphalt binder will be a

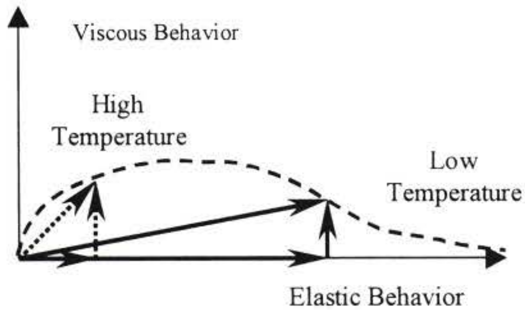

FIGURE 11 Change of asphalt binder stiffness with temperature change.

brittle solid, almost entirely elastic. At normal service temperatures asphalt binder is partly viscous and partly elastic.

At low service temperatures, the asphalt binder stiffness is high and the phase angle is low, as shown by the solid arrows in Figure 11. At high service temperature, stiffness is low and the phase angle is high, as shown by the dashed arrows.

Combined Behavior

Asphalt mixtures have non-linear elastic, linear elastic, and viscous behavior. Depending on the mixture temperature and rate of loading, the relative proportion of each behavior will change. Conceptually, for a typical highway pavement the transition of properties with temperature is shown in Figure 12. At low temperatures the mixture behavior is elastic and most is linear elastic. As temperature increases, the linear elastic behavior becomes less pronounced and non-linear elastic behavior increases. As the temperature increases further, elastic behavior continues to decrease and viscous behavior starts to appear.

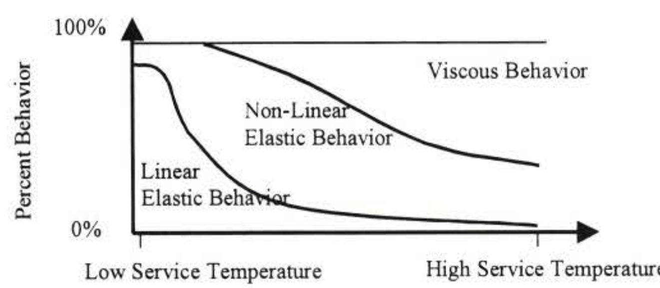

FIGURE 12 Conceptual change in asphalt mixture behavior with change in temperature.

In winter when temperatures are low, no rutting occurs because the mixture is elastic and all energy input into the pavement by traffic is returned in elastic rebound. For pavements with adequate structural thickness the stresses generated by traffic do not approach failure stress and minor fatigue damage occurs. Therefore fatigue damage is also not a problem at low temperature.

At mid-range temperature, typical in spring or fall, the mixture retains much elastic behavior. Viscous behavior occurs but the stiffness remains high enough to limit rutting. Under these conditions fatigue damage does occur.

At high temperature in the summer, mixture stiffness decreases and strains imposed by traffic increase. A portion of strain within the viscous component is not recovered and permanent deformation occurs. Usually the magnitude of unrecovered strain is low enough that rutting is not a problem. If the strains are excessively high or the mix behavior is dominated by viscous behavior, permanent deformation can be excessive.

Transition from one behavior to another for a specific mixture is determined by the material. The transition temperature can be shifted by the mix design and construction technique used. Ideally, at high service temperatures elastic behavior should be sufficient to prevent rutting.

The aggregate skeleton contributes to elastic behavior. Particle shape and texture of the aggregate will influence the elastic properties. The amount of compaction during construction will influence how tightly the particles are compacted and influence elastic behavior. If the aggregate skeleton is well compacted, elastic properties will be increased. If it is loosely compacted, elastic properties will not be as well developed. The elastic component of binder stiffness also contributes to elastic behavior. An asphalt binder with a high elastic stiffness will increase the elastic behavior of the mixture. A poorly crushed aggregate with smooth rounded surfaces will decrease elastic behavior and produce a pavement that will deform under load and not return to the original shape.

Asphalt binder influences viscous properties of the mixture. Increased amounts of asphalt binder in the mixture will increase viscous behavior. The grade of asphalt binder also influences viscous behavior. A stiffer binder, say PG 70-22 instead of PG 64-22 has more elastic behavior at a given temperature. Air voids influence how much the asphalt binder contributes to the mixture properties. At air void contents above 3 percent, the elastic properties of the aggregate skeleton are maintained. At low voids, the viscous property of the binder becomes more predominant as the elastic behavior of the aggregate skeleton is decreased.

RELATIONSHIP OF RUTTING AND DURABILITY

Durability of an asphalt mixture is generally defined as the ability to resist changes that are detrimental to long-term performance of the pavement. Two durability problems are predominant. Aging, defined as hardening of the asphalt binder, causes the mixture to become brittle. Moisture damage that weakens the asphalt aggregate bond causes a weakening and softening of the asphalt mixture. Aging and moisture damage are not regarded as distresses in the same fashion as permanent deformation and fatigue cracking. Instead, they are factors that change the engineering properties of the mixture. For example, if the HMA ages prematurely, fatigue cracking or raveling may occur (*14*). Aging caused the pavement failure indirectly.

Generally, design of hot-mix asphalt has been viewed as a balance between durability and permanent deformation. Ideally the HMA in service should have air voids between three and seven percent. As air voids decrease, the rate of asphalt binder aging decreases, but as air voids fall below three percent, the chance of rutting increases. The pavement will be more rut resistant if air voids are maintained high but the greater access of air into the pavement accelerates aging, which leads to cracking (*15*).

DURABILITY

Aging

Aging of hot-mix asphalt is a general term that describes the change in properties over time, including changes in asphalt binder properties and absorption of asphalt into aggregate. Other changes in mixture properties that occur with time and traffic loading include increased mixture density caused by traffic-induced compaction, and changes in air voids. Traffic compaction will reduce air void content. Air voids in areas of the pavement subjected to little traffic, such as between the wheelpaths, remain unchanged or increase slightly from freeze-thaw action.

Changes in Asphalt Binder Properties

Changes in asphalt binder properties over time occur because of chemical reactions with oxygen (*16*). Evaporation of lighter molecules is a minor contributor of change to the bulk asphalt properties. The performance-graded (PG) specification includes a maximum weight loss during the rolling thin film oven test. PG asphalt binders contain little material capable of evaporating at pavement service temperature.

Oxygen from air diffuses through the asphalt binder film reacting to form new or increased levels of sulfoxides, carbonyls, carboxylic acid, and ketones. The amount of products formed depends on the chemistry of the unaged binder.

The aging chemical reaction depends on temperature and time. The higher the temperature, the more rapid the oxidative aging. The longer the time, the more complete the reaction. The aging reactions increase the viscosity (stiffness) of the asphalt binder but the aging products have the same affinity for aggregate as the unaged asphalt. In other words, aged asphalt molecules bond to aggregate with the same energy. Hence, moisture resistance remains unchanged as the aging process occurs.

Absorption of Asphalt into Aggregate

Porous aggregate will absorb asphalt into the rock particles. The rate of absorption increases with increasing temperature. During plant mixing when the mixture is at high temperature, absorption occurs quite rapidly. Absorption begins very rapidly then slows. Within one to two hours at mixing temperature the aggregate will have absorbed most of the asphalt it will ultimately absorb.

Usually, hot-mix asphalt experiences at least an hour of elevated temperature storage, from mixing in the plant until laydown through the paver. Absorption is therefore quite stable immediately after construction and changes little thereafter. Properties of the mixture are not strongly influenced by absorption after the pavement is constructed.

Moisture Damage

Moisture damage refers to the loss of adhesion between the asphalt and aggregate induced by the presence of water. Moisture damage weakens the asphalt mixture leading to rutting, cracking, and, in extreme cases, disintegration of the pavement.

Adhesion

Asphaltenes in the asphalt binder adhere or stick to aggregates at active sites (*16*). These adhesive bonds are not chemical bonds in the sense of hydrogen and oxygen bonding to form water. Instead they are bonds of attraction.

Hydrogen bonding, electrostatic forces, and Van der Walls interactions form the bond. Asphaltenes are attracted to active sites. These bonds are not permanent but exist in a quasi-equilibrium state. Conditions near the bonding site influence and change the bonding strength. Asphalt contains many different chemical compounds that actively compete for an active site. The molecules that form the strongest bond, that is release the most energy during bonding, will be the successful competitor for the active site.

Water substantially affects the pH of the local environment near the bond, making it either more acidic or more basic. As the pH changes the strength of the adhesive bonds can change. Different components in the asphalt that were previously not bonded can become the successful competitor for an active site. Amphoteric molecules were identified during the Strategic Highway Research Program as molecules that display both acidic and basic behavior. This type of molecule is large enough that different parts can have different behavior. Amphoteric molecules are less affected by pH because they can bond in both acidic and basic conditions.

Aggregate has the greatest effect on the aggregate asphalt bond. The number and density of active sites on the surface of aggregates is similar for different aggregate types but there are large differences in the bonding energy. Surface charge varies for different aggregate types. Quartz is the strongest electron acceptor. Other siliceous aggregates are less strong but still are predominantly electron acceptors. Carbonate aggregates have a range of donor-acceptor behavior. The more an aggregate is an electron acceptor the more favorable the aggregate will be to accepting the partial electron charge of hydrogen in a water molecule. The aggregate will accept water in favor of asphalt.

Stripping Mechanism

If stripping is complete the matrix of the asphalt aggregate structure will be destroyed and the material will disintegrate. More commonly, the matrix of asphalt and aggregate will be weakened and traffic loading will exceed its structural capacity, causing cracking. Three types of stripping are identified (*16*).

1. Cohesive failure within the asphalt mix is a failure in the asphalt binder. A thin brownish film of asphalt remains on the aggregate but the bulk of the asphalt has been removed or lost from the aggregate. Water diffusing through the asphalt softens the asphalt binder creating a weak point in the mixture matrix.
2. Structural failure in the aggregate occurs when the aggregate breaks away near the surface and a thin layer of minerals will remain attached to the asphalt binder. This mode of failure occurs in soft limestone. In siliceous aggregates soluble salt near the interface may dissolve in acidic conditions, hence dissolving the stratum to which the asphalt binder is attached.
3. Adhesive failure of the bond between asphalt and aggregate occurs when water competes for active sites. Water entry to the aggregate surface can be through cracks in the pavement or along the uncoated edges of aggregates near the surface where traffic and weather have removed the asphalt film. The most common method of entry is diffusion through the asphalt film.

Water molecules not only compete for active sites but they change the environment at the adsorption sites and hence change the bonding equilibrium. Water changes the pH at the interface and changes the equilibrium of the adsorptive bonds. Adhesive failure of the bond occurs between asphalt and aggregate.

Methods to Prevent Stripping

Research done during the Strategic Highway Research Program has provided the most fundamental knowledge available of the mechanism of bonding and stripping of asphalt (*16*). A thorough understanding of the role of anti-strip agents was not obtained but some insight was gained.

Polyamine anti-strip agents improve the ability of asphalt to wet aggregate. Reduced surface tension allows bonds to form more easily. Tensile strength ratios that increase after use of polyamine reflects the increased strength of asphalt aggregate bond.

Cations such as iron, magnesium, and calcium can be added to prevent stripping. These cations increase the number of active sites and increase the strength of the asphalt aggregate bond. Calcium added as lime, the most common form of addition, also produces a high pH environment.

To increase bonding strength, cations must be present on the surface of the aggregate. When lime is mixed in the asphalt prior to coating aggregate, the benefit is almost negligible. When the same amount of lime is added to the aggregate surface prior to coating the effect is dramatic.

Tests for Stripping Potential

The most commonly used form of stripping test is AASHTO T-283 or a variation thereof. AASHTO T-283 uses six Marshall sized specimens. Three specimens are vacuum saturated, soaked overnight at 60°C, frozen and thawed, then tested for indirect tensile strength. Three specimens remain unconditioned; they are also tested for indirect tensile strength. A minimum ratio of conditioned to unconditioned tensile strength is specified. Some agencies also specify a minimum tensile strength after conditioning.

During the Strategic Highway Research Program two tests were developed for possible implementation. The net adsorption test evaluated the amount of asphalt that was adsorbed onto aggregate particles when a solution containing dissolved asphalt binder was circulated through a bed of aggregate. A small amount of water was injected into the solution and the amount of asphalt removed from the aggregate was measured. The test was developed as an indicator of the compatibility of an asphalt binder and aggregate combination. It was never intended to be used on a mix and cannot account for the effect of changing gradation or asphalt content on stripping potential. It is intended to be used only as a screening test and not as part of a mix design.

A second test protocol was developed called the environmental conditioning system (ECS). The ECS uses compacted specimens 100 mm diameter by 100 mm high. The specimens are conditioned by drawing water through them and subjecting the specimens to pulse compressive loads at 60°C. After each conditioning cycle, the specimen temperature is equilibrated to 25°C and an axial dynamic modulus is measured. The protocol uses three hot conditioning cycles and a freeze cycle. The dynamic modulus decreases as moisture damage occurs. The specimens are broken open at the end of conditioning and inspected for stripping. The ECS test is specifically designed to measure mixture properties as part of a mix design.

The ECS was not selected for Superpave. When compared, the ECS and AASHTO T-283 both had approximately the same ability to identify known stripping mixtures. Therefore, AASHTO T-283 was selected because it was already in use in the industry. For Superpave, the specimen diameter was changed to 150 mm to allow use of the Superpave gyratory compactor. There is discussion concerning the severity of the test using larger size samples, but a clear picture is not yet available.

CHAPTER THREE

DESIGN OF RUT-RESISTANT MIXTURES

SUPERPAVE APPROACH

The Superpave system, a SHRP product, is composed of a performance-graded asphalt binder specification, a mixture design specification, and a mixture analysis method. The objective of SHRP was to identify performance-based properties that control the behavior of asphalt binders and asphalt mixtures (*17,18*). The performance-based properties were to be used in new specifications and in a new mixture design method.

Engineering properties are used to evaluate asphalt mixtures. A performance-based property is defined as an engineering property that controls response to load. To be a performance-based property there must be a direct connection between the engineering property and the predicted performance. Currently, Superpave mix design is based on volumetric properties. Performance-based tests are under additional development awaiting inclusion into Superpave.

Many properties that have been used in the past are performance-related properties. A performance-related property may be an engineering property or the result of a simulative test that is related to performance through empirical regression analysis. For example, Marshall stability is the result of a simulative test. Marshall stability is not fundamentally related to rutting performance. That is, there is no fundamental method of using Marshall stability to predict the reaction of HMA to load. The connection of Marshall stability to rutting is an empirical relationship.

Superpave mix design as delivered by SHRP contained three vertically integrated levels of design. Level 1, which has become known as the Superpave volumetric mix design method, is based on empirical properties. Levels 2 and 3, which have become known as the Superpave mixture analysis method, contain performance-based tests.

Currently the Superpave volumetric mix design method is being implemented by states. The AASHTO Lead States Task Force performed a survey on Superpave implementation. In 1996, 28 states used Superpave binder and mixture specifications on 98 projects containing 2.9 million Mg of mixture, about 2 percent of total U.S. mixture. In 1997, the number of states to use Superpave increased to 40 and the number of awarded projects was 355, containing 15.5 million Mg of mixture. In 1998, 47 states planned to use Superpave in awarding 1339 projects containing 44.3 million Mg of mixture, about 30 percent of total U.S. mixture.

Superpave volumetric mix design addresses permanent deformation in asphalt binder properties, aggregate properties, and gyratory compactor requirements.

The Superpave asphalt binder specification uses performance-based properties to specify the contribution to permanent deformation resistance (*19*). The asphalt binder specification uses the rolling thin film oven test (RTFOT) to simulate asphalt binder aging during construction. The specification requires a minimum value of $G^*/\sin\delta$, basically the asphalt stiffness, which is the performance-based property for rutting. The stiffer the binder, that is the higher the $G^*/\sin\delta$, the more resistant to permanent deformation the mixture will be (*20*).

The asphalt binder after RTFOT must have a minimum $G^*/\sin\delta$ value of 2.2 kPa. Two items influence $G^*/\sin\delta$: the total stiffness of the asphalt binder can be increased, which will increase G^*, or the asphalt binder can be made more elastic, which will decrease the phase angle, δ.

Selecting a higher binder grade for a specific project, for example PG 70-xx instead of PG 64-xx, will approximately double the asphalt binder stiffness in service. In other words, at 60°C the PG 70-xx binder will be about twice as stiff as a PG 64-xx binder and will double the asphalt binder contribution to asphalt mixture stiffness.

Superpave promotes rut-resistant mixtures by specifying aggregate properties (*21*). Coarse aggregate requires a minimum percent of crushed faces that varies depending on traffic and depth within the pavement structure. Fine aggregate requires a minimum value of angularity in the National Aggregates Association test for fine aggregate angularity. The intent is to specify a minimum amount of inter-particle friction in the fine aggregate. Aggregate gradation is recognized to influence the rutting potential of mixtures. The Superpave gradation specification uses control points and a restricted zone to restrict the amount of fine aggregate and promote the use of larger stone particles in the mixture to develop a coarse skeleton.

The method of compaction influences mixture properties (*22*). In Superpave, mixture compaction is done with a gyratory compactor that monitors the rate of densification during compaction. Strong aggregate skeletons will produce a densification curve with a higher slope. Superpave requires that the design mixture have a density of 96 percent, four percent air voids, at the design number of gyrations. At a low number of gyrations the density cannot be above 89 percent, in effect limiting the minimum acceptable slope.

ASPHALT BINDER

Asphalt binder properties influence the permanent deformation resistance of asphalt mixtures. Stiffer binders at high service temperatures have less rutting (*23*).

Asphalt binders are graded using three specifications. In the most common method, asphalt is specified by viscosity at 60°C (*24*). The higher the viscosity, the stiffer the binder. The second method, more common 20 years ago than today, some asphalt is still purchased based on penetration at 25°C. The lower the penetration, the stiffer the binder. The third specification for purchasing asphalt is based on viscosity of a laboratory-aged residue instead of the unaged asphalt. The aged residue specification is used mostly by states in the West.

SHRP developed a performance-graded specification that is being implemented by many states. The specification is distinctly different from existing specifications. Properties are related to specific distresses and each property has a different test temperature (*25–27*).

Binder Relationship to Permanent Deformation

Asphalt binder contributes to mixture stiffness, which increases resistance to permanent deformation (*28*). Generally the stiffer the asphalt binder, the stiffer the mixture and the more resistant to permanent deformation. The performance-graded specification recognizes the contribution of the asphalt binder by requiring a minimum value related to the complex shear modulus, that is, total stiffness of the asphalt binder and the phase angle, which relates to the ratio of elastic and viscous behavior.

Modifiers are added to asphalt binder to increase high temperature stiffness. Most modifiers have little effect on low-temperature properties of the asphalt binder. Low-temperature properties come from the base asphalt. The effect of the modifier is to increase high-temperature stiffness. Typically stiffness of the modifying material is less affected by temperature than is the asphalt binder. The effect on the asphalt binder is to physically increase stiffness according to the concentration of modifying material. Therefore most modifiers have a threshold that must be exceeded before beneficial effects of the modifier can be seen.

In the literature there are numerous studies comparing the laboratory properties of modified and unmodified asphalt mixtures (*29–44*). Beneficial effects are typically shown for properties such as resilient modulus, creep stiffness, and dynamic stiffness. Torture tests such as rolling wheel rut testers also indicate beneficial effects of modified asphalt binders.

In 1992 Button reported that states were evaluating many different types of modified asphalt (*45,46*). Table 4 contains a list of asphalt binder modifiers used or tested in pavements.

TABLE 4

ASPHALT BINDER ADDITIVES USED OR TESTED IN PAVEMENTS

Modifier Family	Example Modifiers
Polymers	SBR, SBS, Polyethylene, EVA
Extenders	Sulfur
Mineral Fillers	Carbon Black
Natural Asphalt	Trinidad, Gilsonite
Antistripping Agents	Amines
Antioxidants	Diethyldithio Carbamates
Hydrocarbons	Tall Oil, Aromatics
Fibers	Polypropylene, Polyester
Others	Gelling Agents

Williamson and Gaughan report results of trial sections containing modified and unmodified asphalt binders (*47*). The road carries 44,500 vehicles per day on six lanes with an accumulated ESAL rate of 1.3 million per year. Test sections, each 100 meters in length, were built at approaches to traffic lights. Table 5 lists the different asphalt types used in the experiment, the rut depth and age. Two penetration-graded unmodified binders were replaced 28 months after construction. The 170-pen asphalt binder had 18-mm rut depths and the 320-pen asphalt had ruts 27 mm deep.

All of the modified asphalt binders have performed better than the unmodified. Many of the sections containing modified asphalt are twice as old as the unmodified sections that were taken out of service at age 28 months. Maximum rut depth of the modified asphalt section range from 17 to 60 percent of the control Class 170 binder.

The Federal Highway Administration performed an experiment using an accelerated loading facility (ALF) to test an asphalt mixture with different asphalt binder grades. A 19.0-mm Superpave mixture was used with five performance-graded binders, both modified and unmodified, ranging from PG 58-34 to PG 82-22 (*48*). The pavement was tested at a constant temperature of 60°C. Each asphalt binder was tested to determine the high temperature limit at which $G^*/\sin \delta$ just meets the specification minimum. The result provides a more specific theoretical grade between actual PG grades that are used. Mixture

TABLE 5

THE EFFECT OF VARIOUS TYPES OF MODIFIED ASPHALT BINDERS ON RUTTING PERFORMANCE *(6)*

Asphalt Type	Age (months)	Average Rut (mm)	Maximum Rut (mm)
SBS Grade 60 Binder	56	4	11
SBS Grade 60 Binder	56	3	5
EVA, 5%, Grade 170 Binder	44	4	10
EVA plus terpene resin, high percent	56	2	3
EVA plus terpene resin, medium percent	56	2	4
EVA plus terpene resin, low percent	28	3	6
Ethylene acrylic, 5%, Grade 170 Binder	28	3	9
Gilsonite, 10%, Class 320 Binder	56	2	8
Gilsonite, 5%, Class 170 Binder	56	4	11
SBS, 15%, Class 320 Binder	56	2	6
SBS, 20%, Class 320 Binder	56	6	10
Class 320 Binder	28	11	27
Class 170 Binder	28	7	18
Class 170 Binder with lime, 0.85:1 ratio	34	5	16

performance in the ALF was normalized by determining the number of repetitions to create a 20-mm rut. For the two highest PG grades, the values were extrapolated *(49)*. Figure 13 shows the number of repetitions from the ALF and the high temperature PG binder grade. As the PG grade increases, the number of passes increases greatly.

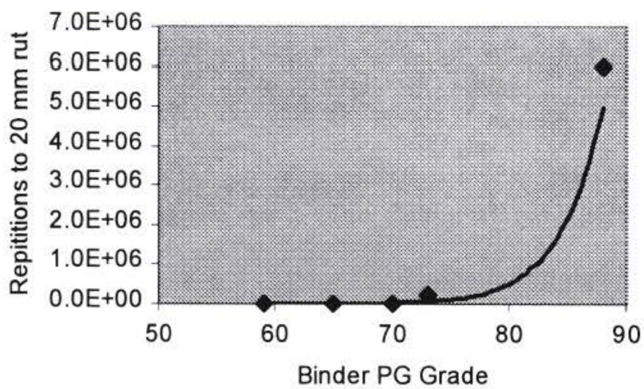

FIGURE 13 Relation of rut depth and binder high temperature properties for mixtures tested with ALF *(47, 48)*.

In Nantes, France, LCPC performed a full-scale experiment using a circular test track with a wheel speed of 40 km/h. The effect of asphalt binder stiffness can be seen in Figure 14. The section containing 60/70 penetration asphalt had ruts 12.6 mm deep after 175,000 passes while the section containing the stiffer 10/20 pen asphalt had ruts 3.7 mm deep. The two modified asphalt sections fell in between. Both the SBS modified asphalt and the Multigrade asphalt had ruts 7.2 mm deep.

State Agency Practice

Most of North America uses viscosity to grade asphalt binder, although some areas use penetration. For the purposes of this discussion 80- to 100-pen asphalt binder is considered AC 20, 120- to 150-pen asphalt is considered AC 10, and 150- to 200-pen is considered AC 5. According to the questionnaire responses, the most commonly used asphalt binder is an AC 20. Most of the United States uses AC 20 as the asphalt binder in the surface layers of pavements built on high-volume roadways. Exceptions occur in hotter and colder parts of the country.

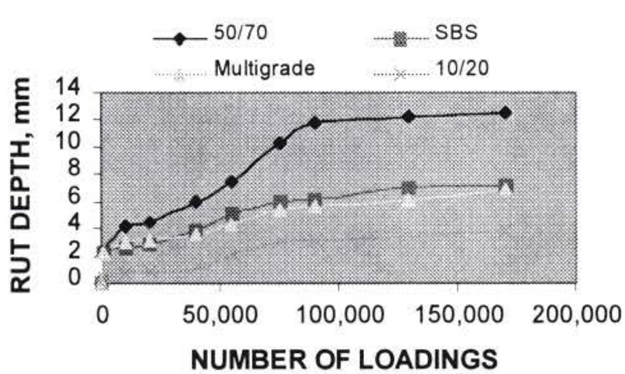

FIGURE 14 LCPC test track results: development of rut depth *(43)*.

In the Southeast, Florida, Georgia, Mississippi, and Tennessee use AC 30. In the Southwest, Arizona uses AC 40 and Nevada uses AC 30P. In the North, Michigan, Minnesota, North Dakota, and Nebraska use AC 10 and Alberta, Saskatchewan, Manitoba, and New Brunswick use AC 5.

Most agencies use the same asphalt binder in the lower layers as in the surface layer. Thirty-seven agencies report use of the same asphalt binder in all layers. Nine agencies use a different grade in the surface layer. Of the nine, five report using modified asphalt in the surface layer and an unmodified one in the lower layers. The other four agencies use a stiffer grade of unmodified asphalt in the surface layer.

Modified asphalt is commonly used by agencies on high traffic roadways. Only 20 percent (9 of 46) report no usage of modified asphalt. Fifty-six percent (26 of 46) use modified asphalt binders on 1 to 20 percent of projects and 17 percent (8 of 46) use modified binder on 21 to 60 percent of projects. Three agencies, seven percent, use modified binder on most projects.

Superpave asphalt binder specifications are currently being implemented. Five questionnaire respondents indicated the use of performance-graded asphalt binders when indicating grades commonly used. The AASHTO Task Force on Superpave Implementation indicates four states adopted performance-graded binder specifications in 1996 and 22 states implemented them in 1997 (*50*), 11 more planned to adopt them in 1998. By the end of 1998, 37 states will have adopted Superpave binder specifications.

AGGREGATES

Aggregate properties are known to influence the permanent deformation resistance of asphalt mixtures. Crushed faces, sand properties, gradation, and aggregate size can change the properties of an asphalt mixture (*51–62*).

Aggregate Gradation

Aggregate gradation, i.e. particle size distribution, will influence hot mix asphalt properties (*63–67*). Dense-graded mixtures with the same nominal maximum size can have a fine gradation, a coarse gradation, or anything in between. Mixtures composed mostly of fine particles will not be as rut resistant (*68–70*). The experience of states has indicated that coarsening the gradation of HMA has increased the pavement rut resistance.

Some contradictory evidence exists that fine-graded mixtures perform at least as well, maybe better, than coarse-graded mixtures. Geller (*63*) indicated that the angle of friction in aggregates is influenced by fracture properties not by the particle size. In accelerated testing at the WesTrack facility, fine-graded mixtures have outperformed coarse-graded mixtures.

All states plot aggregate gradation on 0.45 power charts. Either percent passing or percent retained is plotted on the vertical axis. The horizontal axis is the sieve size opening raised to the power 0.45. Research by Nijboer in the 1940s (*71*), and Goode and Lufsey in the 1950s (*72*) confirms that gradations that are a straight line when plotted on a 0.45 power chart produce the densest possible packing for the aggregate. Huber and Shuler found that drawing a maximum density line on a 0.45 power chart worked only when the line was drawn from the maximum aggregate size to the origin (*73*). Maximum aggregate size is defined as one sieve larger than the nominal maximum size, which is one sieve larger than the first sieve to retain more than 10.0 percent.

George et al. (*65*) performed an evaluation of mixtures with gradations above the maximum density line, near the line, and below the line. They found that gradations near the maximum density line were not desirable based on Hveem stability. The stability of coarse gradations was less sensitive to asphalt content than fine gradations.

The French LCPC investigated the effect of increased amounts of ground sand using an LCPC wheel tracking machine. The reference mixture is a 14-mm maximum sized mixture with 32 percent manufactured sand, 7.5 percent dust, and 5.7 percent asphalt. The percent sand was changed to 28, 30, 32, and 36 percent. Rut depths after 30,000 passes were nearly the same, about 9 mm, for 28 and 30 percent sand. The limit for discontinuing the test is 10 mm. For 32 percent sand the test was stopped at 9,000 passes and for 36 percent sand the test was stopped at 2,500 passes. The more sand added to the mixture, the more sensitive the mixture to rutting.

Ruth (*64*) evaluated 18 gradations of coarse aggregate only; the minus 2-mm material was removed. Shear strength of the aggregate skeleton was found to increase for coarser gradations. Gradations near the maximum density line developed shear strength equal to the coarse gradations but at the expense of a lower VMA. Skeletons that had a large gap grade in the sieve size one size smaller than the nominal maximum size had lower shear strengths. In particular, coarse gradations with very large gaps, 30 to 35 percent change on the sieve lower than the nominal maximum size sieve, suffered reduced shear strength.

States report coarsening of HMA gradations has improved rutting resistance of pavements. Eighteen percent of questionnaire respondents report that recent specification changes to coarsen mixture gradations have reduced rutting. Current mixtures tend to be fine with about 40 to 45 percent passing the 2.36-mm sieve, as shown in Figure 15.

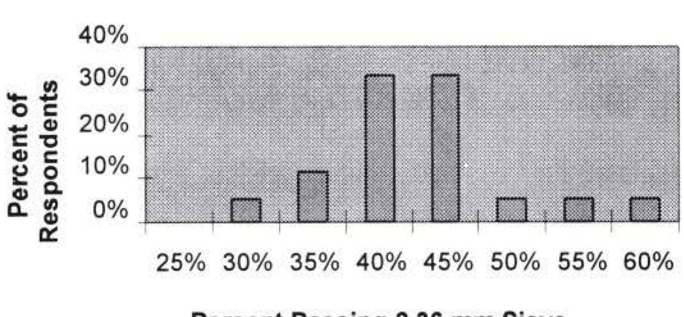

FIGURE 15 State specified average gradation for 2.36 mm sieve.

Superpave places a restricted zone on the maximum density line from the 2.36-mm sieve to the 0.300-mm sieve and recommends for high-volume traffic that gradations plot below the restricted zone. Gradations below the restricted zone will have 28 to 39 percent passing the 2.36-mm sieve. These gradations are coarser than most gradations currently used by states.

Coarse Aggregate

Fractured faces are an indicator of inter-particle friction that can be generated in the aggregate skeleton (*74,75*). If the surfaces of adjacent aggregate particles are rough and angular, more force will be required to make the contact slip. On the other hand, smooth surfaces will slide past one another even if the particles are forcibly pushed together (*76*).

Crushed faces tend to increase skeleton strength but there are situations where crushed faces do not add strength to a skeleton. Some aggregate types will fracture with a smooth surface and the fractured surface will produce no more friction than an unfractured surface. Such aggregates are not common; most produce a rough-textured fractured surface.

In coarse aggregate, particles greater than 4.75 mm, crushed faces are usually used to describe the texture of the particles (*77*). Many states have increased percent crushed face requirements. Wisconsin is a typical example (*78*). Much of the aggregate supply in Wisconsin is gravel, and crush particle count in mixtures tended to be low. In the 1980s, Wisconsin suffered a rash of rutted pavements. An increase in required crush faces was one of the changes made in response. The requirement for interstate pavement was increased to 90 percent crushed one-face and 60 percent crushed two-face. A similar situation occurred in Pennsylvania (*79*), where the requirement was increased to 95 percent crushed one-face and 85 percent crushed two-faces.

Conceptually, it is easy to visualize a fractured face. Measuring fractured faces is less straightforward and many states have their own test method. The recently adopted ASTM standard D 5821 provides a definition of a fractured face and offers a uniform method of measuring fracture. ASTM defines fracture as

> An angular, rough, or broken surface of an aggregate particle created by crushing, by other artificial means, or by nature. A face will be considered a "fractured face" only if it has a projected area (maximum cross-sectional area) of the particle and the face has sharp and well-defined edges; this excludes small nicks.

Superpave requires 95 percent crushed one-face, 90 percent crushed two-faces for 10 to 30 million ESALs and 100 percent crushed one- and two-faces for traffic above 30 million. On high-volume roads, questionnaire respondents indicate an average of 88 percent crushed one-face and 83 percent crushed two-faces. A distribution of the amounts specified is shown in Figure 16.

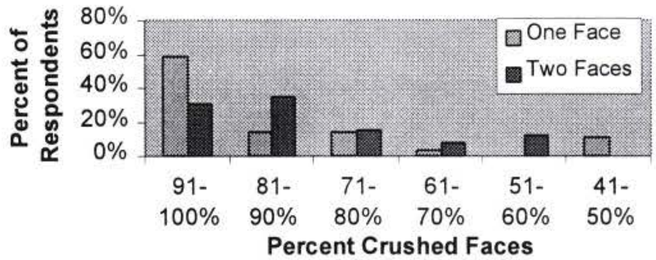

FIGURE 16 State specified percent crushed faces for high-volume roads.

Sand

Angularity of sand is an important factor contributing to skeletal strength of the mixture. In dense-graded hot-mix asphalt, sand particles are actively carrying load. Throughout the mixture, sand particles are sandwiched between coarse aggregate particles. Smooth rounded sand particles will allow the coarse particles to slip easily. Rough, irregular shaped particles will lodge between coarse particles maintaining a strong skeleton (*80*).

Several field investigations of rutting pavements have indicated natural sand as a contributing factor to the poor resistance to loading (*81*). Perdomo and Button measured the effect of natural versus manufactured sand (*82,83*). In a 12.5-mm nominal maximum sized mixture that contained 40 percent sand, the ratio of natural and manufactured sand was varied from 100 percent natural sand to 100 percent manufactured sand. Resistance to permanent deformation was measured using a static creep test and a cyclical creep test. A substantial increase in deformation occurred as the percentage of natural sand was increased from zero to 20 percent. For the mixture studied 20 percent natural sand is excessive.

The shape and texture of sand is often specified by requiring a manufactured sand, which is composed of sand sized particles that are the result of crushing massive rock. According to the questionnaire done for this synthesis, 33 percent (15 of 46) of the respondents specify the minimum percentage of the fine aggregate that must be manufactured sand. Figure 17 shows the limits specified by the states.

Some methods attempt to quantify the amount of inter-particle friction in fine aggregate. These methods include fine aggregate angularity, crushed faces, and particle index.

Particle index is an ASTM test method, ASTM D 3398, which measures the air voids of aggregate compacted in

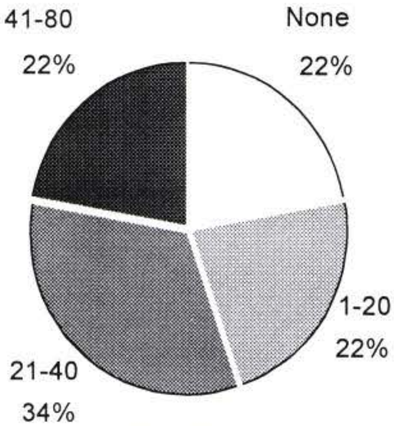

FIGURE 17 Maximum amount of natural sand allowed as percentage of fine aggregate for high-volume roads.

standard molds. Individual particle sizes are compacted separately. The weighted average particle index is calculated from the individual sizes. A particle index value of 14 was found to separate natural and manufactured sand.

The National Aggregates Association developed the fine aggregate angularity (FAA) test, a flow test selected by SHRP for use in Superpave. The test is based on the principle that sand with high inter-particle friction will not pack as tightly when allowed to free fall into a container. Using Method A of the FAA test, a value of 44.5 separated natural and manufactured sand. Fine aggregate angularity results correlate well with the particle index test *(84–85)*.

The FAA test is used to measure inter-particle friction. Method A is used because the gradation is specified. Hence, the FAA value is not influenced by gradation. Increasing the FAA value by changing gradation will not change the particle shape and texture or improve the performance of the mix.

The National Center for Asphalt Technology performed two research projects that determined FAA as an indicator of rutting potential *(86)*. A national rutting study selected 42 sites across the country where various aggregate and mixture properties were correlated with the amount of rutting observed. Fine aggregate angularity measured on aggregate recovered from mixture on the road had the highest correlation, R-square of 0.67. The study identified a threshold value of 44.1 as separating good and poor performing pavements.

Mogawer and Stuart evaluated 10 natural sands and two manufactured sands of known field performance in a trap rock mixture containing 30 percent sand *(87)*. Properties of the sands were measured with the NAA test Method A, particle index, direct shear, and the Michigan fine aggregate test. The investigated sands were dosed at various percents in the mixture and tested in a gyratory test machine (GTM). The direct shear and the Michigan test did not distinguish the good and poor performing sands; both the FAA test and the particle index test did.

Aggregate Texture

Texture of the coarse and fine aggregates has been treated separately. Two studies done by the National Center for Asphalt Technology indicate the combined effect of coarse and fine angularity. Aggregate sources in the state of Alabama tend to be sand and gravel in the south and western part of the state, and limestone in the north and eastern part of the state. Thirteen sites were evaluated, five with good performance, eight with poor performance *(89)*.

Fractured faces and fine aggregate angularity were measured on the recovered aggregate. Figure 18 shows a plot of the results with good performing sites indicated by squares and poor sites indicated by diamonds. All of the sites with less than 75 percent crushed faces and an FAA less than 43 were poor performing sites. Even most of the sites with crushed faces more than 75 percent but with an FAA less than 43 were also poor performers. Most of the sites with FAA greater than 43 performed well.

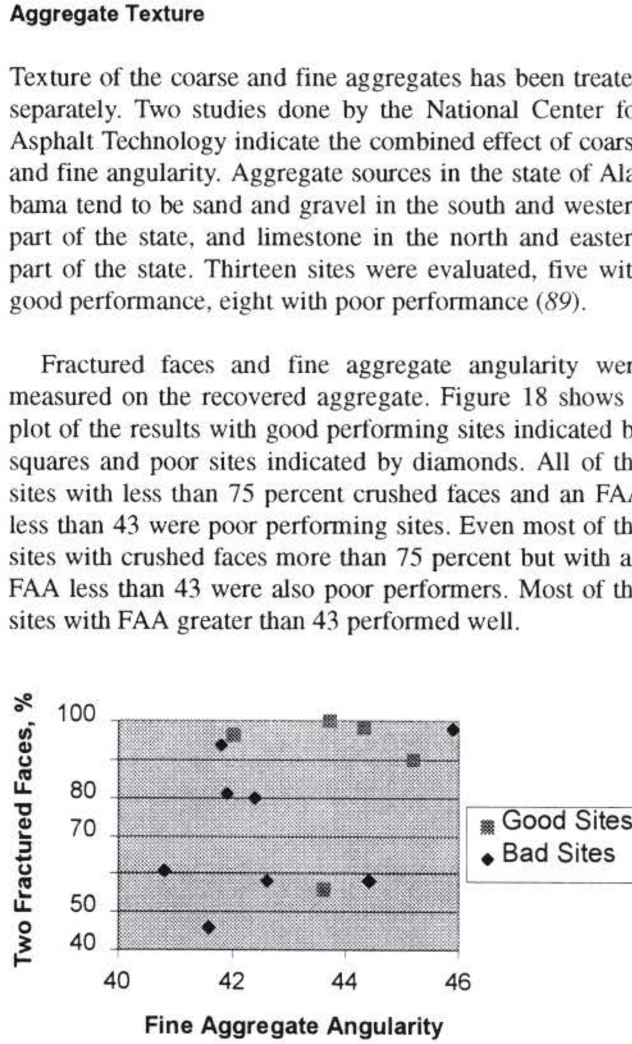

FIGURE 18 Crushed faces and fine aggregate angularity of Alabama research sites *(88)*.

In the national rutting study Cross and Brown evaluated 12 good and 30 poor performing sites *(86–90)*. They found that aggregate properties at sites with low in-place air voids had no effect on rutting. At sites with more than 2.5 percent air voids in place fractured faces and fine aggregate angularity had the most effect. Figure 19 shows the results of a regression model based on two fractured faces and fine aggregate angularity.

If the fine aggregate angularity is low, an increase in percent two-crushed faces is not as effective in increasing the traffic carrying capability of the mixture. If the crushed faces are 60 percent, decreasing the fine aggregate

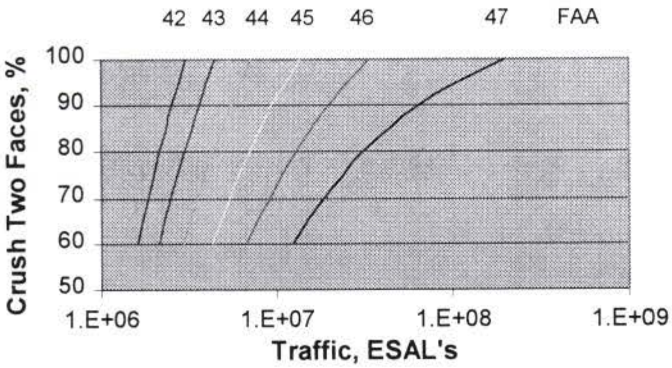

FIGURE 19 Predicted traffic to create a 15-mm rut depth *(91)*.

angularity from 47 to 42 causes a 90 percent drop in the load carrying capability. If the fine aggregate angularity is 42, a decrease in crushed faces from 100 percent to 60 percent decreases the load carrying capability by 50 percent. Fine aggregate angularity is the dominant factor in rut resistance.

PERFORMANCE TESTS

Options for a performance test include a performance-based test, a performance-related test, or an empirical test.

Performance-Based Tests

A performance-based test measures an engineering property that is directly related to performance. The measured engineering property can be used in a mechanics of materials model to predict response to load.

In the Superpave system as developed during SHRP, performance prediction models were designed to use performance-based properties. However, the Superpave models are not ready for implementation. Additional work is required before they can be routinely used.

The FHWA is completing development of the performance prediction models using a series of research contracts. Work is scheduled to be complete in 2007. In the short term, the first contract by the University of Maryland is charged with developing a framework for the performance prediction models and test to measure performance-based properties. Subsequent contracts will complete and validate the performance prediction system.

In the meantime, the University of Maryland is developing indicator tests to check for problematic mixtures. These tests differ from performance prediction models in that no prediction of distress severity can be made. The tests will provide an indication of mix suitability for the intended application.

Performance-Related Tests

A performance-related property is an engineering property that is connected to performance but is not directly used to predict performance. Rather it is used in an empirical model, to predict performance *(92–105)*. Such properties may not be used to predict material response directly but they can be used knowing that there is a fundamental relationship between the property measured and performance on the road. A regression analysis is usually used to link the property with performance. Example performance related properties include mixture stiffness, dynamic creep, repeated shear, and static creep.

Mixture stiffness can be measured using a number of methods, such as shear frequency sweep, dynamic modulus, and indirect resilient modulus. Mixture stiffness is generally accepted as being related to rutting, that is the stiffer the mixture, the less the rutting. Nevertheless, stiffness, by itself, is insufficient to predict rutting directly. Therefore, regression analysis must be used to link mixture stiffness to rutting.

Dynamic creep is an axial repeated load test *(106)*. Typically, the test is performed at elevated temperature, 40°C to 60°C, and the accumulated axial deformation is recorded.

Repeated shear testing is similar to dynamic creep, except that the repeated load is applied to the specimen in shear *(107,108)*. Typically, a specification is based on limiting the percent strain at a specified temperature and number of repetitions.

A static uniaxial load is placed on a specimen for 3,600 seconds and released. Axial deformation is monitored during the loading and for 3,600 seconds thereafter. A specification can be based on maximum allowable axial deformation, unrecovered deformation, and creep modulus.

Empirical Tests

Empirical tests can also be used as a link to expected rutting performance. The difference between an empirical test and a performance-related or performance-based test is the property that is measured. An empirical test does not measure a fundamental engineering property. Hveem stability and Marshall stability are considered empirical tests.

Sometimes empirical tests are designed as a miniature simulation of the road. Test parameters are set up to simulate the road in the laboratory, to the extent possible. The test result is not an engineering property, but merely the response to an applied loading. The response depends

TABLE 6
CHARACTERISTICS OF RUT TESTERS

	Rut Tester		
	French LCPC	Hamburg	Georgia
Wheel load (N)	5,000	705	700
Pressure on sample (kPa)	600	1,500	700
Loading rate, (cycles per minute)	60	60	45
Load mechanism	Pneumatic tire	Steel wheel	Pressurized rubber hose
Test environment	Air	Water	Air
Sample mass (kg)	20	10	5
Sample thickness (mm)	100	80	75
Test temperature °C	60	50	40
Specification cycles	30,000	10,000	8,000
Max rut depth (mm)	10	4	7

on specimen geometry, geometry of the applied load(s), scale effects, etc. Loaded wheel testers, sometimes known as rut testers, are empirical tests.

RUT TESTERS

A rut tester is an empirical laboratory-scale device designed to simulate the action of a wheel rolling on a compacted mixture sample. Three different rut testers have been developed (*109*): the LCPC rut tester, the Hamburg rut tester, and the Georgia loaded wheel tester.

Table 6 lists the individual characteristics, including machine and operating characteristics, of the various testers. There are several varieties of the Hamburg tester. The city of Hamburg, Germany, developed the original machine. The Couch Construction Company uses a similar machine except the steel wheel is replaced by a solid rubber-faced wheel. Purdue University developed another Hamburg variant that can test in water or air using a steel wheel or a rubber-faced wheel.

French (LCPC) Rut Tester

Approximately 72 of these machines are in use, 45 in France, the rest in various parts of Europe and the rest of the world. Five machines are located in North America, one at the FHWA Turner-Fairbank Highway Research Center in Virginia, one at the Colorado DOT in Denver, another at Oregon State University and two machines at the Ministry of Transportation of Quebec in Quebec City, Quebec.

The LCPC rut tester tests two specimens in an air chamber with 5000-N load applied using a 400-mm diameter pneumatic tire inflated to 600 kPa. The samples are 500-mm long by 180-mm wide and 100- or 50-mm thick. One pass or cycle is a complete back-and-forth stroke that occurs each second. Load time at the center of the sample is approximately 0.1 seconds.

A standard test procedure, ISO Standard 5725, is written for the LCPC rut tester. Inter-laboratory testing was done with 12 laboratories on mixture that has 7-mm rut depth. Within lab repeatability is 1.1 mm and inter-laboratory reproducibility is 1.4 mm (*110*).

French standards for the percentage of allowable rutting have been developed. For mixture to be placed at a lift thickness greater than 50 mm, samples are 100-mm thick. For mixtures placed less than 50-mm thick, samples are 50-mm thick. A standard test temperature of 60°C is used without regard to the temperature environment where the pavement is located or the depth the mixture is located within the pavement. At one time, base mixtures were tested at 50°C and the specification limit was 10-m rut depth at 30,000 passes. Currently, base mixtures are tested at 60°C and the specification limit is 10 mm at 10,000 passes.

In 1993, LCPC reported studies in progress to develop rut tester specifications for particularly heavy truck traffic or for severe situations such as uphill grades or signalized intersections. No results of these studies have been located in the literature.

Developers of the LCPC rut tester believe that no statistical correlation of rutting in the rut tester and rutting on the road can be developed because the rut tester simulates the most severe rutting conditions. Much traffic is carried when conditions are less severe. Although no correlation can be developed, they report that no rutting is found on sites where the rutting specifications in the LCPC rut tester are met.

In Colorado, extensive work was done to correlate in situ rut depths on the road to rut depths in the LCPC tester (*111,112*). Thirty-seven sites were selected with traffic ranging from three daily ESALs to 3,127 daily ESALs. Pavement samples were obtained from the road and tested in the laboratory at 60°C. Rut tester results were evaluated using the French criteria. The test appears to be

TABLE 7

COMPARISON OF FIELD PERFORMANCE AND RUT TESTER PREDICTIONS *(61)*

		Number of Sites	
		Acceptable Field Performance	Unacceptable Field Performance
French specification	Acceptable lab performance	4	0
	Unacceptable lab performance	11	16
Modified specification	Acceptable lab performance	10	0
	Unacceptable lab performance	4	16

too severe. All of the poorly performing sites were identified as well as most of the acceptable sections.

To reduce severity, the test temperature was modified. The actual field temperature was defined using the highest monthly mean maximum temperature (HMMMT). Test temperatures were used as follows:

1. Test temperature 60°C for HMMMT from 32 to 38°C
2. Test temperature 55°C for HMMMT from 27 to 32°C
3. Test temperature 50°C for HMMMT less than 27°C

Using the modified temperature, a much closer correspondence was obtained between the rut tester and performance on the road. Table 7 lists both results using the unmodified French test method and results from the modified test temperatures.

In Colorado, Aschenbrener tried to identify a correlation between rutting in the LCPC rut tester and the field. Rutting in the LCPC rut tester was monitored during the test and a correlation was done. Typical results are shown in Figure 20.

Test data were segregated into high, medium, and low traffic. Modified test temperatures of 60, 55, or 50°C were used based on the HMMMT. The correlation of field rut depth to the slope of the rutting curve is shown for high traffic in Figure 21. Correlation coefficients were 0.87 for high traffic and 0.68 for medium traffic (*113*).

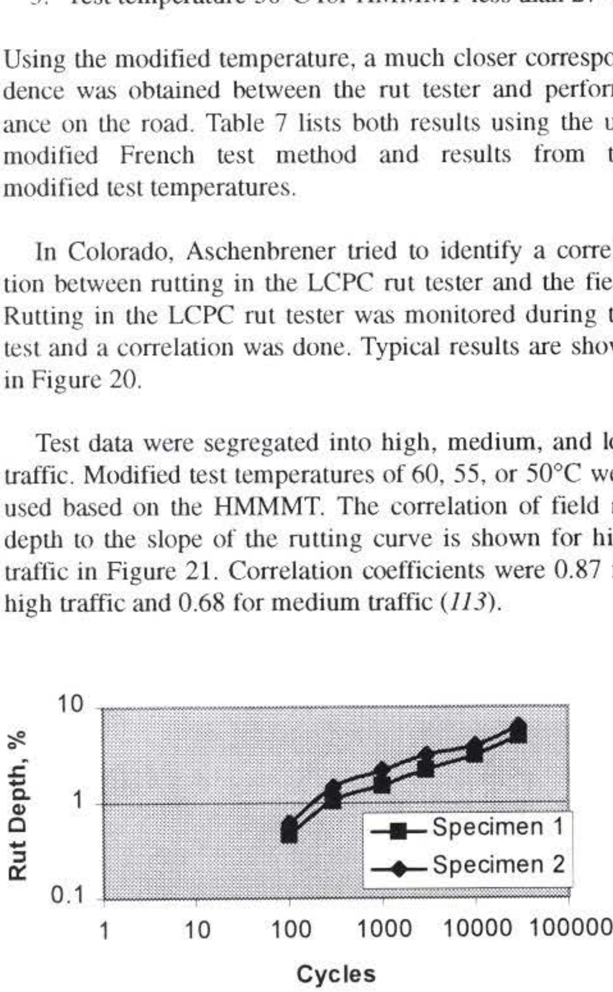

FIGURE 20 Typical rut test results in LCPC tester *(111)*.

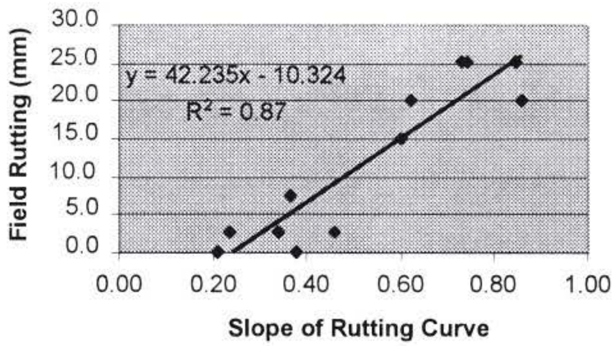

FIGURE 21 Correlation of field rutting to slope of rutting curve in LCPC rut tester for high traffic *(111)*.

Hamburg Rut Tester

The Hamburg rut tester is used to measure moisture damage resistance. An indication of rutting resistance is obtained from the test results, but often the mode of failure in the testing machine is stripping. Figure 22 shows typical results from the machine. Some initial compaction occurs quickly, then the rut rate stabilizes. If the mixture experiences moisture damage, the rate will accelerate and the mixture will fail rapidly.

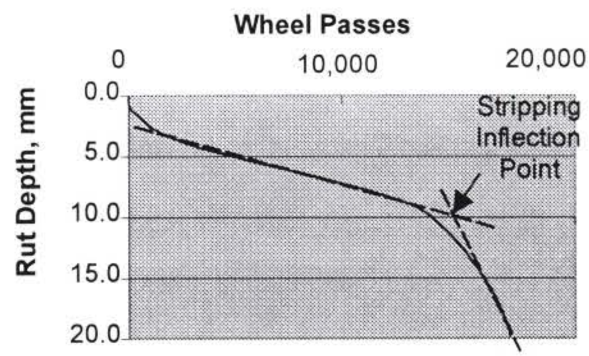

FIGURE 22 Typical deformation in Hamburg rut tester specimen.

The smooth steel wheel produces high point stresses on the mixture surface. If the mixture is able to withstand the loads without moisture damage, the line will not turn down. In such cases, the rutting at 20,000 passes will be relatively small, less than 10 mm. Mixtures that show large rut depths often show a stripping inflection point.

The effect of test temperature on Hamburg test results is shown in Figure 23. The mixture was obtained from a Colorado project on Interstate Highway 70. Rut depth increases with increasing test temperature. Generally, if the temperature is sufficiently warm, the mixture will develop a stripping inflection point and fail rapidly. In the Hamburg test, the development of a stripping inflection point will be delayed if harder asphalt binder is used.

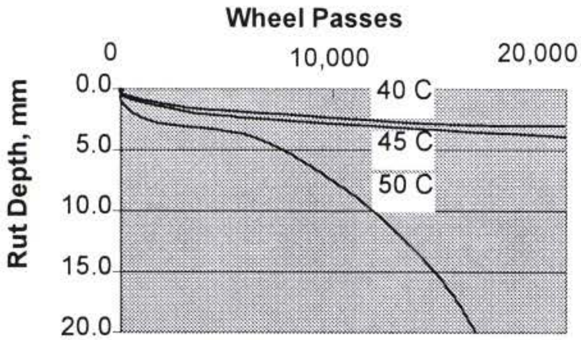

FIGURE 23 Effect of test temperature on deformation in Hamburg rut tester *(112)*.

Others have developed variations of the Hamburg rut tester. The PURWHEEL rut tester uses the same basic principle as the Hamburg tester. The mixture can be tested dry in warm air as well as under water. Also, a rubber-faced wheel can be substituted for the steel wheel. Couch Construction developed another variant of the Hamburg tester that uses a rubber-faced wheel and tests under water only.

Georgia Loaded Wheel Tester

The Georgia Department of Transportation and the Georgia Institute of Technology developed the Georgia loaded wheel tester in the mid-1980s. An aluminum wheel with a saddle-shaped face applies the load. The saddle straddles an inflated rubber hose that lies on the specimen surface. The loaded wheel moves back and forth 45 times per minute.

Test temperature, wheel load, hose pressure, and hose stiffness influence specimen deformation. The parameters reported in the literature indicate a load of 500 N was selected with a stiff hose pressurized to 690 kPa. Testing temperature was selected as 40.5°C.

The results of several studies are reported in the literature but little correlation is reported between performance in the field and in the rutting tester *(113,114)*. Florida DOT tested mixtures from three pavements in the Georgia LWT. Agreement was found between the observed field rutting and rutting in the LWT, as shown in Figure 24.

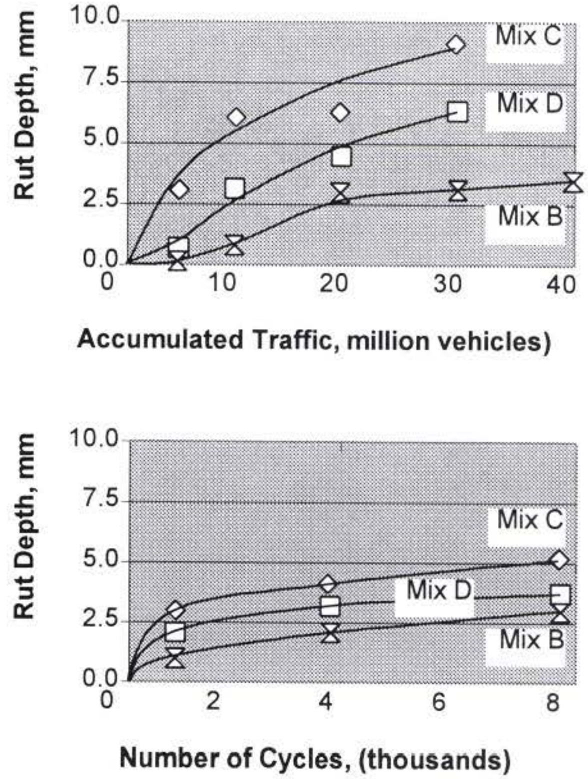

FIGURE 24 Comparison of field rut depths and loaded wheel test ruts for three Florida pavements *(114)*.

To quantify test variability, six state highway agencies participated in round robin testing in 1990 *(115)*. The mean measured rut depth was 3.38 mm with a standard deviation of 1.22 mm. Large scatter in the data was traced to differences in specimen preparation. A follow-up study was not located.

Current Use of Supplemental Rut Tests

Twelve of 46 agencies report the use of a supplementary test to evaluate rutting performance. Five agencies are using rut testers, four agencies are using the gyratory test machine, and two agencies are using Hveem stability.

The five agencies with rut testers use them on high traffic volume pavements during mix design. Alaska, Georgia, Ohio, and Virginia use the Georgia loaded wheel tester. Colorado uses both the LCPC rut tester and the Hamburg rut tester.

Accelerated Pavement Testers

Accelerated pavement testers (APTs) are full-scale testing facilities that apply full-scale loading in an accelerated time *(117)*. Generally, they can be separated into three classesClinear, circular, and test tracks.

Linear testers apply linear loads to a short pavement section, typically 10 to 20 meters. Some linear testers are enclosed in a climate controlled building with the pavement structure built in a test pit. Loads are applied using a loading frame at a slow speed, typically 10 to 15 km/hr.

A second group of linear testers is placed outdoors on pavements that were built using normal construction techniques. These testers are designed to move from place to place. Loads are generally applied at 10 to 40 km/hr. Some of these testers can isolate the section of pavement being tested and maintain constant pavement temperature. Loads are typically applied using one half of a full-size axle.

Circular test tracks apply loads in a circle. Smaller diameter, less than 20-m, tracks are typically housed in buildings that allow climate to be controlled. Larger diameter tracks, typically 20- to 40-m, are located outdoors without artificial climate control.

Test tracks are full-scale outdoor pavements constructed specially for testing. Full-scale trucks are driven on the pavement to apply loads.

APTs have been used for a wide variety of studies, including pavement material studies and structural studies. In studies of mixture components, APTs provide confirmatory evidence of laboratory testing. APTs are particularly useful for the study of pavement structure.

ALTERNATE MIXTURE TYPES

This section discusses asphalt mixtures other than the dense-graded mixtures that have historically been used by highway agencies. This discussion will primarily concern stone matrix asphalt and porous asphalt (open-graded mixtures).

Stone Matrix Asphalt

Stone matrix asphalt was developed in Germany in the late 1960s to resist wear from studded tires. The concept of the mixture is different from normal dense-graded asphalt mixtures. In SMA, a stone skeleton is developed with gap-graded coarse aggregate. The skeleton is intended to carry the vehicle loads. The gap grade in the coarse aggregate provides sufficient room for mastic composed of asphalt binder, fine aggregate, and mineral filler.

The main difference between an SMA mixture and a dense-graded mixture is the role of the fine aggregate. In a dense-graded mixture the fine aggregate is locked between large aggregate particles and load is transferred through the fine aggregate. In an SMA, fine aggregate particles are not intended to carry load, as shown in Figure 25. Fine aggregate particles are intended to form part of the mastic that fills up void space in the skeleton. In short, fine aggregate particles are active particles in dense-graded mixtures and they are inactive in SMA.

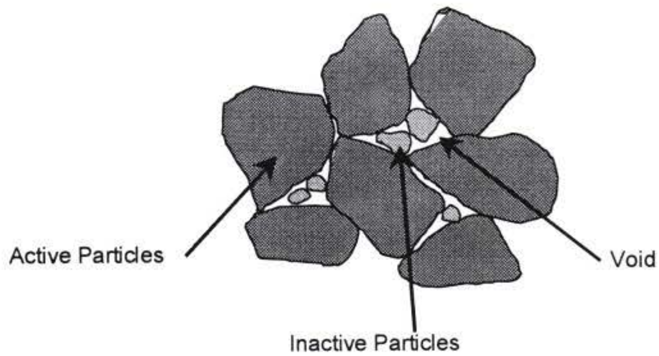

FIGURE 25 Structure and role of aggregate particles in a stone matrix asphalt mixture.

There are three separate and distinct aspects to the SMA mix design. First, the aggregate skeleton must be designed. The aggregate particles must be strong enough to carry load without crushing. Most importantly, the skeleton must be designed to have sufficient room for the mastic that will fill space in the skeleton.

Second, the mastic must be designed. To make the SMA tough, the mastic must be very stiff at high pavement service temperatures. Mineral filler is used to stiffen the asphalt binder. Some fine aggregate is also used to make the mastic resistant to flow at high temperatures. The mastic must be stiff enough at the production temperature to resist drain-down in the storage silo or trucks during transport. Modified asphalt binders are sometimes used to resist drain-down. Otherwise, fibers, usually cellulose or mineral fibers, are used.

Third, the mastic must be proportioned into the skeleton. If excess mastic is added to the SMA, the stone skeleton will be disrupted and rutting resistance will be lost. If the skeleton does not have enough space for the required amount of mastic, the mixture will have a low asphalt content and durability will be lost.

In 1990, members of the European Asphalt Study Tour identified SMA as a high-performance mixture with possible application in North America. The mixtures had been developed in Germany first as a two-step process and later as a single-step process. The two-step process consisted of first compacting a layer of uncoated coarse aggregate that would act as a stone skeleton. Next, mastic composed of asphalt binder, mineral filler, and fine aggregate

TABLE 8

FHWA GRADATION SPECIFICATION FOR STONE MATRIX ASPHALT *(126)*

Property (%)	Minimum (%)	Maximum (%)	Sieve Designation (mm)	Minimum Percent Passing	Maximum Percent Passing
Air voids	3.0	4.0	19.0	100	
Asphalt content	6.0		12.5	85	95
Voids mineral aggregate	17.0		9.5		75
Drain down		0.3	4.75	20	28
One fractured face	100		2.36	16	24
Two fractured faces	90		0.60	12	16
			0.30	12	15
			0.075	8	10

was heated, mixed, and squeegeed into the coarse aggregate. Later, the overall composition of SMA was analyzed and the mixture was manufactured in a single step at a hot-mix plant and was placed using conventional asphalt laydown and compaction equipment.

Several SMA projects were built in the United States (*117–121*). These mixtures were found to react differently than usual dense-graded mixtures (*122,123*). Volumetric properties of SMA did not respond to changes in composition the same as dense-graded mixtures do. Changing sand size aggregate did not change VMA, but larger aggregate did affect the VMA. Air voids were not as sensitive to changes in asphalt content. Adding asphalt led to draindown during construction causing badly flushed pavements that were unacceptable. Removing asphalt stiffened the mixture, making compaction difficult and leading to open porous mixtures.

During construction, the coarse aggregate skeleton was apparent by the reduced amount of roll down. If haul trucks bumped the paver, or if the paver stopped for a period of time, a surface bump was created that was not easily removed by the rollers.

SMA mixtures must be evaluated differently from dense-graded mixtures (*124*). A dense-graded mixture is evaluated as aggregate, asphalt binder, and air voids. An SMA must be considered as stone skeleton, mastic, and air voids (*125*).

Early projects were used to gain experience with the new mixture and to evaluate which European specifications required modification for North America. An FHWA Expert Task Group formed in 1991 developed the specifications that are summarized in Table 8 (*126*). The National Center for Asphalt Technology developed a mix design methodology using the Superpave gyratory compactor that could be used in North America (*127*).

The method includes design of a coarse aggregate skeleton, design of the mastic filling, and proportioning of mastic into the skeleton. Meeting the gradation is not sufficient for an acceptable design. The NCAT design method calculates the voids in the coarse aggregate (VCA) of the aggregate skeleton alone and with mastic included. Comparing VCA with and without mastic indicates whether the mastic is spreading the skeleton. Excess fine aggregate or fine aggregate particles that are too large for the spaces in the coarse aggregate will cause spreading. Spreading of the coarse aggregate is undesirable since fine aggregate particles become active particles and weaken the skeleton.

By 1996, SMA gained widespread use in Maryland and Georgia. In both states, SMA is used on all major highways. Maryland uses SMA as the wearing course as done in Europe. Georgia places a porous asphalt mixture on top of the SMA.

Porous Asphalt Mixtures

Porous asphalt mixtures are identified by their free-draining characteristic; water easily flows into the mixture. They are usually used as a surface layer to remove water from the pavement surface. Underneath, an impermeable layer acts as a floor on which the water flows, moving laterally across the pavement to exit at the pavement edge or to be collected in a subsurface drainage system below the pavement (*128–131*).

On occasions, porous mixtures are used for all layers in a pavement structure where the water is intended to drain completely through the pavement into the ground below. Usually such structures are used for parking areas if limits are placed on the amount of surface runoff that can be generated. Saturation of the subgrade or lower pavement layers limits use to low traffic pavements.

Function

Porous asphalt mixtures have been used in North America for many years. They have been known as open-graded friction courses, plant mix seal coats, and draining asphalt at different times (*130*). Although differences exist in specifications for these different products, one objective is common to all of them. They are intended to be porous and remove water from the pavement surface.

Porous asphalt mixtures are generally applied in a thin surface layer, less than 50 mm. Thicker layers are not stable on high-volume roadways (*132*). When applied in thin layers the mixture has a strong aggregate skeleton with stone-on-stone contact because there is not enough fine aggregate or asphalt binder to spread the coarse aggregate skeleton. In addition to good permanent deformation resistance, the mixture has enhanced safety properties. During wet weather, water is removed from the pavement surface, dramatically increasing driver visibility by reducing surface spray. At the same time, hydroplaning is prevented because free water is not allowed to collect on the pavement surface and cause loss of tire contact at high speed.

Porous asphalt has also been used to reduce traffic noise. Tire noise caused by compressed air underneath a rolling tire is reduced because air pockets in the pavement act as compression relief chambers. For high traffic volume roads in noise-sensitive areas, the application of porous asphalt has reduced noise by 3 to 4 decibels. A 3-decibel reduction in noise is a 50 percent reduction in sound pressure, a noticeable difference to the ear.

Mix Design

Porous asphalt is designed using a recipe mix design (*133*). A gradation is chosen within a specific band and a specified amount of asphalt binder is added. Open-graded friction courses used in the past have typically been designed at 12 to 15 percent air voids. The asphalt binder was typically unmodified asphalt. Draindown during construction was a problem. The asphalt content was kept low to allow constructibility.

Durability was the most common failure mechanism of open-graded friction courses. Low asphalt contents resulted in thin asphalt films that were constantly exposed to air. When the asphalt binder hardened excessively, raveling would begin. The progression to failure was rapid, often less than a few months from the time distress began until complete disintegration of the pavement layer. Compounded with aging, asphalt on the aggregate particles slowly drained down during the hot summer months, further reducing the asphalt binder film thickness and accelerating the aging and raveling.

The current generation of porous asphalt uses modified asphalt binders. Higher asphalt contents and thicker films of asphalt are possible with modified asphalt. The thicker films are more resistant to aging. In addition, modified asphalt binders are less susceptible to draining off the aggregate particles with time; hence the binder film retains thickness and raveling is prevented.

Another change in porous asphalt is an increase in design air voids. Current porous asphalt is designed with 20 percent air voids. The voids in open-graded friction courses tended to fill and plug up with debris, particularly in northern climates where deicing sand and salt are used. Porous asphalt with larger air voids is less prone to plugging. The larger void spaces tend to flush under the action of water being squeezed into and out of the pores. Nevertheless, porous asphalt that is used in northern climates is subjected to special considerations. In near-freezing conditions, porous asphalt tends to develop frost on the pavement surface sooner than impermeable pavements. To prevent clogging of the pores, sand is usually not mixed with deicing chemicals thus increasing winter maintenance costs. When ice forms on the pavement surface additional deicing chemicals are needed to remove the ice since salt brine flows down into the pavement and is prevented from dissolving additional ice.

In general, porous pavements tend to be used in non-freezing wet climates where rainfall is common but where winter maintenance and clogging caused by deicing chemicals are not problems.

The Oregon DOT has commonly used open-graded friction courses (*134,135*) and by experience have found that draindown is prevented if mixing temperature during construction is not above 130°C and if the mixture is not hauled long distances. The mixtures are not used in snow areas because of plow damage. Oregon reports that open-graded mixtures have less permanent deformation than dense-graded mixture and about the same friction value.

Experience in Spain (*129*) is typical of new porous asphalt. Mixtures are designed with more than 20 percent air voids. Asphalt content is 4.5 percent; the asphalt binder is polymer-modified. On high traffic volume roadways, 20,000 vehicles per day one way, a high-quality crushed aggregate is used. The mixture is placed 40 mm thick. Mixing temperature is typically 140°C and the mixture can be laid at temperatures as low as 120°C. Problems with winter maintenance are the only concern expressed.

Georgia found that these mixtures can be produced at about 160°C if polymer-modified asphalt and fiber stabilizers are used. A higher temperature more thoroughly dries the aggregate particles and improves asphalt adhesion. By stabilizing the asphalt film, draindown is no longer a problem.

INTERNATIONAL APPROACH

Internationally, the most commonly used method of mix design is the Marshall method (*136*). Slight variations and modifications are made to the method, not unlike agency specific modifications made to Marshall design in the United States. No examples of Hveem mix design were found outside of the United States.

In France, the LCPC developed a mix design method in the 1970s based around a gyratory compactor (*137–141*). The idea for the LCPC gyratory compactor originated following a technical visit to Texas in 1959 where the Texas gyratory compactor and the gyratory testing machine were observed.

LCPC developed the approach of monitoring specimen density during compaction that was subsequently used in Superpave. The LCPC angle of gyration is 1.00 degrees and the rate of gyration is 6 per minute. In the LCPC method, asphalt content is specified for each type of mixture. The design process consists of selecting an aggregate skeleton that will have 5 percent air voids at the design number of gyrations and air voids more than 11 percent at 10 gyrations.

The design number of gyrations is set based on the construction layer thickness. Pavement that is built in 80-mm thick lifts will use 80 gyrations for the design. Correlation studies were done using a pilot scale (200 kg) batch plant and a 2-meter spreader. Layers of different thickness were compacted with a full-scale single pneumatic tire roller. The number of passes to achieve an in-place air void content of 5 percent was obtained. The mixture rate of densification under the roller was compared to the rate in the gyratory compactor. The comparison is the basis for the number of design gyrations.

Each type of mixture has a specified laydown thickness. Laydown is typically thicker than in North America. Usually mixtures are placed 80- to 100-mm thick. Special mixtures referred to as thin mixtures are placed 50- to 60-mm thick and very thin mixtures are placed 40-mm thick. On high traffic volume and other important highways, specialized tests are used to guard against rutting, fatigue, and low-temperature cracking. Specialized equipment has been developed for each test. A plate compaction bench is used to compact specimens to be tested in the LCPC rut tester. Trapezoidal fatigue specimens are also cut from compacted slabs. The pieces are loaded at a frequency of 20 Hz to evaluate fatigue behavior. Cylindrical specimens for low-temperature cracking are cored from compacted slabs. The specimens are tested in direct tension at low temperature to determine low temperature behavior.

Currently members of the European Union are developing new standards. The Committee European Normalization (CEN) is evaluating test methods and mixture design methods. CEN will issue new standards that will supercede standards currently used in each country and all member countries will use the same standards. Superpave, Marshall, and the LCPC mix design methods are being evaluated. The standards are to be finalized and adopted in 1999.

CHAPTER FOUR

CONSTRUCTION OF RUT- RESISTANT MIXTURES

BUILDING THE MIX DESIGN

Historically, considerable effort has been placed on studies of material properties and mix design criteria to achieve durable pavements. Studies have focused on asphalt binder properties and aggregate properties. The effect of crushed faces, natural sand, and gradation have been evaluated on the mix design properties. Very large efforts have occurred in the area of acceptable asphalt content. Historically, many studies have tried to determine what is optimal asphalt content and how the content should be measured.

Quality control and quality acceptance testing consisted of confirming asphalt content and gradation. It was implicitly assumed that if gradation and asphalt content were correct, then mixture properties would be correct. This assumption proved to be insupportable (*10, 143–145*).

Change in Air Voids

In 1987, the Office of Technology Applications of the Federal Highway Administration (FHWA) developed mobile laboratories that visited active hot-mix plants. The initial objective of the "trailer" program was to verify volumetric properties of asphalt mixtures. At that time, many agencies did not have laboratory equipment to measure theoretical maximum specific gravity or to compact plant mixture in the laboratory compactor.

In attempting to verify 17 mix designs, the trailer found that 14 mixtures had air voids lower than the design air voids even though asphalt content and gradation conformed to the job mix formula (Figure 26) (*145*). Less than a quarter of the mixtures, 3 of 17 designs, had volumetric properties that matched the design. Another 6 of the designs had air voids within one percent of the design, allowing slight adjustments to be made to the asphalt content and gradation to bring volumetric properties into conformance. Another five of the mixtures were more than 2 percent lower than design, necessitating a complete shut down and re-design of the mixture.

If volumetric properties had not been checked, more than one-quarter of the mixtures designed at 4 percent air voids would have been produced at less than 2 percent air voids. Rutting would most likely occur on these projects.

In the 1980s as rutting became more prominent across North America, attention was focused on properties of the mixture in place. In 1987 the National Center for Asphalt Technology started a national study in the United States to identify causes of rutting and to recommend solutions (*87*).

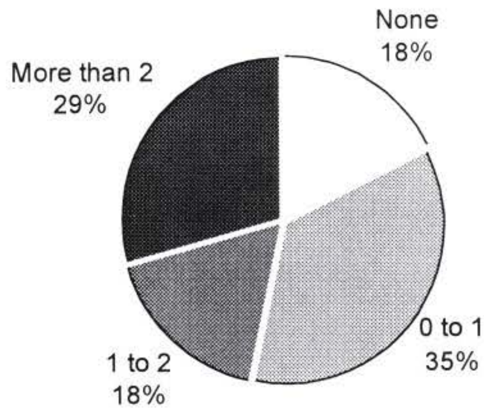

FIGURE 26 Plant mixture decrease in percent air voids from design to production (FHWA Trailer) *(145)*.

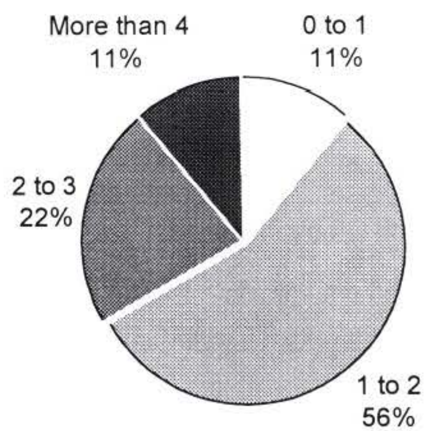

FIGURE 27 Plant mixture decrease in percent air voids (from NCAT National Rutting Study) *(86)*.

Properties of the mixture in place at 42 sites were compared to the mixture design that was used during construction. Most of the selected projects did not use compaction of plant mix during production to verify the mixture design. Cores had to be reheated and compacted in a Marshall compactor. Figure 27 shows the decrease in air voids from the mix design to the compacted reheated mixture. Significant decreases in air voids were found.

Remaining air void contents were low enough to cause rutting.

IN-PLACE PROPERTIES

The 42 sites in NCAT's study were subjectively rated as acceptable or unacceptable. A definition of performance was determined that would allow comparison of different age pavements carrying differing amounts of traffic. The selected parameter is rut depth divided by the square root of ESALs (*146*). Figure 28 shows the parameter for each site with the column shaded according to observed performance. Dark colored bars are acceptable performance; light colored bars are not. A value of 0.0058-mm per square root of ESALs was selected as a threshold between performing and non-performing pavements. This value is similar to the threshold, 0.0050-mm per square root of ESAL's found on pavements in Alabama (*88*).

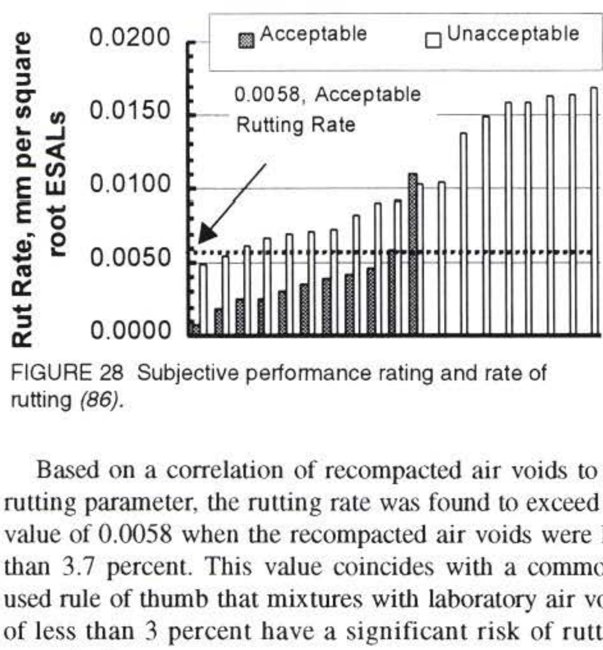

FIGURE 28 Subjective performance rating and rate of rutting *(86)*.

Based on a correlation of recompacted air voids to the rutting parameter, the rutting rate was found to exceed the value of 0.0058 when the recompacted air voids were less than 3.7 percent. This value coincides with a commonly used rule of thumb that mixtures with laboratory air voids of less than 3 percent have a significant risk of rutting excessively.

In the NCAT rutting study, the air voids of recompacted specimens were compared to the air voids of in-place wheelpath specimens (Figure 29). Generally, air voids in-place followed the air voids of the recompacted specimens.

Rutting performance matched in-place air-void content. Sections with good rutting performance had average air voids in the wheelpath of 4.7 percent; air voids of rutted sections were 1.9 percent. In summary, the NCAT rutting study showed the importance of volumetric properties, in particular, air voids for rutting performance. Pavements with low air voids tend to be pavements with rutting problems. The final in-place air voids are closely related

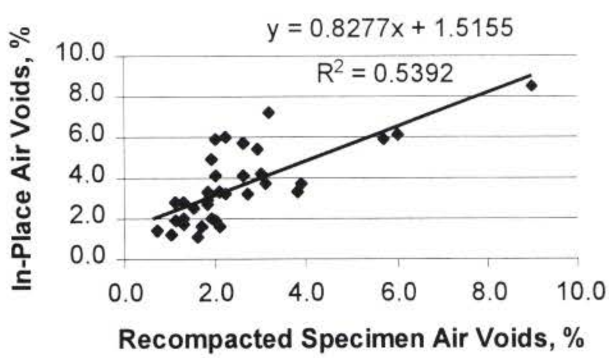

FIGURE 29 Comparison of recompacted air voids and in-place air voids in wheelpath, after NCAT *(86)*.

to air voids of laboratory compacted plant mix. The plant mix must be compacted in the laboratory compactor during production to monitor air voids. Reliance on asphalt content and gradation to match design air voids leads to low air void pavements.

Aschenbrener evaluated 33 sites in Colorado that contain at least one rutting and one non-rutting site from a range of sites encompassing traffic level and temperature environment in Colorado. Mixture was obtained from each site and recompacted in a Texas gyratory compactor. For light traffic load pavements, a correlation, r^2 of 0.68, was found between recompacted air voids and in-place voids. For high-traffic pavements, the coefficient was 0.78.

The Colorado study investigated the change in air voids from design to construction. They found the same trend as the FHWA trailers and the NCAT rutting study. Air voids of laboratory compacted plant mix are lower than the design. They found that sites with low recompacted air voids for the most part were rutting (Figure 30). Of 18 sites that are rutting, 14 have air voids less than 3 percent and four have air voids greater than 3 percent. Of 15 sites that are not rutting, 10 have air voids greater than 3 percent and five have air voids less than 3 percent.

The four best performing pavements in the Colorado study were evaluated to identify common properties. The

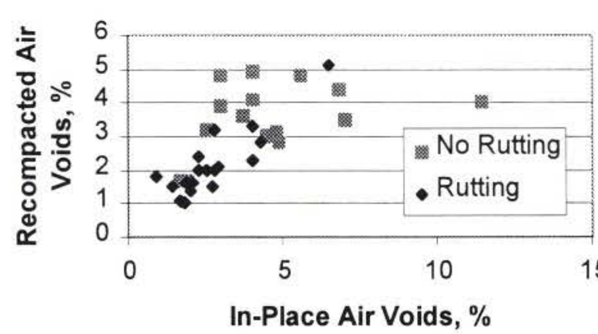

FIGURE 30 Comparison of in-place and recompacted voids, Colorado Study *(153)*.

four sites had no more than 2.5 mm of rutting and were 6 to 9 years old. Gradation and asphalt content were not the same for the mixtures. Some were fine-graded, some were coarse-graded, and others were in between. Asphalt content ranged from 5.0 to 6.1 percent. Two properties common among all the mixtures are air voids and Hveem stability. All of the sections have in-place air voids above 3.0 percent with Hveem stability values above 35.

Another study done on Pennsylvania DOT pavements investigated volumetric, aggregate, and asphalt properties. This study also confirmed the importance of air voids. As the percent of in-place air voids increased, the number of fair or poor sites decreased (*147*).

In-Place Density

Construction of hot-mix asphalt pavements can be considered as a two-step process. First, the hot mix must be manufactured to specification; second, it must be installed correctly. Performance of an asphalt pavement is dependent not only on correct manufacture of the hot mix, but on correct installation as well (*148,149*).

Excess air voids in the compacted mixture on the road creates a weaker skeleton that is permeable to air and water and is susceptible to durability distress (*150*). Air access to the interior causes advanced hardening and brittleness in the mixture. Water ingress can lead to stripping, especially under the action of traffic loads. In freezing climates, water inside the mixture can cause freeze-thaw damage. Ultimately, permeability of a mixture not designed to be open-graded will lead to disintegration and raveling.

Compaction to low air voids can cause rutting problems. Compaction of a verified mixture to 4 percent air voids should be very difficult. If the mixture can be compacted to less than 4 percent air voids on the road, the volumetric properties of the plant-mix are most likely not desirable. A mixture that is easily compactable on the road usually has low air voids in mixture verification specimens.

The performance-related specification experiment at WesTrack is investigating the performance of mixtures installed to different levels of density on the road. Two Superpave mixtures were designed. The mixture was the same; the only difference was the installed density. Properties of the plant-mixture were verified at the plant. As one part of the experiment, the mixtures were installed on the test track at 4, 8, and 12 percent air voids.

Figure 31 shows the difference in rutting that occurred in the different sections. The coarse mixture installed with 12 percent air voids rutted only slightly more than the

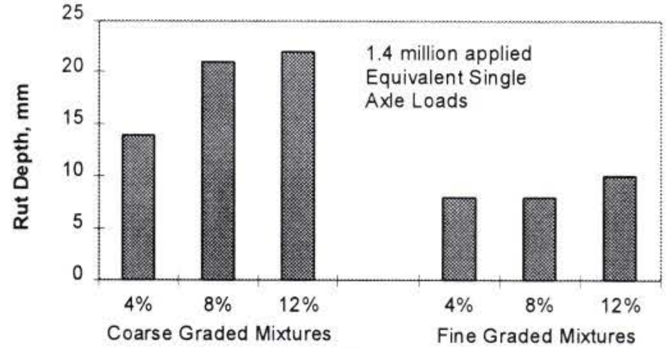

FIGURE 31 Performance of WesTrack mixtures.

same mixture installed at 8 percent air voids. The mixture installed at 4 percent air voids rutted 33 percent less than the 8 percent mixture.

In the fine mixture, the difference between the mixtures had the same trend but the range was less. The 12 percent air void mixture rutted only slightly more than the 8 percent and 4 percent mixture.

There is an improvement in rutting performance as the mixture is compacted to higher density. This improvement occurs only if the properties of the mixture as produced in the plant are kept constant. Increased density on the road does not necessarily produce improved performance if the properties of the plant mix change.

VOLUMETRIC ACCEPTANCE SPECIFICATIONS

Volumetric acceptance specifications are being promoted by FHWA in place of asphalt content and gradation, that is, mixture composition (*151,152*). Little or no emphasis is placed on gradation. Instead of focusing on gradation to determine acceptability of the mixture, gradation becomes a quality control item that the contractor uses to monitor the manufacturing process. Gradation information has value but not as an acceptance item.

Mixture rutting is most strongly related to volumetric properties, air voids in particular (*132*) and for that reason, air voids should be an acceptance item. Mixture composition acceptance is based on asphalt content and gradation. Volumetric mixture acceptance is based on asphalt content and volumetric properties.

Volumetric properties will vary during production. Normal variability inherent as part of the manufacturing process, sometimes referred to as chance variability, is caused by random fluctuations in materials properties and fluctuations in the manufacturing equipment as well as sampling and testing variability. Such variability is part of the manufacturing process.

TABLE 9

STANDARD DEVIATIONS OF VOLUMETRIC PROPERTIES FROM LABORATORY COMPACTED MIXTURE

Source	Compactor	Percent		
		Air Voids	VMA	VFA
NCHRP 9-7	Superpave Gyratory	0.73	0.55	3.25
FHWA (1513)	Superpave Gyratory	0.54	0.42	–
Colorado	Texas Gyratory	0.57	0.31	–
FHWA (AAPT)	Marshall	0.7	0.6	–
Virginia	Marshall	0.86	0.7	–
Virginia	Marshall	0.9	0.9	–

Variability outside the normal range is usually attributable to a variation in the process. For example, gradation of the incoming stockpiles changed, causing air voids to change, moisture content of the cold feed changed causing asphalt content to change, etc. Such changes are not part of normal manufacturing variability and should be corrected to prevent manufacture of poor quality mixture.

If volumetric properties are monitored, boundaries must be set on variability. To set acceptable boundaries, normal variability must be known. A summary of standard deviations from various studies is shown in Table 9. These standard deviations include variability in the manufacturing process and in the sampling and testing process.

Differences in standard deviation among the studies are created by differences in materials manufacturing and handling (e.g. crushing and stockpile management). They are also caused by differences in sampling and testing (e.g. sampling location and sampling method).

If sampling location and method are made constant, the standard deviation for different projects will be less varied. In Colorado, the standard deviation of air voids on four projects ranged from 0.44 to 0.64 percent (*153*).

The data in Table 10 indicated that standard deviation for air voids and VMA is less for mixtures compacted with a gyratory compactor than a Marshall compactor. Specification limits, shown in Table 10, are higher for the Marshall compactor than for a gyratory compactor.

INFLUENCE OF EXISTING CONTRACT SPECIFICATIONS

Contract specifications used to specify asphalt mixtures can influence mixture properties and hence performance even if the same mix design technology is used. This section discusses aspects of different contract specifications used and issues relating to the different approaches.

Mixture Sampling Point

• Samples of asphalt mixture for quality control or quality assurance can be taken at several different places. The most common are discharge chute of hot-mix plant; from truck box at the hot-mix plant; at the road before mix is placed; and behind the screed of the paver.

The issue of where to get a sample is based on operational and technical reasons. Samples obtained at the plant can be taken from the hot-mix plant discharge chute or from a truck about to leave the plant. Sampling at the plant has both advantages and disadvantages.

Advantages of sampling at the plant include:

1. Most convenient place to obtain a sample. Often the field laboratory and the laboratory technicians are located at the plant. Obtaining a sample and transporting to the laboratory simply means walking across the yard.

TABLE 10

VOLUMETRIC PROPERTY SPECIFICATION LIMITS FOR LABORATORY COMPACTED MIXTURE

Source	Compactor	Percent		
		Air Voids	VMA	VFA
NCHRP 9-7	Superpave Gyratory	± 1.0	± 1.0	± 5.0
FHWA (1513)	Superpave Gyratory	± 1.2	± 1.0	–
FHWA (1513)	Marshall	± 1.5	± 1.5	–
Colorado	Texas Gyratory	± 1.2	± 1.2	–

2. Fastest turnaround time for results. For quality control tests, a contractor needs real-time data to make control changes in the plant. Delayed data increases amount of potential out-of-specification mixture produced.

Disadvantages of sampling at the plant include:

1. Sample segregation more common. Requires entering the truck box, cutting a shelf in the mix and sampling using a shovel that has vertical sides. Entering the truck box is an employee safety issue. Leaning over the side of the truck box is safer but mix near the edge of the box is segregated.
2. Absorption process not complete. Absorption is a time-dependant phenomenon. The longer the time at high temperature, the more the absorption. This is not a problem with non-absorptive aggregates.

Samples obtained at the paver can be taken either before the mix has been placed or behind the screed. Sampling at the paver has advantages and disadvantages.

Advantages of sampling at the paver include:

1. All absorption has occurred that will occur in the mixture as placed. If haul time back to the laboratory is long, absorption will have stabilized. Additional absorption will not likely occur. If haul time to the project is short, absorption may still be occurring during sample transport back to the laboratory. The additional absorption is an allowance of absorption that has continued on the road during compaction.
2. Sample segregation is less common. Segregation of the sample to make it unrepresentative of the mixture is less likely to occur. A segregated sample represents segregation that actually did occur since properties of the mixture as placed will be compromised by the segregation.

Disadvantages of sampling at the paver include:

1. Least convenient place to obtain a sample. The sample must be transported from the road back to the field laboratory. Often the travel time can be significant, commonly 30 to 60 minutes, sometimes as much as 2 hours. During transport, the sample loses temperature and must be reheated. Insulated transport containers reduce the amount of cooling.
2. Slowest turnaround time for results. Considering a project 30 to 60 minutes from the plant and allowing for truck queuing time a mixture sample will arrive at the field laboratory more than 2 hours after leaving. If testing time is 1 to 2 hours for complete results, a half-day of production has occurred since the mixture sample was manufactured. If properties are out of compliance, it is impossible to know if the conditions that caused lack of compliance still exist.

The percentages of agencies that sample at the plant and at the project are shown in Figure 32. Thirty agencies report sampling at the plant and 20 report sampling at the project. Two agencies sample at both the project and the plant.

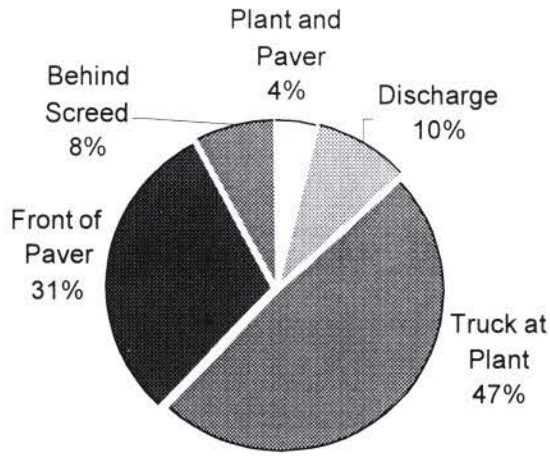

FIGURE 32 Sample location reported by agencies.

The needs of quality control and quality assurance testing differ. Quality control data is needed in time for control of the hot mix plant. Quality assurance data is needed to confirm that the agency has purchased materials as specified.

In QC/QA systems, obtaining quality control samples at the plant should be considered. Quality assurance samples should come from the road.

Contractors are concerned with quality assurance tests since payment is based on QA test results. Quality assurance data cannot be used to operate the hot-mix plant; there is not enough of it and it comes too late to effectively control the plant. Contractors may wish to test companion quality assurance samples to confirm QA test results.

Gradation Sampling Point

Gradation samples for quality control or quality assurance can be taken at several different places. The most common are

- Cold feed belt,
- Hot bins (batch plant),
- Solvent extraction aggregates, and
- Ignition oven extracted aggregates.

The sample locations can be categorized as before or after mixing with asphalt. Sampling before adding asphalt has advantages and disadvantages.

Advantages of sampling before adding asphalt include:

1. Quickest turnaround time. Samples from the cold feed belt need only to be dried and they are ready for testing. Hot bin samples can begin testing as soon as they cool enough to obtain the dry sample weight.

Disadvantages of sampling before adding asphalt include:

1. Degradation may be only partially accounted for. For some aggregates the pugmill in a batch plant will generate significant amounts of minus 0.075-mm material that can affect the mixture properties. For a drum plant, additional minus 0.075-mm material will be developed during heating and mixing in the drum.
2. Unless the plant is equipped with a cold feed sampler, the plant must be stopped to obtain a sample.

Almost all agencies monitor gradation as part of acceptance. Of 39 agencies that use pay factors, 30 agencies have a pay reduction factor for gradation. Ten agencies have a pay incentive factor for maintaining gradation within a tighter tolerance.

Asphalt Content Measurement

Asphalt content can be measured using several different methods. Each has advantages. The most common methods are:

1. Solvent extraction. A time-consuming method. Chlorinated solvents are becoming restricted because of environmental concerns. Solvent and wastes require special handling, storage, and disposal for hazardous materials. Non-chlorinated solvents do not have the environmental concerns but they are slower acting and lengthen test time.
2. Nuclear asphalt content gauge. A rapid method of measuring asphalt content. Requires calibration with mixture samples of known asphalt content. Machines are sensitive to environment. Requires special training, storage, and paperwork to comply with nuclear regulations. Aggregates are not available to perform gradation.
3. Ignition oven. A rapid and accurate method of determining asphalt content. Requires a calibration factor to account for aggregate weight loss that could be counted as asphalt.
4. Calculated from Rice theoretical gravity (Gmm). Easy to obtain as an additional piece of information if Rice test is run as part of air void determination. Assumes that effective specific gravity does not change. Changes in aggregate specific gravity, absorptivity of aggregate, or actual absorption are not accounted for.
5. Asphalt meter reading. An accurate indicator of asphalt content. Measures average asphalt content over the period. No indication of asphalt content variability can be obtained if the period is long.

Agencies allow one or more of these methods to be used on projects. Figure 33 shows the number of agencies that allow each method. Solvent extraction, the oldest method, remains widely accepted. Nuclear asphalt content gauges and ignition furnace ovens, both more recent, are commonly accepted as well. Back-calculation of asphalt content from Rice specific gravity or calculation based on reading of the asphalt meter is not as commonly accepted.

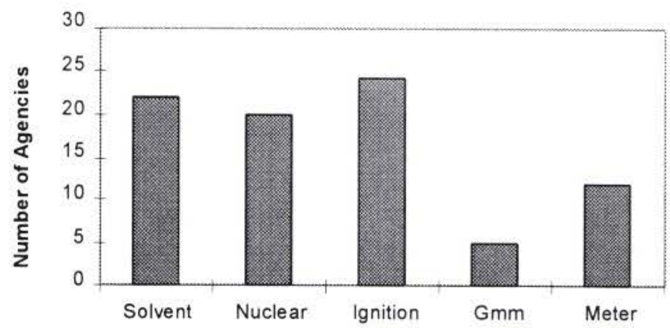

FIGURE 33 Asphalt binder measurement methods allowed by agencies.

Field Compacted Asphalt Mixture Specimens

Samples of plant mixed asphalt are compacted with the same compactor used in the design. Compacted specimens can be used for several purposes including to confirm conformance to mix design and to reference density for mixture compacted on the road.

Some agencies compact specimens to confirm the mix design. Others accept mixture based on results of the compacted specimens. The number of agencies that compact samples and the use of the samples is shown in Figure 34.

Of the 39 agencies that calculate air voids of laboratory compacted plant mix, 36 indicated the range of air voids desired. Twenty of the agencies require 3.0 to 5.0 percent air voids. Another 12 agencies require air voids to be higher. All but one agency report that the same test methods are used to calculate air voids in the field test specimens as in the laboratory design. The Rice theoretical specific gravity (Gmm) used in the calculation is measured in the field by all but five agencies. Two of the five agencies indicate that the Gmm value from the mix design is used to calculate voids. Another two calculate the Gmm value using the effective aggregate specific gravity from the mix design and the percent of asphalt measured in the

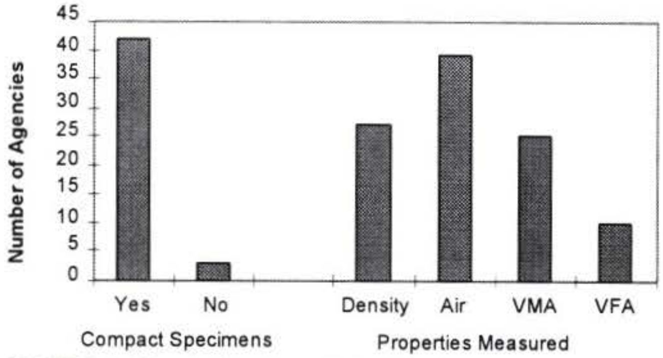

FIGURE 34 Use of laboratory compacted plant mix by agencies.

field. The last agency calculates a Gmm value using the apparent specific gravity from the design and the asphalt content measured in the field.

To obtain a valid determination of air voids in a specimen, a portion of the sample must be tested for Gmm. Depending on changes in aggregate characteristics or the amount of absorption, calculated values may or may not be close to the actual value.

The range allowed for air void specifications currently used by the agencies is shown in Figure 35. The range is similar to the ranges recommended by NCHRP 9-7 and by FHWA. The target air voids and the number of states using each target are listed in Table 11.

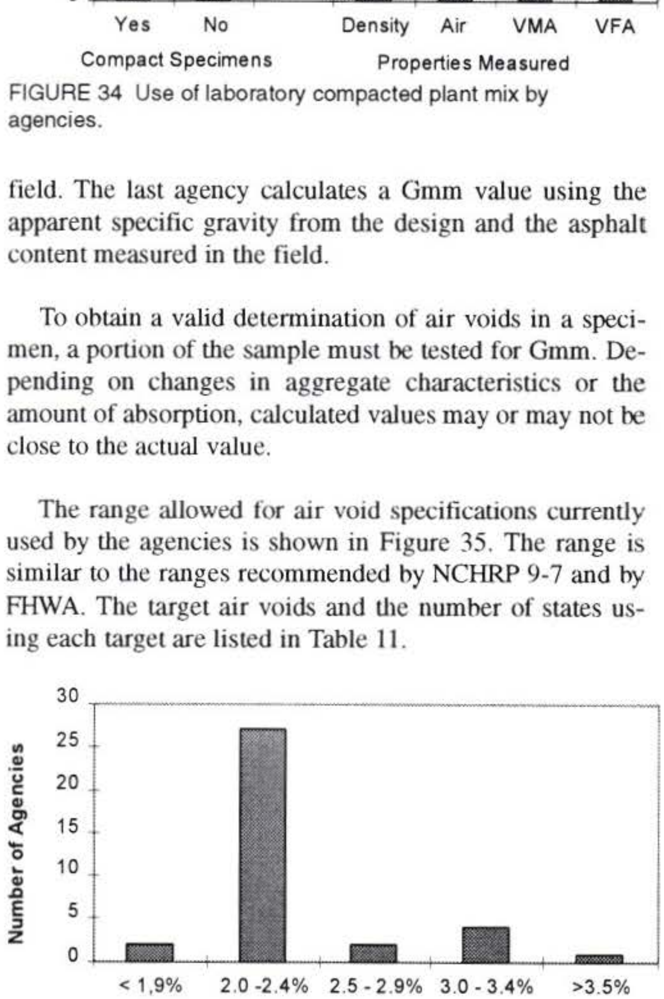

FIGURE 35 Range allowed for air void specifications.

TABLE 11

RANGE OF AIR VOIDS ACCEPTED BY VARIOUS STATES

Range	Number of States	Range	Number of States
2.8–5.2	1	4.0–6.0	3
3.0–5.0	20	3.5–5.0	1
3.0–6.0	4	3.5–5.5	2
3.0–6.5	1		

Road Density Specification

All of the agencies that returned a survey indicated that density on the road was specified. Several methods can be used to specify density on the road. The following methods were listed:

- Percent of job mix formula (JMF) density,
- Percent of JMF Rice theoretical maximum specific gravity,
- Percent of laboratory compacted plant mix,
- Percent of plant mix Rice theoretical specific gravity, and
- Percent of test strip.

Each method has subtle differences.

1. Percent of JMF density. JMF density is an uncomplicated approach to determining target density. It is known before construction begins and it never changes throughout the project. As discussed previously, plant mix tends to have a higher density than laboratory mix. The JMF density is therefore a low target, easier to meet than some other methods. The aggregate skeleton is not as strongly locked together and the pavement will be less rut resistant than it could be.
2. Percent of JMF Rice theoretical maximum specific gravity. The Rice gravity can change from the mix design to production for several reasons. Bulk specific gravity of the aggregate may be different than during design. More commonly, the amount of absorption in the field is different than in the laboratory. At a specified percent of JMF Rice gravity, air voids in the road are slightly higher than anticipated. The aggregate skeleton is not affected. Rut resistance is not impacted, but durability decreases.
3. Percent of laboratory compacted plant mix. Laboratory compacted plant mix specimens are verification tests confirming the volumetric properties. As mix properties change, density of the compacted specimen changes. If the asphalt content decreases, specimen density will decrease. Since the target density is determined as a percent of laboratory compacted density, the target density will decrease as well. There is a possibility of ending up with high in-place air voids but meeting the density specification.
4. Percent of plant mix Rice theoretical specific gravity. Specifying percent of Rice gravity means the target in-place air voids are always the same. Asphalt content may vary and Rice may vary with it, but the final in-place void content should be the same.
5. Percent of test strip. This requirement typically specifies that a test strip should be rolled until the density begins to decrease. The minimum acceptable density can be specified as a percent of laboratory compacted plant mix or plant mix Rice gravity. The target density then becomes a percent of the test strip.

Figure 36 shows the number of agencies that use the different methods to specify density. Most agencies recognize the importance of specifying density based on the actual plant mix produced, not the laboratory design mix. Only 10 agencies specify density based on JMF properties. Three agencies use the JMF density as a target, seven others use the Rice theoretical density of the JMF.

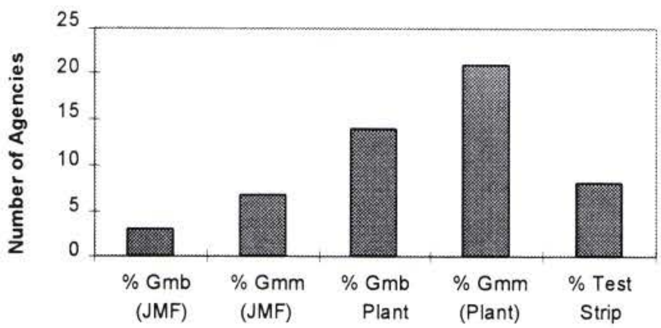

FIGURE 36 Density references used by agencies.

The rest of the agencies use properties of the plant-produced mixture. Fourteen specify density based on laboratory compacted density, 21 specified density based on Rice theoretical density of the plant produced material and another 10 agencies specify density based on roller test strip. The total number of agencies using each method is larger than the total number of agencies since some agencies report using more than one method.

Pay Factors for Plant Mix Properties

Properties of the plant mix that are monitored have already been discussed. This section will discuss pay factors that are used in quality assurance. Pay factors focus extra attention on the property they are attached to. By nature, the contractor focuses on items that influence the amount of income received for the project. Other properties may have an effect on the contract operations but less attention is paid to these items.

For example, most states have problems on occasion with segregation. Yet, there is no specification and no pay factor for segregation. A practical segregation test for use in a specification is not available. Therefore, in most agencies segregation is judged visually. Segregation is a "go/no go" criteria. If the agency believes the segregation is serious enough, the project will be stopped. However, the dividing line between serious and minor segregation is arguable and arguments often occur. The amount of segregation that ultimately occurs on a project sometimes depends on the personality strength of the contractor and the agency supervisor. Therefore, a property that is monitored but has no pay factor is less effective in controlling the project than a property that has a pay factor.

Based on the survey, 39 agencies report that pay factors are used; seven do not use pay factors. Figure 37 shows the number of agencies that use pay factors and the items pay factors are attached to.

Thirty of the 37 agencies use pay reduction factors for asphalt content and gradation. Despite the strong link between air voids and rutting, only 14 agencies use pay factors for air voids.

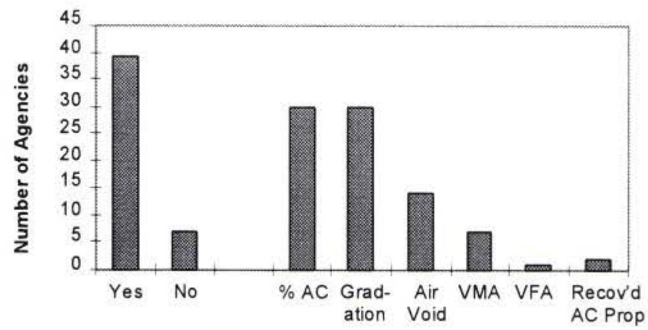

FIGURE 37 Use of pay reduction factors by agencies.

INNOVATIVE CONTRACTING METHODS

The method of contracting most commonly used by highway agencies is the low bid method. The agency writes a set of specifications and describes the work to be done with estimated quantities. Contractors bid a unit price for each of the contract items leading to a total bid price for the contract. The agency receives bids from prospective contractors and by law is required to accept the lowest qualified bid.

Innovative contracting methods have been investigated. In 1990, FHWA initiated Special Experimental Project 14 (SEP 14) to investigate innovative contract methods, including the use of warranties. Warranties may improve rutting resistance and durability performance by focusing contractor attention on materials, design, and construction issues that affect performance.

Warranty Construction

The use of warranties as applied to hot-mix asphalt pavements is just beginning in the United States (*154*). Whereas manufacturers of many consumer products offer warranties against defects that affect performance of the product, the application of warranties to paving materials is not yet commonplace. Introduction of warranties has been hampered for both technical reasons and administrative reasons.

Technically, the link between manufactured material and performance on the road is not well understood.

Construction variability and variances are known to influence performance of an asphalt pavement. Highway agencies have developed and modified specifications aimed at controlling construction quality to obtain a performing product. Yet, current specifications do not guarantee a durable pavement. A small percentage of pavements perform poorly even if construction complied with existing specifications. Hence, the contracting industry has been hesitant to accept responsibility for pavement performance.

Administrative rules have generally prevented warranties on federally funded projects. It was believed that costs would be lowest if the owner agency carried the risk of performance. Therefore, contractors have been content to leave the risk with the owner.

Despite the technical and administrative issues, warranties are being examined for use in asphalt pavements. Three types or levels of warranty are identified in which responsibility and risk are shifted incrementally from the agency to the contractor as follows:

1. The agency is responsible for all plans, specifications, and design criteria. The contractor warrants that all agency requirements have been fulfilled and the agency maintains responsibility for performance of the roadway. The contractor has little or no responsibility for performance of the pavement. This is the process currently used in the North America. Although not referred to as a warranty, the contractor is involved in meeting the requirements for design and construction as set out by the agency. Responsibility for performance of the pavement extends only to the point where the contractor establishes that the contract requirements have been met for materials and workmanship. If so, the consequences of poor performance fall to the agency to correct.
2. The agency is responsible for geometrical standards and structural thickness. The contractor is responsible for mix design criteria, construction specifications, materials selection, and construction quality control. The agency retains responsibility for structural failures or failures related to significant changes in traffic composition. The contractor assumes responsibility for failures related to mixture, materials, or workmanship and assumes the risk of poor performance attributable to mix design or mix construction. The agency retains the risk of poor performance from inadequate structural design. The division of risk is based on responsibility.
3. The contractor is responsible for structural specifications and mixture specifications as well as designs for both. The contractor is also responsible for both structural and mixture failures. This level of warranty is often referred to as design-build. The contractor is able to control the complete construction project. The contractor therefore, accepts all risk of failure.

The above discussion is by no means comprehensive. There are variations of each of the levels of warranty discussed. Generally though, risk shifts to the contractor as the contractor accepts more responsibility. In addition, the contractor requires an increased level of technical oversight as the warranty level increases. At the same time, agency demand for technical expertise decreases.

Prior to 1991, the FHWA had a longstanding policy that restricts the use of warranties on federal-aid projects. The rationale for the restriction was that such contract requirements might indirectly result in federal–aid funds participating in maintenance costs, which is prohibited by law.

The 1991 Intermodal Surface Transportation Efficiency Act permitted a state to exempt itself from FHWA oversight. For projects under these conditions, warranty clauses may be used in accordance with state procedures. In 1995, FHWA allowed warranty projects on all federal-aid projects. Warranty provisions must be for a specific construction product or feature. Routine maintenance is still not eligible and warranties on items not within the control of the contractor are prohibited. The Texas A&M Research Foundation is performing research for NCHRP Study 10-49. They will develop a model HMA warranty specification. The final report is expected in January 1999.

Eleven states have used warranty provisions for hot-mix asphalt (HMA) or rubberized HMA pavements (Ala., Cal., Col., Fla., Ind., Maine, Mich., Mo., Oh., Nev., Wis.). Ten have specified rutting as a performance criteria for the warranty. Data were not available for Nevada.

Definition of Success

The principle benefit of a warranty contract to improving pavement performance is a shift in focus that occurs from specifications to performance. Non-warranty contracts focus on specifications. The definition of a successful contract is one where the specifications were met. Anticipated performance of the finished product is not part of the contract. Of course, standard specifications have been developed to provide a performing product and the anticipation is that the pavement will perform but performance itself is a by-product of the specifications. The project may fail completely in 2 years, yet the original contract would still be considered successful.

In a warranty contract, the definition of success is a pavement that has performed to the level specified.

Performance is the central focus of the contract requirements and is the final specification. Specifications regarding how the project is to be built become less important than the anticipated performance.

QUALITY CULTURE

Quality culture in this synthesis is considered the formal establishment of procedures and practices that lead to quality improvements in pavement. These include:

1. National Quality Initiative (NQI)
2. International Standards Organization (ISO) Certification
3. Training
4. AASHTO Materials Reference Laboratory Inspection and Accreditation
5. Industry award programs.

The National Quality Initiative is a customer driven approach to providing high-quality highways for the public. Agencies and industry have combined to pledge support to improving the quality of products produced. Individual contractors are not members of NQI but specific contractors have spoken at NQI meetings regarding quality processes used in their organizations. Contractor representatives are an integral part of the NQI effort.

A novel approach, which some believe increases quality, is to use warranties. Pavements can be warranted for engineering defects or public perception of quality. Engineering defects are items that a pavement engineer would use to evaluate a pavement such as permanent deformation, fatigue cracking, and raveling. The public's perception of pavement quality includes smoothness and safety. Warranty items that address the public perception include international roughness index (IRI) for smoothness, rut depth to prevent hydroplaning, and friction for safety.

ISO certification is a rigorous set of standards that are used in all manufacturing. The standards developed in Europe, for trade within the European Common Market, have become worldwide standards for manufacturing. No indication was found of the number of asphalt contracting firms that are adopting ISO standards, although some are known to be in the process.

Training is an important part of the quality culture. Specifications alone are not sufficient to obtain durable rut-resistant asphalt pavements. The mix designs must be done correctly to determine the job mix formula. During constructions, skilled technicians who are qualified to perform the required testing must monitor the mixture. Finally, the mixture must be accepted based on test results obtained from a qualified technician.

Training plays a key role in ensuring that design, quality control, and quality assurance data are accurate and valid. Many states have training programs to qualify technicians, both industry and agency. Often, there is more than one level of qualification. An example of requirements is shown below:

Roadway Control	An understanding of important characteristics for an asphalt pavement during placement. The ability to determine quantities, measure density, and correct mat deficiencies.
Plant Control	An understanding of important properties to be monitored during manufacture of hot-mix. The ability to measure gradation and asphalt content, to compact mixture, and control volumetric properties.
Mix Design	An understanding of important characteristics of a mix design and the relation to performance. The ability to perform all tests required in a mix design.

The American Association of State Highway and Transportation Officials (AASHTO) operates a materials reference library (AMRL) and several programs that are aimed at testing laboratories.

The AMRL sample proficiency program allows laboratories to receive semi-annual samples to be tested using AASHTO test methods. Results are collated and the participating laboratory is rated according to deviation from the average test results.

The AMRL inspection program includes an on-site inspection of the testing laboratory. The inspector witnesses sample testing and checks equipment used. The laboratory can then make changes to ensure that AASHTO test standards are being met.

The AMRL accreditation program is the highest quality program available from AASHTO. In addition to a visiting inspector who certifies that equipment meets AASHTO standards and test operators are proficient, the participating laboratory sets up a quality control system. The system ensures that equipment is kept within specification, that test operators are observed for specific tests, and that proficiency is demonstrated routinely. AMRL inspectors confirm that the quality control system is functional and up to date.

State and national associations give industry awards for paving excellence. The National Asphalt Pavement Association presents annual awards for outstanding projects. The highest of these awards, the Sheldon Hayes Award is one of the most coveted awards in the paving industry. State highway agencies also have award programs to select the best examples of paving for various types of projects within the state. Each of these programs is intended to reward quality paving with recognition.

CHAPTER FIVE

CONCLUSIONS

The objective of this synthesis is to identify methods to construct rut-resistant asphalt pavements that are durable and resistant to moisture damage. Surveys indicate that the percentage of rehabilitation budgets spent on rutted pavements has increased in 1997 as compared to 1987. This increase does not indicate a worsening rutting problem because rehabilitation trigger values are lower in 1997. The 1997 survey indicates that rutting is not the major cause of rehabilitation. Expenditure for fatigue-cracked pavements is more than double the expenditure for rutted pavements.

Superpave, the new mix method developed by the Strategic Highway Research Program promises improved rut resistance as compared with existing mix design methods. Performance-graded binders can be specified to match environmental conditions. Aggregate criteria to create a strong rut-resistant aggregate skeleton are specified according to the expected traffic level. Often the aggregate criteria are more stringent than criteria used in existing mix design methods. In Superpave, the gyratory compactor provides an indication of aggregate skeleton strength that is not available from a Marshall or kneading compactor.

Asphalt binder can contribute significantly to mixture rut resistance. Binders that are designed to have greater stiffness at high temperatures outperform binders with low stiffness.

Contradictory evidence exists regarding the effect of gradation on rutting resistance. Mixtures with high sand content, even if the sand is manufactured, are generally less resistant to rutting than coarser mixtures. Similarly, mixtures with stone-rich coarse gradations develop more shear resistance than more finely graded coarse aggregate skeletons. The role of asphalt binder in coarse mixtures is important to achieving rut resistance. A coarse skeleton with a soft asphalt binder may not be as rut resistant as a fine skeleton with the same binder.

Fractured faces is the method most commonly used to specify coarse aggregate angularity. Increased fractured faces increase resistance to rutting.

Fine aggregate angularity is measured by a number of methods, including the fine aggregate angularity test, particle index, and classification as manufactured aggregate. Field experiments indicate that fine aggregate angularity has a greater effect on rutting resistance than coarse aggregate angularity.

The literature contains much about supplementary tests that can be used in conjunction with mix design to identify rut-susceptible mixtures. Rut testers of several varieties, creep tests, stiffness tests, and shear tests are used or are considered by agencies on some mix designs. There is no clear evidence to indicate a preferred method.

Two alternate mixtures were identified that provide increased rut resistance. Stone matrix asphalt and porous mixtures are used by some agencies as the surface mixture. Both of these mixtures have enhanced rut-resistance properties derived from a strong stone skeleton.

An alternate method of mix design, the LCPC method used in France, is distinctly different from Marshall or Hveem mix design. The LCPC method more closely resembles the Superpave method with the gyratory compactor used to control aggregate skeleton strength. The method uses coarsely graded mixtures with stiff asphalt binders and high dust contents to stiffen the asphalt mastic.

Mixture verification is an essential step in constructing a rut-resistant mixture. The first step includes verification of constituent materials. Aggregate and asphalt binder properties must meet the design. The second and equally important verification is ensuring that the volumetric properties of the plant-produced material meet the mix design criteria. Typically, volumetric properties of mixtures will degrade from design to production even if asphalt content and gradation meet the design. Adjustments must be made to bring volumetric properties into compliance or the risk of rutting will increase. For a given set of constituent materials, performance of the mixture will be directly dependent on the volumetric properties. Acceptance specifications should be based on volumetric properties with less emphasis on gradation.

Density of the placed mix influences the rut resistance of plant-verified mixture. Typically, specifications require mixture to be compacted to six to eight percent air voids on the road. If the mixture is placed with higher air voids, the aggregate skeleton will be less solidified and the rutting susceptibility will increase. At higher in-place air voids there is risk of water entering the pavement causing potential moisture damage. Interconnected air voids also allow intrusion of air into the pavement and result in accelerated aging of the asphalt binder film.

The influence of contract specifications on mixture accepted by highway agencies is not well-discussed in published literature. Most published literature deals with material properties or mix design or mixture properties. Asphalt pavements are construction sensitive; that is, performance is strongly influenced by construction practices. Specifications used for asphalt mixtures have a direct influence on construction practices and mixture properties on the road.

Pay factors are an important part of contract specifications. Key properties that influence mixture performance should have a pay factor attached. A property that is monitored but has no pay factor is a less effective control than one that has a pay factor attached.

During mix production, asphalt content and laboratory compacted air voids have the most impact on rutting and durability. Pay factors, if used, should at least be placed on these two parameters.

Some innovative contracting methods shift responsibility for performance. Warranties, for example, shift responsibility from the highway agency to the contractor. Warranty periods are typically 3 to 5 years. In effect, economic implications to the contractor of poor performance replace the highway agency as a control of quality during construction.

A review of all the information leads to the following conclusions:

- The Superpave performance-graded asphalt binder specification offers a better-defined specification with properties that are directly related to performance. Many agencies are switching to PG specifications and several more indicate that they will be. Modified asphalt binder use will most likely continue to grow. Modified binders, whether required under current specifications or under PG specifications, are more expensive than unmodified. Evidence continues to mount that the modified binders offer improved performance as compared with unmodified binders.
- Agencies are requiring more crushed faces in coarse aggregate. The trend is increasing and Superpave volumetric mix design generally calls for more fractured faces than some agencies are currently using. In areas of the country dominated by gravel sources, an increase in fracture requirement is costly. Generally, agencies have increased fracture contents with reported beneficial results. The trend will be to increase fracture content for high truck volume roads where the benefits justify the cost.
- Agencies have developed more stringent specifications for fine aggregate. Fine aggregate angularity and particle index have been used to categorize fine aggregates. Many agencies have placed limits on natural sand in mixtures used on high-volume roads. The cost of limiting natural sand in areas that are predominantly sand and gravel can be high.
- Aggregate gradations have become coarser over the last decade as agencies require the use of more rock and less sand in mixtures. The gradations commonly used are generally fine as compared to the coarse Superpave gradations. Current findings at the WesTrack research facility may affect the current move toward coarser gradations.
- Marshall and Hveem mix design remain the most common mix design methods in use.
- Some agencies are tentatively implementing Superpave mix design. Others are more aggressive. Generally, a move is expected toward implementation of Superpave mix design. Although the oldest Superpave projects are only 6 years old, experience with Superpave is encouraging.
- Other approaches to mix design have been researched, such as the Asphalt Aggregate Mixture Analysis System (AAMAS), developed in the late 1980s, but implementation has been very limited, practically nonexistent.
- Currently the greatest need in the asphalt community is the ability to accurately judge the adequacy of a mixture for a proposed application. Research in this area is very active and many tests have been evaluated. Although many tests are proposed, there are no performance tests with criteria developed for different traffic and environments.
- The SHRP program tried to develop a mixture performance test with performance prediction models. Performance tests and prediction methods, developed as part of Superpave, offer promise but none is ready for implementation.
- Rut testers have been developed to simulate the kneading action of rolling tires and identify potential rut-prone mixtures. Several varieties of rut testers have been developed and locally they are used by some agencies. The ability to identify rut-prone mixtures for all varieties of rut testers is mixed, based on a national study of WesTrack and FHWA mixtures at the Turner-Fairbank laboratory. Agencies will most likely continue to investigate use of rut testers based on the simulative nature of the test and the lack of a suitable alternative.
- Acceptance of mixtures based on asphalt content and gradation has been the most common method used in the past and remains a common method among many states. Evidence continues to mount to show that acceptance based on gradation and asphalt content may systematically build rut-prone pavements. Agencies have been switching to acceptance based on volumetric property, which is better linked to rutting performance.
- Pay factors are an important part of the acceptance system. Pay factors should be placed on properties important

to performance. Items with pay factors attached receive the greatest attention during construction and so should include properties important to quality assurance. The NCHRP project 9-7 recommends the use of a volumetric based acceptance system for Superpave mixtures and the use of a percent-within-limits approach to accept mixtures.

• Warranted asphalt pavement is an innovative contracting method that offers promise to improve performance of pavements. The responsibility for performance is shifted from the agency to the industry. Several obstacles must be overcome to implement this system on a widespread basis. On the limited number of projects that have used warranty specification, the results have been encouraging.

• France has developed a system that is most remarkably different from the system used in North America and the rest of Europe. The entire approach to design is different, including the design philosophy and the construction techniques used. Many similarities exist between France and North America, which suggests that the system could be applied in North America.

• Stone matrix asphalt (SMA) and porous asphalt are highly rut-resistant mixtures. Many agencies have constructed trial sections of SMA and some routinely use SMA in high traffic volume applications.

• Open-graded mixtures are used by some agencies on high traffic volume roads for both rut resistance and drainage of water from the pavement surface.

Further research is needed in the following areas:

• A performance test is needed to identify mixtures prone to rutting.

• Development of a simple fatigue test is needed.

• Performance models that can be used to predict the degree of distress accumulation are needed.

• Contracting methods need to be developed to supply the benefits of warranted construction without some of the drawbacks.

• Changes in construction methods to achieve higher density, such as thick lift construction, should be evaluated.

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APPENDIX A

Survey Questionnaire

National Cooperative Highway Research Program

NCHRP Synthesis Topic 28-06

QUESTIONNAIRE

METHODS TO ACHIEVE RUT RESISTANT DURABLE ASPHALT PAVEMENTS

PURPOSE OF THIS SURVEY

Permanent deformation continues to be a leading form of pavement distress in the United States. Several state and federal agencies have performed research projects or developed specifications aimed at addressing the permanent deformation and durability problem. Several states have recently made significant changes in their specifications to minimize permanent deformation. A synthesis of current practice, including recent research findings, is being developed to aid owner agencies and the construction industry in developing high-performance, rut resistant, durable pavements.

Thank you for filling out this survey. Please complete the following information:

Agency: ______________________________

Address: ______________________________
____
____

City: ______________ State: __________ ZIP________

Questionnaire Completed By: ______________________________

Position/Title: ______________________ Date: ______________
Telephone: ______________________________
Fax: ______________________________
E-Mail: ______________________________

RETURN QUESTIONNAIRE AND SUPPORTING DOCUMENTS BY *May 2, 1997*

TO: Gerry Huber
5698 North 375 East
Pittsboro, IN 46167

For questions contact him on e-mail: gahuber@aol.com

THE SIZE OF THE PROBLEM

How much of a problem is permanent deformation? Answers to the following questions will provide a national perspective. This section looks at the extent of rutting as a problem in the high traffic volume (i. e. National Highway System) pavements of your network. You may need to consult your pavement management system for the basic information. I hope that your capital programming people will know the cause of rehabilitation.

1. How many miles of asphalt surfaced pavements are in your entire road network? (Answer in centerline miles or lane miles, as you prefer.)

   ____________ ❑ Centerline miles
   ____________ ❑ Lane miles

2. How many miles of asphalt surfaced pavements are in your NHS network? (Answer in centerline miles or lane miles, as you prefer.)

   ____________ ❑ Centerline miles
   ____________ ❑ Lane miles

3. How many miles of NHS asphalt pavements are resurfaced or rebuilt in a typical year? (Answer in centerline miles or lane miles, as you prefer.)

   ____________ ❑ Centerline miles
   ____________ ❑ Lane miles

4. How many miles of NHS asphalt pavement are resurfaced or rebuilt each year where excessive rutting is the main rea son? (Answer in centerline miles or lane miles, as you prefer.)

   ____________ ❑ Centerline miles
   ____________ ❑ Lane miles

5. On the National Highway System, what severity of rutting is considered a threshold to program a highway segment for rehabilitation?

   ❑ <0.2" ❑ 0.2" to 0.4"
   ❑ <0.4" to 0.8" ❑ >0.8"

6. What is the average age of resurfaced or rebuilt NHS pavements where rutting is the main reason for construction? This may be a guess but the intent is to find out how much of a problem early and mid-life rutting is.

   _____ Years, OR __% 1 to 3 yrs __% 3 to 5 yrs
   __% 5 to 10 yrs __% 10 to 15 yrs
   __% 15 to 20 yrs __% > 20 yrs

7. Is stripping, (water damage to an asphalt mixture) a cause of pavement rehabilitation in your agency?

   ❑ yes ❑ no

8. How many miles of NHS asphalt pavement are resurfaced or rebuilt each year where stripping is the main reason? This may be a guess but the intent is to find out how much of a problem stripping is. (Answer in centerline miles or lane miles, as you prefer.)

   ____________ ❑ Centerline miles
   ____________ ❑ Lane miles

9. Is fatigue cracking a cause of pavement rehabilitation in your agency?
   ❑ yes ❑ no

10. How many miles of NHS asphalt pavement are resurfaced or rebuilt each year where fatigue cracking is the main reason? This may be a guess but the intent is to find out how much of a problem fatigue cracking is. (Answer in centerline miles or lane miles, as you prefer.)

    ____________ ❑ Centerline miles
    ____________ ❑ Lane miles

MATERIALS

This section looks at the materials typically specified for use. All of the questions are directed to the high traffic volume pavements, i.e., the National Highway System or the Interstate Highway System. The objective is to develop a current picture of materials used in the country. If there is more than one answer because of different geographic locations, you may provide more than one answer for each question.

11. What grade of asphalt binder is typically used in the upper layers? __________

12. What grade of asphalt binder is typically used in the lower layers? __________

13. Is modified asphalt used on:

all projects (100%)	❑ yes	❑ no
most projects (61 to 99%)	❑ yes	❑ no
some projects (21 to 60%)	❑ yes	❑ no
a few projects (1 to 20%)	❑ yes	❑ no
no projects (0%)	❑ yes	❑ no

14. Is crushed two faces an aggregate requirement? ❑ yes ❑ no

    If so, what is the requirement for surface mixtures on interstate pavements? __________

15. Is crushed-one-face an aggregate requirement? ❑ yes ❑ no

    If so, what is the requirement for surface mixtures on interstate pavements? __________

16. Is sand equivalent an aggregate requirement? ❑ yes ❑ no

    If so, what is the requirement for surface mixtures on interstate pavements? __________

17. Is another method used for aggregate cleanliness? ❑ yes ❑ no __________

If yes, what is the test? ____________________

What value is used for interstate pavement? ____________________

18. Is fine aggregate angularity a requirement? ❑ yes ❑ no

If so, what is the requirement for surface mixtures on interstate pavements? __________

19. Is another method used to control inter-particle friction of fine aggregate? ❑ yes ❑ no

If yes, what is the test? ____________________

What value is used for interstate pavement? ____________________

20. Have significant changes been made in the recent past to material requirements in your agency?

❑ yes ❑ no

If yes, what changes to improve rutting resistance have been made, when were they made and how effective do you think they are?

MIXTURE DESIGN

This section looks at the mixture design specified for use. All of the questions are directed to the high traffic volume pavements, i.e., the National Highway System or the Interstate Highway System. The objective is to develop a national view of current mix design practice.

21. What type of mix design is typically used

❑ none
❑ 50 blow, static base Marshall hammer
❑ 75 blow, static base Marshall hammer
❑ 50 blow, rotating base Marshall hammer
❑ 75 blow, rotating base Marshall hammer
❑ Hveem mix design with kneading compactor
❑ Hveem mix design with gyratory compactor
❑ other, __________

22. Are any additional tests used to indicate the rutting potential of an asphalt mix design?

❑ yes ❑ no

If yes, what is the test? ____________________

What value is used for interstate pavement? ____________________

22. Are any additional tests used to indicate the fatigue resistance of an asphalt mix design?

❑ yes ❑ no

If yes, what is the test? ____________________

What value is used for interstate pavement? ____________________

24. Are any additional tests used to indicate the stripping potential of an asphalt mix design?

❑ yes ❑ no

If yes, what is the test? ______________________

What value is used for interstate pavement? ______________________

25. Are air voids a specified design requirement?

❑ yes ❑ no

If so, what is the range of air void values allowed?

minimum __ % maximum ____ %

26. How is maximum theoretical specific gravity determined?

❑ measured Rice method
❑ Calculated based on aggregate bulk specific gravity and asphalt specific gravity
❑ Calculated based on aggregate effective specific gravity and asphalt specific gravity
❑ Calculated based on aggregate apparent specific gravity and asphalt specific gravity
❑ other,______________________

27. Are voids in the mineral aggregate (VMA) a specified requirement?

❑ yes ❑ no

If so, what is the range of VMA values allowed for a ½ inch nominal size mix? (i.e. 100% passing ¾ inch and 90 - 100% passing ½ inch)

minimum __ % maximum ____ % (if used)

28. Does any other properties influence the design criteria for VMA?

❑ yes ❑ no

If yes, what is it? ______________________

What effect does it have on the design requirement?______________________

29. How are voids in the mineral aggregate calculated?

❑ Based on aggregate bulk specific gravity.
❑ Based on aggregate effective specific gravity.
❑ Based on aggregate apparent specific gravity.
❑ other, ______________________

30. If VMA is not used, is a minimum asphalt content specified? ❑ yes ❑ no

If yes, does the requirement vary with mixture size? ❑ yes ❑ no

Does the requirement vary with gradation (i.e. coarse, fine) ❑ yes ❑ no

31. Are any other volumetric properties specified during design that you believe are important for good permanent deformation performance?

❑ yes ❑ no

If yes, what are they? ____________________

What value is used for interstate pavement? ____________________

32. What is a typical gradation used for a surface mixture? (Use standard sieves or fill in sieve sizes used in your state.)

Sieve	% Passing
1 in	
¾ in	
½ in	
3/8 in	
#4	
#8	
#16	
#30	
#50	
#100	
#200	

Sieve	% Passing

33. What is the predominant aggregate parent material in a typical surface mixture?

❑ granite
❑ basalt
❑ limestone
❑ dolomite
❑ slag
❑ diorite
❑ other, ____________________

34. What is the source of the predominant aggregate parent material?

❑ gravel source
❑ quarry
❑ other, ____________________

35. Have significant changes been made in the recent past to mix design requirements in your agency?

❑ yes ❑ no

If yes, what changes to improve rutting resistance have been made, when were they made and how effective do you think they are?

MANUFACTURE OF HOT MIX ASPHALT AT THE PLANT

This section considers production issues at the hot mix plant. The questions are directed at quality assurance testing rather than quality control testing. All of the questions are directed to mixture produced for high traffic volume pavements, i.e., the National Highway System or the Interstate Highway System. Just as the mix design is important to specify the design properties of a mix, the methods of quality assurance influence properties of the mixture actually produced.

36. Where are the hot mix samples taken for quality assurance testing?

- ❑ at plant discharge chute
- ❑ rom truck at hot mix plant
- ❑ from truck at paver
- ❑ from loose mat behind the screed.
- ❑ other, ______________________

37. Is gradation of the hot mix monitored for quality assurance? ❑ yes ❑ no

If yes, how is it measured?

- ❑ combined cold feed aggregates
- ❑ solvent extracted aggregates
- ❑ ignition oven extracted aggregates
- ❑ other, ______________________

38. Is asphalt content of the hot mix monitored for quality assurance? ❑ yes ❑ no

If yes, how is it measured?

- ❑ solvent extraction
- ❑ nuclear asphalt content gage
- ❑ ignition oven
- ❑ back calculated from theoretical maximum density
- ❑ asphalt plant meter readings
- ❑ other, ______________________

39. Are specimens of plant mix compacted using the same type of laboratory compactor used in the mix design?

❑ yes ❑ no

If yes, which properties are monitored for quality assurance?

- ❑ mixture density
- ❑ air voids
- ❑ VMA
- ❑ voids filled with asphalt
- ❑ other, ______________________

40. If air voids are measured in the laboratory-compacted plant-mix specimens, what is the allowable range?

minimum ___ maximum ___ how many tests ___

41. Are the air voids calculated using the same test methods used in the mix design? ❑ yes ❑ no

42. Is the theoretical maximum specific gravity measured on plant mix? ❑ yes ❑ no

If no, what value is used for theoretical maximum specific gravity? ______________________

43. If VMA is measured in the laboratory-compacted plant-mix specimens, what is the allowable range?

minimum_____ % maximum_____% how many tests________

44. Are any additional tests used to indicate the rutting potential of an asphalt mix design? ❑ yes ❑ no

If yes, what is the test? ___________________________________

What value is used for interstate pavement? ___________________________________

45. If VFA is measured in the laboratory-compacted plant-mix specimens, what is the allowable range?

minimum_____ % maximum_____% how many tests________

46. Have significant changes been made in the recent past to hot mix-manufacturing requirements in your agency?

❑ yes ❑ no

If yes, what changes to improve rutting resistance have been made, when were they made and how effective do you think they are?

PLACEMENT PROPERTIES

This section considers the hot mix as compacted on the roadway. All of the questions are directed to mixture place on high traffic volume pavements, i.e., the National Highway System or the Interstate Highway System. Properties of asphalt mixtures as placed strongly influence mixture performance. The objective of this section is to obtain a national view of placement requirements.

47. Is the density of the compacted mixture on the road specified? ❑ yes ❑ no

If yes, what reference value is used?

❑ % laboratory density of the job mix formula
❑ % density of the lab-compacted plant mix
❑ % theoretical maximum specific gravity of the job mix formula
❑ % theoretical maximum specific gravity of the lab-compacted plant mix
❑ % test-strip density
❑ other, ___________________________________

If no, how is the rolling pattern governed?

❑ not controlled
❑ standard specified rollers and passes
❑ rollers and passes based on test strip
❑ other, ___________________________________

48. How is segregation controlled on the project?

❑ Visually
If so, what visual method is used? ___________________________________

❑ Measurement
If so, what measurement method is used? ___________________________________

49. Have significant changes been made in the recent past to hot mix placement requirements in your agency?

❑ yes ❑ no

If yes, what changes to improve rutting resistance have been made, when were they made and how effective do you think they are?

PAYMENT FACTORS

Payment factors can cause a shift in emphasis during construction causing attention to be focused on items that could reduce payment for the mixture. This section considers payment factors for hot mix placed on high traffic volume pavements, i.e., the National Highway System or the Interstate Highway System. The objective is to obtain a national view of payment factors that are used during construction.

50. Are pay factors used to adjust the unit bid prices for hot mix? ❑ yes ❑ no

If yes, which items have a pay adjustment factor?

❑ asphalt content
❑ recovered asphalt properties
❑ air voids
❑ voids in mineral aggregate
❑ voids filled with asphalt
❑ in-place density
❑ segregation
❑ other, ____________________

51. If a payment factor is used for gradation:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

52. If a payment factor is used for asphalt content:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

53. If a payment factor is used for recovered asphalt properties:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ____________

54. If a payment factor is used for air voids:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

55. If a payment factor is used for voids in mineral aggregate:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

56. If a payment factor is used for voids filled with asphalt:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

57. If a payment factor is used for inplace density:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

58. If a payment factor is used for segregation:

can the factor be less than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

can the factor be greater than 1.00? ❑ yes ❑ no

If yes, for what values? ______________

59. How is asphalt mixture paid for?

Inclusive price for mix including aggregate and asphalt? ❑ yes ❑ no
Separate price for aggregate and asphalt? ❑ yes ❑ no

60. Have significant changes been made in the recent past to payment factors in your agency?

❑ yes ❑ no

If yes, what changes to improve rutting resistance have been made, when were they made and how effective do you think they are?

We hope to develop a national view of what we are currently doing in materials, mix design and placement of asphalt mixtures to achieve durable rut resistant pavements. Your information will help define the practice. If you have any questions, you may contact Gerry Huber at:

Telephone (317) 390 3141 during the day or preferably by

E-mail gahuber@aol.com

THANK YOU FOR YOUR TIME AND EFFORT

APPENDIX B

Summary of Permanent Deformation Causes and Solutions

MATERIALS SPECIFICATIONS	DESIGN	CONSTRUCTION
Coarse Aggregate:		
specify enough fracture	use representative material for design	ensure fracture meets design
limit flat and elongated	avoid smooth texture fracture	ensure flat and elongated particles do not exceed design
Fine Aggregate:		
specify limits on natural sand	use representative sample for design	ensure natural sand percent is maintained
specify minimum fine aggregate angularity	avoid gradations humped in the sand size	ensure fine aggregate cleanliness is consistent
specify cleanliness (sand equivalent)		
Asphalt Binder		
specify binder grade high enough for high temperature environment	use correct mixing and compaction temperature	ensure correct asphalt binder is used
ensure PG grade covers traffic level as well as environment	use short term oven aging	avoid contamination of different asphalt binders
Compactive Effort		
match compactive effort to traffic	use correct compaction level	use same compaction level as design
use 20 year design traffic to select compaction level		
Air Voids		
set design criteria at a minimum of 3.5%	use a minimum design level of 3.5%	ensure a minimum of 3.0% air voids is met in laboratory compacted specimens
set design criteria at a maximum of 5.0%	use a maximum design level of 5.0%	ensure a maximum of 5.0% air voids is obtained in laboratory compacted specimens
Voids in Mineral Aggregate		
specify VMA based on aggregate bulk specific gravity	meet specification requirements	meet specification requirements
use Asphalt Institute recommended minimum based on nominal maximum aggregate size	allow for VMA collapse during production	change gradation to maintain VMA
limit maximum VMA to 2% above the minimum		
if effective aggregate specific gravity used, criteria should be adjusted up depending on aggregate absorption		
minimum VMA should be adjusted if other than 4% air voids is used for design		
Filler		
specify non-plastic filler	include baghouse fines in the design	ensure fines to asphalt ratio meets specification
specify filler to effective asphalt content ratio up to 1.6		ensure filler is uniformly fed
specify minimum filler to effective asphalt content ratio of 0.6		ensure baghouse fines are uniformly fed into the mixture
Moisture Damage		
specify a minimum tensile strength ratio	ensure a minimum tensile strength ratio is met	check tensile strength ratio is obtained
In-Place Density		
specify minimum density on % of maximum theoretical density (Gmm)	design layer thickness to be 4 times nominal maximum size	ensure specifications are uniformly met
specify minimum of 92% Gmm for fine graded mixtures		
specify minimum of 94% Gmm for coarse graded mixtures		

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