A Manual for Design of Hot-Mix Asphalt with Commentary

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 673

A Manual for Design of Hot Mix Asphalt with Commentary

Advanced Asphalt Technologies, LLC
Sterling, VA

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FOREWORD

By Edward T. Harrigan
Staff Officer
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This report presents a mix design manual for hot mix asphalt (HMA) that incorporates the many advances in materials characterization and mix design technology developed since the conclusion of the Strategic Highway Research Program (SHRP). The report will be of immediate interest to materials engineers in state highway agencies and industry.

At the conclusion of SHRP in 1993, the Superpave system of HMA mix design and analysis was envisioned to include both novel volumetric design procedures and a suite of performance tests supported by performance models for judging the quality of candidate mix designs on the basis of their predicted long-term in-service behavior. Since that time, the Superpave volumetric design method has been thoroughly validated and widely implemented in routine practice throughout the HMA industry as AASHTO R 35, Superpave Volumetric Design for Hot Mix Asphalt (HMA): AASHTO M 323, Superpave Volumetric Mix Design; and their supporting standard specifications and methods of test. Unfortunately, however, the SHRP performance models proved unreliable, the performance test equipment was complex and expensive, and the planned Superpave performance evaluation was not fully or widely implemented.

In the past decade, more robust and reliable HMA performance tests, models, and equipment were delivered through coordinated research projects sponsored by NCHRP and FHWA. Of particular utility are the Mechanistic-Empirical Pavement Design Guide (MEPDG) and software developed and implemented in NCHRP Projects 1-37A and 1-40 and the simple performance tests and equipment developed in NCHRP Projects 9-19 and 9-29. Thus, the opportunity was presented to develop a new mix design method incorporating (1) features that were intended to be part of the Superpave system, including performance testing and performance predictions, and (2) the products of research on HMA materials characterization and mix design. This new mix design method is foreseen as an eventual successor to the Superpave system.

The objective of NCHRP Project 9-33 was to develop a new, improved mix design method for HMA (including dense-graded, open-graded, and gap-graded mixes) in the form of a manual of practice for use by engineers and technicians in the public and private sectors. The project was carried out by Advanced Asphalt Technologies, LLC, Sterling, Virginia. The manual also includes a mix design method for warm mix asphalt (WMA) that codifies the key findings of the recently completed NCHRP Project 9-43.

In the course of the research, the project team critically reviewed the worldwide literature on HMA materials and mix design since the conclusion of SHRP and, when necessary to resolve specific issues, conducted limited laboratory testing and analysis. Several detailed drafts of the manual were critically evaluated by the NCHRP project panel and several FHWA expert technical groups and subsequently revised by the project team. This report presents the final version of the manual; the manual's key features are (1) a single mix design method applicable to dense-graded, open-graded, and gap-graded HMA and WMA; (2) the application of a range of performance tests and criteria to estimate potential permanent deformation, fatigue cracking, and low-temperature cracking behavior of HMA and WMA mix designs; (3) integration of mix and structural design through the use of the MEPDG software to validate HMA performance for specific combinations of pavement structure, climate, and traffic; and (4) the extensive use of examples to illustrate all facets of the mix design method. Detailed technical discussion to support the methodologies and processes included in the manual are presented in the commentary included herein as Appendix A.

Besides the manual and commentary, the project team delivered

- HMA Tools a comprehensive software program written in Microsoft Excel spreadsheet format that is capable of carrying out all computations and analyses required to conduct and document a mix design;
- A 1-day training course to introduce the mix design method to practitioners; and
- A draft specification and practice in AASHTO standard format for volumetric mix design of dense-graded HMA that reflect the procedures presented in the manual.

The project final report included the manual and commentary as Appendixes A and B and four additional appendixes:

- Appendix C: Course Manual
- Appendix D: Draft Specification for Volumetric Mix Design of Dense-Graded HMA
- Appendix E: Draft Practice for Volumetric Mix Design of Dense-Graded HMA
- Appendix F: Tutorial

The project final report and Appendixes C through F are available as *NCHRP Web-Only Document 159*. HMA Tools and the training course materials are available for download at *http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=967*.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

CHAPTER 1

Introduction

This manual includes chapters on all the primary aspects of hot-mix asphalt (HMA) mix design:

- General background on HMA mixtures and pavements
- Asphalt binders
- Aggregates
- Volumetric composition of HMA mixtures
- Performance
- Mix type selection
- Design of dense-graded HMA
- Recycled asphalt pavement
- Gap-graded HMA mixtures
- Open-graded friction courses
- Field production and quality control of HMA

Engineers and technicians already experienced in HMA technology need not read many of these chapters in detail, though they may find them useful as references. The most important chapters for those already experienced in HMA mix design are Chapter 8, Design of Dense-Graded HMA Mixtures, and Chapter 9, Reclaimed Asphalt Pavement. Chapter 8 contains all the essential information needed to develop HMA mix designs. Because of the complexity of incorporating recycled or reclaimed asphalt pavement (RAP) into HMA mix designs, this topic is discussed in a separate chapter in this manual. Using RAP is becoming increasingly important in the HMA industry, and engineers and technicians responsible for HMA mix design should read Chapter 9 of this manual carefully. Laboratory personnel involved in developing designs for other mix types may also find Chapters 10 and 11 useful, which discuss the design of gap-graded HMA mixtures and open-graded friction course mixtures, respectively. Readers who have not yet gained significant experience in developing HMA mix designs may find a careful reading of most or all of the chapters in the manual useful.

The manual may also be incorporated into technician training and certification programs; training materials that instructors will find useful in presenting the information contained in this manual are available on the TRB website by searching for "NCHRP Report 673." These training materials focus on Chapter 8, since the design of dense-graded mixtures is the most important part of the manual and this chapter also discusses many of the most important parts of the other chapters. Furthermore, many technician training programs emphasize the design of dense-graded HMA.

Although this manual was developed for use by practicing engineers and technicians, its format lends itself for use in college courses on construction materials. The chapters included in such a course will depend on the format selected by the instructor. In such a situation it is likely that information contained in some chapters would either be excluded from the course or covered using other text books. An introductory course on construction materials might include the following chapters from the manual:

- Chapter 3, Asphalt Binders
- Chapter 5, Mixture Volumetric Composition
- Chapter 8, Design of Dense-Graded HMA Mixtures

An upper level course on construction materials or HMA technology might include all of the chapters except for Chapter 4 (Aggregates), since this is likely covered by a more general text on construction materials. Such an upper level course might either omit or simply quickly review material covered in the prerequisite courses.

This manual is supported by a comprehensive spreadsheet (HMA Tools) for performing dense-graded HMA mix designs. This spreadsheet is among the materials available on the TRB website and can be found by search under "NCHRP Report 673." HMA Tools includes all calculation tools needed for performing a dense-graded mix design, including those required for mix designs containing RAP. Such calculations include determining (1) the effective binder grade of the mixture or (2) the new binder grade that should be added to a RAP-containing mixture in order to meet the requirements of the specified binder grade. The spreadsheet includes utilities for blending aggregates, performing volumetric calculations, determining weights for laboratory batches, and estimating specification properties for aggregate mixes from test data on individual aggregate stockpiles. In many cases, this manual refers directly to the HMA Tools spreadsheet and describes how HMA Tools is used to perform various steps in the mix design process. This is especially true in Chapter 9, given that the mathematics involved in proper design of HMA containing RAP can be exceedingly complex. Engineers and technicians already satisfied with their current spreadsheet or computer program can certainly continue using it to perform mix designs following the procedure given in this manual, although some of the specification values may need to be modified.

This manual also includes a Commentary, which is designed to provide technical background information supporting specific parts of this manual. Such commentary is needed because this manual itself was intended for use by technicians and engineers, including those with little knowledge or experience with HMA mixtures. Detailed technical references and equations would make this manual difficult to read and use as a practical reference by laboratory personnel. On the other hand, excluding such information would make it difficult for engineers to review the mix design procedure and specifications to determine if such procedures and specifications might need to be adjusted to account for unusual local conditions or special projects. Information in the Commentary will also make it much easier to evaluate and revise this manual when such revisions become necessary. Engineers and technicians primarily concerned with developing or evaluating HMA mix designs need not read the Commentary.

The information presented herein, for the most part, is meant to follow as accurately as possible current practice as well as recommendations made by researchers and reviewed and approved by industry panels. This manual reflects research and engineering work performed by a wide range of individuals and organizations and is not simply a statement of the authors' opinions and judgments concerning HMA mix design. When possible, current AASHTO standards have been followed and included as references at the end of each chapter. In some cases, such as the chapters on RAP and HMA performance, AASHTO standards have not yet caught up with practice in the industry, and the information reflects research reports and, in some cases, state agency standards, rather than AASHTO standards. Differences in the procedures given in this manual compared to previous practice are significant but should not be too difficult to implement:

- Although this mix design procedure closely follows many of the procedures and standards used in the Superpave system of mix design and analysis, it is not meant to be the "next generation" of Superpave. For this reason, the term Superpave is not used to describe the technology contained herein.

- There are some minor differences in terminology compared to previous usage. In the Superpave system, aggregate specification properties are called "consensus" properties, since they represented the consensus of a panel of experts. These properties have now been studied and reviewed by a wide range of engineers, and no longer simply represent such a consensus. Therefore, they are simply referred to as aggregate specification properties. There are also a few small changes in these aggregate specification properties, making it easier to meet some requirements at the highest design traffic levels.
- In this manual, the control points that define aggregate blend gradations are meant to be guidelines, rather than strict specifications. This distinction provides engineers and technicians with additional flexibility in adjusting aggregate gradations to meet volumetric requirements when developing mix designs. Furthermore, there is little research suggesting that aggregate gradation *in and of itself* affects HMA performance.
- As discussed above, this manual contains a comprehensive treatment of RAP, including the relatively new issue of variability and how it may limit the amount of RAP that can be incorporated into a mix design.
- This manual includes a chapter on HMA performance, which describes specific tests to be used in evaluating the rut resistance of HMA mixtures. Guidelines are given for interpreting these tests, but they are just that—guidelines. These guidelines are likely to be modified through research, experience, and judgment by individual agencies to suit local materials and conditions.
- Perhaps the most significant change is in the overall philosophy of mixture design. Traditionally, when developing an HMA mix design, several trial mixtures with different aggregate gradations were evaluated, and then one was picked for further refinement by evaluating a range of asphalt binder contents. The final mix design was selected as the binder content that provided the proper volumetric composition and best met other pertinent requirements. This manual proposes a somewhat different approach in which the proper asphalt binder content is determined early in the mix design process and maintained throughout the various trial mixtures. Volumetric properties are adjusted in these trial mixes not by adjusting binder content, but by adjusting aggregate gradation. This approach is suggested primarily because it ensures that the final mix design will contain the proper amount of asphalt binder. It is also a simpler approach, requiring fewer trial mixtures to finalize a mix design. However, this approach to mix design is only a suggestion—engineers and technicians are free to use whatever procedures they deem appropriate to develop HMA mix designs, as long as the final results meet the requirements given in this manual.

This manual is designed to be a complete, up-to-date reference that will complement other manuals available to engineers and technicians for mix design. Such other resources include the Asphalt Institute's *Superpave Mix Design* (SP-2) and the National Asphalt Pavement Association's (NAPA's) *Hot Mix Asphalt Materials, Mixture Design and Construction.*

Bibliography

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The Asphalt Institute (1997) *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types* (MS-2), 6th Ed., 141 pp.

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CHAPTER 2

Background

To thoroughly understand the mix design procedures and test methods presented herein, a basic knowledge of construction materials and paving technology is needed. This information is summarized below. Engineers and technicians with a broad range of experience in materials testing and the design of HMA mixtures and flexible pavement structures need not read this chapter in detail. Individuals who are relatively new to asphalt pavement technology will find the information on materials, asphalt pavements, asphalt concrete mixtures, and mix design methods helpful when reading the later chapters of this manual.

Materials Used in Making Asphalt Concrete

Asphalt concrete is composed primarily of aggregate and asphalt binder. Aggregate typically makes up about 95% of an HMA mixture by weight, whereas asphalt binder makes up the remaining 5%. By volume, a typical HMA mixture is about 85% aggregate, 10% asphalt binder, and 5% air voids. Small amounts of additives and admixtures are added to many HMA mixtures to enhance their performance or workability. These additives include fibers, crumb rubber, and anti-strip additives. Figure 2-1 shows a typical HMA laboratory specimen and the materials used to produce it.

Asphalt binder holds the aggregate in HMA together—without asphalt binder, HMA would simply be crushed stone or gravel. Asphalt binder is the thick, heavy residue remaining after kerosene, gasoline, diesel oil, and other fuels and lubricants are refined from crude oil. Asphalt binder consists mostly of carbon and hydrogen, with small amounts of oxygen, sulfur, and several metals. The physical properties of asphalt binder vary tremendously with temperature. At high temperatures, asphalt binder is a fluid with a consistency similar to that of motor oil. At room temperature most asphalt binders will have the consistency of putty or soft rubber. At subzero temperatures, asphalt binder can become very brittle—asphalt samples stored in a freezer will shatter like glass if dropped on a hard surface. Many asphalt binders contain small percentages of polymer to improve their physical properties; these materials are called polymer-modified binders. Much of the current asphalt binder specification used in the United States was designed to control changes in consistency with temperature. This specification and the associated test methods are discussed in more detail in Chapter 3.

Because HMA mixtures are mostly aggregate, aggregates used in HMA must be of good quality to ensure the resulting pavement will perform as expected. Aggregates used in HMA mixtures may be either crushed stone or crushed gravel. In either case, the material must be thoroughly crushed, and the resulting particles should be cubical rather than flat or elongated. Aggregates should be free of dust, dirt, clay, and other deleterious materials. Because aggregate particles carry most of the load in HMA pavements, aggregates should be tough and abrasion resistant. Properties of aggregates and the tests that technicians use to evaluate them are discussed in detail in Chapter 4 of this manual.

Figure 2-1. A compacted HMA laboratory specimen and the aggregate and asphalt used to prepare it.

All HMA mixtures contain small amounts of air voids. In the laboratory, HMA mixtures are usually designed to contain about 4% air voids, with a range of about 3 to 5%, depending on the type of mixture being designed and the design procedure being used. Properly constructed HMA pavements will usually contain about 6 to 8% air voids immediately after placement and compaction. After construction, as traffic passes over a pavement, the HMA in the wheel paths will normally gradually compact to air void levels approaching the design value of 3 to 5%. However, if the pavement is not compacted adequately during construction, compaction under traffic will fail to reduce the air void content to the design value and, as a result, the pavement will be permeable to air and water, potentially leading to moisture damage and excessive age hardening.

Asphalt Concrete Pavements

Asphalt concrete pavements are not simply a thin covering of asphalt concrete over soil—they are engineered structures composed of several different layers. Because asphalt concrete is much more flexible than portland cement concrete, asphalt concrete pavements are sometimes called flexible pavements. The visible part of an asphalt concrete pavement, the part that directly supports truck and passenger vehicles, is called the surface course or wearing course. It is typically between about 40 and 75 mm thick and consists of crushed aggregate and asphalt binder. Surface course mixtures tend to have a relatively high asphalt content, which helps these mixtures stand up better to traffic and the effects of sunlight, air, and water. Surface course mixtures also are usually made using maximum aggregate sizes less than 19 mm, which helps to provide for a quiet ride. Also, using aggregate sizes larger than 19 mm can make it more difficult to obtain mixtures with sufficient asphalt binder contents to provide adequate durability for surface course mixtures, since the lower aggregate surface area of these aggregates results in a lower demand for asphalt binder. On the other hand, the lower binder content needed for these mixtures can make them more economical than mixtures made using smaller aggregates.

Below the surface course of a flexible pavement is the base course. The base course helps provide the overall thickness to the pavement needed to ensure that the pavement can withstand the projected traffic over the life of the project. Base courses may be anywhere from about 100 to 300-mm thick. In general, the higher the anticipated traffic level on a pavement, the thicker the pavement must be, and the thicker the base course. Thicker pavements will deflect less than thinner ones under traffic loading, which reduces strains within the pavement and makes them more resistant to fatigue cracking. Traditionally, base course mixtures have been designed using larger aggregate sizes than surface course mixtures, with maximum aggregate sizes ranging from about 19 to 37.5 mm. This helps to produce a lean mixture with low asphalt binder content, which helps keep the cost of these mixtures low. Also, using larger aggregate sizes allows base course mixtures to be placed in thicker lifts, which can reduce construction costs. However, many engineers have recently been designing base course mixtures more like surface course mixtures—with smaller aggregate sizes and higher asphalt binder contents. Using these types of mixtures in base course mixtures can help improve both fatigue resistance and resistance to moisture damage, since increased asphalt binder contents in HMA tends to improve fatigue resistance and will also reduce permeability to water.

Sometimes an intermediate course is placed between the surface and base courses of a flexible pavement system. This is sometimes called a binder course. Typically 50 to 100 mm in thickness, it consists of a mixture with intermediate aggregate size and asphalt binder content.

The surface, base, and intermediate courses together are referred to as bound material or bound layers, because they are held together with asphalt binder. In a typical asphalt concrete pavement, the bound layers are supported by a granular subbase that in turn lays over the subgrade. Granular subbase is crushed stone or gravel, usually 100 to 300 mm in thickness. The nominal maximum aggregate size varies, but it should always be well compacted prior to placement of the base course. The subgrade is the soil on which the pavement is constructed. If the soil is stable and strong, it may only need compaction prior to placing the granular subbase and remaining pavement layers. However, some soils, including many soils containing clay and clay-like minerals are unstable—that is, they shrink and swell significantly when their moisture content changes, and they can also become very weak when the moisture content becomes too high. Before pavement construction, such a subgrade should be stabilized by blending in lime, portland cement, or other additives, or treating it with asphalt emulsion, and then thoroughly compacting the soil.

Sometimes the granular subbase is omitted from a pavement and a relatively thick base course is placed directly on the subgrade soil. Such a pavement structure is called a full-depth asphalt pavement. The advantage of this type of construction is that the overall pavement can be thinner because of the increased strength and stiffness of the supporting pavement. However, it should be remembered that such a base course mixture will be significantly more expensive than granular subbase, since it contains asphalt binder. Figure 2-2 is a cross section of a typical flexible pavement system.

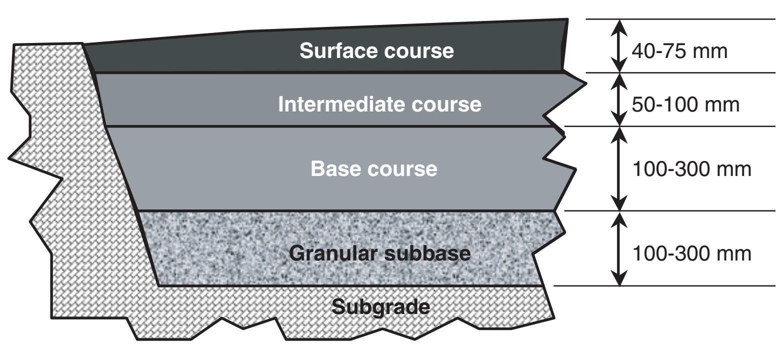

Figure 2-2. Typical asphalt concrete pavement structure.
In many cases, the intermediate course is omitted; full-depth asphalt pavements do not include a granular subbase.

How Asphalt Concrete Pavements Fail

Rutting

Rutting (often referred to as permanent deformation) is a common form of distress in flexible pavements. When truck tires move across an asphalt concrete pavement, the pavement deflects a very small amount. These deflections range from much less than a tenth of a millimeter in cold weather—when the pavement and subgrade are very stiff—to a millimeter or more in warm weather—when the pavement surface is hot and very soft. After the truck tire passes over a given spot in the pavement, the pavement tends to spring back to its original position. Often, however, the pavement surface will not completely recover. Instead, there will be a very small amount of permanent deformation in the wheel path. After many wheel loads have passed over the pavement—perhaps only a few thousand in a poorly constructed pavement, to 10 million or more for one properly designed and constructed for heavy traffic loads—this rutting can become significant. Severely rutted pavements can have ruts 20 mm or more in depth. Rutting is a serious problem because the ruts contribute to a rough riding surface and can fill with water during rain or snow events, which can then cause vehicles traveling on the road to hydroplane and lose control. Rut depths of about 10 mm or more are usually considered excessive and a significant safety hazard. Figure 2-3 is diagram of rutting in an HMA pavement.

Other related forms of permanent deformation include shoving and wash boarding. Shoving occurs at intersections when vehicles stop, exerting a lateral force on the surface of the hot mix causing it to deform excessively across the pavement, rather than within the wheel ruts. Wash boarding is a similar phenomenon but, in this case, the deformation takes the form of a series of large ripples across the pavement surface.

Rutting, shoving, and wash boarding can be the result of permanent deformation in any part of the pavement—the subgrade, the granular subbase, or any of the bound layers. Excessive permanent deformation in one or more of the bound layers is the result of an asphalt concrete mixture that lacks strength and stiffness at high temperatures. Several problems with a mix design, such as selecting an asphalt binder that is too soft for the given climate and traffic level, can make it prone to rutting and other forms of permanent deformation. Relationships between mixture composition and pavement performance are discussed in detail in Chapter 7.

Fatigue Cracking

Like rutting, fatigue cracking results from the large number of loads applied over time to a pavement subject by traffic. However, fatigue cracking tends to occur when the pavement is at moderate temperatures, rather than at the high temperatures that cause rutting. Because the HMA at moderate temperatures is stiffer and more brittle than at high temperatures, it tends to crack under repeated loading rather than deform. When cracks first form in an HMA pavement,

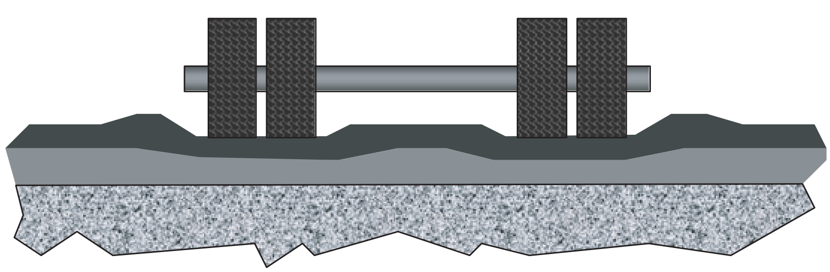

Figure 2-3. Sketch of rutting in a flexible pavement.

they are so small that they cannot be seen without a microscope. The cracks at this point will also not be continuous. Under the action of traffic loading, these microscopic cracks will slowly grow in size and number, until they grow together into much larger cracks that can be clearly seen with the naked eye. Severe fatigue cracking is often referred to as "alligator cracking," because the pavement surface texture resembles an alligator's back. These large cracks will significantly affect pavement performance, by weakening the pavement, contributing to a rough riding surface, and allowing air and water into the pavement, which will cause additional damage to the pavement structure. Eventually fatigue cracking can lead to extensive areas of cracking, large potholes, and total pavement failure.

Traditionally, pavement engineers believed that fatigue cracks first formed on the underside of the HMA layers, and gradually grew toward the pavement surface. It has become clear during the past 10 years that pavements are also subject to top-down fatigue cracking, where the cracks begin at or near the pavement surface and grow downward, typically along the edges of the wheel paths. It is likely that most HMA pavements undergo both bottom-up and top-down fatigue cracking. However, as HMA pavements have become thicker and as HMA overlays on top of portland cement concrete pavements have become more common, top-down cracking has become more commonly observed than bottom-up cracking. Figure 2-4 illustrates both bottom-up and top-down fatigue cracking.

Although fatigue cracking in HMA pavements is still not completely understood, most pavement engineers agree that there are several ways mixture composition can affect fatigue resistance in HMA pavements. One of the most important factors affecting fatigue resistance is asphalt binder content—HMA mixtures with very low asphalt contents tend to be less fatigue resistant than richer mixtures. Poor field compaction also contributes significantly to surface cracking by reducing the strength of the pavement surface. High in-place air void content will also increase pavement permeability, which will then allow air and water into the pavement, both of which can damage the pavement and increase the rate of fatigue cracking. Relationships between fatigue cracking and HMA mix design are discussed in more detail in Chapter 6.

Low-Temperature Cracking

Temperature has an extreme effect on asphalt binders. At temperatures of about 150°C (300°F) asphalt binders are fluids that can be easily pumped through pipes and mixed with hot aggregate. At temperatures of about 25°C (77°F), asphalt binders have the consistency of a stiff putty or soft rubber. At temperatures of about –20°C and lower, asphalt binders can become very brittle.

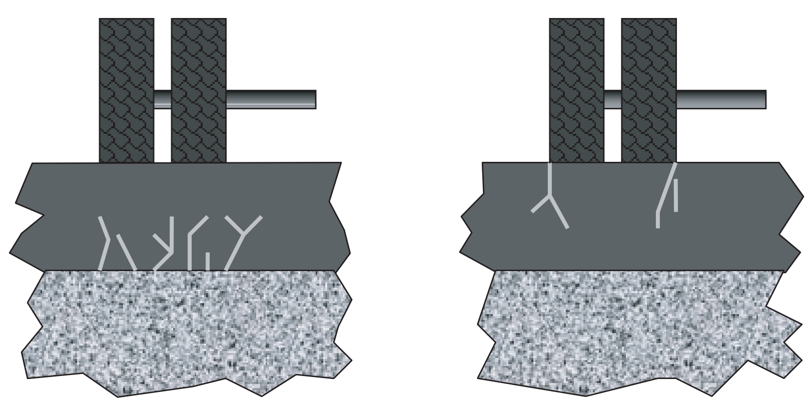

Figure 2-4. Bottom-up (left) and top-down (right) fatigue cracking.

As a result, HMA pavements in many regions of the United States and most of Canada will become very stiff and brittle during the winter. When cold fronts move through an area causing rapid drops in temperature, HMA pavements can quickly cool. Like most materials, HMA tends to contract as it cools. Unlike portland cement concrete pavements, flexible pavements have no contraction joints and the entire pavement surface will develop tensile stresses during rapid drops in temperature in cold weather. When the pavement temperature drops quickly enough to a low enough temperature, the resulting tensile stresses can cause cracks in the embrittled pavement. These low-temperature cracks will stretch transversely across part or all of the pavement, their spacing ranging from about 3 to 10 meters (10 to 40 feet). Although low-temperature cracking may not at first cause a significant problem in a pavement, the cracks tend to become more numerous and wider with time and cause a significant performance problem after several years.

Low-temperature cracking in HMA pavements can be minimized or even eliminated by proper selection of asphalt binder grade. In fact, one of the main reasons for the development of the current system for grading asphalt binders was to help prevent low-temperature cracking. This grading system, and how it is used to select binders that are both resistant to rutting and low-temperature cracking, is discussed in Chapter 3.

Moisture Damage

Water does not flow easily through properly constructed HMA pavements, but it will flow very slowly even through well-compacted material. Water can work its way between the aggregate surfaces and asphalt binder in a mixture, weakening or even totally destroying the bond between these two materials. This moisture damage is sometimes called stripping. Moisture damage can occur quickly when water is present underneath a pavement, as when pavements are built over poorly drained areas and are not properly designed or constructed to remove water from the pavement structure. Even occasional exposure to water can cause moisture damage in HMA mixtures prone to it because of faulty design or construction or poor materials selection. The physicochemical processes that control moisture damage are complex and only now are beginning to be understood. Different combinations of asphalt binder and aggregate will exhibit widely varying degrees of resistance to moisture damage. It is very difficult to predict the moisture resistance of a particular combination of asphalt and aggregate, although HMA produced with aggregates containing a large amount of silica, such as sandstone, quartzite, chert, and some granites, tend to be more susceptible to moisture damage. Proper construction, especially thorough compaction, can help reduce the permeability of HMA pavements and so significantly reduce the likelihood of moisture damage. Anti-stripping additives can be added to HMA mixtures to improve their moisture resistance. Hydrated lime is one of the most common and most effective of such additives.

The moisture resistance of HMA mixtures is often evaluated using AASHTO T 283 (often referred to as the Lottman procedure). In this test, six cylindrical HMA specimens are compacted in the laboratory. Three of these are subjected to conditioning—vacuum saturation, freezing, and thawing—while the other three are not conditioned. Both sets of specimens are then tested using the indirect tension (IDT) test (see Figure 2-5). The percentage of strength retained after conditioning is called the tensile strength ratio (TSR) and is an indication of the moisture resistance of that particular mixture. Many highway agencies require a minimum TSR of 70 to 80% for HMA mixtures. Engineers and technicians should keep in mind that this test is not 100% accurate and only provides a rough indication of moisture resistance. Research is underway on improved procedures for evaluating the moisture resistance of HMA mixtures. The use of moisture resistance testing in HMA mix design is discussed in greater detail in Chapter 8.

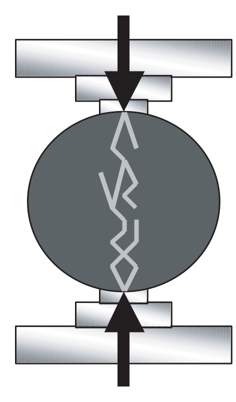

Figure 2-5. Indirect tension test, as used in moisture resistance testing of HMA mixtures.

Raveling

Raveling occurs when tires dislodge aggregate particles from the surface of an HMA pavement. Many of the same factors that contribute to poor fatigue resistance will also contribute to raveling, including low asphalt binder contents and poor field compaction. Because the pavement surface is exposed to water from rain and snow, poor moisture resistance can also accelerate raveling in HMA pavements.

Asphalt Concrete Mixtures

Asphalt concrete mixtures can be classified in many different ways. Perhaps the most general type of classification is by whether or not the mix must be heated prior to transport, placement, and compaction. HMA concrete, or simply HMA, must be thoroughly heated during mixing, transport, placement, and compaction. The asphalt binder used in HMA is quite stiff at room temperatures, so that once this type of asphalt concrete cools it becomes stiff and strong enough to support heavy traffic. Cold mix asphalt, on the other hand, is normally handled, placed, and compacted without heating. This material can be handled cold because it uses liquid asphalts in the form of emulsions and cutbacks that are fluid at room temperature. Asphalt emulsions are mixtures of asphalt, water, and special chemical additives called surfactants that allow the other two materials to be blended into a stable liquid. When blended with aggregate, the emulsion "breaks," meaning the asphalt separates from the water and thoroughly coats the aggregate. Cutback asphalts are blends of asphalt binder and petroleum solvents. Once placed, cold mix made with cutback asphalts gradually cure as the solvent evaporates from the asphalt concrete. Many engineers now avoid the use of cutback asphalts because of environmental concerns. Cold mix is economical because it does not require large amounts of energy to heat the mix during production and placement. However, it is difficult to compact thoroughly and in general is not as durable as HMA. Cold mix is sometimes used for base course construction and is also commonly used for patching and repairing pavement.

A new, third type of mix—called warm-mix asphalt (WMA)—has recently become increasingly popular. In this type of mixture, various different methods are used to significantly reduce mix production temperature by 30 to over 100°F. These methods include (1) using chemical additives to lower the high-temperature viscosity of the asphalt binder; (2) techniques involving the addition of water to the binder, causing it to foam; and (3) two-stage processes involving the addition of hard and soft binders at different points during mix production. WMA has several benefits, including lower cost (since significantly less fuel is needed to heat the mix), lower emissions and so improved environmental impact, and potentially improved performance because of decreased age hardening. There is some concern that WMA might in some cases be more susceptible to moisture damage, but this has yet to be clearly demonstrated.

This manual deals exclusively with HMA of which there are three different major types—dense-graded mixtures, gap-graded mixtures, and open-graded mixtures. Dense-graded mixtures are the most common HMA mix type. The term dense-graded refers to the dense aggregate gradation used in these types of mixtures, which means that there is relatively little space between the aggregate particles in such mixtures. Historically, dense-graded mixtures were popular because they required relatively low asphalt binder contents, which kept their cost down. However, experience has shown that HMA with binder contents that are too low can be difficult to place and compact and may be prone to surface cracking and other durability problems. Therefore, many "dense-graded" HMA mixtures do not use a true maximum density gradation, but use somewhat "open" gradations that deviate slightly from maximum density; such mixtures have more space between the aggregate particles and can be designed to contain more asphalt binder. Mixtures that are somewhat coarser than the maximum density gradation are called coarse-graded mixtures, while mixtures somewhat finer than the maximum density gradation are called fine-graded mixtures. This terminology can be somewhat confusing, since both coarse- and fine-graded mixtures should be considered variations of dense-graded HMA. A more appropriate terminology is to refer to the three types of dense-graded HMA as dense/dense-graded, dense/coarse-graded, and dense/fine-graded mixtures. When engineers and technicians first began developing mix designs using the Superpave system in the 1990s, there was a clear trend toward dense/coarse-graded mixtures, in order to increase the rut resistance of HMA pavements. However, in the past few years, many agencies have shifted back toward finer mixtures (dense/dense or dense/fine), to help improve the durability of surface course mixtures. Also, recent research has suggested that dense/fine HMA mixtures can, in most cases, be designed to have just as much rut resistance as dense/coarse mixtures. The procedure for designing dense-graded HMA mixtures given in this manual (in Chapter 8) suggests that a range of gradations be evaluated during the mix design process and that the gradation most effective in meeting the given mixture specifications should be selected. The suggested volumetric requirements do include a slight increase in the allowable range for dust-to-binder ratio and an optional table for high-durability mixtures that includes an even higher dust-to-binder ratio and an increase in minimum VMA; both of these changes will probably reduce the number of dense/coarse-graded HMA mixtures being designed under this system.

During the past 20 years, stone-matrix asphalt (SMA) has become increasingly common in the United States and Europe. SMA is a special type of HMA designed specifically to hold up under very heavy traffic. SMA is composed of high-quality coarse aggregate, combined with a large amount of mastic composed of a high-performance asphalt binder, mineral filler, and a small amount of fibers. The aggregate used in SMA contains a large amount of coarse aggregate and a large amount of very fine material (called mineral filler), but not much sand-sized material. For this reason, such aggregates are called gap-graded, and SMA and similar HMA types are referred to as gap-graded mixtures, or gap-graded HMA (GGHMA)—the term used in this manual. A well-developed coarse aggregate structure in combination with a relatively large volume of high performance binder helps ensure that a properly designed SMA mixture will exhibit excellent performance. SMA is usually only used on very heavily trafficked roadways, where its excellent performance makes it cost-effective despite the high initial investment required to construct SMA pavements. The design of GGHMA mixtures is discussed in detail in Chapter 10 of this manual. Figure 2-6 shows an SMA surface course on a dense-graded HMA base.

Figure 2-6. SMA surface course on dense-graded HMA base.

Another type of HMA mixture is open-graded friction course (OGFC). OGFC mixtures contain very large amounts of coarse aggregate, with very little fine aggregate or mineral filler. The air void content is much higher than in conventional HMA, typically 15 to 20%. A large amount of high-performance binder is needed to provide adequate stability in these mixtures. OGFC mixtures are usually used as thin overlays, where they can help control noise and limit spray on wet pavements. OGFC mixtures are discussed in detail in Chapter 11 of this manual.

HMA Mix Design Methods

When HMA pavements first began to be constructed, mixture composition was determined based on the judgment and experience of the contractor or used proprietary mix designs. Some of these pavements performed well, others did not. In the 1930s, Bruce Marshall, an engineer working for the Mississippi Highway Department, developed a more rational system for designing HMA mixtures, which became known as the Marshall mix design method. This method of mix design became common by the 1950s and continued to be widely used through the 1980s. It was adopted for use by the U.S. Army Corps of Engineers (USACE) during World War II and was modified by that agency and by the many state highway departments that eventually chose the Marshall method of HMA mix design. Briefly, the Marshall mix design procedure relies on compacting specimens using a standard drop hammer over a range of asphalt binder contents. The binder content is selected to produce proper air void content and voids in the mineral aggregate (VMA). An essential part of the Marshall design method is the stability and flow test, which is an empirical procedure used to evaluate the strength and flexibility of the HMA mixture. Figure 2-7 shows a mechanical Marshall drop hammer and several Marshall specimens prepared in the laboratory.

Francis Hveem, an engineer working for the California Division of Highways at about the same time Bruce Marshall was developing his procedure, developed an HMA mix design method adopted

Figure 2-7. Marshall compactor and laboratory specimens for use in the Marshall mix design method.

by many agencies in the western United States. The Hveem method of mix design is unique in its use of the centrifuge kerosene equivalent (CKE) test to determine an initial estimate of the asphalt binder content for a given aggregate. Laboratory specimens are prepared using a kneading compactor and then evaluated using a stabilometer test and a swell test. At one time, a cohesiometer test was also used to evaluate HMA properties in the Hveem method. As with the Marshall mix design method, evaluation of mixture volumetrics was an important part of the Hveem procedure.

In the late 1980s and early 1990s, the Strategic Highway Research Program (SHRP) was conducted by engineers and researchers at various universities and research organizations throughout the United States. The SHRP Asphalt Research Program was a 5-year, $50 million-dollar research program to develop improved test methods and specifications for asphalt binder and aggregates and an improved procedure for HMA mixture design and analysis. The performance grading system now used to specify binders in the United States and other countries was one of the products of SHRP. Another SHRP product was the Superpave system of mix design and analysis, often simply referred to as Superpave. This method is similar to the Marshall system in its use of volumetrics, but laboratory specimens are prepared using a gyratory compactor rather than a drop hammer. Superpave also includes a comprehensive set of requirements for aggregate gradations and property requirements. The Superpave method of mix design was meant to include mixture test methods and an associated computer program that would predict the performance of HMA pavements, as an aid in the design and analysis of mixtures. However, this computer program never produced reliably accurate predictions of rutting and fatigue cracking, and this aspect of Superpave was never implemented, although the Mechanistic-Empirical Pavement Design Guide (MEPDG) is in many ways a continuation of the SHRP efforts for flexible pavements. Figure 2-8 shows the Superpave gyratory compactor (SGC).

In many ways, the Superpave system was a success. The performance grading system appears to do a better job of ensuring that asphalt binders have adequate stiffness at high temperature

Figure 2-8. Superpave gyratory compactor *(Courtesy of Pine Equipment Company).*

while remaining flexible at low temperatures for a wide range of applications, helping to provide resistance to both rutting and low-temperature cracking in HMA pavements. Very few pavements using mixtures designed using the Superpave system have exhibited excessive rutting. However, recently some highway agencies have expressed concern over surface cracking and high permeability in pavements using HMA designed using the Superpave system. Many highway agencies have modified the Superpave system to address these problems and others associated with local materials and conditions. A major research effort, continued since the implementation of SHRP, is under way to refine various aspects of this system, including volumetric requirements, compaction levels, and specifications for aggregate properties and gradation and finally implement practical mixture performance tests and methods of analysis. This mix design manual is largely based on the Superpave system of mixture design and analysis, but an attempt has been made to address the perceived performance problems associated with some mixtures designed using this system. Also, the results and recommendations of some recent research projects have been adopted where such results have been published and presented to the paving technology community and favorably received. Because local conditions and materials vary significantly throughout the United States and Canada, an attempt has been made to provide more flexibility in the procedure used to design dense-graded HMA mixtures, compared to the Superpave system. This manual is also much more comprehensive than the procedures given in the original Superpave system; it includes design procedures not just for dense-graded HMA, but also for gap-graded or SMA-like mixtures and OGFC types. Because of these changes, the term "Superpave" is not used to describe the procedure, tests, and requirements used in the mix design methods presented in this manual.

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CHAPTER 3

Asphalt Binders

Asphalt binders, sometimes referred to as asphalt cement binders or simply asphalt cement, are an essential component of asphalt concrete—they are the cement that holds the aggregate together. Asphalt binders are a co-product of refining crude petroleum to produce gasoline, diesel fuel, lubricating oils, and many other petroleum products. Asphalt binder is produced from the thick, heavy residue that remains after fuels and lubricants are removed from crude oil. This heavy residue can be further processed in various ways, such as steam reduction and oxidation, until it meets the desired set of specifications for asphalt binders. For demanding, high-performance applications, small amounts of polymers are sometimes blended into the asphalt binder, producing a polymer-modified binder.

Asphalt binders have been mixed with crushed aggregate to form paving materials for over 100 years. They are a very useful and valuable material for constructing flexible pavement worldwide. However, asphalt binders have very unusual engineering properties that must be carefully controlled in order to ensure good performance. One of the most important characteristics of asphalt binders that must be addressed in test methods and specifications is that their precise properties almost always depend on their temperature. Asphalt binders tend to be very stiff and brittle at low temperatures, thick fluids at high temperatures, and leathery/rubbery semi-solids at intermediate temperatures. Such extreme changes in properties can cause performance problems in pavements. At high temperatures, a pavement with a binder that is too soft will be prone to rutting and shoving. On the other hand, a pavement that contains a binder that is too stiff at low temperatures will be prone to low-temperature cracking. Figure 3-1 illustrates the extreme change in modulus that occurs in asphalt binders over the range of temperatures likely to occur in pavements; at −30°C the modulus of this particular asphalt binder was about *37,000 times* greater than its modulus at 50°C. Specifications for asphalt binders must control properties at high, low, and intermediate temperatures. Furthermore, test methods used to specify asphalt binders usually must be conducted with very careful temperature control; otherwise, the results will not be reliable. Asphalt binders are also very sensitive to the time or rate of loading. When tested at a fast loading rate, an asphalt binder will be much stiffer than when tested at a slow loading rate. Therefore, time or rate of loading must also be specified and carefully controlled when testing asphalt binders.

Another characteristic of asphalt binders that complicates specification and testing of these materials is that, for various reasons, such binders tend to harden with time. For example, when asphalt binders are heated to high temperatures, as happens when mixing and transporting HMA, some of the lighter volatile oil fractions of the asphalt vaporize, which can harden the remaining asphalt binder. At the same time, some of the chemical compounds making up asphalt binders can oxidize, which can also result in an increase in stiffness. Some oxidation occurs during mixing, transport, and placement of the HMA. However, slow, long-term oxidation will continue to occur in the asphalt binder in a pavement for many years, resulting in a slow but sometimes very

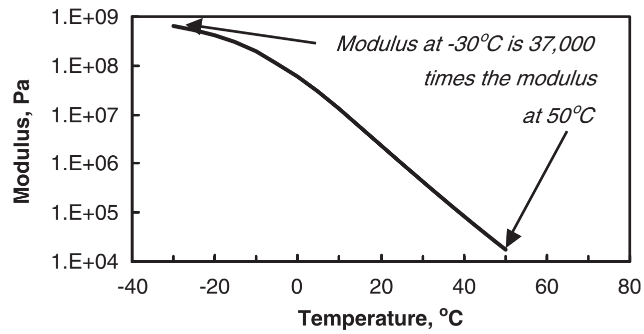

Figure 3-1. Change in dynamic shear modulus with temperature for typical asphalt binder (frequency = 10 rad/s).

significant increase in stiffness. Sometimes asphalt binder age hardening can be so severe that it can lead to serious premature surface cracking of the pavement surface. Several other types of hardening occur in asphalt binders without any loss of volatiles or oxidation; these include steric hardening and physical hardening. These phenomena are not yet well understood, but appear to be caused by a slow rearrangement of the molecules in the asphalt binder over time, resulting in a gradual increase in stiffness. Unlike other types of hardening, steric hardening and physical hardening are reversible—if the asphalt is heated until fluid and then cooled, all or most of the hardening will be removed. This is one of the reasons it is important to thoroughly heat and stir asphalt samples prior to performing any laboratory tests.

Asphalt binders are complex materials that are difficult to specify and test. Pavement engineers and technicians have struggled for over 100 years to develop simple tests and effective specifications for asphalt binders. One of the earliest tests for asphalt binders was the penetration test, in which a small lightly weighted needle was allowed to penetrate the asphalt for a set period of time (typically 5 or 60 seconds). The distance the needle penetrated into the asphalt was measured and was used as an indication of its stiffness. Other such empirical tests were the ring and ball softening point temperature, and the ductility test. These tests were useful (many are still used in specifications in Europe and other parts of the world), but had shortcomings. They did not measure any fundamental property of the asphalt binder, like modulus or strength. The results were also sometimes highly variable and were not always in close agreement from laboratory to laboratory. In the 1960s, specifications based on viscosity measurements began to be adopted by many highway agencies. Viscosity tests are superior to the earlier empirical tests—they provide information on a fundamental characteristic of the asphalt binder and provide reasonably repeatable results among laboratories. However, there are drawbacks to viscosity testing. First, it is best used at high temperatures, where the behavior of the asphalt binder approaches that of an ideal fluid. At low and intermediate temperatures, viscosity tests become difficult to perform and even more difficult to interpret. Second, viscosity tests only provide a limited amount of information on the flow properties of a material. Two different asphalt binders can have identical viscosity values at a given temperature but might behave very differently because of differences in the degree of elasticity exhibited in their behavior. When loaded, the asphalt binders might deform the same amount, but when the load is removed, one might spring back, or *recover,* to nearly its initial shape. The other might hardly recover at all, staying in its deformed shape. The asphalt binder that showed more recovery—that behaved in a more elastic fashion—would tend to provide better rut resistance in paving applications compared to the other binder with poor recovery. However, viscosity tests provide no information about recovery or about the degree of elasticity exhibited by a material under loading. The shortcomings in both older empirical tests and in the newer viscosity tests eventually led to the development of a more effective system

(a)

(b)

Figure 3-2. Traditional test for asphalt binders: (a) penetration test; and (b) viscosity test on asphalt binders.

of grading asphalt binders, as described in the remainder of this chapter. Photographs of the penetration test and the capillary viscosity test are shown in Figure 3-2.

Performance Grading of Asphalt Binders—Overview

Performance grading of asphalt binders was developed during SHRP. The main purpose of this way of classifying and selecting asphalt binders is to make certain that the binder has the correct properties for the given environment. Performance grading was also meant to be based more soundly on basic engineering principles—earlier methods of grading binders often used empirical tests, which were useful but did not provide any information on the fundamental engineering properties of the binder. Performance grading uses various measurements of the binder's flow properties to establish its grade, which is expressed as two numbers, for example "PG 64-22." In this example, the "64" represents the maximum pavement temperature for which this binder can be used at low [moderate?] traffic levels. The second number, "–22," signifies the minimum temperature for which the binder can be used without likelihood of failure. It is essential to understand when using the performance grading system that the numbers in the grade designation represent the most extreme temperatures for which that binder is suited. For example, if a given application requires a PG 58-16 binder, there are many other grades that could meet the requirements: PG 58-22, PG 58-28, PG 64-16, PG 64-22, etc. This is only a simple explanation of the basic features of performance grading; the details of the system are more complicated and are explained below.

Performance Grading—Test Methods

Most of the tests used in performance grading of asphalt binders involve rheological tests. Rheology is the study of the way materials flow, so a rheological test is one that measures one or more aspects of the way in which a material flows. The dynamic shear rheometer (DSR) and the bending beam rheometer (BBR) both measure the flow properties of asphalt binders—the DSR at temperatures ranging from about 10 to 82°C, the BBR at temperatures ranging from −20 to 0°C. It may at first be surprising that asphalt binders flow at temperatures below 0°C, where it normally appears to be a brittle, glassy material. But asphalt binder will in fact flow even at very low temperatures, although this flow might take months or even years before it is noticeable to the naked eye.

Asphalt binders can be characterized as they are produced at the refinery using these rheological tests. Unfortunately, this is not enough to give technicians and engineers a good idea of how a binder will perform in a pavement, since asphalt binders in a pavement harden during mixing, transport, and placement of the mix, and even after the pavement has been placed. Therefore, laboratory procedures are needed to estimate the amount of such age hardening. Two laboratory age-hardening procedures are used in the performance grading system: the rolling-thin film oven test (RTFOT) and the pressure aging vessel (PAV). The way in which these aging tests are used in combination with the rheological tests is illustrated in Figure 3-3. DSR tests at high temperature are performed on both the unaged binder and on the residue from the RTFOT aging procedure. Residue from the RTFOT procedure is then aged in the PAV, and additional tests are performed on the residue from this procedure. The DSR test at intermediate temperature and the BBR test are always performed. An optional test, the direct tension test, is also sometimes performed on the PAV residue. Unlike the other performance grading tests, the direct tension procedure does not measure flow properties, but instead measures the fracture properties of the asphalt binder at low temperatures. The direct tension test is useful for grading some modified binders with unusually high strength and toughness, since it will improve the low temperature grade.

The sections below describe in additional detail each of the grading procedures. The aging procedures are discussed first, followed by the actual binder tests. This is followed by a discussion of the grading procedure and a section covering practical aspects of performance grade selection for HMA mix design.

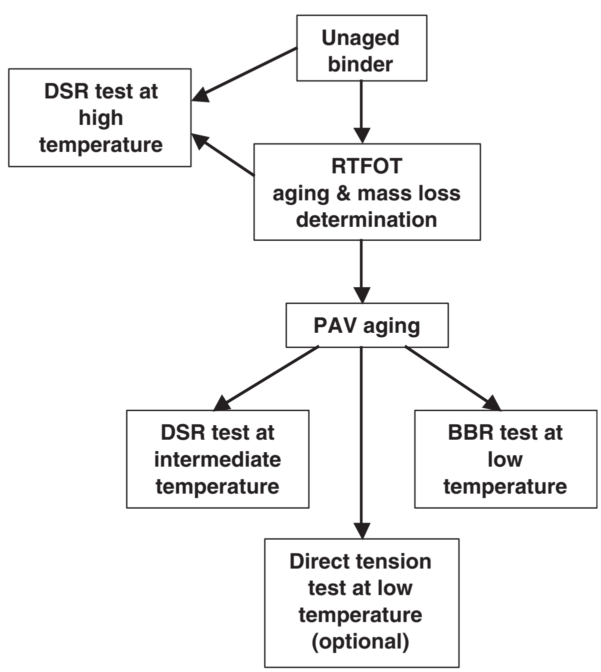

Figure 3-3. Flow chart for performance grading tests.

RTFOT Aging

RTFOT aging is meant to simulate asphalt binder age hardening as it occurs during mixing, transport, and placement. In this procedure, 35 grams of asphalt are carefully weighed into glass bottles, which are placed in a circular rack in a specially designed oven. The rack slowly rotates the bottles while the oven maintains the test temperature of 323°C. During the test, a jet of air is blown into the bottles for a few seconds once every rotation. The test is continued for 75 minutes; the bottles are then removed from the oven, cooled, and weighed. The percent mass loss is calculated from the initial and final weight of the asphalt binder in the bottle. High values of mass loss mean that a significant amount of light oils have volatized during aging and, as a result, the asphalt binder might be prone to excessive age hardening, shrinkage, and cracking. Current performance grading standards require that mass loss during RTFOT aging be no more than 1.0%. After mass loss determination, the bottles are heated, and the asphalt is poured either into a tin for further testing, or into PAV pans for additional aging. Figure 3-4 shows an RTFOT oven.

PAV Aging

In the PAV aging test, the technician fills 125-mm-diameter stainless steel pans with asphalt that has already been aged in the RTFOT test. Six of these pans are placed in a vertical rack, which is then placed in the pressure vessel, which in turn is placed inside an oven. The pressure vessel is a heavily constructed steel chamber, designed to withstand the high pressure and temperature used in the PAV test. These high temperatures and pressures help accelerate aging of the asphalt binder. At the end of the PAV test, the asphalt binder has aged about as much as would typically occur in a pavement after several years of service. Figure 3-5 shows the various pieces of equipment used in performing the PAV aging procedure.

DSR Test at High Temperature

The primary purpose of the DSR test at high temperature is to ensure that a properly specified asphalt binder will have the proper engineering properties at high temperature and, when used in an HMA mix, will keep it from rutting and shoving under traffic. The DSR is a torsional test in which a thin specimen of asphalt binder is sheared between two circular plates. It is also

Figure 3-4. RTFOT aging oven.

(a) (b)

Figure 3-5. PAV aging test: (a) pan; and (b) rack filled with pans.

a dynamic test, meaning that the specimen is sheared very quickly in a back and forth cycle of loading. In the high-temperature test, the steel plates are 25 mm in diameter, and the specimen is about 1-mm thick. Figure 3-6 is a sketch of the DSR test, showing both the high temperature setup and the smaller plates used for the intermediate temperature test (described in detail in the following section). The applied strain varies depending on the stiffness of the binder. The test is performed at various temperatures, depending on the grade of the asphalt: 46, 52, 58, 64, 70, 76, and 82°C are the standard temperatures for high-temperature DSR tests. The first number in a performance grade is the standard high temperature for DSR testing for that binder. For example, one of the most common grades of asphalt binder is PG 64-22; the high-temperature DSR test on this binder would be performed at 64°C.

The DSR test measures both modulus and phase angle. Modulus is a measure of stiffness—the higher the value, the stiffer the binder. Because the DSR test is a shear test, the modulus value is called the dynamic shear modulus, abbreviated with the symbol G^*. The "G" indicates that the modulus is a shear value, and the "*" indicates that it is a dynamic modulus. In rheological tests like the DSR, the phase angle is a measure of how fluid a material is. The more a material behaves like a fluid, the higher the phase angle. Materials that behave like an elastic solid—that spring back quickly after loading—have a low phase angle. Phase angle is often abbreviated using the Greek letter δ ("delta"). When a material with a high phase angle is loaded and deforms, and the load is removed, the material will tend to stay in its deformed shape—it will not spring back.

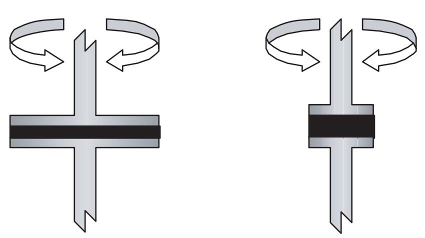

Figure 3-6. Diagram of DSR test at high and intermediate temperature.

Phase angle should not be confused with stiffness or modulus. A very stiff clay might have a higher modulus than a soft rubber, but a higher phase angle. This means it will deform less than the rubber under loading, but will not recover any of this deformation once the load is removed. In the high-temperature DSR test, the quantity specified is $G^*/\sin\delta$, in units of kPa. By using both G^* and $\sin\delta$ in the specification, the stiffness and elasticity of the asphalt binder are simultaneously controlled. Stiff, elastic binders will have a higher $G^*/\sin\delta$ value than soft, fluid binders. Performance-graded binders must have a $G^*/\sin\delta$ value of at least 1.0 kPa at the specified grading temperature in the unaged condition, using a test frequency of 10 rad/s. After RTFOT aging, the minimum value of $G^*/\sin\delta$ is 2.2 kPa.

DSR Test at Intermediate Temperature

The DSR test at intermediate temperature uses the same basic principles as the high-temperature test, but there are a few important differences. The DSR test at intermediate temperatures is designed to prevent binders from becoming too stiff at intermediate temperatures, which can contribute to premature fatigue cracking in pavements. This also helps to control the overall flow properties of the asphalt binder. Because the asphalt binder is much stiffer at the lower test temperatures, the plates must be smaller and the specimen thicker, as shown in Figure 3-6. For the intermediate temperature test, 8-mm-diameter plates are used, and the specimen is 2 millimeters thick. Instead of $G^*/\sin\delta$, the specified quantity for the intermediate temperature test is $G^* \cdot \sin\delta$, since many pavement researchers have found a relationship between $G^* \cdot \sin\delta$ and fatigue resistance for HMA mixtures. The DSR test at intermediate temperatures is run after RTFOT and PAV conditioning, at temperatures ranging from 4 through 40°C, in 3° increments (4, 7, 10°C, etc). The maximum allowable value for $G^* \cdot \sin\delta$ is 5,000 kPa, at a frequency of 10 rad/s.

BBR Test

The purpose of the BBR test is to make sure that asphalt binders do not become too stiff and brittle at low temperatures, since this can contribute to transverse cracking in HMA pavements. The BBR test is a flexural stiffness test—a small beam of asphalt is loaded for 1 minute and the deflection is measured. From the applied load and resulting deflection, the creep stiffness of the asphalt binder is calculated. In analyzing the BBR data, another quantity, called the *m-value,* is also calculated. The m-value is the log-log slope of the creep curve at a given loading time. The BBR specimen is 125 millimeters long, 12.5 millimeters wide, and 6.25 millimeters thick. The test can be run at test temperatures of –36, –30, –24, –18, –12, –6 and 0°C. When performance grading an asphalt binder, the BBR test is run at a temperature 10°C higher than the low grading temperature. A performance 64-22 binder, for example, would be tested using the BBR at –12°C. The maximum allowable stiffness in the BBR test is 300 MPa at 60 seconds, and the minimum m-value is 0.300 at the same loading time. Figure 3-7 is a sketch of the BBR test.

Direct Tension Test

The direct tension test is unique among the binder specification tests in that it is a fracture test, and not a rheological test. In this procedure, a small specimen of asphalt is slowly pulled apart in tension until it fails. Figure 3-8 illustrates the direct tension test. This test should not be confused with the older ductility test, which is performed at higher temperatures at much higher strains, and is an empirical test that does not provide any useful information on engineering properties. The results of the direct tension test are strain and stress at failure, and the test is performed at low temperatures at very low strains and strain rates. These results can be used to perform an analysis of low-temperature thermal stresses that produces an estimated cracking temperature for the binder, as outlined in AASHTO Provisional Standard PP 42. This temperature is then used

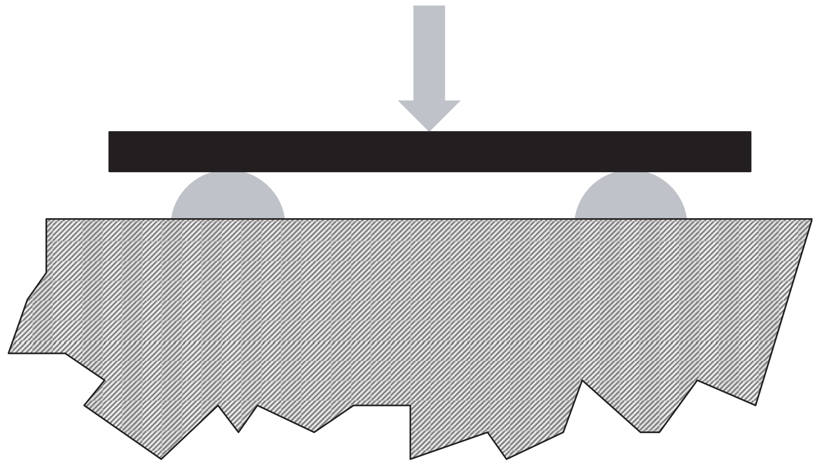

Figure 3-7. Sketch of BBR test.

to determine the low-temperature performance grade for the given binder. The main advantage of the direct tension test and associated analysis compared to BBR grading is that many polymer-modified binders have enhanced fracture properties that will result in a lower grading temperature using the direct tension test compared to that produced by BBR grading.

Performance Grading—Specification

Table 3-1 lists the various requirements for performance-graded asphalt binders, as described in AASHTO M 320 Table 1. In AASHTO M 320 Table 1, the low-temperature grade of the binder is based on creep stiffness and the m-value of the binder from the BBR. If the binder has a creep stiffness between 300 and 600 MPa, the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. There is also a Table 2 in AASHTO M 320 which uses the critical low cracking temperature from the direct tension test to determine the low-temperature grade of the binder. AASHTO M 320 Table 1 is the most commonly used specification for PG binders.

(a) (b)

Figure 3-8. Photographs of the direct tension test: (a) specimen and mold and (b) test device.

Table 3-1. Specification for performance-graded asphalt binders.

Binder Performance Grade:	PG 46			PG 52							PG 58				
	–34	**–40**	**–46**	**–10**	**–16**	**–22**	**–28**	**–34**	**–40**	**–46**	**–16**	**–22**	**–28**	**–34**	**–40**
Design high pavement temperature, °C:	<46			<52							<58				
Design low pavement temperature, °C:	≥34	≥40	≥46	≥10	≥16	≥22	≥28	≥34	≥40	≥46	≥16	≥22	≥28	≥34	≥40
	Test on Original Binder														
Flash Point Temperature (T 48), Min., °C	230														
Viscosity (T 316) **Maximum value of 3 Pa-s at test temperature, °C**	135														
Dynamic Shear (T 315) **G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C**	46			52							58				
	Tests on Residue from Rolling Thin Film Oven (T 240)														
Mass Loss, Maximum, %	1.00														
Dynamic Shear (T315) **G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C**	46			52							58				
	Tests on Residue from Pressure Aging Vessel (R 28)														
PAV Aging Temperature, °C	90			90							100				
Dynamic Shear (T 315) **G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C**	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13
Creep Stiffness (T 313) **Stiffness, maximum value 300 Mpa** **m-value, minimum value 0.30, at 60 sec and Test Temperature,°C**	–24	–30	–36	0	–6	–12	–18	–24	–30	–36	–6	–12	–18	–24	–30
Direct Tension (T 314) **Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C**	–24	–30	–36	0	–6	–12	–18	–24	–30	–36	–6	–12	–18	–24	–30

Binder Performance Grade:	PG 64						PG 70					
	–10	**–16**	**–22**	**–28**	**–34**	**–40**	**–10**	**–16**	**–22**	**–28**	**–34**	**–40**
Design high pavement temperature, °C:	<64						<70					
Design low pavement temperature, °C:	≥10	≥16	≥22	≥28	≥34	≥40	≥10	≥16	≥22	≥28	≥34	≥40
	Tests on Original Binder											
Flash Point Temperature (T 48), Min., °C	230											
Viscosity (T 316) **Maximum value of 3 Pa-s at test temperature, °C**	135											
Dynamic Shear (T 315) **G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C**	64						70					
	Tests on Residue from Rolling Thin Film Oven (T 240)											
Mass Loss, Maximum, %	1.00											
Dynamic Shear (T 315) **G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C**	64						70					
	Tests on Residue from Pressure Aging Vessel (R 28)											
PAV Aging Temperature, °C	100						100 (110)					
Dynamic Shear (T 315) **G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C**	31	28	25	22	19	16	34	31	28	25	22	19
Creep Stiffness (T 313) **Stiffness, maximum value 300 Mpa** **m-value, minimum value 0.30, at 60 sec and Test Temperature,°C**	0	–6	–12	–18	–24	–30	0	–6	–12	–18	–24	–30
Direct Tension (T 314) **Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C**	0	–6	–12	–18	–24	–30	0	–6	–12	–18	–24	–30

(continued on next page)

Table 3-1. (Continued).

Binder Performance Grade:	PG 76					PG 82				
	–10	–16	–22	–28	–34	–10	–16	–22	–28	–34
Design high pavement temperature, °C:	<76					<82				
Design low pavement temperature, °C:	≥10	≥16	≥22	≥28	≥34	≥10	≥16	≥22	≥28	≥34
	Tests on Original Binder									
Flash Point Temperature (T 48), Min., °C	230									
Viscosity (T 316) Maximum value of 3 Pa-s at test temperature, °C	135									
Dynamic Shear (T 315) G*/sin δ, minimum value 1.00 kPa, at 10 rad/s and Test Temperature, °C	76					82				
	Tests on Residue from Thin Film Oven (T 240)									
Mass Loss, Maximum, %	1.00									
Dynamic Shear (T 315) G*/sin δ, minimum value 2.20 kPa, at 10 rad/s and Test Temperature, °C	76					82				
	Tests on Residue from Pressure Aging Vessel (R 28)									
PAV Aging Temperature, °C	100 (110)					100 (110)				
Dynamic Shear (T 315) G* sin δ, maximum value 5,000 kPa, at 10 rad/s and Test Temperature, °C	37	34	31	28	25	40	37	34	31	28
Creep Stiffness (T 313) Stiffness, maximum value 300 Mpa m-value, minimum value 0.30, at 60 sec and Test Temperature,°C	0	–6	–12	–18	–24	0	–6	–12	–18	–24
Direct Tension (T 314) Failure strain, minimum value 1.0%, at 1.0 mm/min and Test Temperature, °C	0	–6	–12	–18	–24	0	–6	–12	–18	–24

Critical Temperatures, Specification Values, and Reliability

A unique feature of the performance grading system is that it is based not on the values of a given property at a given temperature, but on at what temperature a critical value of that property is achieved. A PG 58-28 binder has a G*/sin δ value of at least 1.0 kPa at 58°C and 10 rad/s in the unaged condition and a maximum flexural creep stiffness of no more than 300 MPa at –18°C at 60 s. The two numbers in the performance grade (PG) refer to extreme high and low pavement temperatures at which the binder is expected to perform adequately. It is important to understand how these extreme pavement temperatures are defined. The high temperature is defined as the yearly, 7-day average maximum pavement temperature, measured 20 millimeters below the pavement surface (referred to as design high pavement temperature). This may seem straightforward, but because high pavement temperatures are quite variable, the design high pavement temperature will vary from year to year and cannot be defined in a precise, single value. Instead, statistical methods must be used through the concept of reliability. The reliability of a given high pavement temperature refers to the probability that it will not be exceeded in any given year. For example, in Saint Louis, MO, the average value of the design high pavement temperature is 52.9°C. That means that in any given year, there is a 50% chance that the actual high pavement temperature will be lower than this, and a 50% chance that it will be higher. Therefore, the design high pavement temperature at a 50% level of reliability for Saint Louis is 52.9°C. At a 98% level of reliability, the design high pavement temperature is 60.0°C. In other words, in any given year there is a 98% chance that the maximum pavement temperature in Saint Louis will be less than 60°C.

The same approach is used in low-temperature performance grading. In this case, the low pavement temperature is defined simply as the minimum pavement temperature at the pavement

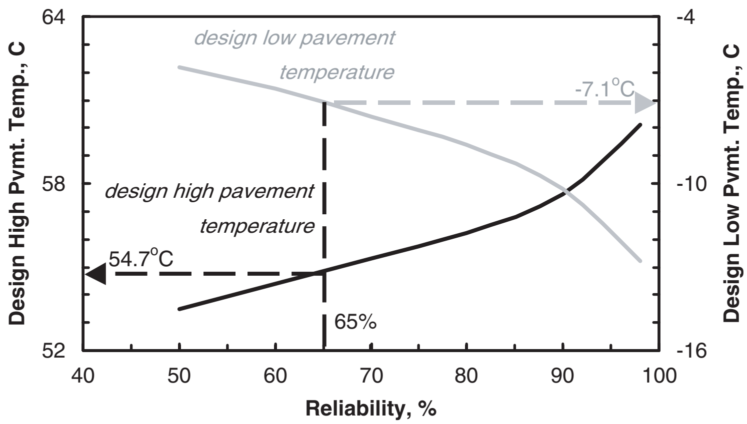

Figure 3-9. Example of PG binder grade reliability for Atlanta, GA.

surface experienced at a given location in a given year. For Salt Lake City, UT, the average low pavement temperature is −13.6°C. Thus, the design low pavement temperature at 50% reliability is −13.6°C. At a 98% reliability level, the design low pavement temperature at Salt Lake City is −21.3°C. It should be emphasized that the design low pavement temperature is not the same as the minimum air temperature. Typically, the design low pavement temperature is significantly higher than the minimum air temperature for a given location. In Salt Lake City, for example, the average minimum air temperature is −19.6°C, 6 degrees colder than the average design low pavement temperature.

Figure 3-9 is a plot of performance grade reliability for design high and low pavement temperatures for Atlanta, GA. In the example illustrated in this plot, at a 65% reliability level, the design high pavement temperature is 54.7°C, and the design low pavement temperature is 7.1°C.

Calculation of design high and low pavement temperatures at different reliability levels involves compilation of a wide range of weather data and analysis of this data to produce both average values and standard deviations for design high and low pavement temperatures for thousands of sites throughout the United States and Canada. Fortunately, the software package LTPPBind has been developed to perform these calculations for pavement engineers and technicians. The values in the examples given above were taken from LTPPBind, Version 2.1. LTPPBind also can generate various useful plots, including reliability plots like that shown in Figure 3-9. At the time this manual was being compiled, a new version of LTPPBind—Version 3.0—was in beta release. This newer version of LTPPBind differs substantially from Version 2.1. The most important of these differences is that in Version 3.0, critical high temperatures are based not just on pavement temperatures calculated from historical weather data, but from damage analyses performed using a newly developed rutting model. Version 3.0, once in full release, should provide better estimates for design high pavement temperatures in hot, dry climates—situations where earlier versions of LTPPBind appeared to under-predict high pavement temperatures. The LTPPBind program can be downloaded from the LTPPBind website maintained by the FHWA.

An important question is what level of reliability should be used when selecting binders. Engineers and technicians should keep in mind that if a PG binder is selected at a 50% reliability level, there is a 50-50 chance in any year that the high and/or low pavement temperature will exceed those for which the binder has been developed. That is, a pavement made using a binder selected at a 50% reliability level is likely to exhibit rutting and or low-temperature cracking within a few years. Therefore, high reliability levels should be used when selecting binders. For lightly traveled rural and residential roads, reliability levels of at least 90% should be used. For interstate highways and other major construction projects, reliability levels of at least 95% should be used when selecting performance-graded binders.

Practical Selection of PG Binder Grades for HMA Mix Design

Although the LTPPBind computer program is very useful, in practice most highway agencies have, through experience, developed their own systems for selecting binder performance grades depending on traffic level and location. This has been done in part because refineries are able to produce only a limited number of binder grades, so engineers must determine two or three performance grades that can be used to meet most or all of the paving needs in a given region. This is sometimes referred to as a "binder slate" for a given state or region. For example, a common binder slate in the Mid-Atlantic states involves only three performance grades: 58-28, 64-22, or 76-22. Other binders might be occasionally used in this region, but typically only for small demonstration projects. Engineers selecting performance-graded binders for paving applications should refer to the appropriate specifications for their state or, if there are none, to those in neighboring states with similar climates and conditions. Binder producers may also be useful in providing information concerning what binder performance grades are available locally and which might be most appropriate for a given application. Engineers and technicians using the LTPPBind program without referring to the binders used by the local highway agency may find that the binder they have specified for a given application is not locally available.

In selecting performance-graded binders from an available slate, it must be remembered that a given performance grade will meet the requirements of many less extreme situations. For example, in many areas of the Mid-Atlantic, LTPPBind (version 3.1) indicates that a PG 58-22 binder should be used for light traffic. However, this binder may not be available in some Mid-Atlantic states. If the PG 58-22 binder cannot be found (or found at a reasonable price), a PG 64-22 binder would be selected and would perform perfectly well, since its extreme high and low temperature ratings meet or exceed those for these applications. Care should however be used in selecting binders that are much stiffer than required for a given application. Recently, many highway agencies have noticed an increase in surface cracking in HMA pavements. Although such top-down cracking is not yet fully understood, using unnecessarily stiff binders may contribute to the problem. Additional details concerning the selection of asphalt binders for HMA mixtures are given in Chapter 8 of this manual.

Bibliography

AASHTO Standards

M 320, Performance-Graded Asphalt Binder
M 323, Superpave Volumetric Mix Design
PP 42, Determination of Low-Temperature Performance Grade (PG) of Asphalt Binders
R 28, Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
R 29, Grading or Verifying the Performance Grade of an Asphalt Binder
R 35, Superpave Volumetric Design for Hot-Mix Asphalt (HMA)
T 48, Flash and Fire Points by Cleveland Open Cup
T 49, Penetration of Bituminous Materials
T 51, Ductility of Bituminous Materials
T 53, Softening Point of Bitumen (Ring-and-Ball Apparatus)
T 202, Viscosity of Asphalts by Vacuum Capillary Viscometer
T 240, Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)
T 313, Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)

T 314, Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT)
T 315, Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
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CHAPTER 4

Aggregates

Because about 85% of the volume of dense-graded HMA is made up of aggregates, HMA pavement performance is greatly influenced by the characteristics of the aggregates. Aggregates in HMA can be divided into three types according to their size: coarse aggregates, fine aggregates, and mineral filler. Coarse aggregates are generally defined as those retained on the 2.36-mm sieve. Fine aggregates are those that pass through the 2.36-mm sieve and are retained on the 0.075-mm sieve. Mineral filler is defined as that portion of the aggregate passing the 0.075-mm sieve. Mineral filler is a very fine material with the consistency of flour and is also referred to as mineral dust or rock dust.

Gravel refers to a coarse aggregate made up mostly of rounded particles. Gravels are often dredged from rivers and are sometimes mined from deposits. Because of the rounded particle size, gravels are not suitable for use in HMA mixtures unless they are well crushed. Poorly crushed gravels will not interlock when used in HMA, and the resulting mixture will have poor strength and rut resistance. Crushed stone is coarse aggregate that is mined and processed by mechanical crushing. It tends to be a very angular material and, depending on its other properties, can be well suited for use in HMA pavements. One potential problem with crushed stone is that the particles sometimes will tend to be flat, elongated, or both, which can cause problems in HMA mixtures. Ideally, the particles in crushed stone aggregate should be cubicle and highly angular.

The fine aggregate, or sand, used in HMA can be natural sand, manufactured sand, or a mixture of both types. Natural sand is dredged from rivers or mined from deposits and is then processed by sieving to produce a fine aggregate having the desired particle size distribution. Manufactured sand is produced by crushing quarried stone and, like natural sand, sieving to produce the desired gradation. The particles in manufactured sands tend to be more angular than those in natural sand and often will produce HMA mixtures having greater strength and rut resistance compared to those made with natural sand. However, this is not always true, and care is needed when selecting fine aggregate for use in HMA mixtures. The fine aggregate angularity test described later in this chapter, although not always reliable, can help to evaluate the angularity of both natural and manufactured sands.

Pavement engineers have worked for many years to relate specific aggregate properties to HMA performance. Rutting, raveling, fatigue cracking, skid resistance, and moisture resistance have all been related to aggregate properties. It is essential that engineers and technicians responsible for HMA mix design thoroughly understand aggregate properties, how they relate to HMA pavement performance, and how aggregate properties are specified and controlled as part of the mix design process.

Aggregate Particle Size Distribution

Perhaps the most widely specified aggregate property is particle size distribution. Although only indirectly related to HMA performance, controlling particle size distribution, also called aggregate gradation, is critical to developing an effective mix design. The maximum aggregate size in an aggregate must be matched to the lift thickness used during construction, otherwise the pavement will be difficult to place and compact properly. The distribution of particle sizes in an aggregate must have just the right density so that the resulting HMA will contain the optimum amount of asphalt binder and air voids. Because the shape and texture of aggregate particles vary significantly depending on the aggregate type and the way it is mined and processed, specification limits for aggregate gradation tend to be very broad. This breadth helps technicians and engineers achieve the right blend of aggregates for different applications. The section below describes general terminology used when discussing aggregate particle size distribution and the relationship between aggregate gradation and different HMA mix types. Because suggested limits for aggregate gradation vary depending on the type of mix design being developed, only a few examples are given here—a complete listing of aggregate gradation requirements are given in the various chapters discussing mix design procedures for the various HMA mix types: dense-graded HMA, gap-graded HMA, and open-graded friction courses.

Nominal Maximum Aggregate Size

Nominal maximum aggregate size (NMAS) is a way of specifying the largest aggregate size in an aggregate. In the mix design procedure described in this manual, as in the Superpave system, NMAS is defined as one sieve-size larger than the first sieve size to retain 10% or more of the total aggregate by mass.

Aggregate Sieve Analysis

The particle size distribution of construction aggregates is usually determined and specified by performing a sieve analysis. In this test, an aggregate is passed through a stack of sieves of decreasing size. The amount of aggregate on each sieve is weighed, and the percent passing each sieve size is calculated as a percent by weight. Sometimes, for aggregates made up of different minerals or rocks having widely different specific gravities, the results of the sieve analysis are given as percent passing by volume. For HMA mix design and analysis, an aggregate sieve analysis uses the following standard sieve sizes: 37.5 mm, 25.0 mm, 19.0 mm, 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 0.60 mm, 0.30 mm, 0.15 mm, and 0.075 mm. Other sieve sizes are sometimes used for special purposes or in other aggregate test procedures.

A schematic of a simplified sieve analysis is shown in Figure 4-1. An aggregate sample is collected and placed through a stack of sieves. The sieves are usually shaken mechanically, until the aggregate has been separated completely on the various sieves. At the bottom of the sieve stack is a pan, in which material is collected that has completely passed through the stack of sieves. The aggregate on each sieve is then weighed, and calculations are performed to determine the percent passing for each sieve.

In performing a sieve analysis, there are several important considerations:

- The weight of the test sample must be large enough to produce reliable test results. Larger aggregate sizes will require larger sample sizes. Table 4-1 lists minimum weights for test samples for sieve analysis for different NMAS values.
- The sieve opening sizes selected should be appropriate for the aggregate being tested. Past data or the gradation specification for the aggregate being tested can be used to determine the sieves needed for a given aggregate.
- The physical size of the sieves—the diameter or area—increases with increasing aggregate size. For fine aggregate, 203-mm-diameter sieves are often used. The maximum amount of aggregate retained on sieves of this size should be limited to about 194 g; larger amounts may result in inaccurate sieve analyses because the aggregate can longer flow freely through the stack of sieves.

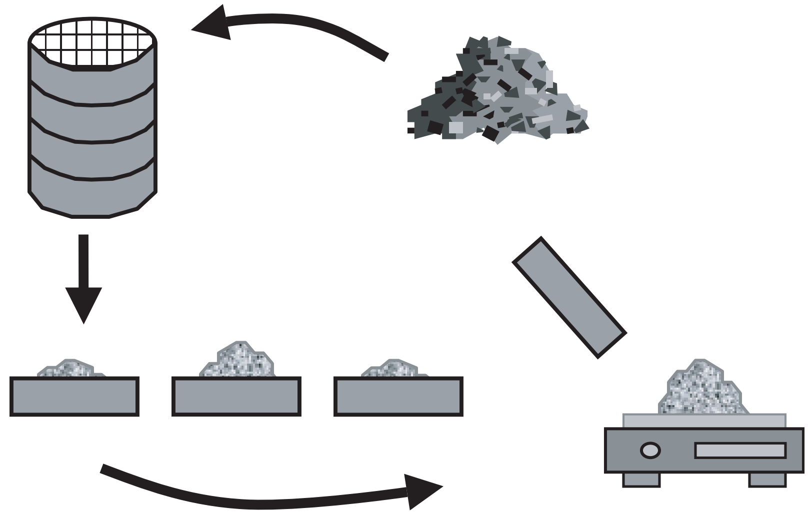


Figure 4-1. Schematic of an aggregate sieve analysis.

- Overloading sieves can sometimes be prevented by placing intermediate sieve sizes in a stack of sieves. For example, a specification may only require the determination of percent passing the 9.5- and 19-mm-diameter sieves, but using these sieves only might overload the 9.5-mm sieve. Inserting a 12.5-mm-diameter sieve between the 9.5- and 19-mm-diameter sieves will help prevent overloading of the 9.5-mm-diameter sieve.
- The amount of time a stack of sieves is shaken should be long enough to ensure that the aggregate particles have been completely sorted through the stack, but not so long that significant aggregate degradation might occur.
- Accurate determination of mineral filler—material finer than 0.075 mm—will in general require a washed sieve analysis.

Figure 4-2 shows a stack of sieves for fine aggregate assembled in a mechanical sieve shaker. Engineers and technicians should refer to appropriate specifications for details on performing aggregate sieve analyses: AASHTO T 27, Sieve Analysis of Fine and Coarse Aggregate; AASHTO T 11, Materials Finer than 75-μm Sieve in Mineral Aggregates by Washing; and AASHTO T 30, Mechanical Analysis of Extracted Aggregates.

Calculations for Aggregate Sieve Analyses

The results of an aggregate sieve analysis in HMA technology are usually presented as weight percent passing. Calculation of percent passing from the results of a sieve analysis is straightforward and is best explained through an example. Table 4-2 gives the results of a sieve analysis of a fine aggregate, along with the calculations of percent retained, cumulative percent retained, and

Table 4-1. Minimum test sample size for sieve analysis of aggregate as a function of nominal maximum aggregate size.

Nominal Maximum Aggregate Size, mm	Minimum Weight for Test Sample, kg
9.5	1
12.5	2
19.0	5
25.0	10
37.5	15

Figure 4-2. Stack of sieves for fine aggregate assembled in a mechanical sieve shaker.

percent passing. The weight retained, as shown in Column 2, is the weight in grams of the aggregate separated onto each sieve. The total of these values, 1143.6 g, is slightly less than the original sample weight of 1146.0 g. The difference is due to material lost, either as dust lost to the air, particles trapped within the mesh of the sieves, or particles fallen from the sieves without being weighed. The percent error is calculated as the difference between the total weight retained and the original sample weight, expressed as a percent of the original sample weight:

$$Error = \frac{1146.0 - 1143.6}{1146.0} \times 100\% = 0.21\% \tag{4-1}$$

The % retained is calculated by dividing the weight retained for each sieve by the original sample weight and, again, expressing the result as a weight percentage. For the 2.36-mm-diameter sieve

$$\%\ retained\ 2.36\ mm\ sieve = \frac{231.7}{1146.0} \times 100\% = 20.2\% \tag{4-2}$$

Table 4-2. Example sieve analysis.

(1) Sieve Size, mm	(2) Weight Retained, g	(3) % Retained, Wt. %	(4) Cumulative % Retained, Wt. %	(5) % Passing, Wt. %
19.0	0.0	0.0	0.0	100.0
12.5	0.0	0.0	0.0	100.0
9.5	97.5	8.5	8.5	91.5
4.75	214.6	18.7	27.2	72.8
2.36	231.7	20.2	47.5	52.5
1.18	215.8	18.8	66.3	33.7
0.60	116.3	10.1	76.4	23.6
0.30	90.4	7.9	84.3	15.7
0.15	75.2	6.6	90.9	9.1
0.075	57.8	5.0	95.9	4.1
pan	44.3	3.9	99.8	---
Total:	1143.6	99.8		
Original Sample Size:	1146.0			
Error, Wt. %:	0.21			

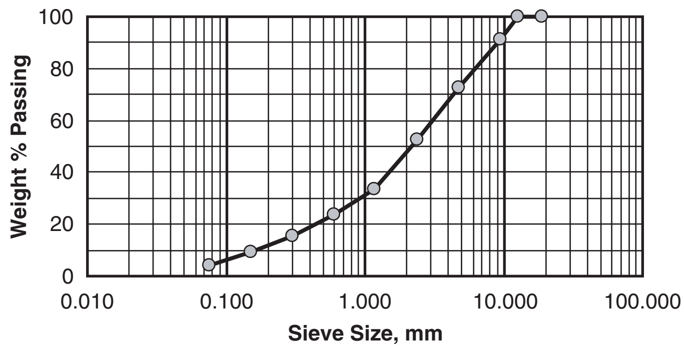


Figure 4-3. % passing plotted against sieve size for example sieve analysis given in table 4-2.

The cumulative % retained is calculated by summing all the values for % retained up to the given sieve size. For the 0.60-mm sieve:

$$\textit{cumulative } \% \textit{ retained } 0.60 \textit{ mm sieve} = 8.5 + 18.7 + 20.2 + 18.8 + 10.1 = 76.4\% \quad (4\text{-}3)$$

The % passing is calculated as 100%—the cumulative % retained. For the 0.60-mm-diameter sieve, for example, the % passing is calculated as $100 - 76.4 = 23.6\%$.

It should be pointed out that there are slightly different ways of calculating these values for sieve analyses, and those responsible for HMA mix design and associated testing should follow the procedures as required by their state agencies.

The results of aggregate sieve analyses are usually presented graphically, by plotting percent passing against sieve size in mm. Sieve size is often plotted on a logarithmic scale. Figure 4-3 is a plot of the results of the example sieve analysis given in Table 4-2.

Aggregate Gradation

The plot given in Figure 4-3 is an example of an aggregate gradation. For purposes of classifying HMA mix types, there are four types of aggregate gradation: dense-graded, fine-graded, coarse-graded, and open-graded. As explained in Chapter 8, when designing HMA, two, three, four, or even more aggregates are combined in specific proportions to create an aggregate blend to which asphalt binder is added forming an HMA mixture. Because fine and coarse aggregate gradations as used in HMA are really slight variations of dense gradations, a more accurate description of these would be dense/fine and dense/coarse aggregate gradations or blends. The densest possible aggregate gradation, called the maximum density gradation (or sometimes the Fuller maximum density curve), can be approximately calculated using the following formula:

$$\%PMD = \left(\frac{d}{D}\right)^{0.45} \times 100\% \quad (4\text{-}4)$$

where

$\% PMD$ = % passing, maximum density gradation
d = sieve size in question, mm
D = maximum sieve size, mm

Figure 4-4 illustrates the different types of HMA aggregate gradations and includes the maximum density gradation calculated using Equation 4-4 for a maximum aggregate size of

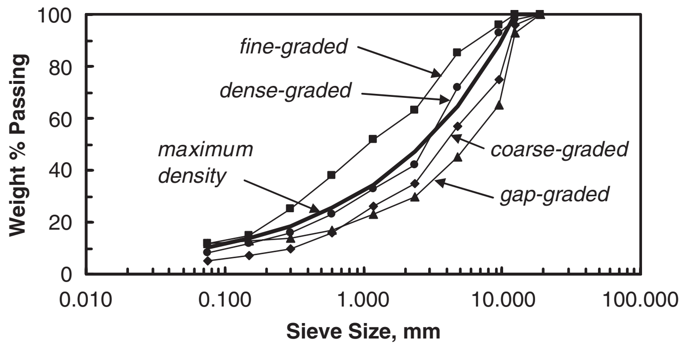


Figure 4-4. Types of HMA aggregate gradations; heavy black line represents maximum density gradation as calculated using equation 4-4.

12.5 mm. Coarse-graded and gap-graded aggregates for HMA mixes can be very similar; the main difference is that gap-graded aggregate blends tend to contain more large aggregate particles compared to coarse gradations, but they also contain larger amounts of very fine material, especially mineral filler. Methods for blending aggregates in HMA mix design are discussed in detail in Chapter 8 of this manual.

Specifications for Aggregate Gradation

There are two types of gradation specifications or requirements used in HMA technology: (1) specifications for the gradation of processed aggregate, as supplied by aggregate producers and (2) requirements for blends of aggregate developed as part of the HMA mix design process and used in the production of HMA. Requirements for aggregate blends are listed in the chapters of this manual specifically dealing with HMA mix design: Chapter 8 for dense-graded mixtures, Chapter 10 for gap-graded mixtures, and Chapter 11 for open-graded friction course mixtures. AASHTO M 43, Standard Specification for Sizes of Aggregate for Road and Bridge Construction, gives the gradation requirements for a wide range of coarse aggregate sizes used in the development of HMA mix designs. Table 4-3 is a shortened version of these requirements, listing the most commonly used aggregate gradations. Table 4-4 lists AASHTO specifications for fine aggregates for bituminous paving mixtures, as given in AASHTO M 29, Fine Aggregate for Bituminous Paving Mixtures.

It should be noted that although many agencies may follow the AASHTO grading requirements listed in Tables 4-3 and 4-4, some agencies may use different specifications. In some cases, there may be minor modifications to the AASHTO requirements; in other cases, the specifications may be significantly different. Engineers and technicians responsible for developing HMA mix designs should obtain current specifications for aggregate gradations from the applicable state highway department (or other agency as appropriate).

Aggregate Specific Gravity and Absorption

When designing HMA, both the mass and volume of the aggregates and asphalt binder going into the mixture must be known. The mass and volume of a material are related through the values of density or specific gravity. Density refers to the mass of a material per unit volume. Density values for most construction materials, including aggregates, are usually reported in

Table 4-3. Standard sizes of coarse aggregates for road and bridge construction as adapted from AASHTO M 43.

AASHTO Size No.	% Passing (mass %) for Sieve Size:									
	50 mm	37.5 mm	25.0 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.300 mm
4	100	90 to 100	20 to 55	0 to 15	---	0 to 5	---	---	---	---
467	100	90 to 100	---	35 to 70	---	10 to 30	0 to 5	---	---	---
5	---	100	90 to 100	20 to 55	0 to 10	0 to 5	---	---	---	---
56	---	100	95 to 100	40 to 85	10 to 40	0 to 15	0 to 5	---	---	---
57	---	100	100	---	25 to 60	---	0 to 10	0 to 5	---	---
6	---	---	100	90 to 100	20 to 55	0 to 15	0 to 5	---	---	---
67	---	---	100	90 to 100	---	20 to 55	0 to 10	0 to 5	---	---
68	---	---	---	90 to 100	---	30 to 65	5 to 25	0 to 10	0 to 5	---
7	---	---	---	100	90 to 100	40 to 70	0 to 15	0 to 5	---	---
78	---	---	---	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	---
8	---	---	---	---	100	85 to 100	10 to 30	0 to 10	0 to 5	---
89	---	---	---	---	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5
9	---	---	---	---	---	100	85 to 100	10 to 40	0 to 10	0 to 5

units of g/cm^3; density values for HMA and other types of concrete are often reported in units of kg/m^3. High-density materials feel heavy for their size.

Water has a density of about 1.0 g/cm^3, while many construction aggregates have density values between 2.5 and 3.0 g/cm^3. Steel has a density of about 7.8 g/cm^3. Table 4-5 lists density values for various materials, including common construction aggregates.

The term specific gravity is often used interchangeably with density, but has a different meaning. Specific gravity is defined as the ratio of the mass of a material to the mass of an equal volume of water. It can also be defined as the ratio of the density of a material to the density of water. Because water has a density of 1.0 g/cm^3 at room temperature, the values of density in units of g/cm^3 are equal to specific gravity values. However, these terms should be used carefully. It is especially important to make sure that the units are included when reporting density values. Because specific gravity values are ratios of two numbers with the same units, specific gravity is dimensionless. The typical density of granite is 2.65 g/cm^3, while the typical value for the specific gravity of granite is 2.65.

Table 4-4. Standard sizes of fine aggregates for bituminous paving mixtures as adapted from AASHTO M 29.

AASHTO Grading No.	% Passing (mass %) for Sieve Size:							
	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.60 mm	0.30 mm	0.150 mm	0.075 mm
1	100	95 to 100	70 to 100	40 to 80	20 to 65	7 to 40	2 to 20	0 to 10
2	---	100	75 to 100	50 to 74	28 to 52	8 to 30	0 to 12	0 to 5
3	---	100	95 to 100	85 to 100	65 to 90	30 to 60	5 to 25	0 to 5
4	100	80 to 100	65 to 100	40 to 80	20 to 65	7 to 40	2 to 20	0 to 10
5	100	80 to 100	65 to 100	40 to 80	20 to 65	7 to 46	2 to 30	---

Table 4-5. Typical density values for various materials, including common construction aggregates.

Material	Density, g/cm^3
Aluminum	2.71
Asphalt binder	1.03
Basalt	2.86
Concrete	2.40
Diabase	2.96
Dolomite	2.70
Glass	2.50
Gneiss	2.74
Granite	2.65
Iron	7.87
Lead	11.35
Limestone	2.66
Marble	2.63
Nylon	1.14
Portland cement	3.15
Quartz	2.65
Quartzite	2.69
Sandstone	2.54
Shale	1.85-2.50
Steel	7.80
Teflon	2.17
Wood	0.50

Aggregate specific gravity is determined using different techniques for coarse and fine aggregate. Obtaining accurate specific gravity values for aggregates prior to performing an HMA mix design is essential, and engineers and technicians responsible for mix designs must develop proper laboratory techniques for these procedures. For coarse aggregate, specific gravity is determined using the weight-in-water method. In this procedure, coarse aggregate is weighed in air, and then in water, in a mesh basket suspended from a balance. A sketch of a weight-in-water apparatus is shown in Figure 4-5. The bulk specific gravity of a sample of coarse aggregate is calculated using the following equation:

$$Bulk\ Sp.\ Gr. = A/(B-C) \qquad (4\text{-}5)$$

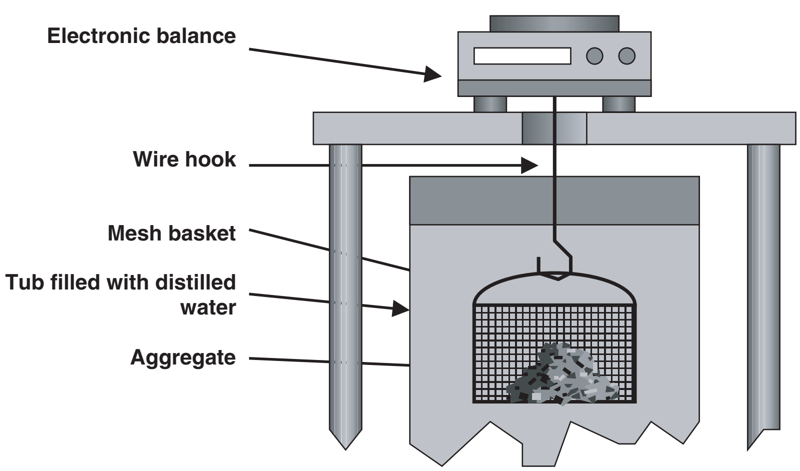


Figure 4-5. Weight-in-water apparatus for determining specific gravity of coarse aggregate.

where

$Bulk\ Sp.\ Gr.$ = bulk specific gravity
A = weight of dry aggregate in air, g
B = weight of saturated, surface-dry aggregate in air, g
C = weight of saturated aggregate in water, g

The term saturated, surface-dry (SSD) as used in Equation 4-5 means that the aggregate specimen has been saturated with water (usually by overnight soaking in water) and then quickly dried with a clean cloth or towel just until the aggregate surfaces are no longer visibly wet and shiny. Aggregate particles contain a large number of microscopic voids or pores, both within the aggregate particle and on and near the aggregate surface. Voids that connect to the surface become filled with water when saturated, while internal voids do not. When a saturated aggregate particle is quickly dried with a cloth or towel, water remains in the permeable voids, while the surface is dry, as shown in Figure 4-6. The SSD condition is important in HMA mix design because this approximately represents the condition of the aggregate in HMA mixtures, except that the permeable voids are now filled with asphalt binder instead of water. Sometimes, two other specific gravity values are reported: bulk specific gravity, SSD basis, and apparent specific gravity. These two specific gravity values are sometimes used in the design and analysis of portland cement concrete mixtures, but are rarely used in HMA technology, where the term "specific gravity," when applied to aggregates, should be taken to mean bulk specific gravity.

Because fine aggregate would fall through the wire mesh basket used in determining weight-in-water, this approach cannot be used in specific gravity measurements. Instead, the pycnometer method is used. This technique requires the use of a pycnometer, which is simply a container that can be repeatedly filled with the same—or nearly the same—volume of water. Usually, a volumetric flask is used for performing fine aggregate specific gravity measurements. The flask is partially filled with distilled water and then about 500 g of saturated sand is placed in the flask. The flask is rolled and gently shaken to remove all air bubbles and then more water is added until the flask is filled just to the calibration mark. The flask is weighed, and the contents carefully poured into a metal pan which is then dried in an oven. The bulk specific gravity of the fine aggregate is then calculated using the following equation:

$$Bulk\ Sp.\,Gr. = A/(B+W-C) \tag{4-6}$$

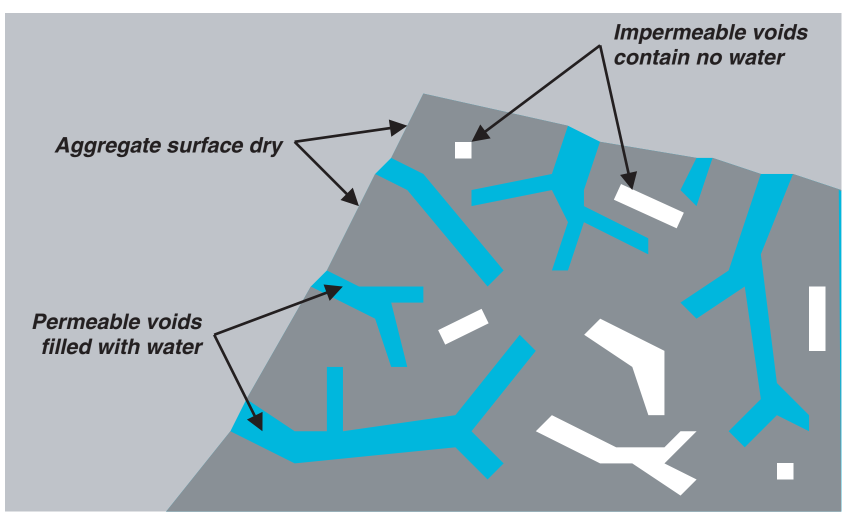


Figure 4-6. Sketch showing saturated, surface-dry condition in an aggregate particle.

where

Bulk Sp. Gr. = bulk specific gravity
A = weight of oven-dry aggregate in air, g
B = weight of pycnometer filled with water to calibration mark, g
W = weight of saturated, surface-dry aggregate in air, g
C = weight of pycnometer with aggregate and filled with water to calibration mark, g

Figure 4-7 is a diagram of a volumetric flask as used in determining the specific gravity of fine aggregate.

Just as is done for coarse aggregate, part of the specific gravity test for fine aggregate involves weighing the aggregate in air in the SSD condition. This is much more difficult for fine aggregate than for coarse aggregate—the surfaces of fine aggregate particles cannot be quickly dried with a cloth or towel because the particles are too fine. Traditionally, the cone test has been used to determine if a fine aggregate is in the SSD condition. In this procedure, saturated fine aggregate is dried using a blow dryer. Every few minutes, a small conical metal mold is filled with fine aggregate and tamped down. The mold is then removed; when the sand first slumps when the mold is removed, it is assumed to be in the SSD condition. Unfortunately, the cone test is not very accurate. Engineers have been working on developing alternative procedures, but at this time, none are yet being implemented.

For both coarse aggregate and fine aggregate, absorption is an important property. It is calculated using the following equation:

$$Absorption = (A - B)/B \times 100\% \qquad (4\text{-}7)$$

where

Absorption = water absorption in weight, %
A = weight of saturated, surface-dry aggregate in air, g
B = weight of dry (air-dry or oven-dry) aggregate in air, g

The absorption calculated using Equation 4-7 is the amount of water absorbed into the permeable voids of the aggregate. When aggregate is mixed with asphalt binder to produce HMA, asphalt will be absorbed into the permeable voids in the same way that water is. However, the amount of asphalt binder absorbed by the aggregate will in general not be as great as the water

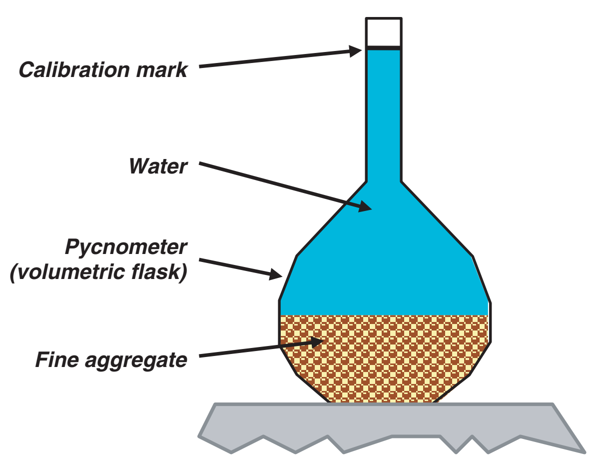


Figure 4-7. Pycnometer method of determining fine aggregate specific gravity.

absorption. In HMA mix design it is often assumed that asphalt absorption will be one-half of the water absorption. It is important to account for this absorption, since asphalt absorbed into the permeable voids of the aggregate will not be available to fill the voids between the aggregate particles. For this reason, the asphalt binder content for HMA mixtures made using aggregates having high absorption values will tend to be significantly higher than those made using aggregates with lower absorption values.

Aggregate Specification Properties

Angular and rough-textured aggregates are desirable within HMA to resist permanent deformation and fatigue cracking. Very angular and rough-textured aggregates provide better interlock between the aggregate particles which helps prevent plastic deformation (rutting) within HMA layers. Angular and rough-textured aggregates also help improve the strength of HMA mixtures, which can help prevent fatigue cracking. Angular aggregates with good surface texture also improve the frictional properties of pavement layers, an important safety consideration in the design of HMA for pavements.

The presence of flat or elongated particles within HMA is undesirable because these particles tend to break down during production and construction. Aggregates that break during production and construction will reduce the durability of the HMA layer, leading to raveling, pop-outs, and potholes.

Another aggregate characteristic related to performance is cleanliness and the presence of deleterious materials. Cleanliness is a term used to characterize the coatings on some aggregate particles. These coatings are often very fine clay-like materials and can affect the adhesion between the asphalt binder and aggregate particles leading to an increased potential for moisture damage. Deleterious materials are particles in an aggregate stockpile that are weak, prone to freeze-thaw damage or damage through repeated wetting and drying, or that otherwise can cause a pavement to deteriorate. Some examples of deleterious materials are clay lumps, friable particles, shale, coal, free mica, and vegetation. These types of materials are not as strong as mineral aggregates and break down during the life of a pavement layer. When this happens, pop-outs and potholes can occur.

Aggregate toughness and abrasion resistance have also been shown to be related to pavement performance. Aggregate particles that are tough and resistant to abrasion will not break down during the construction process, which helps ensure that an HMA mix can be properly constructed, placed, and compacted. Tough, abrasion-resistant aggregates also tend to produce a mix that is resistant to pop-outs and raveling. Because aggregate pop-outs and broken aggregate particles near the pavement surface make it easier for water to flow into a pavement, tough and abrasion-resistant aggregates help improve the moisture resistance of HMA pavements. Aggregates with poor abrasion resistance can also polish under the action of traffic. This can cause the pavement surface to lose skid resistance, especially when wet.

Another aggregate property that is closely related to toughness and abrasion resistance is durability and soundness. Freeze-thaw cycles and alternate periods of wetting and drying in a pavement can weaken poor-quality aggregates, causing pop-outs and raveling. Aggregates that possess good durability and soundness will resist the actions of wet-dry and freeze-thaw cycles during the life of the pavement.

Superpave Consensus and Source Aggregate Properties

During the development of the Superpave mix design system for dense-graded HMA, aggregate requirements were specified based on the experiences of a group of experts. Properties that were identified as important within HMA included the angularity of coarse and fine aggregates, aggregate shape, cleanliness of the aggregates, toughness, soundness, and the proportion of dust within the mixture. After some discussion, these experts reached an agreement, or *consensus,* that four aggregate properties were most important to HMA performance and should be specified as part of the Superpave system. A test method and specification limits were identified for each of these consensus properties. The four Superpave consensus aggregate properties are coarse aggregate angularity (CAA), fine aggregate angularity (FAA), clay content, and flat and elongated particles.

The expert panel identified several other aggregate properties as important to HMA pavement performance, but could not reach agreement on the specification limits. These aggregate properties are toughness (Los Angeles Abrasion test), soundness (Sodium or Magnesium Sulfate Soundness test), and deleterious materials. Test values for these properties vary significantly across the United States and Canada, depending on the type of aggregates locally available. The panel of experts therefore labeled these aggregate properties as "source aggregate properties" and recommended that specification values for these properties be developed by individual highway agencies.

The group of experts developed the aggregate requirements for HMA without the benefit of a formalized research program. Since the early 1990s, when the group of experts met to develop the aggregate requirements for the Superpave mix design system, a significant amount of work has been conducted to evaluate various aggregate tests and their relationship with pavement performance. This chapter provides aggregate requirements for the design of dense-graded HMA. These requirements build on both the experiences of the group of experts and research that has been conducted since the Superpave mix design system was developed. Because the Superpave consensus aggregate properties are now firmly grounded in both experience and research, rather than simply the consensus of an expert panel, the term "consensus properties" is no longer accurate. For this reason, the term "primary aggregate specification properties" is used herein to describe these four critical characteristics. The term source aggregate properties is still appropriate for the other aggregate tests, since specification values for these are still to be determined by individual agencies.

Specification limits for the various primary aggregate specification properties are not uniform for all HMA mixtures. Instead—as in the Superpave system—the specification requirements for these test values are based on the expected amount of traffic over a 20-year pavement life, the position of the layer being designed within the pavement structure, or both. Traffic is characterized as equivalent single-axle loads (ESALs) and more stringent specification limits are provided for pavements that will be subjected to higher traffic loads. Pavement layers that will encounter lower traffic volumes or are within the lower portion of the pavement structure have less stringent requirements.

The primary aggregate specification properties are designed to evaluate four critical characteristics for aggregates used in HMA mixtures. These four characteristics are coarse aggregate angularity, fine aggregate angularity, coarse aggregate particle shape, and cleanliness. Just as in the Superpave system, requirements for these characteristics are intended to be enforced on the aggregate blend and not on individual stockpiles. The following sections summarize the test methods and specification limits for the four primary aggregate specification properties.

Coarse Aggregate Fractured Faces

Several research studies have shown that increasing the number of particles in an aggregate blend that have been mechanically crushed increases resistance to permanent deformation. The test recommended in this manual is the same as that used in the Superpave system—a simple "crush count." The term coarse aggregate fractured faces (CAFF) is used instead of coarse

Table 4-6. Coarse aggregate fractured faces requirements.

Design ESALs (million)	Percentage of Particles with at Least One/Two Fractured Faces, for Depth of Pavement Layer[A], mm	
	0 to 100	Below 100
< 0.30	55 / ---	--- / ---
0.3 to < 3	75 / ---	50 / ---
3 to < 10	85 / 80	60 / ---
10 to < 30	95 / 90	80 / 75
30 or more	98 / 98[B]	98/ 98[B]

[A]Depth of pavement layer is measured from pavement surface to top of pavement layer within the pavement containing the given mixture.
[B]The CAFF requirement for design traffic levels of 30 million ESALs or more may be reduced to 95/95 if experience with local conditions and materials indicate that this would provide HMA mixtures with adequate rut resistance under very heavy traffic.

aggregate angularity because "fractured faces" is simpler and clearer, since it is the common term used for this type of test. The procedure is described in ASTM D 5821, Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate. Aggregate particles larger than 4.75 mm are visually examined to determine the percentage of particles that has at least one fractured face, and the percentage that has at least two. A CAFF value of 76/53, for example, means that 76% of the particles in a coarse aggregate have at least one fractured face, and 53% have at least two fractured faces. Table 4-6 outlines the required minimum values for CAFF as a function of traffic level and depth within the pavement structure. Note that the values given in Table 4-6 are slightly different from the values currently specified within the Superpave system; for the highest traffic level, the values for all mixtures are 98/98, whereas in the Superpave system the required values for coarse aggregate angularity are 100/100. A footnote allows further reduction of the CAFF requirement for this traffic level to 95/95 if local experience suggests that the resulting HMA will have adequate rut resistance under very heavy traffic. These slightly lower requirements for CAFF mean that high-quality crushed gravel can be used in HMA for high traffic applications. In the Superpave system, because of the very high requirements for CAFF at the highest traffic level, only crushed stone could be used for these applications. Experience over the past 5 to 10 years suggests that high-quality crushed gravels will usually perform quite well in properly designed HMA mixtures, even under extremely high traffic levels. Furthermore, the mix design system described in this manual includes performance testing for HMA mixtures designed for traffic levels of 10 million ESALs and greater. This performance testing provides additional assurance that HMA mixtures will have adequate rut resistance. The slightly lower values for CAFF recommended here should not be used unless performance testing is included as part of the mix design process.

Fine Aggregate Angularity

The angularity of the fine aggregate fraction is as important as the angularity of the coarse aggregate fraction to the performance of dense-graded HMA. In combination, the coarse and fine aggregates provide strength to HMA, which helps minimize the potential for permanent deformation. AASHTO T 304, Method A, Uncompacted Void Content of Fine Aggregate, is used to measure fine aggregate angularity. A graded sample of fine aggregate (passing the 2.36-mm sieve) is placed within a specially made funnel which allows the aggregate particles to freely drop into a cylinder of known volume (Figure 4-8). Using the combined bulk specific gravity of the fine aggregate blend, the percent voids between the aggregate particles is determined. Results from the fine aggregate angularity test represent this percent of uncompacted voids in the fine aggregate; higher values of uncompacted voids indicate greater angularity of the fine aggregate. Requirements

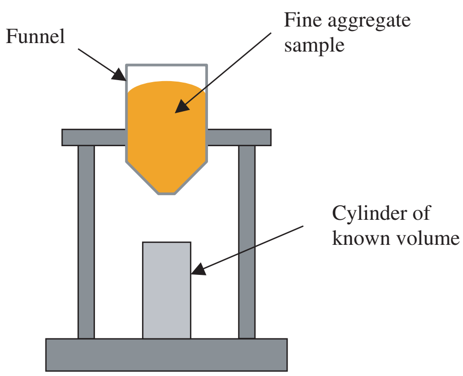


Figure 4-8. Fine aggregate angularity test.

for fine aggregate angularity are given in Table 4-7. The requirements in this table are nearly identical to those given in the Superpave system. The only exception is that in Table 4-7 where the minimum FAA value is 45, it can be lowered to 43 if experience with local conditions and materials suggests that this will produce mixtures with adequate rut resistance.

Flat and Elongated Particles

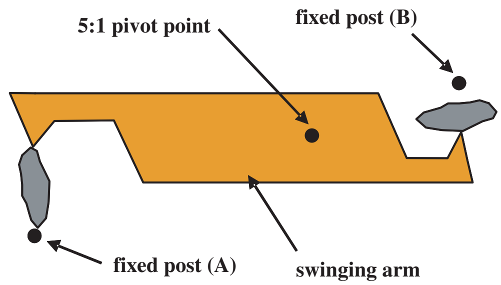
The percentage of flat and elongated particles in a coarse aggregate is determined using procedures described in ASTM D 4791, Flat Particles, Elongated Particles, or Flat and Elongated Particles. As in the Superpave system, this manual recommends a maximum value of 10% for flat and elongated particles exceeding a 5:1 ratio. To conduct this test, aggregate particles are measured with a proportional caliper (Figure 4-9) using a specified ratio of 5:1. The larger caliper opening is set to the particle length; if the width of the particle can fit within the smaller opening, it is considered flat and elongated. Coarse aggregates that fail this requirement are rare. Some state agencies use slightly different versions of this test—using different limits, or specifying a maximum value for flat *or* elongated particles for coarse aggregates. Technicians should check the applicable specifications to make sure they are using the proper test and limits when evaluating aggregates for an HMA mix design.

Table 4-7. Fine aggregate angularity requirements.

Design ESALs (million)	Depth of Pavement Layer from Surface[A], mm	
	0 to 100	Below 100
< 0.30	---[B]	---
0.3 to < 3	40	---
3 to < 10	45[C]	40
10 to < 30	45[C]	40
30 or more	45[C]	45[C]

Criteria are presented as percent air voids in loosely compacted fine aggregate.

[A]Depth of pavement layer is measured from pavement surface to top of pavement layer within the pavement containing the given mixture.
[B]Although there is no FAA requirement for design traffic levels below 0.30 million ESALS, consideration should be given to requiring a minimum uncompacted void content of 40 % for 4.75-mm nominal maximum aggregate size mixes.
[C]The FAA requirement of 45 may be reduced to 43 if experience with local conditions and materials indicate that this would produce HMA mixtures with adequate rut resistance under the given design traffic level.



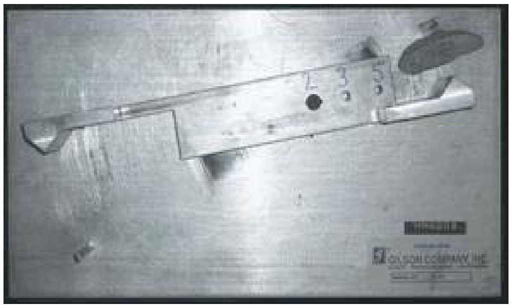


Figure 4-9. Flat or elongated test.

Requirements for flat and elongated particles are not based on the traffic level or the anticipated depth within the pavement structure. Flat and elongated particles are considered to be detrimental within an HMA mixture during production and construction, irrespective of traffic loadings or depth within the pavement; therefore, a single maximum percentage of flat and elongated particles is required. Table 4-8 presents the requirements for flat and elongated particles.

Clay Content

The presence of dust or clay coatings on aggregates can prevent the asphalt binder from properly coating the aggregates within an HMA. This can lead to water penetrating the asphalt binder film and, therefore, stripping of the asphalt binder from the aggregate. The Sand Equivalent test (AASHTO T 176, Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test) is used to evaluate the cleanliness of aggregates to identify when harmful clay-sized particles exist in an aggregate blend. The procedure is conducted on the aggregate fraction of the blend that passes the 4.75-mm sieve. If hydrated lime is used in the mixture, it should not be included in the fine aggregate used during the sand equivalent test. The aggregate sample is placed within a graduated, transparent cylinder that is filled with a mixture of water and flocculating agent. The combination of aggregate, water, and flocculating agent is then agitated for 45±5 seconds. After agitation, the combination is allowed to sit at room temperature for 20 minutes. After the 20 minutes, the heights of the sand particles and the sand plus clay particles are measured (Figure 4-10). The sand equivalent value is then calculated as the ratio of the height of the sand to the height of sand plus clay, expressed as a percentage.

High sand equivalent values are desirable, since this indicates that the aggregate is relatively free of dust and clay particles. Therefore, minimum values for sand equivalency are specified. These minimum values do not change with depth within the pavement, but do vary somewhat with design traffic level. Table 4-9 summarizes the requirements for the sand equivalent test.

Table 4-8. Criteria for flat and elongated particles.

Design ESALs (million)	Maximum Percentage of Flat and Elongated Particles at 5:1
< 0.30	---
0.3 to < 3	10
3 to < 10	10
10 to < 30	10
30 or more	10

Criteria are presented as percent flat and elongated particles by mass.

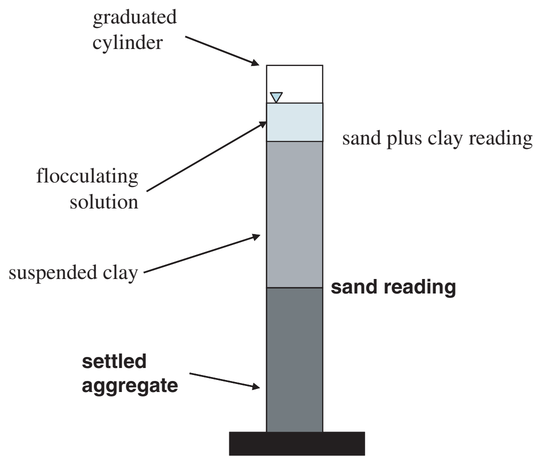


Figure 4-10. Sand equivalent test.

Source Aggregate Properties

Some aggregate properties were identified by the expert group as important, but about which a consensus could not be reached on specification limits. These aggregate properties were called "Source Properties." Test methods were recommended; however, development of specification limits was left to local agencies that had experience with area materials. These properties are generally used during source approval and, therefore, requirements are not applied to the aggregate blend as with the then consensus properties. Source properties deemed important include

- Toughness,
- Soundness, and
- Deleterious materials.

Toughness

The term toughness is used to describe the ability of an aggregate to withstand the abrasion and degradation that occurs during handling, production, construction, and in-service use. Toughness is measured using the Los Angeles Abrasion Test, described in AASHTO T 96, Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.

In performing the Los Angeles Abrasion test, a graded sample of aggregate is placed in a large steel drum (Figure 4-11). Six to twelve steel charges (depending on gradation of the aggregate stockpile) are placed within the drum in addition to the aggregate sample. The drum is then rotated which subjects the aggregates to impact and abrasion by the steel balls. Results from the test are reported as a percent loss, which is the mass percentage of aggregate lost during the test

Table 4-9. Clay content requirements.

Design ESALs (million)	Minimum Sand Equivalency Value
< 0.30	40
0.3 to < 3	40
3 to < 10	45
10 to < 30	45
30 or more	50

Criteria are presented as Sand Equivalent Value.

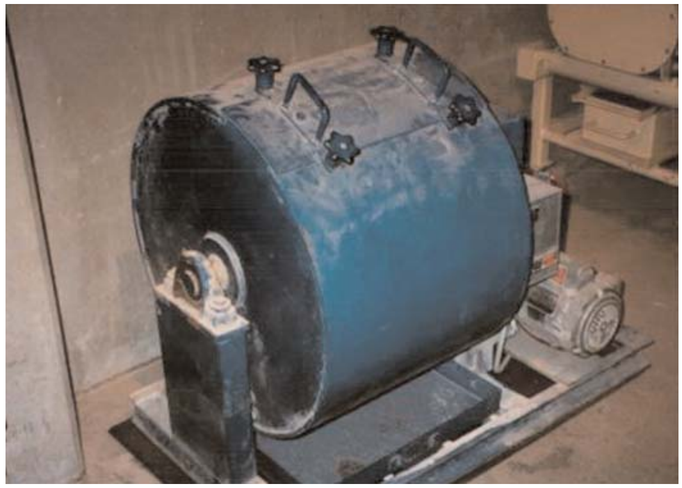


Figure 4-11. Los Angeles abrasion drum.

due to degradation and abrasion. Low Los Angeles Abrasion loss values are desirable, since this indicates that an aggregate is tough and resistant to abrasion. Typical values for Los Angeles Abrasion loss are listed in Table 4-10.

Soundness

Soundness is used to describe the ability of an aggregate to withstand the effects of weathering. To evaluate the soundness of aggregates, AASHTO T 104, Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate, is used. As stated in the title of the test method, either sodium sulfate or magnesium sulfate is used to subject an aggregate sample to the effects of freezing and thawing. This test method can be used to evaluate the soundness of both coarse and fine aggregates.

To perform the test, an aggregate sample is washed and dried to a constant mass and then separated into specified size fractions. The test is performed by alternately exposing an aggregate sample to repeated immersions in the prescribed sulfate solution followed by oven drying. During the period of immersion, the sulfate solution is absorbed into the permeable voids of the aggregates and rehydrates creating forces that simulate the expansive forces of water freezing. During the drying phase, the sulfate solution precipitates similar to the action of thawing. One immersion and drying is considered a soundness cycle. Typically, five soundness cycles are specified by agencies. Results from the soundness testing are the percent loss of material after the five cycles.

Low values of soundness loss are desirable since this suggests that an aggregate is not susceptible to weathering. Soundness test results obtained using sodium sulfate and magnesium sulfate solutions are not interchangeable, since the expansive forces generated by these salt solutions are

Table 4-10. Typical values for Los Angeles abrasion test.

Aggregate Mineralogy	Typical Los Angeles Abrasion Loss Values, %
Basalt	10 to 20
Dolomite	15 to 30
Gneiss	30 to 60
Granite	25 to 50
Limestone	20 to 30
Quartzite	20 to 35

different. Generally, use of magnesium sulfate solution will result in slightly higher loss values than use of sodium sulfate solution. As such, typical specification limits are a maximum of 10% loss when sodium sulfate is used and a maximum of 15% when magnesium sulfate is used, though specification limits can vary by agency.

Deleterious Materials

Deleterious materials are those materials within an aggregate stockpile that are weak, reactive, or unsound. Examples of materials that can be considered deleterious include clay lumps, friable particles, shale, coal, free mica, and vegetation. The test method for evaluating deleterious materials is AASHTO T 112, Clay Lumps and Friable Particles in Aggregate. In this test, fractions of aggregates are wet sieved over prescribed sieves. The mass percentage of material lost as a result of the wet sieving is reported as the percent of clay lumps and friable particles. High mass percentages of clay lumps and friable particles are detrimental to an HMA mixture; therefore, maximum values are generally specified. A wide range of permissible percentages of clay lumps and friable particles are specified by different agencies.

Bibliography

AASHTO Standards

M 29, Fine Aggregate for Bituminous Paving Mixtures
M 43, Standard Specification for Sizes of Aggregate for Road and Bridge Construction
M 323, Superpave Volumetric Mix Design
R 35, Superpave Volumetric Design for Hot-Mix Asphalt
T 2, Sampling of Aggregates
T 11, Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
T 19M/T 19, Bulk Density ("Unit Weight") and Voids in Aggregate
T 27, Sieve Analysis of Fine and Coarse Aggregate
T 30, Mechanical Analysis of Extracted Aggregates
T 84, Specific Gravity and Absorption of Fine Aggregate
T 85, Specific Gravity and Absorption of Coarse Aggregate
T 96, Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
T 104, Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
T 112, Clay Lumps and Friable Particles in Aggregate
T 176, Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test
T 248, Reducing Samples of Aggregate to Testing Size
T 304, Uncompacted Void Content of Fine Aggregate

Other Standards

ASTM D 4791, Flat Particles, Elongated Particles, or Flat and Elongated Particles
ASTM D 5821, Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate

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CHAPTER 5

Mixture Volumetric Composition

Understanding the terminology used to describe the volumetric composition of asphalt concrete and the ability to perform a volumetric analysis are two of the most important skills that engineers and technicians must master in order to develop effective asphalt concrete mix designs. Most of the specifications for HMA and related materials used throughout the United States and Canada are expressed, at least in part, in terms of volumetric composition. This chapter describes the basic terminology used in volumetric analysis, including such important concepts as air void content, voids in the mineral aggregate, and apparent film thickness. Besides basic definitions of such terms, an effort has been made to briefly describe how each of the main mix composition factors affect pavement performance. Because accurate bulk and maximum theoretical specific gravity data are essential to performing volumetric analysis, these tests are described in some detail in this chapter. Equations are presented for performing a complete volumetric analysis, and a detailed example problem is given at the end of the chapter. Because most engineers and technicians use spreadsheets and similar tools to perform the calculations involved in volumetric analysis of HMA mixtures, detailed reading of the equations and example problems will not be necessary for many readers of this manual; however, the equations and example problem are presented to (1) make the chapter complete and (2) for engineers and advanced technicians who wish to have a thorough understanding of volumetric analysis or who are interested in developing their own customized spreadsheets for performing volumetric analysis.

Composition Factors

Asphalt concrete primarily consists of three different components or phases: aggregate, asphalt binder, and air. Materials like concrete, which consist of particles held together by a cement of any type, are called composites. Some asphalt concrete mixtures contain small amounts of other additives, such as cellulose fibers, mineral fibers, ground rubber, and polymers. Although such additives may affect workability and performance significantly, these additives almost always represent a very small percentage of the overall volume and mass of the asphalt concrete. Engineers and technicians should remember the three major components of asphalt concrete—aggregate, asphalt, and air. These three components are the key to understanding volumetric analysis.

The composition of asphalt concrete can be described in terms of either weight or volume. The asphalt binder content of a mixture, for example, is often given in terms of percent of total mix weight, whereas air void content is always given as a percent of total volume—it must be given this way, since the mass of air voids in an asphalt concrete specimen is essentially zero. Although composition of asphalt concrete mixtures can be given in terms of weight, traditionally the most common and most important method of describing and analyzing asphalt concrete composition is by volume. This is what is meant by the term "volumetrics" or "volumetric analysis" of asphalt concrete—characterizing the composition of an asphalt concrete mixture by relative proportions

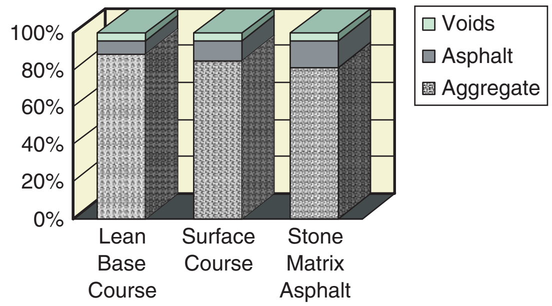


Figure 5-1. Typical composition by volume of different HMA mix types.

by volume of aggregate, asphalt, and air voids. Although this may sound like a simple task, it can become quite complicated when absorption of asphalt by the aggregate must be accounted for or when only incomplete information on a mixture is available. It is essential that engineers and technicians responsible for developing asphalt concrete mix designs or performing quality control operations have a thorough understanding of volumetric analysis.

Typical asphalt concrete mixtures, as designed in the laboratory, contain about 84 to 90% aggregate, 6 to 12% asphalt binder, and about 4% air voids by volume. Figure 5-1 illustrates the typical composition of three different types of HMA—in all cases, almost all of the mixture volume is made up of aggregate. Asphalt concrete is mostly composed of aggregate. If the volume percentage of two of these components is known, the other can be determined by subtraction. For example, if we know that a mixture is to be designed with 4% air voids and 10% asphalt binder by volume, we can calculate the amount of aggregate required as 100 – (4 + 10) = 86% by volume. It is important to remember this when specifying the composition of asphalt concrete. It would be impossible, in this example, to specify a mixture with 4% air voids, 10% minimum binder content and 87% minimum aggregate content by volume—a mixture with a minimum of 87% aggregate by volume could have no more that 13% total volume of air voids and asphalt binder. This is only one example of the complex interrelationships involved in characterizing and specifying the volumetric composition of asphalt concrete. More examples are given throughout this chapter. But first, we will discuss in more detail different volumetric factors used in volumetric analysis.

Air Voids

"Air voids," when applied to asphalt concrete, means small pockets of air that exist within the asphalt binder and between aggregate particles. Air void content does not include pockets of air within individual aggregate particles, or air contained in microscopic surface voids or capillaries on the surface of the aggregate. Figure 5-2 shows the different ways in which air exists in asphalt concrete mixtures. Designing and maintaining the proper air void content in HMA and other mix types is important for several reasons. When air void contents are too high, the pavement may be too permeable to air and water, resulting in significant moisture damage and rapid age hardening. When air void contents are too low, the asphalt binder content may be too high, resulting in a mixture prone to rutting and shoving.

When discussing the air void content of asphalt concrete mixtures, it is first necessary to specify what type of specimen or sample we are testing or analyzing. We can measure the air void content of asphalt concrete in the following types of specimens:

- Specimens compacted in the laboratory when developing a mix design
- Specimens compacted in the laboratory from material produced at the plant as part of quality assurance testing

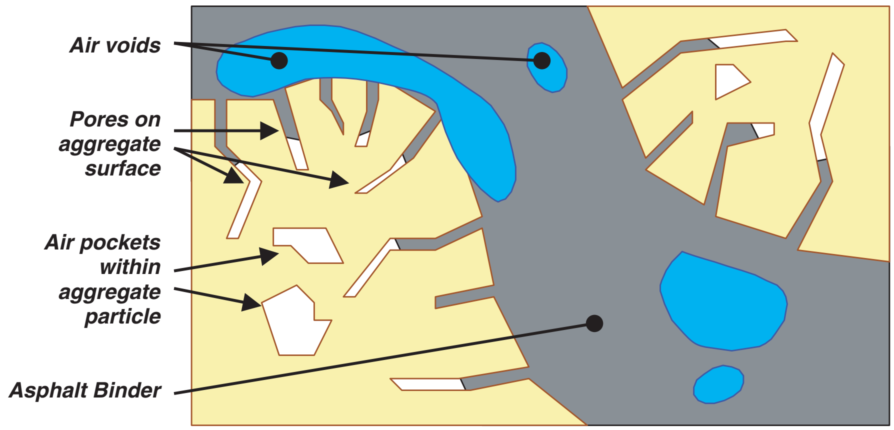


Figure 5-2. Air in asphalt concrete. *Air can exist in pores on the aggregate surface, pockets within aggregate particles, or voids within the asphalt binder or between the binder and aggregate particles. Only the last type of air is included in the air void content of asphalt concrete mixtures.*

- Specimens taken from the roadway as cores immediately after construction as part of quality assurance testing
- Specimens taken from a wheel path of a roadway as cores after several years or more of traffic loading
- Specimens taken from between the wheel paths of a roadway as cores after several years or more of traffic loading

The typical air void content of these different types of specimens will usually vary substantially, so it is important when discussing or specifying the air void content of asphalt concrete to make sure it is clear what type of specimen has been tested or is required to be tested.

In the Superpave method of designing HMA, the air void content in laboratory mix designs is held constant at 4.0%. Some agencies have, however, expanded the allowable range for air void content to as wide as 3.0 to 5.0%, which was the design range used in the Marshall mix design method. The in-place air void content of HMA pavement is often assumed to be about 7%, but recent research (as reported in *NCHRP Report 573*) suggests that immediately after construction the air void content of HMA pavements typically ranges from about 6 to 11%, with a median value between 8 and 9%. Cores taken from a newly constructed pavement will generally have air void contents in this range. However, once the pavement is opened to traffic, the repeated loading as trucks pass over the pavement will tend to further compact the material in the wheel paths of the pavement. Many engineers assume that the air void content in the wheel paths of an asphalt concrete pavement should, within a few years, reach about the same value as was used in the laboratory mix design, but, as documented in *NCHRP Report 573,* this is not always the case. Cores taken outside the wheel path will undergo little or no change in air void content with time, since their location is not subjected to traffic loading. Therefore, if there is a question about how well an asphalt concrete pavement has been constructed several years after construction, this can only be determined from cores taken from between the wheel paths. Samples of HMA loose mix produced at the plant and compacted in the laboratory (commonly termed plant-produced/laboratory-compacted) should have air void contents close to the design value. However, plant-produced HMA often contains more mineral dust than was used in the mix design, which tends to reduce the air void content of laboratory-compacted specimens. This reduction can be avoided by making sure that aggregate gradations used in preparing mix designs accurately reflect what is typically produced at the plant.

Determining air void content is one of the main purposes of volumetric analysis. Unfortunately, there is no simple direct way to determine the air void content of an asphalt concrete specimen. Air void content is determined by comparing the specific gravity (or density) of a compacted specimen with the maximum theoretical density of the mixture used to make that specimen. For example, if the compacted density of an asphalt concrete specimen is 95.3% of the theoretical maximum specific gravity, the air void content is $100 - 95.3 = 4.7\%$. Mixture-specific gravity and the volumetric analysis of asphalt concrete are discussed in more detail later in this chapter.

Binder Content

Binder content is one of the most important characteristics of asphalt concrete. Use of the proper amount of binder is essential to good performance in asphalt concrete mixtures. Too little binder will result in a dry stiff mix that is difficult to place and compact and will be prone to fatigue cracking and other durability problems. Too much binder will be uneconomical, since asphalt binder is, by far, the most expensive component of the mixture and will make the mixture prone to rutting and shoving. Typical asphalt binder contents range from 3.0% or less (for lean base course mixtures) to over 6.0% (for surface course mixtures and rich bottom layers), which are designed for exceptional durability and fatigue resistance.

As mentioned earlier in this chapter, asphalt binder content is most often stated and specified as a percentage of total mix weight. A ton of hot mix that is 5.2% asphalt binder will contain $2{,}000 \times 0.052 = 104$ pounds of binder. However, there are two problems with this way of stating asphalt content. First and most important, it is the asphalt content by volume and not by weight that dictates performance, and asphalt content by total mix weight is a function of both asphalt content by volume and aggregate specific gravity. Consider two asphalt concrete mixtures, both with an air void content of 4.0% and an asphalt binder content of 12.0% by volume. One mixture is made with a limestone aggregate with a specific gravity of 2.50, and the other with a dense diabase having a specific gravity of 3.20. If no asphalt binder is absorbed by the aggregate (as we will discuss below, not a very good assumption), the asphalt content of the limestone mix will be 5.35% by total mix weight, while the asphalt content of the diabase mixture will be 4.23% by total mix weight. The difference in asphalt binder content by weight is over 1.0%, even though these mixtures contain identical asphalt contents by volume! To avoid this problem, many agencies now specify minimum binder content by weight as a function of aggregate specific gravity.

The second problem with stating asphalt binder content by total mix weight is that most aggregates tend to absorb asphalt binder. Asphalt binder absorbed by an aggregate is tightly held in microscopic pores on the aggregate surface and does not significantly contribute to the durability of a mixture. The amount of absorption varies widely, depending on aggregate type. Dense igneous rocks, such as diabase and basalt, might only absorb a few tenths of a percent of asphalt binder from a mixture, while porous sandstones and slags might absorb from 1 to as much as 4% of the asphalt binder from a mixture. The term "effective binder content," abbreviated as Vbe, is used to describe the amount of asphalt binder in a mixture not including that absorbed by the aggregate (see Figure 5-3). For example, if the total asphalt content of a mixture is 5.3% by weight, and the aggregate absorbs 0.4% binder by total mix weight, the effective binder content of this mixture is $5.3 - 0.4 = 4.9\%$. If a mixture is to be designed to have 11.0% asphalt content by volume, not including the 1.0% of the binder absorbed by the aggregate, then the total asphalt binder content must be $11.0 + 1.0 = 12.0\%$ by volume.

Theoretically, the most effective way of characterizing and specifying asphalt binder content is effective binder content by volume, since this avoids the two problems described above. However, effective asphalt content by volume can only be determined through volumetric analysis and cannot be determined with a high degree of precision. Asphalt concrete plants are almost always designed to control asphalt binder content as a percentage of total mix weight. For these reasons,

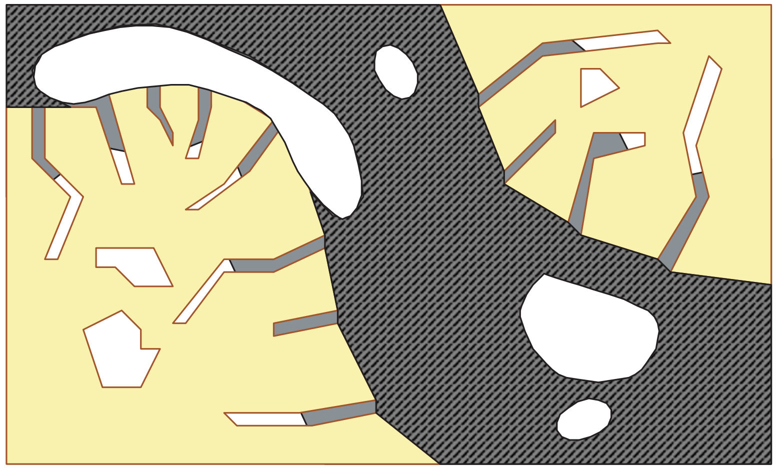


Figure 5-3. Effective asphalt content. *Effective asphalt includes asphalt binder not absorbed by the aggregate—in this sketch, the cross-hatched gray area represents effective binder.*

many agencies specify minimum total binder content by weight and give tables, graphs, or formulas adjusting this minimum according to the aggregate specific gravity. Unfortunately, there is no simple way to account for aggregate absorption when specifying asphalt binder content, since absorption varies so widely among aggregate types, and even substantially within aggregate from a given quarry. As discussed below, one way to specify effective binder content by volume is to control both air void content and voids in the mineral aggregate at the same time.

Voids in the Mineral Aggregate

Voids in the mineral aggregate (VMA) refers to the space between aggregate particles in an asphalt concrete mixture (see Figure 5-4). VMA is also often used to characterize loose aggregate,

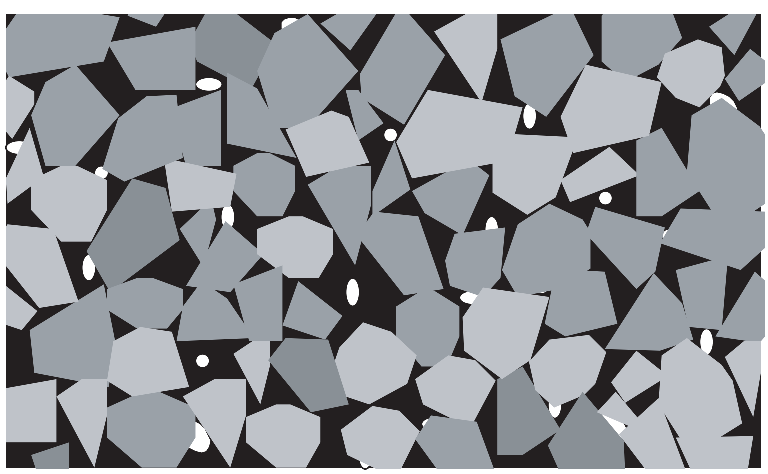


Figure 5-4. Voids in mineral aggregate. *Dark and light gray areas represent aggregate particles, black area asphalt binder and white areas air voids; voids in mineral aggregate (VMA) is composed of asphalt binder and air voids—black and white areas.*

but its meaning is exactly the same—the volume percentage of space between aggregate particles. VMA is numerically equal to the air void content plus the effective binder content by volume. Therefore, establishing a single design air void content (such as the 4.0% used in Superpave mixtures) and then controlling VMA is the same as controlling effective binder content. For example, a Superpave 12.5-mm mixture designed at 4.0% air voids with 14.0% minimum VMA has a minimum effective binder content of 14.0 – 4.0 = 10.0% by volume.

Some engineers and agencies have proposed that VMA should be defined as total binder content plus air void content, both by volume. The only advantage to using this definition is that it makes aggregates with high absorption appear to be more economical than they are, since defining VMA in this way includes the large volume of binder absorbed by such aggregate. Defining VMA in this way is a non-standard practice and can result in mixtures that are deficient in asphalt binder, difficult to place and compact, and prone to fatigue cracking.

Voids Filled with Asphalt

Voids filled with asphalt (VFA) is the percentage of VMA filled with asphalt binder—the balance is air voids. An asphalt concrete mixture with a VMA of 16% and an effective asphalt content of 12% has a VFA value of $(12/16) \times 100\% = 75\%$. In this case, 25% of the VMA is air voids. Consider a second mixture, with 15% VMA and 5% air voids. The effective asphalt content is then $15 - 5 = 10\%$, and the VFA is $(10/15) \times 100\% = 67\%$. VFA is calculated by dividing the effective binder content by the VMA and multiplying by 100%.

In designing asphalt concrete mixtures, VFA is closely related to both VMA and Vbe. This is because with the design air void content constant at about 4.0%, as VMA increases, Vbe increases and VFA also increases. Therefore, in most cases VFA should be thought of as simply an indicator of mix richness, like VMA or Vbe. If design voids are fixed or allowed to vary only over a narrow range, there is little point in simultaneously controlling VMA, Vbe, and/or VFA. In fact, simultaneous control of strongly interrelated volumetric factors can lead to confusion and conflict during the mix design process and during construction. Figure 5-5 shows the relationships between air void content, VFA, and VMA (Figure 5-5a) and Vbe (Figure 5-5b).

It is not entirely clear what aspects of performance are related to VFA that are not also strongly related to other volumetric factors, especially Vbe. Some engineers have proposed that fatigue resistance increases with increasing VFA. However, VFA and Vbe are strongly related. Recent research strongly suggests that Vbe is a somewhat better overall indicator of fatigue resistance in asphalt concrete mixtures. Therefore, in order to control or evaluate fatigue resistance, engineers and technicians should either use Vbe, or VMA at a constant design air void content. There is then little need to independently specify VFA. Relationships between mixture composition and performance are discussed in more detail in Chapter 6 of this manual.

Apparent Film Thickness

"Film thickness," when applied to asphalt concrete mixtures, refers to the average thickness of binder coating aggregate particles in the mixture. Some engineers and researchers have proposed that this is an important characteristic related to several aspects of pavement performance—mixtures with low film thickness will be brittle and prone to durability problems, while mixtures with high film thickness will have too much asphalt and may be prone to rutting and shoving. Film thickness is, however, a controversial concept among paving engineers. Many engineers strongly oppose the use of this term, since there are, in fact, no real films of asphalt binder within an asphalt concrete mixture; the asphalt binder exists as a single homogenous phase binding the aggregate particles together. Critics of film thickness point out that there is no way to physically

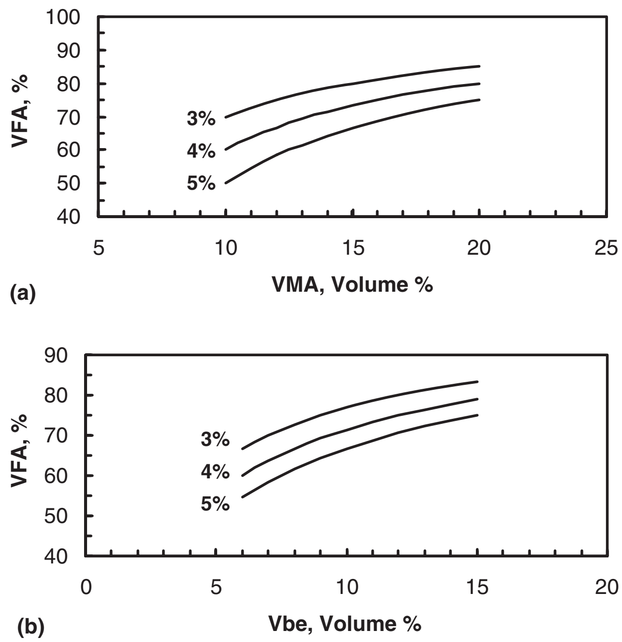


Figure 5-5. Relationship between air void content (labels to the left of curves), VFA and (a) VMA, and (b) Vbe.

separate an aggregate particle with an intact asphalt binder film from a compacted asphalt concrete mixture. However, this criticism does not address the issue of whether or not calculated film thickness values are correlated to pavement performance. For this reason, it is suggested that engineers and technicians use the term "apparent film thickness" rather than "film thickness," thereby avoiding the main objection of many critics that such films do not physically exist. Figure 5-6 illustrates the concept of apparent film thickness.

Recent research (documented in *NCHRP Report 567*) strongly suggests that there are reasons why apparent film thickness should relate to performance, especially rut resistance. Rut resistance of asphalt concrete mixtures increases as VMA decreases and aggregate fineness increases. Because binder content decreases with decreasing VMA, this means that rut resistance should increase

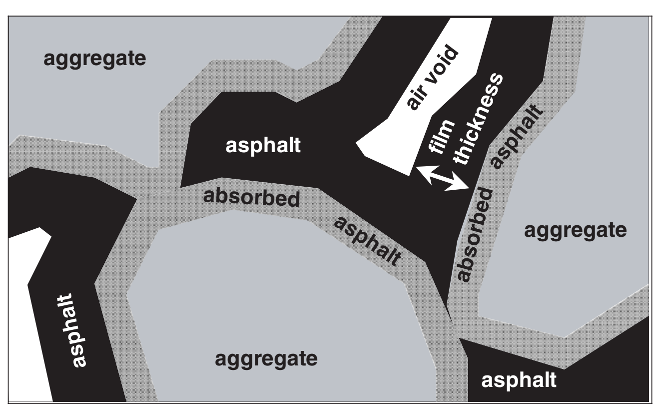


Figure 5-6. Concept of apparent film thickness.
Calculation of apparent film thickness should not include asphalt binder absorbed by the aggregate.

with decreasing apparent film thickness, as has been suggested by many engineers and observed in various field studies. However, the relationship between rut resistance and film thickness is not an inherent mechanism of asphalt films, but instead appears to be an indirect but useful relationship. *NCHRP Report 567* suggests that HMA mixtures with apparent film thickness values greater than 9 to 10 microns can be prone to excessive rutting.

Although physically distinct films of asphalt binder cannot be separated from a compacted specimen of asphalt concrete, such films do have physical meaning in loose uncompacted HMA. Furthermore, these films serve an important purpose—they lubricate the aggregate particles and allow the HMA to be placed and compacted properly. Apparent film thickness values that are too low are indicative of mixtures that are difficult to place and compact, which in turn can cause segregation, high in-place air void content, and a pavement that is permeable and prone to raveling and surface cracking. Again, this relationship is not a direct one; the lack of durability is not the result of low film thickness, but of segregation and/or poor compaction brought about by the poor workability resulting from low film thickness in the loose hot mix. Unfortunately, research linking apparent film thickness to the workability of HMA has not yet been performed. Different ranges for minimum apparent film thickness have been suggested since this concept was first proposed. It is suggested that mixtures with values below about 6 to 7 microns may be difficult to place and compact properly.

The discussion above suggests that apparent film thickness can be a useful tool for designing and analyzing asphalt concrete mixtures. Film thickness values in the range of 7 to 9 microns appear to provide the best compromise between workability and rut resistance. Values below 6 microns or above 10 microns should be avoided. Although apparent film thickness is a potentially useful concept, the relationships between apparent film thickness and performance are, at best, indirect. Furthermore, equally good means of controlling mixture composition as it relates to performance are available that do not involve the use of apparent film thickness. Controlling VMA, design air void content, and aggregate fines is essentially equivalent to controlling film thickness. Agencies that choose to specify film thickness for asphalt concrete mixtures should take special care to ensure that there are no unintended conflicts with any simultaneous requirements for VMA, design air void content, or aggregate gradation.

Mixture-Specific Gravity

Specific gravity has the same meaning when applied to asphalt concrete mixtures as it does when applied to aggregates and other materials—the ratio of the density of a material to the density of water at 25°C and at standard air pressure. Because the density of water under these conditions is 1.000 gm/cm^3, specific gravity is interchangeable with density in these circumstances. However, specific gravity, since it is a ratio, is dimensionless. A mixture with a bulk specific gravity of 1.352 has a bulk density of 1.352 g/cm^3. Some agencies use density in units of kg/m^3, which will be 1,000 × the specific gravity. The specific gravity of the mixture in the previous example could be given as 1.352 g/cm^3 or 1,352 kg/m^3. There is no link between specific gravity and performance. However, measuring and calculating specific gravity and understanding how specific gravity is used in volumetric analysis is critical to developing asphalt concrete mix designs and analyzing paving mixtures.

Bulk Specific Gravity

The bulk specific gravity of a mixture refers to the specific gravity of a specimen of compacted mixture, including the volume of air voids within the mixture. It is equivalent to the mass of a given specimen in grams, divided by its total volume in cubic centimeters. The bulk specific gravity

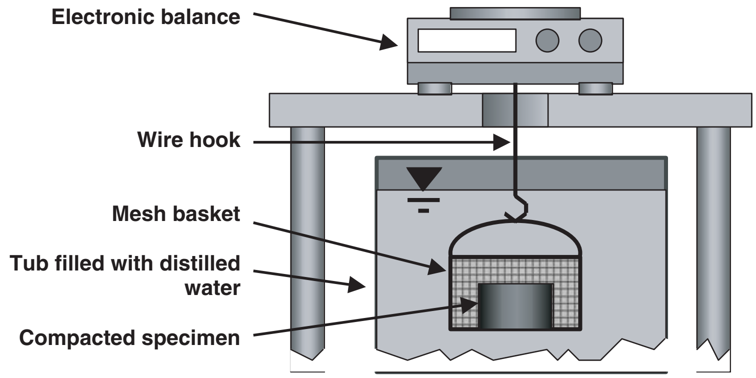


Figure 5-7. Determining weight in water of compacted specimen of HMA.

of an asphalt concrete mixture can be determined using either laboratory-compacted specimens or cores or slabs cut from a pavement.

The standard procedure for determining the bulk specific gravity of compacted asphalt concrete involves weighing the specimen in air and in water. Two slightly different laboratory techniques are used, depending on the absorption of the specimen. For low absorption (less than 2.0%), saturated surface-dry specimens are used (AASHTO T 166). For specimens having high absorption, paraffin-coated specimens should be used in the specific gravity determination (AASHTO T 275). The following formula is used for calculating bulk specific gravity of a saturated surface-dry specimen:

$$G_{mb} = \frac{A}{B - C} \tag{5-1}$$

where

G_{mb} = bulk specific gravity of compacted specimen
A = mass of the dry specimen in air, g
B = mass of the saturated surface-dry specimen in air, g, and
C = mass of the specimen in water, g

As a general rule, if the water absorption of a compacted HMA mixture is above 2.0%, the bulk specific gravity should be determined using paraffin-coated specimens. The calculation of specific gravity in this case is more complicated because the mass and volume of the paraffin film must be accounted for; details can be found in AASHTO T 275. Figure 5-7 is a sketch of a typical weight-in-water determination. Some agencies determine mixture bulk specific gravity using a pycnometer method, which involves calibrating a container and then determining the weight of the container with the compacted specimen and filled with water. Although theoretically this method will provide results equivalent to the weight-in-water method, no AASHTO standard exists for this procedure. Furthermore, use of multiple procedures for determining specific gravity of aggregates and HMA mixtures should be discouraged, since this can only increase the variability in the test results and in the subsequent volumetric analyses.

Theoretical Maximum Specific Gravity

The theoretical maximum specific gravity of an asphalt concrete mixture is the specific gravity of the mixture at zero air void content. It is one of the most difficult tests performed in paving materials laboratories and also one of the most important. Like bulk specific gravity, theoretical maximum specific gravity in and of itself does not affect the performance of a paving mixture. However, it is essential in determining volumetric factors that are good indicators of performance, such as air void content and VMA.

Maximum specific gravity is determined by measuring the specific gravity of the loose paving mixture, after removing all of the air entrapped in the mixture by subjecting the mixture to a partial vacuum (vacuum saturation). The loose mix is prepared by gently heating the sample in an oven until it can be easily broken apart. The mixture is then removed from the oven and occasionally stirred while cooling, to make sure that it remains broken up as much as possible into separate particles of asphalt-coated aggregate. After determining the weight in air of the sample, it is placed in a tared, calibrated vacuum container. The container is then connected to a vacuum pump, and the pressure in the container gradually reduced to 30 mm Hg or less—about 4% of normal atmospheric pressure. This partial vacuum is maintained for 5 to 15 minutes, and the container is occasionally tapped or rolled to help release entrapped air from the loose mixture. The vacuum is then carefully released, the container topped off with water to the calibration mark, and the weight of the container, specimen, and water determined. The theoretical maximum specific gravity of the specimen is calculated using the following formula:

$$G_{mm} = \frac{A}{A + D - E} \tag{5-2}$$

where

G_{mm} = theoretical maximum specific gravity of loose mixture
A = mass of oven-dry specimen in air, g
D = mass of container filled with water at 25°C to calibration mark, g
E = mass of container with specimen filled with water at 25°C to calibration mark, g

Because of the importance of theoretical maximum specific gravity determinations, and because the measurements are difficult to perform with great precision, some additional comments concerning this procedure are warranted. In a well-run laboratory, every effort should be made to perform this procedure as much as possible in the same way every time it is run. Many laboratories use small sieve shakers to agitate the specimen while the vacuum is applied, because of the variability in hand rolling and tapping. Although the procedure allows for a time range of 5 to 15 minutes for vacuum saturation, a much narrower time range should be used. Initial tests with typical local materials and with the specific vacuum saturation equipment to be used in running the procedure will help determine the most effective time for applying the vacuum. Because this test involves pulling a vacuum on a container filled with water, care should be made to ensure that the pump is suitable for this use. Many vacuum pumps are quickly damaged by water vapor when water condenses within the interior of the pump, mixing with the vacuum oil and ruining its effectiveness. If such a pump is not available, a series of traps should be installed between the specimen and the pump to prevent water vapor from entering the pump. Because water will boil at 30 mm Hg at a temperature of 29°C (84°F), the area in which this test is performed and the water used in the procedure should be kept cool. It will be impossible to reach a vacuum of 30 mm Hg if the temperature of the water within the container is 84°F or higher. Laboratory personnel should also make certain that the pump used can quickly reach a partial vacuum of at least 30 mm Hg. Good-quality, accurately calibrated gages should be used to monitor the vacuum during the procedure. The complete procedure for performing the theoretical maximum specific gravity test can be found in AASHTO T 209. Figure 5-8 is a diagram of a sample of loose mix being vacuum saturated as part of this procedure.

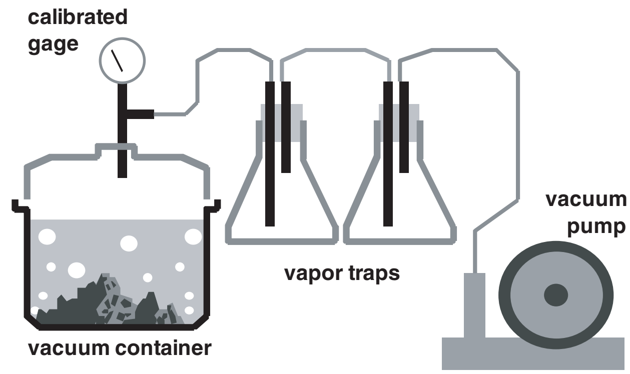


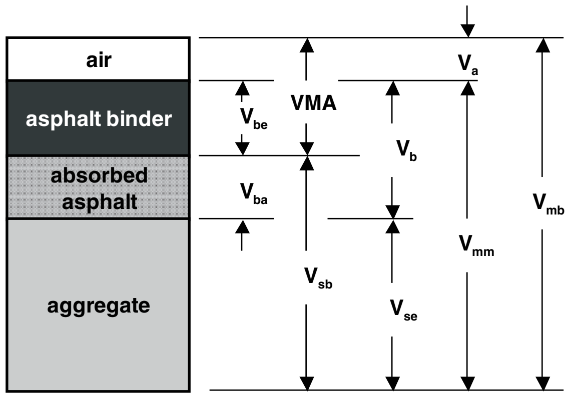
Figure 5-8. Example of apparatus for vacuum saturation of loose hot mix, part of the procedure for determining maximum specific gravity of asphalt concrete mixtures.

Volumetric Analysis

The previous sections give the reader the background needed to understand the primary factors involved in volumetric analysis: mixture bulk and theoretical maximum specific gravity; air void content; VMA; VFA; and effective binder content. This section describes the calculations involved in performing an actual volumetric analysis of asphalt concrete. It is similar but not identical to the discussion of volumetrics given in the Asphalt Institute publications *Superpave Mix Design* (SP-2) and *Mix Design Methods* (MS-2), both of which are good references for technicians and engineers responsible for HMA mix design and analysis. This section has been kept relatively short, since almost all laboratories currently have personal computers that can be loaded with software for performing volumetric analysis. The *Mix Design Tools* spreadsheet included with this manual includes software for performing volumetric analysis. The basic equations are presented here as background for interested engineers and senior technicians and for those interested in putting together their own spreadsheets for mix design and analysis. Figure 5-9 illustrates the definitions of variables used to define various volumes as used in volumetric analysis. The volume of permeable pores in the aggregate surface containing asphalt shows up in three different terms: the aggregate bulk volume (V_{sb}), the total asphalt volume (V_b), and the absorbed asphalt volume (V_{ba}). Also, in this manual the convention adopted for volume terms is that the capital letter V followed by a subscript denotes the absolute volume of a particular component, whereas V followed by capital letters denotes a percentage by volume. Thus, V_{ma} represents the absolute volume of voids in the mineral aggregate (in units of cm^3, for example), whereas VMA indicates the voids in the mineral aggregate as a volume percentage.

A set of variables similar to those given in Figure 5-9 can be defined for the mass terms used in volumetric analysis:

M_{be} = Mass of effective asphalt binder
M_{ba} = Mass of absorbed asphalt binder
M_s = Mass of aggregate, total
M_b = Mass of asphalt binder, total
M_{se} = Mass of aggregate, effective (excluding surface pores filled with asphalt)
M_a = Mass of air voids
M_{mb} = Mass of specimen, total



V_{be} = Volume of effective asphalt binder
VBE = Effective asphalt content, percent by volume
V_{ba} = Volume of absorbed asphalt binder
VBA = Absorbed asphalt binder, percent by total mix volume
V_{ma} = Volume of voids in mineral aggregate
VMA = Voids in mineral aggregate, percent by volume
V_{sb} = Volume of aggregate, bulk (including all permeable surface pores)
V_b = Volume of asphalt binder, total
VB = Total asphalt binder content, percent by volume
V_{se} = Volume of aggregate, effective (excluding surface pores filled with asphalt)
V_a = Volume of air voids
VA = Air void content, volume percent
V_{mm} = Volume of aggregate and asphalt
V_{mb} = Volume of specimen, total

Figure 5-9. Definition of volume terms used in volumetric analysis.

The set of variables for mass are slightly different than the ones for volume because the air voids and the permeable pores in the aggregate surface have no mass. For the same reason, there is no variable representing the mass of air voids or the mass of the asphalt binder plus air voids—which would of course be the same as the mass of the binder alone.

In addition to these variables representing various volumes and masses, two additional variables are used to represent percentages by weight [mass?] of asphalt binder and aggregate: P_b and P_s, respectively. These are normally both percentages by mass [weight?] of the total mixture weight.

Equations

Average Aggregate Specific Gravity. Because the aggregate used in producing asphalt concrete is almost always a blend of two or more aggregates, usually having different values for bulk specific gravity, volumetric calculations such as the ones described below must be done using an average bulk specific gravity for the aggregate blend. This average value can be calculated using the following equation:

$$G_{sb} = \frac{P_{s1/A} + P_{s2/A} + P_{s3/A} + \cdots}{\left(\frac{P_{s1/A}}{G_{sb1}}\right) + \left(\frac{P_{s2/A}}{G_{sb2}}\right) + \left(\frac{P_{s3/A}}{G_{sb3}}\right) + \cdots} \tag{5-3}$$

where

G_{sb} = overall bulk specific gravity for aggregate blend
$P_{s1/A}$ = volume % of aggregate 1 in aggregate blend
G_{sb1} = bulk specific gravity for aggregate 1

$P_{s2/A}$ = volume % of aggregate 2 in aggregate blend
G_{sb2} = bulk specific gravity for aggregate 2
$P_{s3/A}$ = volume % of aggregate 3 in aggregate blend
G_{sb3} = bulk specific gravity for aggregate 3

Air Void Content. Air void content is calculated from the mixture bulk and theoretical maximum specific gravity:

$$VA = 100\left[1 - \left(\frac{G_{mb}}{G_{mm}}\right)\right] \tag{5-4}$$

where

VA = Air void content, volume %
G_{mb} = Bulk specific gravity of compacted mixture
G_{mm} = Theoretical maximum specific gravity of loose mixture

Asphalt Binder Content. Asphalt binder content can be calculated in four different ways: total binder content by weight, effective binder content by weight, total binder content by volume, and effective binder content by volume. Total asphalt content by volume is calculated as the percentage of binder by total mix mass:

$$P_b = 100\left(\frac{M_b}{M_s + M_b}\right) \tag{5-5}$$

where

P_b = Total asphalt binder content, % by mix mass
M_b = Mass of binder in specimen
M_s = Mass of aggregate in specimen

Total asphalt binder content by volume can be calculated as a percentage of total mix volume using the following formula:

$$VB = \frac{P_b G_{mb}}{G_b} \tag{5-6}$$

where

VB = Total asphalt binder content, % by total mix volume
P_b = Total asphalt binder content, % by mix mass
G_{mb} = Bulk specific gravity of the mixture
G_b = Specific gravity of the asphalt binder

The absorbed asphalt binder content by volume is also calculated as a percentage of total mix volume:

$$VBA = G_{mb}\left[\left(\frac{P_b}{G_b}\right) + \left(\frac{P_s}{G_{sb}}\right) - \left(\frac{100}{G_{mm}}\right)\right] \tag{5-7}$$

where

VBA = Absorbed asphalt content, % by total mixture volume
G_{mb} = Bulk specific gravity of the mixture

P_b = Total asphalt binder content, % by mix mass
G_b = Specific gravity of the asphalt binder
P_s = Total aggregate content, % by mix mass
$= 100 - P_b$
G_{sb} = Average bulk specific gravity for the aggregate blend
G_{mm} = Maximum specific gravity of the mixture

The effective asphalt by volume is found by subtracting the absorbed asphalt content from the total asphalt content:

$$VBE = VB - VBA \tag{5-8}$$

where

VBE = Effective asphalt content, % by total mixture volume
VB = Total asphalt binder content, % by mixture volume
VBA = Absorbed asphalt content, % by total mixture volume

The effective and absorbed asphalt binder contents can also be calculated as percentages by weight, once the volume percentage has been calculated:

$$P_{be} = P_b\left(\frac{VBE}{VB}\right) \tag{5-9}$$

$$P_{ba} = P_b - P_{be} \tag{5-10}$$

where

P_{be} = Effective asphalt binder content, % by total mass
P_b = Asphalt binder content, % by total mass (see Equation 5-5)
VBE = Effective asphalt binder content, % by total mixture volume (see Equation 5-8)
VB = Asphalt binder content, % by total mixture volume (see Equation 5-6)
P_{ba} = Absorbed asphalt binder, % by total mixture mass

VMA is simply the sum of the air void content and the effective asphalt binder content by volume:

$$VMA = VA + VBE \tag{5-11}$$

where

VMA = Voids in the mineral aggregate, % by total mixture volume
VA = Air void content, % by total mixture volume (Equation 5-4)
VBE = Effective binder content, % by total mixture volume (Equation 5-8)

VFA is the effective binder content expressed as a percentage of the VMA:

$$VFA = 100\left(\frac{VBE}{VMA}\right) \tag{5-12}$$

where VFA is the voids filled with asphalt, as a volume percentage.

Apparent Film Thickness. Apparent film thickness can be calculated using the following formula:

$$AFT = \frac{1{,}000VBE}{S_s P_s G_{mb}} \tag{5-13}$$

where

AFT = Apparent film thickness, μm
VBE = Effective binder content, % by total mix volume (see Equation 5-8)
S_s = Aggregate specific surface, m³/kg
P_s = Aggregate content, % by total mix weight
= $100 - P_b$
G_{mb} = Mixture bulk specific gravity

Aggregate Specific Surface. The surface area of aggregate contained in a mixture, expressed as specific surface, is needed to calculate apparent film thickness. Specific surface values used in asphalt concrete mix design and analysis are not true specific surface values—they are effective specific surface values, in which some portion of the finest mineral dust is eliminated from the calculation. Unfortunately, aggregate specific surface as it applies to mix design technology cannot be precisely defined; traditional, highly empirical methods for calculating aggregate specific surface became thoroughly embedded in mix design practice, but were largely based on engineering judgment and experience and were never well documented. The methods presented here are taken from *NCHRP Report 567* and have been devised to provide values consistent with traditional aggregate specific surface values. A very easy and accurate method to estimate aggregate specific surface is to add the % passing the 0.30-, 0.15- and 0.075-mm sieves and divide by 5:

$$S_s \cong \frac{P_{0.30} + P_{0.15} + P_{0.075}}{5} \tag{5-14}$$

where

S_s = Aggregate specific surface, m²/kg
$P_{0.30}$ = % of aggregate passing 0.30-mm sieve
$P_{0.15}$ = % of aggregate passing 0.15-mm sieve
$P_{0.075}$ = % of aggregate passing 0.075-mm sieve

A more rigorous calculation requires calculation of the contribution of each size fraction to the total specific surface of the aggregate:

$$S_s = \left(\frac{1}{1{,}000G_{sb}}\right)\begin{bmatrix} 1.4(P_{50} - P_{37.5}) + 2.0(P_{37.5} - P_{25}) + 2.8(P_{25} - P_{19.5}) + 3.9(P_{19.5} - P_{12.5}) \\ + 5.5(P_{12.5} - P_{9.5}) + 8.9(P_{9.5} - P_{4.75}) + 17.9(P_{4.75} - P_{2.36}) \\ + 36.0(P_{2.36} - P_{1.18}) + 71.3(P_{1.18} - P_{0.60}) + 141(P_{0.60} - P_{0.30}) \\ + 283(P_{0.30} - P_{0.15}) + 566(P_{0.15} - P_{0.075}) + 1{,}600(P_{0.075}) \end{bmatrix} \tag{5-15}$$

In Equation 5-15, the Ps represent percent passing for the sieve size in mm represented by the subscript for each P. The calculation appears complicated, but simply involves multiplying the percent of material between each successive pair of sieves by a factor (1.4, 2.0, 2.8, etc.), summing the results, and then dividing by 1,000 times the aggregate bulk specific gravity. Equation 5-15 is quite tedious, but can be entered into a spreadsheet for use in routine calculations. However, given the empirical nature of aggregate specific surface area, it is not clear that there is any advantage in using Equation 5-15 compared to the much simpler Equation 5-14.

Example Problem 5-1. Volumetric Analysis of an HMA Mixture

An example problem in mixture volumetric analysis is given below. The data needed for the problem is first presented in Tables 5-1 through 5-3. Then, the calculations are shown in the typical order in which they would be performed. A table summarizing the results of the analysis is presented at the end of the example.

Table 5-1. Mixture composition for example problem.

Material	Specific Gravity	Percent by Mass in Aggregate Blend	Percent by Mass in Total Mix
12.5-mm limestone	2.621	28.0	26.6
12.5-mm sandstone	2.668	28.0	26.6
Manufactured sand	2.595	44.0	41.8
Asphalt binder	1.030	---	4.9

Table 5-2. Gradation data for example problem.

Sieve Size	% Passing
19 mm	100
12.5 mm	92
9.5 mm	82
4.75 mm	55
2.36 mm	32
1.18 mm	24
0.600 mm	18
0.300 mm	11
0.150 mm	9
0.075 mm	5.5

Table 5-3. Mixture bulk and maximum specific gravity data for example problem.

Measurement	Mass, g
Bulk Specific Gravity	
Dry weight in air	4,299.3
Saturated surface-dry weight in air	4,333.7
Weight in water	2,510.0
Maximum Specific Gravity	
Dry weight in air	4,295.0
Weight of container filled with water	7,823.1
Weight of container with specimen filled with water	10,365.5

(continued on next page)

Example Problem 5-1. (Continued)

Solution

Step 1. Determine the aggregate average bulk specific gravity using Equation 5-3:

$$G_{sb} = \frac{28 + 28 + 44}{\left(\frac{28}{2.621}\right) + \left(\frac{28}{2.668}\right) + \left(\frac{44}{2.595}\right)} = 2.622$$

Step 2. Determine the mixture bulk specific gravity using Equation 5-1:

$$G_{mb} = \frac{4{,}299.3}{4{,}333.7 - 2{,}510.0} = 2.357$$

Step 3. Determine the mixture maximum specific gravity using Equation 5-2:

$$G_{mm} = \frac{4{,}295.0}{4{,}295.0 + 7{,}823.1 - 10{,}365.5} = 2.451$$

Step 4. Calculate the air void content using Equation 5-4:

$$VA = 100\left[1 - \left(\frac{2.357}{2.451}\right)\right] = 3.8\%$$

Step 5. Calculate the total asphalt binder content by mix volume using Equation 5-6:

$$VB = \frac{4.9 \times 2.357}{1.03} = 11.2\%$$

Step 6. Calculate the absorbed asphalt binder content by mix volume using Equation 5-7:

$$VBA = 2.357\left[\left(\frac{4.9}{1.03}\right) + \left(\frac{95.1}{2.622}\right) - \left(\frac{100}{2.451}\right)\right] = 0.5\%$$

Step 7. Calculate the effective asphalt binder content by volume by subtracting the absorbed asphalt from the total asphalt content (Equation 5-8):

$$VBE = 11.2 - 0.5 = 10.7\%$$

Step 8. Calculate the effective and absorbed asphalt contents by total mix weight using Equations 5-9 and 5-10:

$$P_{be} = 4.9\left(\frac{10.7}{11.2}\right) = 4.7\%$$

$$P_{ba} = 4.9 - 4.7 = 0.2\%$$

Example Problem 5-1. (Continued)

Step 9. Calculate the VMA by adding the air void content and the effective asphalt binder content by volume (Equation 5-11):

$$VMA = 3.8 + 10.7 = 14.5\%$$

Step 10. Calculate the VFA using Equation 5-12:

$$VFA = 100\left(\frac{10.7}{14.5}\right) = 73.8\%$$

Step 11. Estimate the specific surface of aggregate using Equation 5-14:

$$S_s \cong \frac{11 + 9 + 5.5}{5} = 5.1\ \text{m}^2/\text{kg}$$

Step 12. Calculate the apparent film thickness using Equation 5-13:

$$AFT = \frac{1{,}000 \times 10.7}{5.1 \times 95.1 \times 2.357} = 9.4\ \mu\text{m}$$

Table 5-4 summarizes the results of the volumetric analysis example problem.

Table 5-4. Summary of volumetric analysis example problem.

Mixture Composition Factor	Value
Total asphalt binder content, % by mix weight	4.9
Absorbed asphalt binder, % by mix weight	0.5
Aggregate content, % by mix weight	95.1
Average aggregate bulk specific gravity	2.622
Mixture bulk specific gravity	2.357
Mixture maximum specific gravity	2.451
Air void content, % by total mix volume	3.8
Effective asphalt binder content, % by total mix volume	10.7
VMA, % by total mix volume	14.5
VFA, % by total mix volume	73.8
Aggregate specific surface, m^2/kg	5.1
Apparent film thickness, µm	9.4

Requirements for Asphalt Concrete Composition

Specific values for volumetric mix factors for different mix types are not presented here. Instead, they are given in the chapter covering the design of each type of material: Chapter 8, dense-graded HMA mixtures; Chapter 10, gap-graded HMA, and Chapter 11, open-graded friction course mixtures.

Bibliography

AASHTO Standards

R 35, Superpave Volumetric Design for Hot-Mix Asphalt (HMA)
T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens

T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
T 269, Percent Air Voids in Compacted Dense and Open Asphalt Mixtures
T 275, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens

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The Asphalt Institute (1997) *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types* (MS-2), 6th Ed., 141 pp.

The Asphalt Institute (2001) *Superpave Mix Design* (SP-2), 128 pp.

Christensen, D. W., and R. F. Bonaquist (2006) *NCHRP Report 567: Volumetric Requirements for Superpave Mix Design,* Final Report for NCHRP Projects 9-25 and 9-31, TRB, National Research Council, Washington, DC, 57 pp.

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CHAPTER 6

Evaluating the Performance of Asphalt Concrete Mixtures

This chapter presents an introduction to evaluating the performance of HMA mixtures in pavement systems. It provides mixture designers with a concise compilation of information relating HMA properties to the failure mechanisms discussed in Chapter 2:

- Rutting and permanent deformation
- Fatigue cracking
- Low-temperature cracking
- Moisture damage
- Durability

HMA performance is strongly influenced by the composition of the mixture and the in-place density. The effect of binder properties, aggregate properties, and mixture volumetric properties on performance, which serves as the basis for the mixture design criteria in Chapters 8 and 10, are discussed. The important role of field compaction in the performance of dense-graded and SMA mixtures cannot be over emphasized. Marginally designed mixtures may perform adequately when properly compacted, but even the best designed mixture will not perform adequately when poorly compacted.

In recent years there has been a growing interest in using performance testing and performance prediction models in HMA mixture design and acceptance. For higher traffic level designs, the three-level Superpave mixture design and analysis system developed during the Strategic Highway Research Program included a series of performance tests and models to evaluate the expected performance of an HMA mixture. Unfortunately due to complexity, high equipment costs, and the lack of field calibration for the models, the mixture analysis portion of this system was not fully implemented. Over the last 15 years, additional progress in performance testing has been made, and recently the Mechanistic-Empirical Pavement Design Guide (MEPDG) has been completed and made available to the profession. The MEPDG includes field-calibrated performance prediction models for rutting and cracking that can be used to predict the performance of an HMA mixture in a particular pavement section.

Although performance is directly addressed by the mixture design processes presented in this manual through the material selection and volumetric criteria used in design, some agencies and mixture designers may desire confirmation through performance testing and modeling. This chapter also discusses various performance tests that can be used to assess HMA mixture performance and presents an introduction to the MEPDG and how this software can be used to complement the mixture design process.

Mixture Composition and Performance

The mixture design criteria for dense-graded and SMA mixtures presented in Chapters 8 and 10 place several requirements on mixture composition that are related to performance. These requirements include

- Binder grade
- Aggregate angularity
- Nominal maximum aggregate size
- Mineral filler content
- Design air void content
- Design compaction level, and
- Design voids in the mineral aggregate (VMA)

These requirements are included in the design procedure to ensure that the resulting mixtures will exhibit adequate performance when properly compacted in the field. It is important to emphasize that field compaction is a critical factor affecting every aspect of pavement performance and that the design procedures presented in this manual for dense-graded and SMA mixtures assume that the HMA mixture will receive proper compaction during construction. Table 6-1 summarizes the effect of mixture composition on pavement performance.

In Table 6-1, an upward arrow indicates that a particular performance indicator improves with an increase in the compositional factor. A downward arrow indicates that the performance indicator deteriorates with an increase in the compositional factor. The relationship between binder stiffness and fatigue resistance depends on the pavement structure; for thin pavement structures, increasing binder stiffness will decrease fatigue resistance, while for thick pavement

Table 6-1. Effect of mixture composition on performance.

Typical Effects of Increasing Given Factor within Normal Specification Limits While Other Factors Are Held Constant within Normal Specification Limits

Component	Factor	Resistance to Rutting and Permanent Deformation	Resistance to Fatigue Cracking	Resistance to Low Temperature Cracking	Resistance to Moisture Damage	Durability/ Resistance to Penetration by Water and Air
Asphalt Binder	Increasing High Temperature Binder Grade	↑↑↑				
	Increasing Low Temperature Binder Grade			↓↓↓		
	Increasing Intermediate Temperature Binder Stiffness		↑↓			
Aggregates	Increasing Aggregate Angularity	↑↑				
	Increasing Proportion of Flat and Elongated Particles					
	Increasing Nominal Maximum Aggregate Size		↓	↓		↓
	Increasing Mineral Filler Content and/or Dust/Binder Ratio	↑↑				↑
	Increasing Clay Content				↓	
Volumetric Properties	Increasing Design Compaction Level	↑↑	↑↑			
	Increasing Design Air Void Content	↑↑				
	Increasing Design VMA and/or Design Binder Content	↓↓	↑	↓		
	Increasing Field Air Void Content	↓↓	↓↓	↓	↓↓	↓↓↓

structures the reverse is true—that is why there are two arrows going in different direction for this entry in Table 6-1. The relative importance of each of the factors is indicated by the number of arrows shown in the table. The information presented in Table 6-1 and the more detailed discussion on the relationships among binder properties, aggregate properties, mixture composition, and pavement performance that is given later in this chapter are based on several sources, most importantly *NCHRP Report 539: Aggregate Properties and the Performance of Superpave-Designed Hot Mix Asphalt* and *NCHRP Report 567: Volumetric Requirements for Superpave Mix Design.*

Binder Characteristics and Performance

Performance Grading System

As discussed in more detail in Chapter 3, the Performance Grading system for asphalt binders, AASHTO M 320, controls the properties of asphalt binders that are related to pavement performance. This grading system includes requirements for the properties of the binder at high, intermediate, and low pavement temperatures to address rutting, fatigue cracking, and low-temperature cracking in pavements. Two conditioning procedures are used to simulate the effects of binder aging during construction and the service life of the pavement.

In the Performance Grading system, asphalt binders are specified by two numbers, for example PG 64-22. The first number, 64 in this example, is called the high temperature grade. It is the pavement temperature in degrees Centigrade up to which the binder provides adequate stiffness to resist excessive rutting for a properly designed HMA mixture exposed to a moderate volume of fast-moving highway traffic. The second number, –22 in this example, is called the low temperature grade. It is the pavement temperature in degrees Centigrade down to which the binder provides adequate flexibility to resist low-temperature cracking. In the Performance Grading system, the critical value of the performance-related property remains the same, but the temperature where the binder provides the minimum or maximum property changes. A PG 70-22 must meet the minimum high temperature properties at 70°C compared to 64°C for a PG 64-22. Similarly a PG 64-28 must meet the low temperature properties at –28°C compared to –22°C for a PG 64-22. Both of these binders provide a wider performance range than the PG 64-22. The performance-related properties used in the Performance Grading system are summarized in Table 6-2. For low temperature cracking, two criteria corresponding to Table 1 and Table 2 of AASHTO M 320 are given. Table 1 of AASHTO M 320 is based on low temperatures properties from the bending beam rheometer, while Table 2 of AASHTO M 320 is based on the computed critical cracking temperature of the binder. This computation uses data from both the bending beam rheometer and the direct tension test. Most agencies use the Table 1 criteria. The test methods used in the Performance Grading system were described in Chapter 3.

Rutting and Permanent Deformation

The high temperature grade of the asphalt binder is one of several important factors affecting the rutting resistance of HMA. For a given pavement and HMA mixture, resistance to permanent

Table 6-2. Performance-related properties and criteria used in the performance grading system, AASHTO M 320.

Distress Mode	Performance Related Binder Property	Criteria
Rutting	$G^*/\sin\delta$	Minimum of 1.00 kPa for unaged binder at 10 rad/sec Minimum of 2.20 kPa for RTFOT aged binder at 10 rad/sec
Low Temperature Cracking, Table 1	Creep stiffness, S	Maximum of 300 kPa for PAV aged binder at 60 s.
	m-value	Minimum of 0.300 for PAV aged binder at 60 s
Low Temperature Cracking, Table 2	Critical cracking temperature	Equal to or lower than specified low temperature grade.
Fatigue Cracking	$G^*\sin\delta$	Maximum of 5,000 kPa for PAV aged binder at 10 rad/sec

deformation increases as the high temperature performance grade increases. Additionally, recent research has shown that for the same high temperature performance grade, the rutting resistance of HMA made with polymer-modified binders is significantly improved over that for neat (that is, undiluted or not mixed with other substances) asphalt binders.

Rutting in asphalt pavements increases with increasing traffic volume and decreasing traffic speed. To counteract these effects, the high temperature binder grade must be increased or "bumped" for pavements exposed to high traffic levels (trucks) and slow-moving traffic. Table 6-3 presents the recommended high temperature grade changes included in the mixture design procedures presented in Chapters 8, 10, and 11.

When using grade bumping for different traffic levels, it is important to avoid binders that are excessively stiff at the intermediate temperature for the binder when selected on the basis of environmental conditions alone. As the high temperature performance grade increases, the temperature where the binder is required to meet the maximum value of $G^*\sin\delta$ also increases. This may result in the binder being too stiff for the intermediate temperature conditions under which the pavement is expected to perform. For example, when bumping two grades from a PG 64-22 to a PG 76-22, the temperature where the intermediate stiffness is normally tested increases 6°C, from 25°C to 31°C. The appropriate intermediate temperature based on environmental conditions is 25°C, not 31°C, and the binder should be expected to have a value of $G^*\sin\delta$ that is less than or equal to 5,000 kPa at 25°C. The Performance Grading system does not ensure that bumped binders will meet the intermediate temperature conditions required for the base binder. Agencies usually place additional language in the HMA specification to require intermediate temperature testing at the temperature obtained on the basis of environmental conditions alone.

Fatigue Cracking

The intermediate temperature stiffness of the asphalt binder is one of several factors affecting fatigue cracking in pavements. Top-down fatigue cracking was identified as an important form of distress in thick asphalt pavements and overlays of portland cement concrete pavements in the mid 1990s. Although this form of distress is not yet completely understood, it appears that binder stiffness is at least a contributing factor. Surface courses made with binders that become excessively stiff due to rapid age hardening are more susceptible to top-down cracking. The Performance Grading system places a maximum limit on the stiffness of the binder after simulated long-term aging. Although there is much debate over this requirement and its relationship to traditional

Table 6-3. Recommended high temperature performance grade changes to account for traffic volume and speed.

Design Traffic (MESALs)	*Grade Adjustment for Average Vehicle Speed in kph (mph):* Very Slow	Slow	Fast
	< 25 (< 15)	25 to < 70 (15 to < 45)	≥ 70 (≥ 45)
< 0.3	---	---	---
0.3 to < 3	12	6	---
3 to < 10	18*	13	6
10 to < 30	22*	16*	10
≥ 30	---	21*	15*

** Consider use of polymer-modified binder. If a polymer-modified binder is used, high temperature grade may be reduced one grade (6 °C) provided rut resistance is verified using suitable performance testing.*

bottom-up fatigue cracking, the requirement serves to limit age hardening of the binder and the potential for top-down cracking.

Low-Temperature Cracking

The low-temperature cracking performance of asphalt pavements is almost completely controlled by the environmental conditions and the low temperature properties of the asphalt binder. Binder grade selection is, therefore, the most critical HMA design factor affecting the low temperature performance of asphalt pavements. Since transverse thermal cracks cannot be repaired and quickly reflect through future overlays, it is critical that binders be selected to have a high reliability against thermal cracking. Reliability as applied to binder grade selection was discussed in detail in Chapter 3.

Durability

Excessive age hardening of the binder during the service life of the pavement is a contributing factor to several pavement distresses including raveling, top-down fatigue cracking, thermal cracking, and moisture damage. The primary factors affecting age hardening are the environment, the permeability of the HMA, and the characteristics of the binder. Age hardening is most severe in high-temperature climates. Age hardening also occurs more rapidly in pavements that are more permeable; therefore, it is critical to ensure that a high level of in-place density is achieved to minimize the potential for interconnected air voids in the HMA. The Performance Grading system includes tests on the binder after simulated long-term aging to control the age-hardening characteristics of the binder.

Moisture Damage

Some combinations of asphalt binder and aggregate exhibit greater potential for moisture damage than others. For the same aggregate type, resistance to moisture damage improves marginally with the use of a stiffer binder, particularly those modified with polymers.

Aggregate Characteristics and Performance

Excellent-performing pavements have been constructed using a wide variety of aggregate types. Several characteristics of aggregates that are related to pavement performance are controlled in the mixture design procedures presented in Chapters 8, 10, and 11.

Aggregate angularity and mineral filler content are important aggregate characteristics affecting the rutting resistance of HMA. Resistance to rutting and permanent deformation improves with increasing aggregate angularity and increasing mineral filler content—although excessive mineral filler content will tend to produce a mixture that is very stiff and sticky and difficult to compact. Rutting resistance also improves as the nominal maximum aggregate size (NMAS) of the HMA increases because the design VMA decreases with increasing NMAS and the design VMA has a major influence on the rutting resistance of HMA.

Aggregate characteristics are also important factors affecting the durability of HMA and its resistance to moisture damage. Aggregates that are flat or elongated tend to break during compaction, leaving uncoated surfaces, which decrease durability and increase the potential for moisture damage. Clay particles disrupt the adhesion of the asphalt binder to the aggregates making the HMA less durable and more susceptible to moisture damage. Durability improves with decreasing NMAS because the design VMA increases with decreasing NMAS and, as discussed in the next section, increasing the design VMA increases the effective binder content of the mixture, which improves durability. Finally, increasing the mineral filler content of the HMA decreases permeability for the same in-place air void content (again, understanding that there are practical limitations to how much mineral filler can be used in HMA mixtures). Binder age hardening and water infiltration are reduced in mixtures with lower permeability, leading to improved durability and greater resistance to moisture damage.

Volumetric Properties and Performance

The volumetric properties of HMA have a major influence on the performance of HMA. Volumetric properties affect an HMA mixture's resistance to rutting and fatigue cracking. They also affect the durability of the mixture and its resistance to moisture damage.

Rutting and Permanent Deformation

Several volumetric factors affect the resistance of HMA to rutting and permanent deformation. Although individually these factors are less important than high-temperature binder grade, aggregate angularity, and mineral filler content, the volumetric factor effects are additive and, if these act together in the same way, the results can be significant.

Rutting resistance tends to improve with decreasing design VMA and in-place air void content. As discussed in Chapter 5, VMA is the volume of air and asphalt binder in the mixture. These are the components of HMA that deform easily upon loading; therefore, rutting resistance improves as VMA and in-place air void content decrease. Rutting resistance also improves with increasing design compaction level. The resistance of the aggregate structure to deformation improves as the number of gyrations used in the design increases. Finally, the rutting resistance improves as the design air void content increases. At first, this effect might seem counter-intuitive, but by increasing the design air void level while maintaining the in-place air void content constant, the energy of compaction required to construct the pavement is increased significantly. Conversely, decreasing design air void content under constant in-place air void content decreases the energy required for field compaction. Even though decreasing VMA and increasing design air void content will, in general, improve rut resistance, as discussed below, VMA values that are too low and design air void values that are too high will often produce mixtures with poor durability. This is why there are both minimum and maximum values for VMA and air void content. These requirements are discussed in detail in Chapter 8 of this manual.

Fatigue Cracking

Several volumetric factors also affect the resistance of HMA to fatigue cracking. The most important of these is the in-place air void content of the pavement. The fatigue life of typical HMA pavements decreases with increasing in-place air void content. This occurs for several reasons. Lower air void content will tend to produce a stronger pavement more resistant to cracking. Lower air void content also will in general produce a pavement with lower permeability to both air and water. This will reduce the amount of binder age hardening in the pavement and will tend to minimize moisture damage, which can render the pavement weak and more prone to fatigue damage.

The primary HMA mixture design factor affecting fatigue life is the effective volumetric binder content of the mixture (VBE). For a given pavement, fatigue life increases with increasing VBE; therefore, controlling VBE is an important consideration in mixture design. Since VBE is equal to VMA minus the air void content, the mixture design procedures presented in Chapters 8 and 10 control VBE by controlling VMA and the design air void content of the mixture. As discussed above, increasing VMA or decreasing air void content too much can significantly decrease rut resistance; therefore, the requirements given in Chapters 8 and 10 provide both upper and lower limits for VMA and design air void content. For dense-graded mixtures, the design procedure in Chapter 8 provides the flexibility to increase the design VMA requirements up to 1.0% to produce mixtures with improved fatigue resistance and durability. SMA mixtures, because they have extremely high VMA, tend to produce mixtures with excellent fatigue resistance.

The design compaction level and the design air void content also affect the fatigue resistance of HMA. HMA fatigue resistance increases with increasing compactive effort. Mixtures that are produced with greater compaction energy have improved performance in fatigue. For constant in-place air void content, fatigue resistance improves with increasing design air void content. This effect may at first seem counterintuitive, but for constant in-place air void content, increasing the design air void content mostly has the effect of increasing the compaction effort during construction. Because mixtures with very high air void content will be difficult to compact to an acceptably low in-place air void content and because mixtures with design air void contents that are too low may exhibit poor rut resistance, design air void contents are controlled within a narrow range for HMA mixtures, typically 4.0 ± 0.5% for surface course mixtures.

Durability

Design VMA and in-place air void content have a major effect on the durability of HMA mixtures. Mixture durability improves as VBE increases. For a constant design air void content, VBE increases with increasing VMA. All else being equal, smaller NMAS mixtures have higher VMA and, therefore, have improved durability compared to larger NMAS mixtures. The durability of dense-graded mixtures can be improved by increasing the design VMA (within limits) as discussed in Chapter 8. SMA mixtures have even greater durability due to their even higher design VMA.

In-place air void content has a major effect on the durability of HMA. Mixture permeability increases with increasing in-place air void content. As permeability increases, binder age hardening and moisture infiltration increase, making the pavement less durable and more susceptible to moisture damage. Proper field compaction is therefore essential to producing durable pavements.

Laboratory Testing

Several laboratory tests have been developed to evaluate the performance of HMA. Tests have been developed to assess the resistance of HMA to rutting, fatigue cracking, thermal cracking, and moisture sensitivity. Additionally, laboratory-conditioning procedures have been developed to simulate the effects of short-term aging that occurs during construction and long-term aging that occurs during the service life of the pavement. Although numerous laboratory performance tests have been developed, only a few have been standardized and are routinely used for evaluation of HMA.

Rut Resistance Testing and HMA Mix Design

Several laboratory tests are available for evaluating the rutting resistance of HMA. These include tests that measure engineering properties, such as modulus or permanent deformation, and proof tests, such as the Asphalt Pavement Analyzer or Hamburg Wheel-track Test. Some specifying agencies and mixture designers have developed a level of confidence in specific tests and criteria for their local mixtures and pavements. In recent years, a major effort was undertaken to develop a rutting performance test and associated criteria that could be applied universally to HMA mixtures throughout the United States. The resulting device is the asphalt mixture performance tester (AMPT), previously called the simple performance test (SPT) system; because of its anticipated high level of future support by specifying agencies, this device is one recommended in this manual to measure rut resistance. Rut resistance can be evaluated in the AMPT using the dynamic modulus test, the flow number test, or the flow time test. Use of the dynamic modulus test to evaluate rut resistance was developed in conjunction with the MEPDG and is discussed in detail in *NCHRP Report 580: Simple Performance Tests for Permanent Deformation of Hot Mix Asphalt—Volume 1: The E* Specification Criteria Program.* Use of both the flow number test and flow time test to evaluate rut resistance during the mix design process are discussed in this manual. Four other tests are recommended in this manual as candidates for performance tests for the evaluation of rut resistance:

- The repeated shear at constant height (RSCH) test performed with the Superpave shear tester (SST).
- The high-temperature indirect tension (IDT) strength test.
- The asphalt pavement analyzer (APA).
- The Hamburg Wheel-track Test.

These six tests for evaluating rut resistance are discussed below. Specific information on using these tests in the mix design process are given in Chapter 8.

The Asphalt Mixture Performance Tester

Figure 6-1 shows the AMPT. It is a relatively small, computer-controlled test machine that can perform various tests on HMA over a temperature range of 4 to 60°C. The machine is available in the United States from several manufacturers who have demonstrated compliance with a detailed equipment specification prepared as part of National Cooperative Highway Research Program (NCHRP) Project 9-29 and contained in *NCHRP Report 513: Simple Performance Tester for Superpave Mix Design: First Article Development and Evaluation.* Two of the tests that can be performed in the AMPT have been related to the rutting performance of HMA. These are the dynamic modulus and the flow number tests. Ruggedness testing with the AMPT has demonstrated that it can control both of these tests with sufficient accuracy for use in specification testing. An interlaboratory study to establish precision statements for the dynamic modulus and flow number tests will be completed in the Fall of 2010. As this manual was being finalized, procedures for

Figure 6-1. Photograph of a simple performance test system.

these tests were available in an AASHTO Provisional Standard, TP 79: Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT). A National Highway Institute (NHI) training course on the AMPT was also being planned.

Dynamic Modulus Test—in the dynamic modulus test, an HMA specimen is subjected to a sinusoidal compressive load. The resulting stress and strain are recorded and used to calculate the dynamic modulus and phase angle for the mixture. The dynamic modulus is abbreviated |E*| (pronounced E-star; E for elastic modulus, and * for dynamic). |E*| is the peak stress in the test divided by the peak strain and represents the overall stiffness of the mixture. The phase angle is the amount that the strain lags the stress and is a measure of the elasticity of the mixture. The lower the phase angle, the more elastic the response. The stresses and strains in the dynamic modulus test are intentionally kept small to keep the response of the HMA in the linear range.

Dynamic modulus testing can be conducted at different temperatures and loading frequencies to evaluate the effect of temperature and traffic speed on the mixture stiffness. |E*| data from different temperatures and loading rates can be combined into a master curve that describes the mixture stiffness for any combination of temperature and loading rate. A dynamic modulus master curve is the primary HMA materials input needed for the design of HMA pavements using the MEPDG. AASHTO TP 79-09, Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT), which was developed in NCHRP Project 9-29, is the standard test method for obtaining dynamic modulus measurements on HMA with the AMPT. AASHTO PP 61-09, Developing Dynamic Modulus Master Curves for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT), also developed in NCHRP Project 9-29, is the recommended practice for developing dynamic modulus master curves for pavement structural design using the AMPT.

Criteria for using the dynamic modulus to judge the rutting resistance of an HMA mixture for a specific pavement can be obtained from the |E*| AMPT Specification Criteria Program developed in NCHRP Project 9-19 and described in *NCHRP Report 580.* This software uses the calibrated rutting model included in the MEPDG to determine project-specific testing conditions and dynamic modulus criteria to limit rutting to a specified level. The MEPDG is discussed in more detail later in this chapter. The |E* |AMPT Specification Criteria Program requires the user to enter information about the specific pavement, including HMA layer thicknesses, design traffic level, design traffic speed, environmental conditions at the project site, and the allowable rut depth in each HMA layer. The software then returns, for each HMA layer, the recommended testing conditions (temperature and frequency), and the minimum |E*| that the mixture must have to limit rutting to the specified level. Specifying agencies choosing to use dynamic modulus as the measure of rutting resistance can use this software to establish |E*| values and testing conditions based on the location of the mixture in the pavement (i.e., surface, intermediate or base), traffic level, and temperature conditions.

Flow Number Test—the flow number is an alternative to the dynamic modulus test for evaluating rutting resistance. In this test, a sample of the HMA mixture at high temperature is subjected to a repeated compressive stress pulse. This repeated loading produces permanent strain in the specimen, which is recorded by the AMPT for each load cycle. Figure 6-2 is an example of a permanent strain curve that results from a flow number test. The point in the permanent strain curve where the rate of accumulation of permanent strain reaches a minimum value has been defined as the flow number. The flow number has been related to the rutting resistance of HMA. As the flow number increases, rutting resistance also increases. AASHTO TP 79-09 includes the standard test method for using the AMPT to obtain the flow number of HMA.

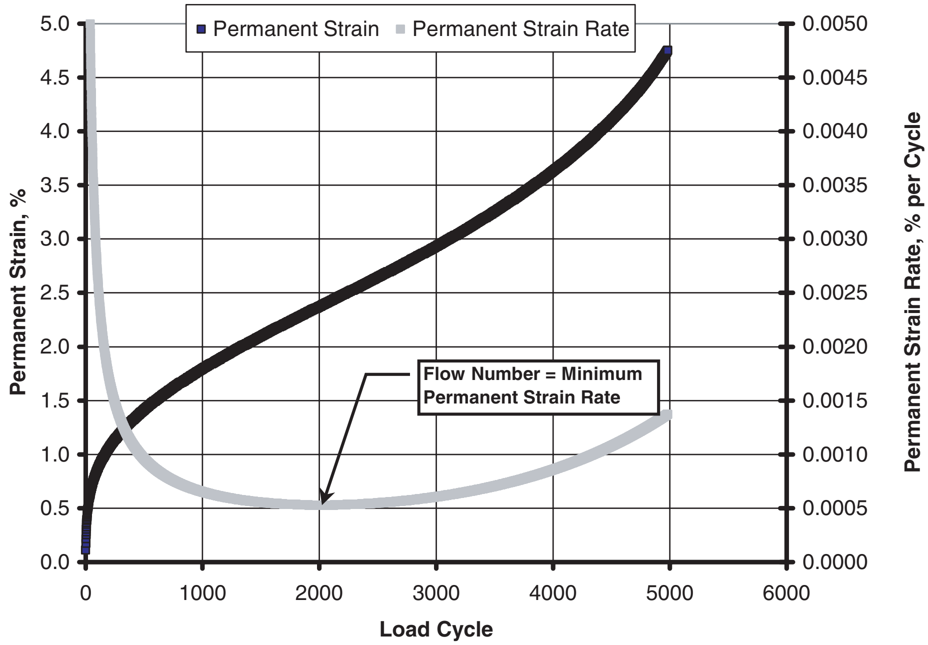

Figure 6-2. Typical data from the flow number test.

Flow Time Test—the flow time test differs from the flow number test in that a constant rather than a repeated load is applied to the specimen and the total deformation is monitored. Thus, the flow time test is simply a static creep test and the flow time is defined as the loading time required to initiate tertiary creep, which is the point at which the rate of deformation begins to increase. The flow time test was envisioned as a simpler alternative to the flow number test.

AMPT Specimens—The AMPT requires a test specimen that is 100 mm (4.0 in) in diameter by 150 mm (6.0 in) high. The specimen is sawed and cored from the middle of a 150 mm in diameter by 175-mm-high gyratory-compacted specimen. Figure 6-3 shows a completed AMPT test specimen and the original gyratory-compacted specimen from which the test specimen was cut. AASHTO PP 60-09, Preparation of Cylindrical Performance Test Specimens Using the

Figure 6-3. Photograph of an AMPT test specimen.

Superpave Gyratory Compactor (SGC), also developed in NCHRP Project 9-29, is the recommended procedure for preparing AMPT specimens.

There are three reasons for using smaller test specimens obtained from larger gyratory specimens in the AMPT:

1. To obtain an appropriate aspect ratio for the test specimens. Research performed during NCHRP Project 9-19 found that a minimum specimen diameter of 100 mm with a height-to-diameter ratio of 1.5 was needed.
2. To eliminate areas of the gyratory specimens with high air void content. Gyratory-compacted specimens typically have high air void content near the ends and around the circumference of the specimen.
3. To obtain relatively smooth, parallel ends for testing, which helps ensure proper stress distribution within the specimen during loading.

The air void content of the AMPT specimen will have a major effect on the properties measured in the AMPT. AMPT specimens used to evaluate rutting resistance should be prepared to the expected average field air void content at the time of construction, not the design air void content. Mixtures should be short-term oven aged for 4 hours at 135°C in accordance with the procedure for Short-Term Conditioning for Mixture Mechanical Property Testing in AASHTO R 30. A reasonable air void tolerance for AMPT specimens is ±0.5%. The AMPT specimen will have a lower air void content than the larger gyratory specimen from which it is produced. AASHTO PP 60-09 contains a procedure for achieving the target air void content for AMPT specimens.

The number of replicates to be tested depends on the repeatability of the test and the desired accuracy of the resulting data. Based on current estimates of coefficients of variation for the dynamic modulus and flow number tests of 13 and 20%, respectively, it is recommended that two replicate specimens be used for dynamic modulus testing and four replicate specimens for flow number testing. These numbers of replicates will result in coefficients of variation for the mean values of dynamic modulus and flow number of approximately 10%.

Other Laboratory Tests for Rut Resistance

Other laboratory tests are available for evaluating the rutting resistance of HMA. Those most often used are the Superpave Shear Tester (SST), the High-Temperature Indirect Tensile Test (IDT), the Asphalt Pavement Analyzer (APA), and the Hamburg Wheel-Track Test.

Repeated Shear at Constant Height (RSCH) Test. This is one of several tests that can be performed with the SST. The RSCH test is designed to evaluate the rutting resistance of HMA by applying repeated shear loading to an HMA specimen at high temperatures. The test has been standardized as AASHTO T 320. The RSCH test is performed on 150-mm-diameter specimens that are 38 to 50 mm thick, depending on the nominal maximum aggregate size. The test specimens are glued to loading platens and subjected to repeated direct shear loading while the vertical load is varied to maintain the specimen at a constant height. In September 1997, during the Superpave implementation effort, the Mixtures Expert Task Group established preliminary guidelines for using the RSCH test to evaluate the rutting resistance of HMA. In the standard performance test—discussed in detail in Chapter 8 of this manual—the RSCH test is performed at the maximum, 7-day average pavement temperature found 20 mm below the pavement surface, as given in LTPPBind Version 3.0. The SST, a relatively expensive device, is available in few laboratories in the United States. Figure 6-4 is a photograph of an SST. The RSCH test is not recommended for routine evaluation of the rut resistance of HMA mixtures in the laboratory—the other tests discussed in this chapter are generally easier to perform and less expensive to conduct and so more widely used than the RSCH test. For those laboratories that have an SST device, specific

Figure 6-4. Photograph of the SST.

recommendations for using it to evaluate mixture rut resistance during the mix design process are given in Chapter 8.

The High-Temperature IDT Strength Test. This test was developed as a quick and inexpensive procedure for evaluating rut resistance using equipment currently available in many HMA design and quality assurance laboratories. Christensen, Bonaquist, and Jack reported an excellent relationship between rutting resistance and the indirect tensile strength at high temperature in a 2000 publication. Additional work confirming these results was reported in 2004 by Zaniewski and Srinivasan. The test is conducted on standard gyratory specimens produced for mixture design or quality assurance with the indirect tensile strength equipment used in AASHTO T 283. Specimens should be compacted to the design gyration level. When testing specimens as part of mixture design, the mixture should be short-term oven aged for 4 hours at 135°C in accordance with the procedure for Short-Term Conditioning for Mixture Mechanical Property Testing in AASHTO R 30. When testing quality assurance specimens from plant production, the short-term aging is not required. The testing temperature is 10°C less than the 50% reliability, 7-day average maximum pavement temperature obtained from LTPPBind Version 3.0.

Figure 6-5. Photograph of the asphalt pavement analyzer.

The Asphalt Pavement Analyzer and Hamburg Wheel-Track Test. The APA (see Figure 6-5) and the Hamburg Wheel-Track tests are proof tests for rutting resistance that are used by some specifying agencies. Both tests attempt to simulate the effect of traffic loading by rolling a small loaded wheel over an HMA specimen at high temperature. In the APA, the load is applied through a rubber hose that can be inflated to a specified pressure. In Hamburg Wheel-Track testing, the load is applied through a steel wheel. Conditioned air is used for temperature control in the APA, while Hamburg Wheel-Track testing uses water to control the temperature of the test specimen.

The Hamburg Wheel-Track test can be used to assess both rutting resistance and moisture sensitivity. Originally, these tests were performed on rectangular specimens; however, in recent years the equipment has been modified to use 150-mm-diameter gyratory-compacted specimens. The standard test method for the APA is AASHTO TP 63-09; the standard test method for Hamburg Wheel-Track testing is AASHTO T 324.

Agencies that specify these tests have established criteria for rutting resistance based on the deformation or impression depth after a specified number of load cycles. For example, for high traffic levels, the Georgia Department of Transportation specifies a maximum deformation of 5 mm in the APA after 8,000 cycles at a test temperature of 64°C. The Texas Department of Transportation specifies the minimum number of wheel passes in the Hamburg Wheel-Track test to reach an impression depth of 12.5 mm when tested at a temperature determined by the performance grade of the asphalt binder. These values are >10,000 for mixes produced with PG 64-XX binder, >15,000 for mixes produced with PG 70-XX binder, and >20,000 for mixes produced with PG 76-XX binder. Additional information on the use of the APA and Hamburg Wheel-Track testing as performance tests for use in the mix design process is given in Chapter 8.

Fatigue Testing

The only standard test method available for fatigue testing of HMA is the flexural fatigue test, AASHTO T 321. In this test a beam sample, 380 mm long by 63 mm wide by 50 mm high, is subjected to strain-controlled, repeated four-point bending. The beam samples are prepared using either a kneading or rolling wheel compaction; there are no AASHTO standards for either of these methods of laboratory compaction. Figure 6-6 shows a device for flexural fatigue testing. The number of laboratories in the United States that can fabricate and test flexural fatigue specimens is limited.

During a flexural fatigue test, the beam is damaged by the repeated flexing. This damage results in a decrease in the modulus of the beam. The beam is considered failed when the modulus decreases to 50% of its initial value. The number of loading cycles applied to the beam can range

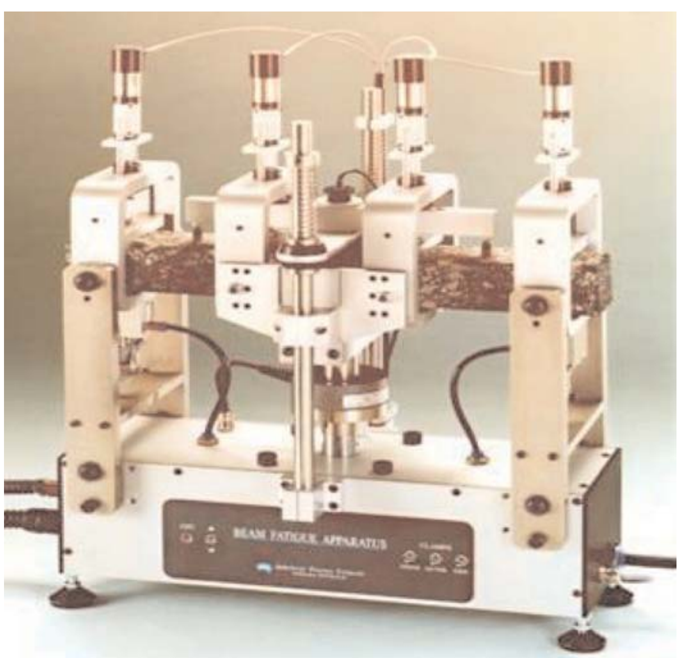

Figure 6-6. Photograph of flexural fatigue apparatus.

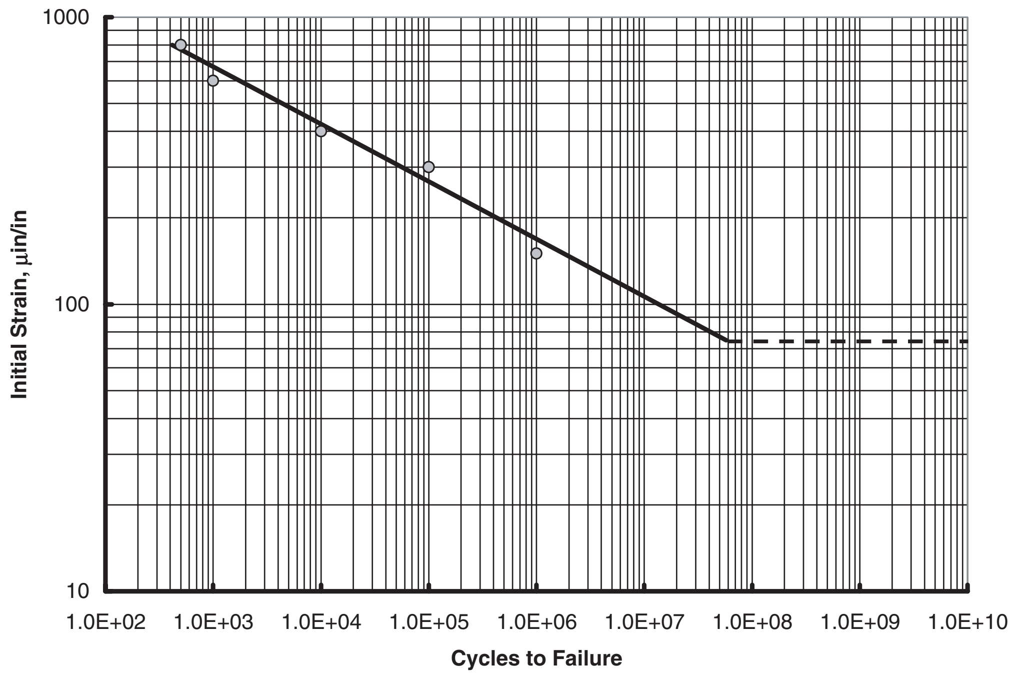

Figure 6-7. Typical S-N diagram for HMA.

from 1,000 to 10,000,000 or more. The results of fatigue tests are presented in the form of S-N diagrams, which are simply plots of the applied strain and the corresponding number of cycles to failure. Figure 6-7 presents a typical S-N diagram for HMA generated from laboratory test data. The point where the fatigue life becomes indefinite is called the fatigue endurance limit. Because of its extreme importance in the structural design of perpetual pavements, research is in progress to better define the endurance limit for HMA.

Generating an S-N curve for HMA requires testing several beams at different strain levels. Due to the high variability of fatigue testing, each strain level requires testing a number of replicate specimens. Because of the high level of effort required to generate S-N curves for HMA, fatigue testing is rarely performed in practice. Instead, relationships between mixture compositional factors and fatigue life that have been developed from databases of tests on a number of mixtures are used. These relationships show that the most important mixture design factor affecting the fatigue life of HMA is the effective volumetric binder content of the mixture, VBE. By controlling VBE, the mixture design process controls the fatigue life of the mixture. As discussed previously, VBE, is controlled in the design method described in this manual by controlling both the VMA and the design air void content.

Thermal Cracking

The MEPDG can predict the amount of thermal cracking that will occur in an asphalt pavement. To perform this analysis, information on the creep and strength properties of the HMA at low temperatures are needed. These properties are measured using the Indirect Tensile Tester (IDT), AASHTO T 322. Low-temperature tests on HMA require an expensive environmental chamber and the capacity to impose high loads on the test specimens. Figure 6-8 shows a specimen being tested in the IDT. Only a few laboratories in the United States have IDT equipment for low-temperature testing.

AASHTO T 322 involves preparing nine IDT test specimens and performing creep and strength tests on three specimens each at temperatures of 0, −10, and −20°C. The results of the creep tests

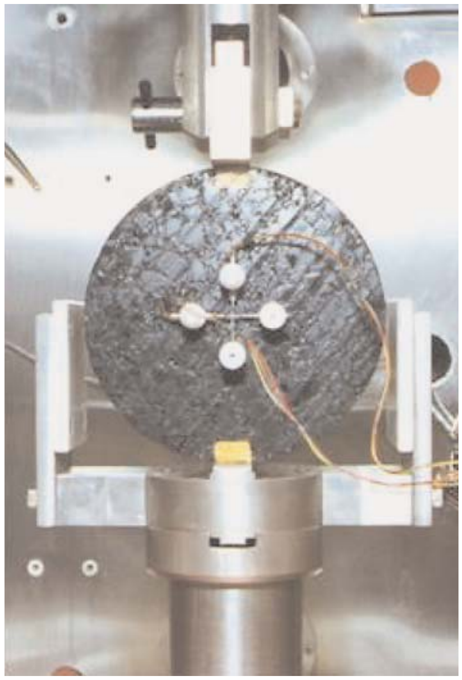

Figure 6-8. Photograph of specimen in the IDT.

are used to generate a compliance master curve for the mixture, which governs the buildup of thermal stresses in the pavement. The potential for thermal fracture depends on the magnitude of the estimated thermal stress relative to the tensile strength of the mixture.

IDT test specimens are 150 mm in diameter by 38 mm thick. They are formed by sawing the test specimen from the middle of a gyratory-compacted specimen. Sawed ends are needed to attach the deformation measuring equipment. IDT testing is usually conducted on specimens compacted to the anticipated in-place air void content and exposed to long-term oven aging in accordance with AASHTO R 30.

Because the resistance to thermal cracking is almost completely governed by the properties of the binder, IDT testing is usually only performed when the binder cannot be tested using the bending beam rheometer and direct tension device. When modifiers are added to the mixture rather than the binder, it may be necessary to conduct IDT testing to evaluate the low temperature properties of the resulting mixtures.

Moisture Sensitivity Testing

Two tests have received acceptance in the United States to evaluate the moisture sensitivity of HMA: the Lottman procedure (AASHTO T 283) and the Hamburg Wheel-Track test (AASHTO T 324). In many cases, the two tests provide different results, likely because they simulate different moisture damage processes. Recent efforts to improve moisture sensitivity testing using the Environmental Conditioning System (ECS) developed during the Strategic Highway Research Program have not yet resulted in a standard test method used by state agencies in the routine design of HMA.

In AASHTO T 283, six laboratory specimens are prepared to an air void content of 7.0 ± 0.5%, then divided into two subsets with approximately equal average air void contents. The tensile strength of one subset is measured dry. The tensile strength of the second subset is measured

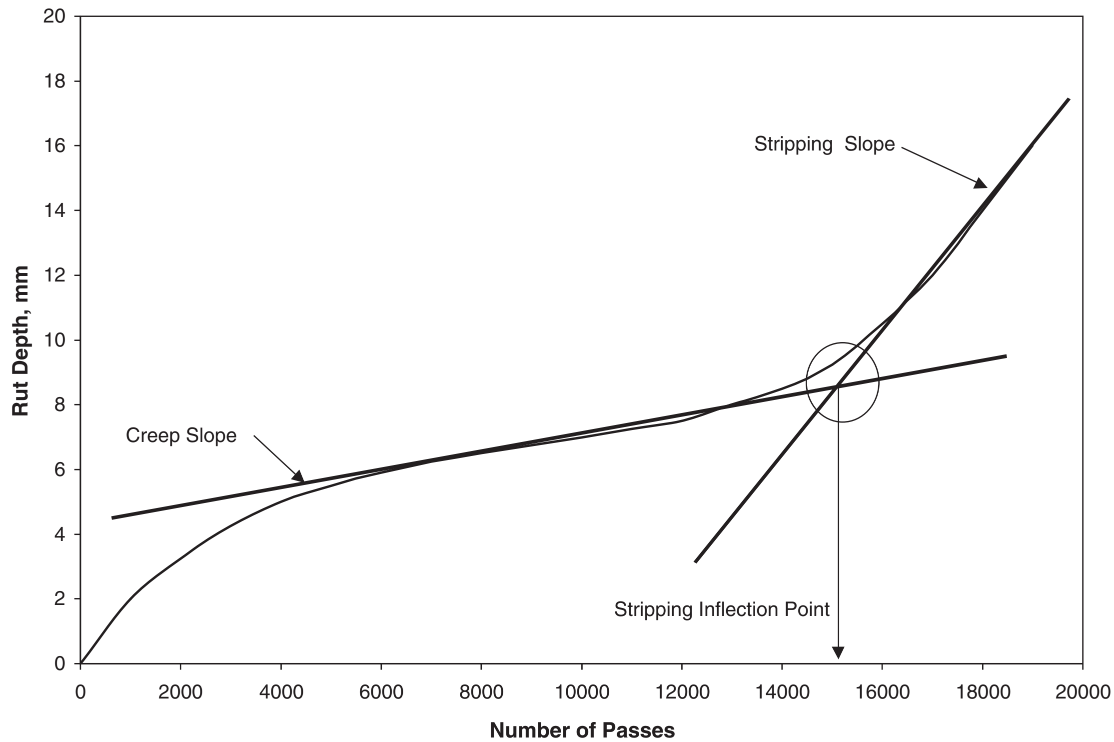

Figure 6-9. Typical rut depth versus wheel pass curve from AASHTO T 324.

after conditioning by vacuum saturation followed by a freeze-thaw cycle and a warm water soak. The ratio of the average tensile strength of the conditioned to unconditioned subsets and a visual assessment of stripping is used to measure moisture sensitivity. A mixture is considered to have an acceptable level of moisture sensitivity if the tensile strength ratio is equal to or greater than 80% and there is no visual evidence of stripping in the conditioned test specimens.

Since the Hamburg Wheel-Track test (AASHTO T 324) tests HMA submerged in water, it can also be used to evaluate the resistance of a mixture to moisture damage. Moisture sensitivity is evaluated by computing the stripping inflection point, which is defined as the intersection of the slopes from the creep and stripping portions of the rut depth versus wheel pass curve as shown in Figure 6-9. The recommended air void content of laboratory-prepared specimens for AASHTO T 324 is 7.0 ± 2.0%.

Criteria for evaluating moisture sensitivity based on AASHTO T 324 place a minimum limit on the stripping inflection point. For example, Aschenbrener et al. suggested for Colorado conditions that mixtures with good performance with respect to moisture damage (life of 10 to 15 years) should have a stripping inflection point greater than 14,000 passes.

Short- and Long-Term Oven Conditioning

When conducting performance tests on HMA, it is important to simulate the effects of (1) short-term aging that occurs during plant mixing and construction and (2) long-term aging that occurs during the service life of the pavement. During production and laydown, some of the binder is absorbed into the aggregate, decreasing the effective binder content, and the binder is aged by the high temperatures that occur in the overall construction process. Further oxidative aging of the binder occurs during the service life of the pavement. During the Strategic Highway Research Program, procedures for short-term and long-term conditioning of mixtures were developed. These were subsequently standardized in AASHTO R 30.

The short-term conditioning procedure consists of conditioning loose mixture in a forced draft oven at 135°C for 4 hours. The mixture is evenly spread in a pan to a thickness between 25 and 50 mm and stirred every hour.

In the long-term conditioning procedure, test specimens prepared from loose mix that was previously short-term conditioned as described above are further conditioned in a forced draft oven before testing. The temperature for this conditioning is 85°C for a period of 120 hours

Because rutting is a distress that occurs early in the life of a pavement, performance testing to assess rutting resistance should be conducted on specimens that have been short-term conditioned. Fatigue and thermal cracking tests should be performed on specimens that have been long-term conditioned. AASHTO T 283 has a different conditioning procedure of 16 hours in a forced draft oven at 60°C.

Evaluating the Need for Performance Testing

It is neither practical nor necessary to perform the full suite of performance tests discussed above when designing HMA using conventional materials, including most modified binders. The test methods for fatigue and thermal cracking require a high level of effort and complex equipment, and substantial research has shown that resistance to these forms of distress can be controlled by controlling the effective binder content of the mixture and the low temperature binder grade, respectively. Rutting resistance is somewhat more difficult to control since several compositional factors affect rutting resistance, and if these act in the same direction, the resulting HMA may exhibit poor performance. Fortunately, tests for rutting resistance using the AMPT are relatively easy to perform and criteria differentiating various levels of performance are available. There is general agreement in the industry that testing the specific combinations of binder, aggregate, and additives used in an HMA mixture is the only way to assess the potential for moisture sensitivity.

Table 6-4 summarizes the performance testing recommended in this manual for HMA made from conventional materials, including most modified binders. It is recommended that all mixtures be evaluated for moisture sensitivity using AASHTO T 283. Equipment for this test is widely available. Rutting resistance should be evaluated for mixtures designed for traffic levels greater than 3 million ESALs. Rutting resistance can be evaluated using either the dynamic modulus test in conjunction with the |E*| AMPT Specification Criteria Program, or the flow number test and the criteria given in Chapter 8. Performance testing is not recommended for fatigue cracking when the mixture design criteria given in Chapters 8 and 10 or 11 are met. Performance testing for thermal cracking is not recommended when binders are selected using the Performance Grading system.

For HMA made with non-conventional materials, performance tests for rutting, fatigue cracking, thermal cracking, and moisture sensitivity should be performed and compared to results from mixtures made with conventional materials. Non-conventional materials might include recycled materials such as ground glass, ground tire rubber, ground or shredded plastic

Table 6-4. Recommended performance tests for HMA made with conventional materials including most modified binders.

Property	Recommended Test	Design Traffic Levels for Which Property Should be Evaluated
Moisture Sensitivity	AASHTO T 283	All
Permanent Deformation	Flow Number or Dynamic Modulus, AASHTO TP 79-09	3 Million ESAL and greater
Fatigue Cracking	None	NA
Thermal Cracking	None	NA

and ground roofing shingles or shingle tabs. Another class of non-conventional materials is novel additives—chemicals, compounds, or other materials—designed to provide some benefit to HMA, but which have not yet been thoroughly evaluated with laboratory tests and field trials. Care should be used in evaluating these mixtures using the criteria presented in the manual or elsewhere for HMA.

Performance Predictions Using the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG)

Some agencies and mixture designers may be interested in using the MEPDG to predict the performance of a pavement incorporating specific HMA mixtures. Such an analysis using the MEPDG will include the effect of both pavement structure and HMA mixture properties on performance. In some cases, such as bottom-up fatigue cracking, layer thicknesses or subgrade support conditions will dominate. In other cases, such as rutting in an overlay of an existing pavement, HMA mixture effects will be more important. This section provides an introduction to the MEPDG and how it can be used to predict the performance of an HMA mixture for a specific pavement section. The MEPDG is a comprehensive, state-of-the-practice tool for the design of new and rehabilitated pavement structures. Readers interested in using this tool are encouraged to study the MEPDG user manual and other detailed documentation for the MEPDG.

The MEPDG is substantially different than most pavement design procedures used in the past by highway agencies. The MEPDG is based on mechanistic-empirical pavement design principles. Critical stresses and strains from vehicle and environmental loading are computed using mechanistic theory. These critical stresses and strains are then empirically related to the occurrence of distresses such as rutting and cracking in the pavement. Most agencies have experience with the 1993 AASHTO Pavement Design Guide, which is based on limited empirical pavement performance equations from the AASHO Road Test conducted in the late 1950s and early 1960s. The distress prediction models in the MEPDG have been calibrated using data for a large number of pavement sections in the Long-Term Pavement Performance (LTPP) database. Pavement sections used in the calibration were located throughout the United States.

The MEPDG is an analysis tool. The output from the MEPDG is the predicted performance of a trial pavement section, not pavement thickness design. As discussed in more detail below, some of the distress prediction models in the MEPDG provide a link between HMA material properties and pavement performance that can be used after the mixture design is completed to verify that an HMA mixture will provide acceptable performance for a specific project.

MEPDG Input Levels

The MEPDG requires a large amount of information about the pavement being analyzed. This includes data concerning traffic, climate, subgrade soils, the condition of existing pavements for rehabilitation design, and the thicknesses and material properties for each layer of the pavement, including existing pavement layers for rehabilitation design. To provide flexibility for users with different capabilities, the MEPDG uses a three-level hierarchical scheme of data input:

- **Level 1.** The input parameter is measured directly. This level provides the most accurate information about the input parameter. The primary Level 1 material property input for HMA is the measured dynamic modulus of the mixture that will be used in the pavement.
- **Level 2.** The input parameter is estimated from correlations or regression equations that are embedded in the MEPDG. For Level 2, the dynamic modulus for HMA materials is estimated from gradation, volumetric properties, and measured binder properties.

- **Level 3.** The input parameter is based on default values provided by the MEPDG software. For Level 3, the dynamic modulus for HMA is estimated from gradation, volumetric properties, and the binder grade.

Testing or data collection costs decrease as the hierarchical level increases from Level 1 to Level 3, but the accuracy of the input data also decreases. If performance predictions from the MEPDG are to be used to verify a specific mixture design, Level 1 inputs should be used for all of the HMA material properties. Other levels can be used for the traffic, climate, and the material properties of the other layers. The overall accuracy of the predicted performance, however, will depend on the accuracy of all of the input data, not just the HMA material properties.

MEPDG Performance Models for HMA

The MEPDG can analyze flexible, semi-rigid, rigid, and composite pavements. For pavements with HMA surfaces, the MEPDG includes performance models to predict the following distresses:

- Rut depth for HMA layers, unbound aggregate layers, and the subgrade
- Transverse thermal cracking
- Alligator cracking due to bottom-initiated fatigue
- Longitudinal wheel path cracking due to surface-initiated fatigue
- Reflection cracking
- Roughness

A detailed discussion of the form of the MEPDG performance models is beyond the scope of this manual. The interested reader should refer to the MEPDG user manual and other detailed documentation for the MEPDG.

The MEPDG does not include models for durability distresses such as raveling or moisture damage. It is assumed that the potential for these forms of distress will be minimized through proper HMA mixture design. The MEPDG also does not include a model to predict changes in the skid resistance of the pavement with time and traffic.

HMA Material Property Inputs

HMA material properties are not direct inputs to some of the distress prediction models. The reflection cracking model included in the MEPDG is an empirical function that delays the appearance of existing joints and cracks depending on the thickness of the HMA and the condition of the underlying pavement. The roughness model predicts the International Roughness Index (IRI) for the pavement based on the initial roughness, the amount of rutting and cracking obtained from the other models, and site factors including pavement age, soil type, freezing index, and precipitation. Table 6-5 summarizes the HMA materials properties used by each of the performance models.

The dynamic modulus and binder grade data are used by the MEPDG to generate a dynamic modulus master curve for each HMA layer. This requires testing the dynamic modulus at multiple temperatures and frequencies as described in AASHTO PP 61-09. The dynamic modulus master curve is used for the computation of the traffic-induced strains that are used by the rutting, alligator cracking, and longitudinal cracking models. It is also a direct input into the damage models used for alligator and longitudinal cracking. The dynamic modulus testing should be conducted on short-term oven-aged mixture (4 hours at 135°C per AASHTO R30), compacted to the expected in-place air void content of the pavement.

The alligator and longitudinal cracking models also require in-place volumetric properties of the HMA layers, specifically the air void content and the effective binder content. These volumetric

Table 6-5. Summary of HMA materials properties used by the MEPDG performance models.

HMA Property	Rutting	Thermal Cracking	Alligator Cracking	Longitudinal Cracking	Reflection Cracking	Roughness
Dynamic Modulus	X		X	X		Indirect*
Binder Grade	X		X	X		Indirect*
In-Place Air Void Content		X	X	X		Indirect*
In-Place Effective Binder Content			X	X		Indirect*
In-Place VMA		X				Indirect*
Low Temperature Creep Compliance		X				Indirect*
Low Temperature Tensile Strength		X				Indirect*

* The MEPDG estimates roughness from rutting and cracking predictions, which in turn depend on various physical properties as noted here.

properties are direct inputs to the models and have a major effect on the predicted load-associated cracking distress. As discuss previously, the resistance of HMA to fatigue cracking increases with an increase in the effective binder content and a decrease in in-place air void content.

For thermal cracking analysis, the in-place VMA of the surface mixture is needed to estimate the coefficient of thermal contraction. Creep compliance and tensile strength data for the surface mixture are obtained from AASHTO T 322. The specimens used in this testing should be compacted to the expected in-place air void content of the pavement, then long-term oven aged in accordance with AASHTO R 30 before testing.

Overview of Using the MEPDG to Verify an HMA Mixture Design

As discussed previously, the MEPDG is a comprehensive pavement analysis tool that can predict the performance of a given pavement section. The accuracy of the predicted performance depends, in part, on the accuracy of the input data. Detailed information on traffic, climate, subgrade soils, and unbound layers, and existing pavement conditions for rehabilitation design are needed in addition to materials properties for the HMA layers. The User Manual for the MEPDG provides guidance for the selection of specific input data.

The MEPDG can be used to predict the amount of rutting, bottom-up fatigue cracking, top-down fatigue cracking, and thermal cracking in a pavement section for mixture-specific HMA properties. The change in roughness caused by these distresses can also be predicted. Any or all of these may be used as criteria to judge the acceptability of the HMA mixture for the specific pavement analyzed. Reflective cracking should not be used as a criterion because the MEPDG model for reflective cracking is empirical and is not affected by the properties of the HMA.

Rut Resistance

To evaluate only the rutting resistance of an HMA mixture, the $|E^*|$ AMPT Specification Criteria Program as described in *NCHRP Report 580*, not the MEPDG, should be used. This program uses the calibrated rutting model included in the MEPDG, but does not require all of the MEPDG input data for traffic, climate, and properties of the other layers of the pavement. In this approach, rutting in the HMA layer is assumed to be insensitive to underlying layer properties. The $|E^*|$ AMPT Specification Criteria Program provides a predicted rut depth in each HMA layer specified by the user.

If rutting will be used in conjunction with other forms of distress to judge the acceptability of the HMA mixture, then a complete analysis using the MEPDG must be performed. In this case, the rut depth in each layer of the pavement will be predicted as a function of time for the pavement section.

Fatigue Cracking

To evaluate the potential for bottom-up (alligator) and top-down (longitudinal) cracking, a complete analysis must be performed using the MEPDG. The user is cautioned that fatigue cracking is more sensitive to traffic, subgrade support conditions, and pavement layer thicknesses than to HMA properties. It may not be possible to adjust HMA properties to obtain acceptable cracking performance if the pavement is not thick enough or the subgrade support is poor. The MEPDG provides separate predictions of alligator and longitudinal cracking with time for the pavement section being analyzed. Alligator cracking is expressed as the percent of the total lane area. Longitudinal cracking is expressed as feet of longitudinal cracking per lane mile.

Thermal Cracking

To evaluate a mixture for resistance to low temperature cracking, a thermal cracking analysis must be performed with the MEPDG. All climatic data needed for this analysis are stored within a database supplied with the MEPDG; therefore, the user need only specify the longitude and latitude of the pavement and the required material properties. The MEPDG provides a prediction of transverse thermal cracking with time for the pavement section being analyzed. Thermal cracking is expressed as feet of transverse cracking per lane mile.

Surface Roughness

Within the MEPDG, the change in roughness in a pavement section depends on the initial roughness, the predicted rutting and cracking, and site factors including pavement age, soil type, freezing index, and precipitation. To analyze changes in roughness, a complete analysis must be performed with the MEPDG. In many cases, the initial roughness and site factors, which are not associated with the HMA mixtures used in the pavement, will dominate the prediction.

MEPDG Predictions

Using the MEPDG as an analysis tool for HMA mixtures is conceptually simple. The basic steps are summarized below. Additional detail for each step is provided in the MEPDG User Manual.

1. **Select a trial pavement section.** For verification of a mixture design, the pavement section will usually be specified based on the original design of the pavement.
2. **Select the performance criteria that will be used.** As discussed above the MEPDG predicts the development of various distresses with time for the trial pavement section. The performance criteria used to evaluate the HMA mixture will generally be determined by the specifying agency based on its pavement management policies.
3. **Obtain the necessary inputs for the trial pavement section.** This is the most time-consuming step. The MEPDG requires a large amount of information about the pavement being analyzed. This includes data concerning traffic, climate, subgrade soils, the condition of existing pavements for rehabilitation design, and the thicknesses and material properties for each layer of the pavement, including existing pavement layers for rehabilitation design. For verification of an HMA mixture, Level 1 material property inputs should be used for the HMA mixture being analyzed. For the remaining inputs, other level data can be used, keeping in mind that the accuracy of the distress predictions depends on the accuracy of the input data.
4. **Run the MEPDG software and examine the inputs and outputs for engineering reasonableness.** The software summarizes the inputs. This summary should be examined to ensure that no errors were made during the data input process. If input errors are discovered, fix the errors and rerun the analysis. The software also summarizes the pavement layer moduli

for each month of the analysis. These summaries should be examined to ensure that they are reasonable. Temperature and aging change the modulus of HMA layers over time. Frost and moisture content change the modulus of unbound materials on a seasonal basis. Finally, the software summarizes all distresses by month over the design life of the pavement. These should be examined carefully to see if they appear reasonable and then compared to the performance criteria.

5. **Modify the HMA mixture properties to improve performance.** The evolution of distress over the design life of the pavement should be studied carefully to identify potential adjustments that can be made to the mixture to improve the predicted performance. The next section presents guidance for adjusting HMA mixtures based on the distresses predicted by the MEPDG.

Mixture Adjustments Based on MEPDG Predictions

The distress prediction models in the MEPDG are driven by the stiffness of the HMA layers (dynamic modulus and creep compliance), the low-temperature strength of the HMA surface layer, and the in-place volumetric properties of the HMA layers. Therefore, to change the predicted level of distress by changing HMA mixture properties, the change must affect the properties listed above. The effects of changing HMA mixture properties are specific to the distress type, and it is not uncommon that actions taken to improve HMA rutting performance result in an adverse effect on cracking performance. The exception to this rule is the in-place air void content. Decreasing the in-place air void content of the mixture will improve the performance for all distresses. Recommended mixture adjustments are presented below for rutting, alligator cracking, longitudinal cracking, and thermal cracking. It is recommended that users of the MEPDG perform a sensitivity analysis for the pavement section to determine the magnitude of adjustment needed. In some cases, the adjustments may not be possible with the mixture types and binder grades available. In such cases, changes to the pavement structure may be needed to obtain acceptable performance predictions.

HMA Layer Rutting

Within the MEPDG, the only way to decrease the predicted rutting within the HMA layers of a pavement is to increase the dynamic modulus of the mixture. The $|E^*|$ AMPT Specification Criteria Program, described in *NCHRP Report 580,* can be used to determine the minimum dynamic modulus needed to keep the predicted rutting below a specified level. The MODULUS spreadsheets in the EXCEL™ workbook that accompanies this manual provide tools for estimating mixture dynamic modulus values from mixture composition. These tools should be used to estimate mixture properties needed to meet the minimum dynamic modulus determined from the $|E^*|$ AMPT Specification Criteria Program. The mixture design factors that affect the dynamic modulus are listed below in order of importance:

- **High-Temperature Binder Grade.** The high-temperature binder grade has the greatest effect on the dynamic modulus of the HMA mixture. Increasing the high temperature binder grade one level will increase the dynamic modulus of the mixture approximately 25%.
- **Design VMA.** VMA is the sum of air void content and effective binder content in the mixture, which are the components of HMA that deform under load. The modulus of HMA increases with decreasing VMA. A 1% decrease in design VMA will increase the dynamic modulus approximately 5%.
- **Nominal Maximum Aggregate Size (NMAS).** Larger NMAS mixtures have lower design VMA. Increasing the nominal maximum aggregate size of the mixture one level will increase the dynamic modulus approximately 5%.
- **In-Place Air Void Content.** In-place air void content affects the in-place VMA of the mixture. The MEPDG predicts performance based on the properties of the in-place mixture. Decreasing in-place air void content 1% will increase the dynamic modulus approximately 5%.

- **Filler Content.** Increasing the filler content of the mixture will increase the dynamic modulus. Within the filler contents allowed for dense-graded mixtures, a 1% increase in the percent passing the #200 sieve will increase the dynamic modulus approximately 1.5%.

Alligator Cracking (Bottom-Up Fatigue Cracking)

Within the MEPDG, alligator cracking depends on pavement thickness, subgrade support conditions, and the properties of the lowest asphalt-bound layer in the pavement. Fatigue cracking can be decreased by increasing the pavement thickness, improving subgrade support, or enhancing the HMA properties for the lowest layer. Alligator cracking is generally more sensitive to changes in thickness and subgrade support than to changes in the properties of the bottom HMA layer.

The effective binder content, VBE, and in-place density are direct inputs to the fatigue cracking model in the MEPDG. Increasing the effective binder content and decreasing the in-place air void content of the lowest HMA layer will substantially decrease the predicted alligator cracking in the pavement. As discussed previously, use of dense-graded mixtures with higher design VMA will decrease the predicted alligator cracking compared to that for mixtures with more typical VMA values. The effective binder content of these mixtures is up to 1% higher than that for the standard mixtures. The effective binder content can also be increased by decreasing the nominal maximum aggregate size of the lowest HMA layer. A one-level decrease in nominal maximum aggregate size also increases the effective binder content by 1%. The effect of decreasing air void content is similar to that of increasing effective binder content.

Alligator cracking is affected to a lesser degree by the dynamic modulus of the lowest HMA layer. For pavements with 5 inches or more of HMA, alligator cracking predicted by the MEPDG decreases with an increase in the dynamic modulus. For pavements with 3 inches or less of HMA, the predicted alligator cracking decreases with a decrease in the dynamic modulus. As discussed above for rutting, changing binder grade is the most efficient way of changing the dynamic modulus of HMA.

Longitudinal Cracking (Top-Down Fatigue Cracking)

Within the MEPDG, top-down cracking depends heavily on the properties of the surface HMA layer. Since effective binder content and in-place density are direct inputs to the fatigue model in the MEPDG, these properties for the surface HMA layer have a major effect on the predicted longitudinal cracking. Increasing the effective binder content and decreasing the in-place air void content of the surface HMA layer will substantially decrease the predicted longitudinal cracking in the pavement. As discussed previously, the use of dense-graded mixtures with higher design VMA will decrease the predicted longitudinal cracking compared to that for the standard mixtures. The effective binder content of these mixtures is up to 1% higher than that for the standard mixtures. The effective binder content can also be increased by decreasing the nominal maximum aggregate size of the surface HMA layer. A one-level decrease in nominal maximum aggregate size also increases the effective binder content by 1%. The effect of decreasing air void content is similar to that for increasing effective binder content.

Longitudinal cracking is affected to a lesser degree by the dynamic modulus of the surface HMA layer. Decreasing the dynamic modulus of the surface layer will decrease the amount of longitudinal cracking that occurs in the pavement. As discussed above for rutting and alligator cracking, changing binder grade is the most efficient way of changing the dynamic modulus of HMA.

Thermal Cracking

Within the MEPDG, the predicted thermal cracking depends on the environment at the project location, the total thickness of the HMA, and the properties of the surface HMA layer.

Table 6-6. Summary of effect of mixture composition on performance predictions.

HMA Property	Rutting	Thermal Cracking	Alligator Cracking HMA ≥ 5 in	Alligator Cracking HMA < 3 in	Longitudinal Cracking
High Temperature Binder Grade	**Increase to improve**		Increase to improve	Decrease to improve	Decrease to improve
Low Temperature Binder Grade		**Decrease to improve**			
Design VMA	Decrease to improve		**Increase to improve**	**Increase to improve**	**Increase to improve**
Design VFA		Increase to improve			
Filler Content	Increase to improve				
In-Place Air Void Content	Decrease to improve	Decrease to improve	**Decrease to improve**	**Decrease to improve**	**Decrease to improve**

For a given project, the predicted thermal cracking can be decreased by increasing the HMA thickness or improving the low-temperature properties of the surface layer. The predicted amount of thermal cracking will decrease with an increase in either the tensile strength or creep compliance of the surface mixture.

Low-temperature tensile strength increases with increasing voids filled with asphalt, VFA. A 5% increase in VFA will increase the low-temperature tensile strength approximately 85 psi. In-place properties of the HMA layer are used in the MEPDG predictions; therefore, the in-place air void content also affects the tensile strength of the mixture. For a given binder content, decreasing the in-place air void content increases VFA and the tensile strength of the mixture. Polymer-modified binders exhibit approximately 8% higher low-temperature strengths compared to neat binders.

The creep compliance of the mixture is affected by the same properties that affect the dynamic modulus of the mixture. Low temperature binder grade is by far the most important factor affecting the creep compliance of the mixture. Decreasing the low temperature grade by one level increases the creep compliance by approximately 25%. VMA and in-place air void content have smaller effects. Increasing the VMA or in-place air void content 1% will increase the creep compliance by approximately 5%.

Summary

Table 6-6 summarizes the effects of mixture composition on performance predictions using the MEPDG. The properties highlighted in bold for each distress have the greatest effect on the predicted performance.

A Note on Modulus, HMA Mix Design, and Pavement Design Using the MEPDG

It is likely that many state highway agencies will eventually adopt the MEPDG for designing flexible pavements. In most cases, pavement designs—at least preliminary designs—will be done well in advance of developing or selecting HMA mix designs for a given pavement. This will involve making assumptions about the type of mixture used, and most importantly, its $|E^*|$ values as a function of temperature. In such situations, the potential performance of the HMA mixture should be verified by comparing the $|E^*|$ value assumed in the pavement design with that developed by the mix design. These latter $|E^*|$ values can be determined in two ways. For pavements subject to relatively low traffic levels (below 3 million ESALs), estimated values for $|E^*|$

can be used; the HMA Tools spreadsheet can be used to provide such estimates at virtually any combination of frequency and temperature. For more critical pavements, an |E*| master curve should be measured and compared with the |E*| values assumed during development of the pavement design. If there are discrepancies, the mix design should be modified—in general, the most effective way of modifying |E*| values for an HMA mix design is to change the binder: the stiffer the binder, the higher the |E*| values will be. The HMA Tools modulus prediction tool can be used to estimate modulus early in the mix design process, even for critical mixtures that will eventually require |E*| testing. Thus, potential mix designs that do not provide proper |E*| values need not be evaluated further. As with other aspects of the MEPDG and related performance testing, there will likely be many differences in the way |E*| values will be used in both the pavement design and mix design process and differences in when these requirements will be implemented. Engineers and technicians responsible for mix design should refer to the appropriate state standards for details of verifying mix design modulus.

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CHAPTER 7

Selection of Asphalt Concrete Mix Type

The selection of an appropriate HMA mixture for a specific paving application is important in designing new pavements and for rehabilitation strategies for existing pavements. The type of mixture selected for the various layers of a pavement has a major effect on the cost, constructability, and long-term performance of the pavement. Mixtures with lower binder contents and lower quality aggregates are less expensive. To facilitate placement and compaction, thinner layers should be made with smaller nominal maximum aggregate size mixtures, while thick base layers should be made with larger nominal maximum aggregate sizes. Mixtures at the surface of a pavement should have relatively high binder content to make them more resistant to the damaging effects of traffic and the environment. Lower binder contents can be used in mixtures for intermediate and base courses because they are protected by the layers above them. Careful consideration of mix type is an important factor when staged construction is used, because the base or intermediate courses must serve temporarily as the surface during the first stages of construction.

This chapter provides recommendations for mixture type selection considering traffic, environment, constructability, and economics. It discusses the appropriate use of the three HMA mix types that can be designed using the procedures presented in this manual: dense-graded, gap-graded (GGHMA), and open-graded friction course (OGFC). Although the types of mixtures to be used in a project are usually selected during the design phase, it is important that mixture designers understand the rationale behind the selection of mixtures for specific applications. In some cases, the engineer responsible for a mix design may be asked to suggest a mix type for a given application. The recommendations presented in this chapter largely follow those contained the National Asphalt Pavement Association (NAPA) Publication IS 128, *HMA Pavement Mix Type Selection Guide.* The interested reader should refer to this publication for additional information concerning mixture type selection.

Pavement Structure and Construction

As discussed in Chapter 2, asphalt concrete pavements are engineered structures consisting of multiple layers or courses of hot-mix asphalt (HMA) and other materials. The structural HMA layers are usually referred to as surface, intermediate, and base courses depending on their location in the pavement structure. The intermediate course is sometimes called the binder course. Some pavements with higher traffic volumes may also include a wearing course composed of OGFC placed over the surface course. Each HMA layer in a pavement is composed of different materials and is placed in one or more lifts using separate paving operations. Each layer has a specific function that affects the type of mixture that should be specified and used. Figures 7-1 and 7-2 show typical cross sections for asphalt pavements commonly encountered in new construction and rehabilitation.

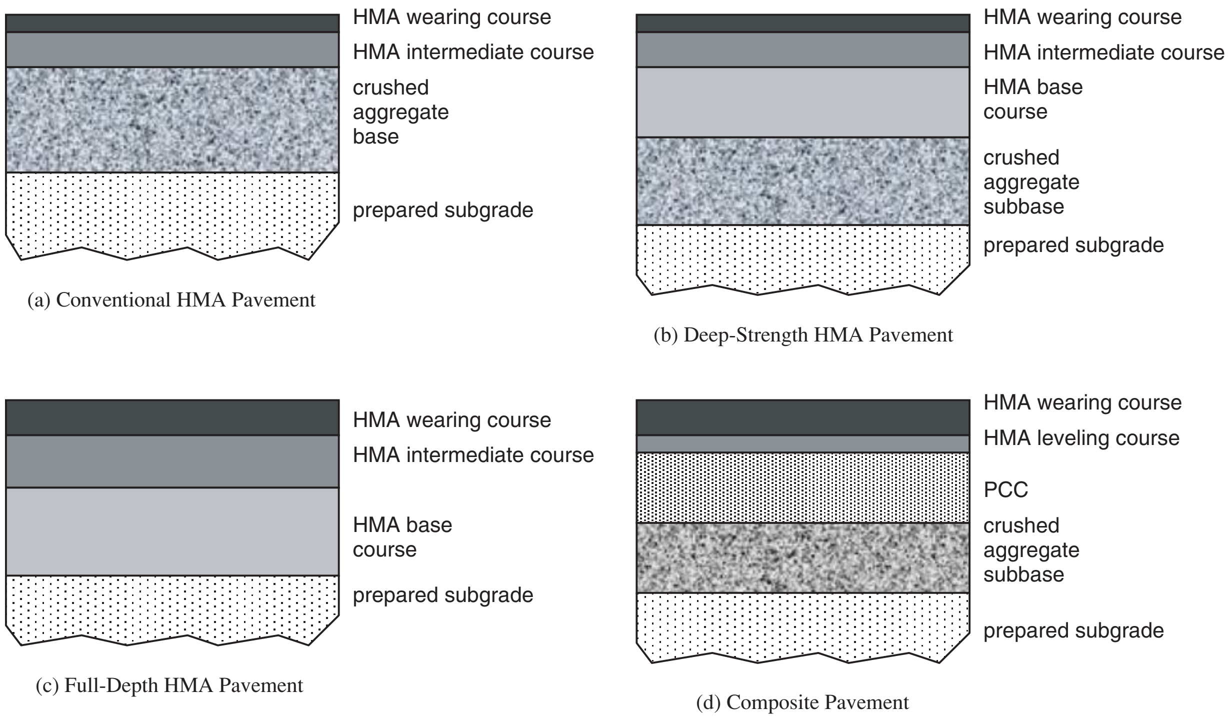

Figure 7-1. Cross-sections for typical asphalt pavements in new construction.

As shown in Figure 7-1, there are four types of new pavements depending on the type of base and the overall thickness of the HMA layers. Conventional flexible pavements, shown in Figure 7-1a, consist of relatively thin layers of HMA constructed over an unbound aggregate base. In this type of pavement, the unbound aggregate base is thick and is the major load-carrying element in the pavement. Conventional flexible pavements are primarily used on roadways with low traffic volumes. Flexible pavements that carry moderate to high traffic volumes are either deep-strength or full-depth. Deep-strength HMA pavements, shown in Figure 7-1b, have a relatively thick HMA base constructed on an unbound aggregate subbase, while in full-depth HMA pavements, shown in Figure 7-1c, all layers above the prepared subgrade are constructed with HMA. The HMA base is the primary load-carrying element in both of these pavement types. The unbound aggregate subbase in deep-strength HMA pavements provides a working platform for paving, and in some areas, additional thickness for frost protection. Composite pavements, shown in Figure 7-1d, consist of an HMA surface constructed on Portland cement concrete (PCC). The PCC is the primary load-carrying element in composite pavements. Composite pavements are constructed by design in some urban areas or during lane widening on PCC rehabilitation projects that include an HMA overlay where it is desired to maintain the same pavement cross section in the new lanes and the existing lanes.

Perpetual pavement, a relatively new concept, is intended to provide a pavement with a very long-lasting underlying structure combined with a durable wearing course. Ideally, the pavement structure should last 50 years or more without replacement, while the surface course might need replacement every 20 years. Selection of mixtures for perpetual pavements is discussed at the end of this chapter.

Pavement rehabilitation with HMA can result in two types of pavements as shown in Figure 7-2. Rehabilitation of existing asphalt pavements, shown in Figure 7-2a, is almost always accomplished using an HMA overlay. Prior to constructing the overlay, areas of the pavement that exhibit alligator or fatigue cracking must be repaired to full depth because the base of the existing pavement remains the primary load-carrying element in the flexible pavement after construction

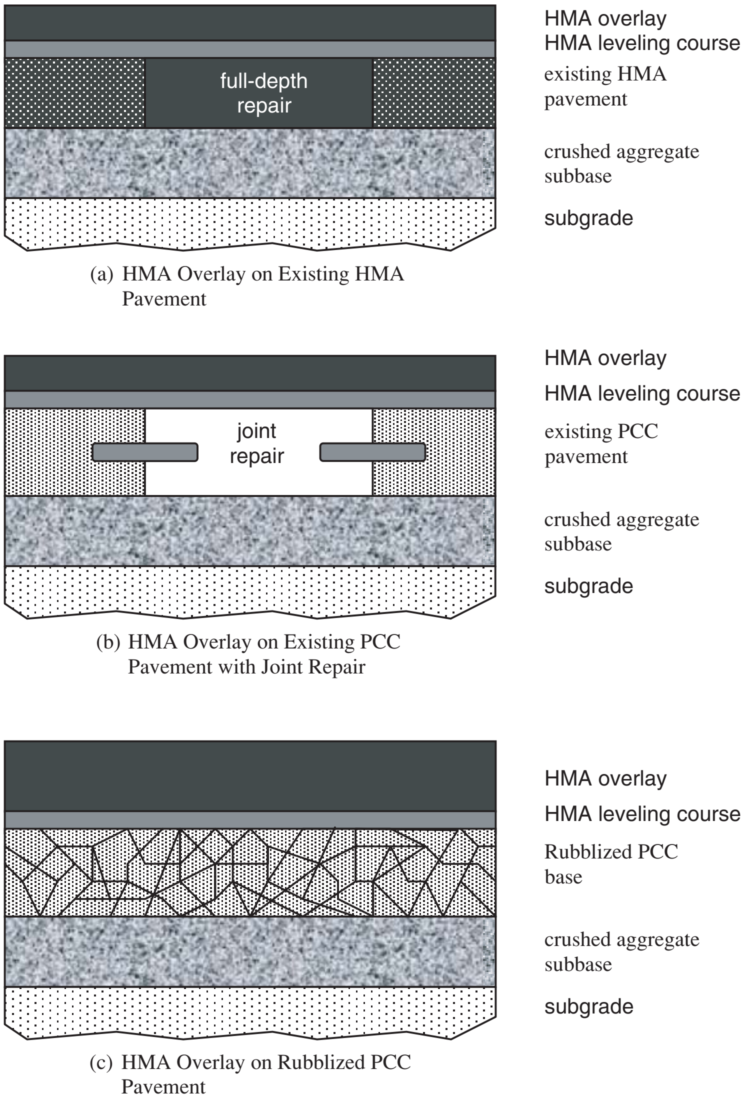

Figure 7-2. Cross-sections for typical asphalt pavements in rehabilitation.

of the overlay. If the existing surface course is in reasonably good condition, there is adequate vertical clearance, and safety hardware can accommodate an increase in pavement elevation, the overlay may be placed directly on the existing surface course. If the existing pavement includes an OGFC; the surface course is rutted, cracked or highly weathered; or it is important to maintain the existing elevation of the pavement, then the existing pavement is milled to an appropriate depth prior to placement of the overlay. A thin leveling or scratch course of variable thickness may be placed on the existing or milled pavement to improve smoothness prior to placing the HMA overlay. If strengthening is required due to an anticipated change in traffic volume, an intermediate course may also be added. Rehabilitation of existing PCC pavements with HMA involves placing one or more layers of HMA over the PCC. The HMA may be placed directly on the existing PCC, shown in Figure 7-2b, after repair of cracked PCC slabs and joints that exhibit poor load transfer. When the HMA is placed directly on the intact PCC, the PCC is the primary load-carrying element of the rehabilitated pavement. The HMA overlay is often saw cut at the location of the PCC joints to control reflective cracking in the HMA. The saw cuts are sealed at the time of construction. Alternatively, as shown in Figure 7-2c, the PCC slab may be broken or rubblized to control reflective cracking. In this case much thicker HMA layers are placed over the broken or rubblized PCC. The new HMA base serves as the primary load-carrying element in the rehabilitated pavement. A thin leveling course of variable thickness may be placed on the broken or rubblized PCC to improve smoothness prior to placing the HMA layers.

The sections that follow describe in greater detail the function and characteristics of each of the HMA layers shown in Figure 7-1 and 7-2. These characteristics are important factors in the selection of appropriate mixture types for each layer.

Surface Course

The surface course is the uppermost structural layer in an asphalt pavement. In most cases it is the top layer of the pavement and also serves as the wearing course. Since it is directly exposed to traffic and environmental forces, it must be produced with the highest quality materials. The surface course provides the following characteristics for an asphalt pavement:

- Adequate wet weather friction for safety
- High resistance to load-induced rutting, shoving, and surface cracking
- High resistance to thermally induced cracking
- Low permeability to minimize surface-water infiltration
- High durability to resist disintegration due to the combined effects of aging, traffic loading, and freeze-thaw effects
- Appropriate surface texture for noise control, safety, and aesthetics
- Smoothness.

Because the surface course is made with the highest quality materials, economics dictate that it be the thinnest pavement layer, typically 25 to 75 mm (1.0 to 3.0 in) thick. Surface course mixtures are typically only one lift thick and made with nominal maximum aggregate sizes of 12.5 mm or less. Smaller nominal maximum aggregate size mixtures can be placed in thinner layers, have higher binder contents, and, when compacted to the same in-place air void content, have lower permeability than larger nominal maximum aggregate size mixtures. Surface courses contain highly angular aggregates and an appropriate performance-graded binder to resist traffic and environmental forces. If the surface course is also the top layer in the pavement, then the aggregates must be resistant to polishing under traffic loading to provide appropriate skid resistance over the service life of the pavement. Dense-graded and GGHMA mixtures are commonly used as surface courses.

OGFC Wearing Course

Some moderate- to high-traffic pavements may include an OGFC as a wearing course on top of the surface course to improve skid resistance, reduce splash and spray, and reduce noise. These characteristics of OGFCs are the result of the open pore structure of these mixtures. OGFCs are made with durable crushed aggregates and often include modified binders and fibers to increase the binder content and improve durability. Because OGFCs are very permeable, the surface course directly beneath them must be impermeable to minimize infiltration of water into the pavement structure. To avoid trapping water in the pavement structure, OGFCs should be daylighted at the shoulders and milled from the pavement before placing future overlays.

Intermediate Course

The intermediate or binder course consists of one or more lifts of HMA between the surface and base courses. Not all pavements have an intermediate course; the need for an intermediate course depends on the overall thickness of the HMA and the thickness of the base and surface courses.

The purpose of the intermediate course is to add thickness to the pavement when additional structural capacity is required in new flexible pavements, rehabilitated asphalt pavements, and rubblized PCC pavements. An intermediate course may also be used in overlays of intact PCC pavement to provide additional thickness to delay reflective cracking or to provide an additional layer to improve pavement smoothness. Since intermediate courses are close to the surface of the pavement, they must be resistant to rutting. However, they can be constructed with mixtures having lower binder contents than surface courses because the intermediate course is not directly subjected to traffic loading or the damaging effects caused by water and oxidative hardening of asphalt binder. Binder courses are typically dense-graded mixtures with nominal maximum aggregate sizes of 19 or 25 mm.

Base Course

The base course consists of one or more lifts of HMA at the bottom of the pavement structure. The base course is the primary load-carrying element in deep-strength flexible pavements, full-depth flexible pavements, and rubblized PCC pavements. Because base courses are deep in the pavement structure, they do not have to be highly rut resistant. Base course mixtures should be relatively easy to compact to ensure that the base course is durable and resistant to bottom-up fatigue cracking. HMA base courses are typically dense-graded mixtures with nominal maximum aggregate sizes ranging from 19 to 37.5 mm.

Leveling Course

A leveling course is a thin layer of variable thickness used in rehabilitation to correct variations in the longitudinal or transverse profile of the pavement. They are referred to as scratch courses in some areas of the United States. Mixtures used for leveling courses are either 9.5- or 4.75-mm dense-graded mixtures to facilitate placement and compaction in thin layers.

Important Factors in Mix Selection

Several important factors should be considered when selecting an HMA mixture for a specific application. These include

- Traffic loading
- Rut resistance
- Fatigue resistance
- Durability
- Environment
- Lift thickness
- Appearance

Traffic Loading

Traffic loading, specifically the amount of truck loading, is a major factor affecting the design and performance of HMA pavements. Traffic loading is normally expressed as the number of 18,000 lb (80 kN) equivalent single-axle loads (ESALs) that the pavement is projected to carry over its design life. Traffic loading is a major factor in pavement structural design; it is used to determine the overall thickness of the pavement. The overall thickness of the pavement increases with increasing traffic loading. It is also a factor in the design of dense-graded mixtures and the selection of the high temperature binder grade for all mixtures. Higher traffic levels place greater demands on the HMA mixture used, particularly for surface and wearing courses. Mixtures

Table 7-1. Traffic levels for HMA mixture design (AASHTO M 323 and R 35).

Traffic Level, ESAL	Description
< 300,000	Applications include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways would be classified as local in nature, not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas may also be included at this level
300,000 to < 3,000,000	Applications include many collector roads or access streets. Medium-trafficked city streets and the majority of county roadways may be included at this level.
3,000,000 to <10,000,000	Applications include many two-lane, multilane, divided, and partially or completely controlled-access roadways. Among these are medium to highly trafficked city streets, many state routes, United States highways, and some rural Interstates.
10,000,000 to < 30,000,000	
≥ 30,000,0000	Applications include the vast majority of the U.S. Interstate system, both rural and urban in nature. Special applications such as truck-weigh stations or truck-climbing lanes on two lane roadways may also be included at this level.

designed for higher traffic loading must have greater resistance to both rutting and fatigue cracking. For dense-graded HMA mixture design, the five traffic levels listed in Table 7-1 have been defined. These traffic levels are also used in the recommendations for mixture type presented later in this chapter. Dense-graded mixtures can be used with all traffic levels. GGHMA and OGFC mixtures are more appropriate for pavements with moderate to high levels of traffic.

Rut Resistance

The required rut resistance of a mixture depends on the traffic level and the location of the mixture in the pavement structure. Pavements with higher traffic levels require greater rut resistance than pavements with low traffic volumes. Surface and intermediate layers require greater rut resistance than base layers. Rut resistance is a consideration in each of the design procedures presented in this manual. For dense-graded mixtures, aggregate angularity, binder grade, compactive effort, and some volumetric properties vary with traffic level and layer depth to provide adequate rut resistance. GGHMA and OGFC mixtures are designed to ensure stone-on-stone contact to minimize the potential for rutting. Binder grade for these mixtures is also selected considering environment and traffic level.

Fatigue Resistance

Another important consideration related to traffic loading is the resistance of the HMA mixture to fatigue cracking. As discussed in Chapter 2, two types of fatigue cracks have been identified in asphalt pavements: top-down and bottom-up. Thus, fatigue resistance is an important consideration for both surface and base course mixtures. Pavements with higher traffic levels require surface and base courses with greater resistance to fatigue cracking. One of the most important mixture design factors affecting fatigue resistance is the effective binder content of the HMA mixture. Fatigue resistance increases with increasing effective binder content; therefore, to resist top-down cracking, dense-graded mixtures of smaller nominal maximum aggregate size and GGHMA mixtures should be considered for high traffic levels. The dense-graded mixture design procedure presented in Chapter 8 provides the flexibility to increase the design VMA requirements up to 1.0% to produce mixtures with improved fatigue resistance and durability. Increasing the VMA requirement increases the effective binder content of these mixtures over that for normal dense-graded mixtures. The use of dense-graded mixtures with higher effective binder content should be considered for base courses in perpetual pavements. One of the structural design considerations for a perpetual pavement is that bottom-up fatigue cracking never occurs in the pavement.

Durability

Durability is the resistance of an HMA mixture to disintegration due to exposure to the combined effects of weathering and traffic. HMA surface and wearing courses have the most severe exposure, because they are subjected directly to damage by both traffic loading and the environment. The exposure for intermediate and base courses is less, except during staged construction when the intermediate or base layer may temporarily carry traffic for extended time periods. Mixtures subjected to more severe exposure conditions must have greater durability. *NCHRP Report 567* summarizes the relationships among HMA composition and performance; for the most durable mixes—ones with good fatigue resistance and low permeability to air and water—high binder contents are needed, along with a reasonable amount of fine material in the aggregate. Perhaps most importantly, the mix should be well compacted during construction. In general, both the binder content and the amount of fines in the aggregate blend will increase with decreasing aggregate nominal maximum aggregate size (NMAS). This is one of the reasons that smaller NMAS mixtures are used in surface courses. The effective binder content of GGHMA mixtures is very high due to the gap-graded structure of these mixtures. OGFC mixtures typically incorporate modified binders and fibers to increase the binder content of these mixtures and improve their durability.

Environment

Environment is a direct consideration in each of the design procedures presented in this manual. The environment in which the pavement will be constructed determines the performance grade of binder that will be used for all mixture types. When considering an OGFC as a wearing course in freezing climates, it is important to recognize that these surfaces may require somewhat different winter maintenance practices. The open structure of OGFCs causes these mixtures to freeze more quickly than dense-graded and GGHMA mixtures, resulting in the need for earlier and more frequent application of deicing chemicals. Additionally, sand should not be used with the deicing chemicals because the sand will plug the pores of the OGFC, decreasing their effectiveness.

Lift Thickness

Proper compaction of HMA is critical to its long-term performance. Unfortunately, many design engineers consider compaction to be a detail to be worked out by the paving contractor at the time of construction. Adequate compaction may not be possible if lift thickness is not properly considered during pavement design and mixture selection. NCHRP Project 9-27 included field studies to evaluate the effect of lift thickness on the density and permeability of HMA layers. One of the recommendations of this study, as given in *NCHRP Report 531,* is that the ratio of the lift thickness to nominal maximum aggregate size be 3.0 to 5.0 for fine, dense-graded mixtures and 4.0 to 5.0 for coarse, dense-graded mixtures and GGHMA. OGFCs are typically constructed 19 to 25 mm (¾ to 1 in) thick. Table 7-2 summarizes the recommendations given in *NCHRP Report 531* considering HMA lift thickness.

Appearance

In some cases, the appearance of the surface is an important consideration. Mixtures with larger aggregate sizes have coarser surface textures, which may not be appropriate for some applications like city streets.

Table 7-2. Recommended lift thicknesses as given in *NCHRP Report 531.*

Mixture Type	Minimum Ratio of Lift Thickness to Nominal Maximum Aggregate Size	Maximum Ratio of Lift Thickness to Nominal Maximum Aggregate Size
Fine, Dense-Graded	3.0	5.0
Coarse, Dense-Graded	4.0	5.0
GGHMA	4.0	5.0

Recommended Mix Types

This manual presents detailed design procedures for three types of HMA mixtures: dense-graded, GGHMA, and OGFC. Table 7-3 presents recommended mixture types based on traffic level and layer.

Dense-Graded

Dense-graded HMA mixtures are the most commonly used mixtures in the United States. They can be used in any layer of the pavement structure for any traffic level. Traffic level is a direct consideration in the design of dense-graded mixtures. Aggregate angularity, clay content, binder grade, compactive effort, and some volumetric properties vary with traffic level in the dense-graded mixture design procedure.

Dense-graded mixtures also provide the mixture designer with the greatest flexibility to tailor the mixture for the specific application. The dense-graded mixture design procedure presented in Chapter 8 provides the flexibility to increase the design VMA requirements up to 1.0% to produce mixtures with improved fatigue resistance and durability. Increasing the VMA requirement increases the effective binder content of these mixtures over that for normal dense-graded mixtures. The use of higher effective binder content dense-graded mixtures should be considered for surface and base layers when the traffic level exceeds 10,000,000 ESALs.

Dense-graded mixtures can also be designed as fine or coarse mixtures. Fine mixtures generally have a gradation that plots above the maximum density line while coarse mixtures plot below the maximum density line. The definition of fine and coarse mixtures used in AASHTO M 323 is summarized in Table 7-4. For each nominal maximum aggregate size, a primary control sieve has been identified. If the percent passing the primary control sieve is equal to or greater than the specified value in Table 7-4, the mixture classifies as a fine mixture; otherwise it classifies as a coarse mixture. Fine mixtures have smoother surface texture, lower permeability for the same in-place density, and can be placed in thinner lifts than coarse mixtures.

Table 7-3. Recommended HMA mixture types.

Traffic Level, ESAL	Surface		Intermediate		Base		Leveling	
	Mix Type	NMAS, mm[a]	Mix Type	NMAS, mm[a]	Mix Type	NMAS, mm[a]	Mix Type	NMAS, mm
< 300,000	Dense-graded	4.75, 9.5	Dense-graded	19.0, 25.0	Dense-graded	19.0, 25.0, 37.5	Dense-graded	4.75, 9.5
300,000 to < 3,000,000	Dense-graded	4.75, 9.5	Dense-graded	19.0, 25.0	Dense-graded	19.0, 25.0, 37.5	Dense-graded	4.75, 9.5
3,000,000 to <10,000,000	Dense-graded	9.5, 12.5	Dense-graded	19.0, 25.0	Dense-graded	19.0, 25.0, 37.5	Dense-graded	4.75, 9.5
10,000,000 to < 30,000,000	Dense-graded[b, c] GGHMA	9.5, 12.5 9.5, 12.5	Dense-graded	19.0, 25.0	Dense-graded[b]	19.0, 25.0, 37.5	Dense-graded	4.75, 9.5
≥ 30,000,0000	Dense-graded[b, c] GGHMA	9.5, 12.5 9.5, 12.5	Dense-graded	19.0, 25.0	Dense-graded[b]	19.0, 25.0, 37.5	Dense-graded	4.75, 9.5

[a]Select nominal maximum aggregate size to meet requirements of Table 7-2

[b]Consider increasing design VMA by 1.0%

[c]May add OGFC wearing course on pavements with high-speed traffic

Table 7-4. Definition of fine, dense-graded HMA mixtures (AASHTO M323).

Nominal Maximum Aggregate Size	Primary Control Sieve	Percent Passing
37.5 mm	9.5 mm	≥ 47
25.0 mm	4.75 mm	≥ 40
19.0 mm	4.75 mm	≥ 47
12.5 mm	2.36 mm	≥ 39
9.5 mm	2.36 mm	≥ 47

GGHMA

GGHMA is a gap-graded, densely compacted HMA designed to maximize rut resistance and durability. The principal design consideration in GGHMA is to maximize the contact between particles in the coarse aggregate fraction of the mixture. This fraction provides stability and shear strength to the mixture. The coarse aggregate fraction is then essentially glued together by a binder-rich mastic consisting of a properly selected asphalt binder, mineral filler, and fibers. The fibers are included to stabilize the mixture during handling and placement. The advantages of GGHMA mixtures over dense-graded mixtures include (1) increased resistance to permanent deformation, cracking, and aging and (2) improved durability, wear resistance, low-temperature performance, and surface texture. GGHMA mixtures generally cost more than dense-graded mixtures due to their higher binder content, high filler content, stringent aggregate requirements, and the use of polymer-modified binders and fibers. GGHMA should be considered for surface courses when the traffic level exceeds 10,000,000 ESALs. The design of GGHMA mixtures is discussed in Chapter 10.

Open-Graded Friction Course (OGFC)

OGFC is a gap-graded mixture with a high air void content. The high air void content and open structure of the mixture provides macrotexture and high permeability to drain water from the tire-pavement interface. This minimizes the potential for hydroplaning, improves wet weather skid resistance, and reduces splash and spray. Other benefits of OGFC include reduced noise levels, improved wet weather visibility of pavement markings, and reduced glare. OGFCs are made with durable, polish-resistant aggregates and usually contain modified binders and fibers to increase the binder content and improve their durability. OGFCs generally cost more than dense-graded mixtures. An OGFC may be considered as a wearing course on high-speed pavement sections when traffic levels exceed 10,000,000 ESALs. High-speed traffic is an important consideration because it helps keep the pores from clogging with debris. The design of OGFC mixtures is discussed in Chapter 11.

Materials Selection for Perpetual Pavements

As discussed in the introduction to this chapter, perpetual pavements are intended to provide an exceptionally long service life—about 20 years for the surface course and 50 years or more for the underlying pavement layers. Figure 7-3 illustrates the typical structure of a perpetual pavement. The base material should be flexible and fatigue resistant, meaning it should be designed as either a 9.5-mm or 12.5-mm NMAS mixture. Improved fatigue resistance will usually be obtained through the use of fine aggregate gradations and increased asphalt binder content—this means increasing the target VMA by 0.5 to 1.0% over typical design values for the given aggregate size. The high temperature asphalt binder grade for the base material should be high enough to prevent any rutting, but no higher. Otherwise the fatigue resistance of the material might be compromised. The low temperature binder grade should, in general, be one grade higher than that required at the surface.

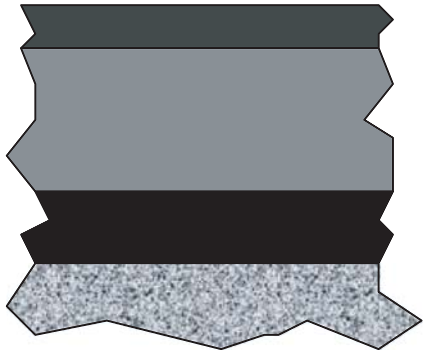

Figure 7-3. Typical structure for perpetual pavement.

The intermediate layer should be a strong, rut-resistant mixture. Although in the past it was believed that relatively coarse-graded mixtures with large NMAS provide optimum rut resistance, more recent research has suggested that equal or even better rut resistance can be obtained using fine-graded mixtures with 9.5- or 12.5-mm NMAS aggregate gradations. Selection of the mix type should be based on obtaining the best rut resistance at a minimum cost. This can probably be best achieved in most cases with a standard, dense-graded HMA mixture. The high temperature binder grade for this layer should be the same as that required for the surface mixture. To ensure that the intermediate layer has a high modulus, the low temperature binder grade should be one grade higher than that used for the surface mixture.

Selection of mix type for the surface coarse mixture will depend on the traffic level. For very heavy traffic levels, GGHMA mixtures will provide the best performance and greatest assurance of a long pavement life. At intermediate to high traffic levels, carefully designed dense-graded HMA mixtures should perform well. Normal procedures for binder grade selection should be followed in designing the HMA for the surface course of a perpetual pavement.

Engineers and technicians performing mix designs for perpetual pavements should keep in mind that this is a relatively new technology that is likely to undergo changes in the near future. The Asphalt Alliance currently maintains a very useful website providing up-to-date information on perpetual pavements. Additional information on perpetual pavements can also be found in *TRB Circular 50: Perpetual Bituminous Pavements*

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CHAPTER 8

Design of Dense-Graded HMA Mixtures

This chapter presents a comprehensive procedure for the design of dense-graded HMA mixtures. Although the procedures described have been specifically selected for use in designing dense-graded mixtures, most can be applied to the design of other mix types with little or no modification. Before reading this chapter, engineers and technicians should make certain they understand the information presented in earlier chapters of this manual. Many steps in the procedure are covered in other chapters of the manual. For example, binder tests and grading are discussed in Chapter 3, aggregate properties and specifications in Chapter 4; Chapter 5 is a detailed discussion of the volumetric composition of HMA; and Chapter 6 deals with the performance of HMA, including tests for evaluating rut resistance and fatigue resistance.

To keep this chapter as simple and direct as possible, the details of handling RAP in an HMA mix design are not addressed here but are covered in Chapter 9. Engineers and technicians using RAP in their mix designs should make sure they read Chapter 9 carefully and understand the proper procedures for incorporating RAP into HMA mixtures. Chapter 6 presents an in-depth discussion of performance testing of HMA mixtures, including background information important in understanding how and why many of these procedures were developed. In Chapter 8, specific tests recommended for use in HMA mix design are summarized and the specific conditions suggested for running these tests are described. Where appropriate, standard test methods are given so that technicians and engineers can refer to them for details on each procedure.

This chapter begins with a short discussion of the history of HMA mix design, including comments on the Marshall, Hveem, and Superpave procedures. The suggested mix design procedure is then summarized, followed by a detailed, step-by-step presentation. Specific examples are included at several points in the discussion. Frequent reference is also made to the HMA Tools spreadsheet, which includes provisions for performing most of the calculations needed in the mix design process.

Other Mix Design Methods

Three HMA mix design methods have been widely used in the United States and Canada during the past 60 years: the Marshall method of mix design, the Hveem method, and the Superpave method of mix design and analysis. The Marshall and Hveem methods were largely developed in the 1940s and were the first systematic and widely used methods of HMA mix design. The Superpave system, developed during the late 1980s and early 1990s, was intended to improve on the Marshall and Hveem procedures. (Leahy and McGennis wrote an excellent article summarizing the evolution of HMA mix design systems, "Asphalt Mixes: Materials, Design and Characterization," which appears in Volume 68A (1999) of the *Journal of the Association of Asphalt Paving Technologists.*) The sections below provide a brief background on these other mix design methods, in order to help provide perspective to the procedure described later in this chapter. Of special significance is the Superpave system, which is the basis for the mix design method presented in this manual. Because of this close relationship, the Superpave system is described in greater detail than the other two mix design methods.

Marshall Mix Design

The Marshall method of HMA mix design was originally developed by Bruce Marshall in the 1940s, while he was working for the Mississippi State Highway Department. The procedure was later adopted and further refined by the U.S. Army Corps of Engineers (USACE). A wide range of engineers and organizations have proposed improvements and variations to this design procedure; publications of the Asphalt Institute are considered by many to be the best references for this and many other mix design methods.

There are four primary features of the Marshall method:

1. Asphalt binders and aggregates should be selected to meet all applicable project specifications.
2. Evaluation of trial mixtures is done using laboratory-compacted specimens 100 mm in diameter by approximately 70 mm thick, compacted using a standardized drop hammer (see Figure 8-1).
3. Laboratory-compacted specimens must meet requirements for air void content and VMA, and, in some cases, VFA.
4. Laboratory-compacted specimens must also meet requirements for stability and flow—properties related to strength and flexibility that are determined in a quick and simple mechanical test.

The specific requirements for air void content, VMA, VFA, and stability and flow varied over time and from agency to agency. In the Asphalt Institute's publication *Mix Design Methods for Asphalt Concrete and Other Mix Types* (MS-2), the requirements are as follows:

- Compaction level (number of "blows") varies with traffic level. For light traffic, the specified compaction level is 35 blows; for medium traffic, 50 blows; and for heavy traffic, 75 blows.

Figure 8-1. Marshall compaction hammer.

- Design air void content ranges from 3 to 5%.
- Minimum values for VMA depend upon nominal maximum aggregate size; for a 9.5-mm mix, the minimum VMA is 14% for an air void content of 3%, 15% for an air void content of 4%, and 16% for an air void content of 5%. For a 12.5-mm mix, minimum VMA values are 1% lower; for a 19-mm mix, minimum VMA values are 2% lower. As aggregate size increases, minimum VMA decreases.
- The allowable range for VFA depends on traffic level (light, medium or heavy). For light traffic, allowable VFA ranges from 70 to 80%; for medium traffic, the allowable range is from 65 to 78%; for heavy traffic, the allowable range is from 65 to 75%.
- Stability and flow values also depend on traffic level. Minimum stability values are 3340 N for light traffic, 5340 N for medium traffic, and 8010 N for heavy traffic. The allowable range for flow (in flow units of 0.25 mm) is 8 to 18 for light traffic, 8 to 16 for medium traffic, and 8 to 14 for heavy traffic.

An important aspect of the Marshall design method is compaction of laboratory specimens over a range of asphalt binder contents and evaluation of the mixture volumetrics over this entire range in order to determine the optimum binder content. Originally, Bruce Marshall recommended producing HMA mixtures at the lowest possible VMA, since this produced the densest, most stable mixtures and required the lowest asphalt contents. However, engineers eventually realized that such mixtures often exhibited durability problems, and minimum VMA values such as those given by the Asphalt Institute were established. Although the current Asphalt Institute version of the Marshall method does not explicitly specify maximum values for VMA, the combination of specifying air void content and VFA in fact indirectly establishes such maximums. For example, at 4% air voids, a maximum VFA value of 75% implies a maximum VMA value of 16%.

The Marshall design method was widely used in the United States and Canada through the early 1990s, at which point the Superpave Method of Mix Design and Analysis (the Superpave system) began replacing it. At the writing of this manual, the FAA and most United States military organizations were changing their mix design methods from the Marshall method to the Superpave system. For example, *Airfield Asphalt Pavement Technology Program (AAPTP) Project 04-03,* started in 2007, funded by the FAA, involves developing an implementation plan for the Superpave mix design system for airfield pavements.

Hveem Mix Design Method

The Hveem method of mix design was developed at about the same time as the Marshall method, by Francis Hveem, who was at the time a materials and research engineer for the then California Department of Highways. The Hveem method was not as widely used as the Marshall method, but was used by many highway departments in the Western United States until the Superpave method became the generally accepted method for HMA mix design.

There are several important features of the Hveem method:

1. Like the Marshall method, asphalt binder and aggregates should meet all applicable project specifications.
2. Evaluation of trial mixtures is done using the same size specimens as those used in the Marshall method (100 mm diameter by 70 mm thick), but the specimens are compacted using a kneading compactor, rather than a Marshall drop hammer.
3. The asphalt binder content for trial mixtures is estimated using the centrifuge kerosene equivalent (CKE) test and a series of charts. This is a somewhat complicated procedure; a detailed description can be found in the Asphalt Institute's *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types* (MS-2).

4. Laboratory-compacted specimens must meet requirements for the stabilometer and swell tests, and sometimes the cohesiometer test.
5. Although volumetric analysis is recommended, the Hveem method does not include specific requirements for air void content, VMA, VFA, etc.

The Superpave System

The Superpave Method of Mix Design and Analysis was developed during the Strategic Highway Research Program (SHRP) and was intended to be an improvement over the Marshall and Hveem procedures. The Superpave method was meant to be based on engineering principles and performance-related properties. The Asphalt Institute publishes an excellent manual covering the Superpave system, *Superpave Mix Design* (SP-2). Several AASHTO standards deal with the Superpave system of mix design, the most important being AASHTO R 35, Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt, and AASHTO M 323, Standard Specification for Superpave Volumetric Mix Design. In many ways, the Superpave system can be viewed as an evolution of the Marshall method of mix design. Like the Marshall method, it places much emphasis on volumetric analysis and requirements for air void content, VMA, and VFA. In fact, the specific volumetric requirements for the Superpave system and for the current Marshall method are quite similar. However, as discussed below, the Superpave system is more comprehensive than either the Marshall or the Hveem method of mix design and includes important innovations.

The five primary features of the Superpave system are as follows:

1. Selection of the asphalt binder grade is made on the basis of local climate and expected traffic level. Binder grading is done using a performance-based system of tests and specification requirements, commonly, but somewhat redundantly, referred to as "PG grading."
2. Aggregate gradations are given for each aggregate size (NMAS), ranging from 4.75 to 37.5 mm. In the early versions of the Superpave system, the aggregate gradations included a restricted zone—a region that should be avoided to ensure against tender mixes—but the most recent Superpave standard (AASHTO M 323-07) no longer includes this feature.
3. Evaluation of trial mixtures is done on laboratory-compacted specimens 150 mm in diameter by about 100 mm thick. These specimens are compacted using the Superpave gyratory compactor (see Figure 8-2). As in the Marshall method, the level of compaction varies with the expected traffic level.
4. Trial mixtures are evaluated on the basis of volumetric composition, with requirements for design air void content, VMA, and VFA.
5. All mixtures must be evaluated for moisture resistance using a standard test (AASHTO T 283). Unlike the Marshall method, there is no final test of stability, flexibility, or strength.

The Superpave mix design process is generally described as consisting of four primary steps:

1. Selecting materials
2. Selecting the design aggregate structure
3. Selecting the design binder content
4. Evaluating moisture resistance

The selection of the design aggregate structure is based on determining an estimated design binder content and preparing test specimens using three different aggregate gradations. The gradation giving a mixture composition closest to the given requirements is selected for continued refinement, the first step of which is the selection of the design asphalt content. This is done by preparing specimens with the design aggregate gradation, using a range of asphalt contents. The design asphalt content is that giving an air void content of 4%. Moisture resistance is evaluated using AASHTO T 283. This procedure is discussed both in Chapter 6 of this manual and toward the end of this chapter.

Figure 8-2. Superpave gyratory compactor *(courtesy of Pine Equipment Company).*

Procedures for volumetric analysis and requirements for air void content, VMA, and VFA are similar to those used in the Marshall method. For example, for a 9.5-mm mix, the minimum VMA value is 15%, exactly the same as the minimum VMA for a 9.5-mm Marshall mix designed at 4% air voids. As in the current version of the Marshall method, VFA values depend on traffic level—at higher traffic levels, the allowable range for VFA in the Superpave system is 65 to 75%.

Compaction of specimens in the laboratory and requirements for compaction properties are more complicated in the Superpave system than for the Marshall method. Currently, three compaction points are defined during the compaction process: $N_{initial}$, N_{design}, and N_{max}, although some engineers and researchers have recently questioned the need for $N_{initial}$ and N_{max}, and these requirements might be eliminated in the near future. Within the original Superpave system a maximum value of density at $N_{initial}$ was specified in order to help ensure proper aggregate structure, since it was believed that mixtures that compacted too quickly had poor aggregate structures. N_{design} was (and still is) the actual point at which air void content, VMA, and VFA are specified—the design compaction level. N_{max} is the maximum number of gyrations applied to the specimen. A maximum density at N_{max} was specified in the original Superpave system in order to ensure that the mix would remain stable at the expected maximum traffic level.

From the mid 1990s to the mid 2000s, the Superpave system gradually replaced the Marshall and Hveem methods of mix design in most highway agencies. However, as it was being adopted, many engineers thought that the specific requirements given in the Superpave system were not providing HMA mixtures with the best performance for their local conditions and climates. For this reason, many state highway agencies have modified some of the requirements for HMA mixtures designed using the Superpave system. Many engineers have also criticized the Superpave system for its lack of a "proof" test, that is, a final test of strength or stability, similar to the Marshall stability and flow test. Since the initial implementation of Superpave, significant research has been done on various aspects of HMA mix design and analysis, suggesting that a number of changes can be made in the Superpave system that would improve the performance of HMA mixtures. Perhaps most importantly, the experience of many engineers suggested that Superpave mixtures were difficult to compact and often exhibited only fair to poor durability. For example, the Virginia Transportation Research Council in *Report VTRC 03-R15* concluded that Superpave mixes often lack sufficient binder content for adequate durability. After the 2003 paving season, New Jersey formed a task force on Superpave durability because of concern about raveling, segregation, and the generally dry appearance of Superpave mixes.

Despite the possible shortcomings of the Superpave system, the mix design procedure presented here is not meant to replace the Superpave system but to build on it—correcting some of its shortcomings and incorporating the findings of recent research dealing with HMA mix design and performance. Many terms have been borrowed from the Superpave system and have identical or very similar meanings in this manual as they do within the Superpave system: N_{design}, binder performance grades, aggregate gradation control points, coarse and fine aggregate angularity. There are, however, several important differences between the Superpave method and the mix design method described in this manual. The Superpave method deals only with the design of dense-graded HMA mixes, while this manual addresses not only dense-graded HMA, but also stone-matrix asphalt (SMA) mixes (Chapter 10) and open-graded friction course (OGFC) mixes (Chapter 11). In the design of dense-graded HMA, there are three important differences between Superpave system and the mix design method described in the remainder of this chapter:

1. Developing mix designs containing RAP is addressed directly and thoroughly (see Chapter 9); the original Superpave system did not address RAP directly, although various modifications to address RAP usage were developed and implemented over the past few years.
2. In the Superpave method, one of the steps of the mix design process is selecting the design asphalt binder content for the mixture. This involves preparing three or four mixtures with the selected aggregate structure (gradation) over a range of binder contents and determining which best meets the mix specifications. In the method described below, the design asphalt content is determined early in the procedure and maintained throughout the design process. To meet the requirements for VMA and air void content, the aggregate gradation is varied, rather than the binder content. This emphasizes the importance of the design binder content and helps ensure that the final mix has the proper amount of binder. It also prevents unnecessary work in preparing and evaluating trial mixes that do not contain the proper amount of asphalt binder.
3. The Superpave method included no provisions for a final performance or "proof" test, such as the Marshall stability and flow. The method described below includes three different options for evaluating the rut resistance of dense-graded HMA mixtures as a final step in the mix design process.

Overview of Design Method

The method described below for designing HMA mixtures is similar to the Superpave method, but uses a larger number of simpler steps as follows:

1. Gather Information
2. Select Asphalt Binder
3. Determine Compaction Level
4. Select Nominal Maximum Aggregate Size
5. Determine Target VMA and Design Air Void Content
6. Calculate Target Binder Content
7. Calculate Aggregate Content
8. Proportion Aggregates for Trial Mixtures

9. Calculate Trial Mix Proportions by Weight and Check Dust/Binder Ratio
10. Evaluate and Refine Trial Mixtures
11. Compile Mix Design Report

Most of these steps are straightforward and easily accomplished. However, Steps 8 through 10 are more complicated and require some experience in order to perform them proficiently. The suggested procedures for performing each of these steps are described below, followed by an example.

Step 1. Gather Information

At the beginning of the mix design process, the technician must gather as much information as possible bearing on the mix design, including the design traffic level, the climate at the place of construction, information on available aggregates and binders, anticipated lift thickness, pavement type (that is, surface, intermediate, or base course), and any special issues pertaining to the mix design or pavement construction. Unfortunately, this information is often incomplete, and the technician must use her or his judgment to fill in the gaps. Frequently, the organization requesting the mix design will provide specific information concerning the aggregates and binders to be used, eliminating these steps from the mix design process and making the process of gathering information somewhat simpler.

Generally, the information below is often useful during the mix design process:

- Site Information
  - Geographic Location
  - Climate Relating to Binder Grade
  - Design Traffic Level
- Construction Information
  - Lift Thickness
- Pavement Information
  - Mix type (dense-graded, SMA, or OGFC)
  - Distance from Pavement Surface to Top of Layer
- Materials Information
  - Information on Available or Recommended Aggregates
    - Nominal maximum aggregate size
    - Gradation
    - Specific gravity and absorption
    - Pertinent specification properties
  - Information on Available or Recommended Asphalt Binders
    - Performance grade (PG)
    - Specification test data, including mixing and compaction temperatures
    - Type of modification, if applicable
    - Other pertinent specification properties
  - Information on Other Mix Materials
    - Anti-strip additives
    - Properties of recycled asphalt pavement (RAP)
    - Properties of other additives such as ground rubber or fibers
- Special Issues Pertaining To Mix Design
  - Unusual Specification Requirements
  - Use of Special Additives or Recycled Materials
  - Unusual Construction Constraints
  - Unusual Performance Constraints

In HMA Tools, the worksheet "General" should be filled out with information about the mix design. This includes the date, the name of the engineer or technician performing the mix design, the project name, the aggregate size (NMAS), the required binder grade, the target VMA, and the target air void content. Completion of this worksheet is essential, because much of this information is used in other parts of HMA Tools.

Step 2. Select Asphalt Binder

Details concerning the properties and specification of asphalt binders are presented in Chapter 3 of this manual; what is presented here is a concise description of the grade selection process. Details of the performance grading system are given in AASHTO M 320, Performance-Graded Asphalt Binder. Engineers and technicians unfamiliar with the performance grading system should review Chapter 3 and AASHTO M 320 before attempting to understand HMA mix design as presented in this manual.

Selection of the asphalt binder grade in an HMA mix design will generally depend on five factors:

1. The base performance grade dictated by the climate
2. The grade adjustment required for traffic level and speed
3. The performance grade specified by the state highway agency or other authority
4. The effect of any RAP in the mix on the final effective performance grade of the binder
5. The available asphalt binders.

In most cases, the performance grade will be specified by the state highway agency or other owner/agency and will not be selected during the mix design process. However, private clients will often leave binder selection up to the contractor or HMA supplier. Engineers and technicians involved in the design of HMA mixes should keep in mind that in many cases the available binder grades in a given geographic area are limited, given that state highway agencies tend to specify a limited number of binder grades, often referred to as a "binder slate." For example, a state might specify only PG 58-28, PG 64-22, and PG 76-22 binder grades, even though some areas could use a PG 70-22 binder grade—in these locations, a PG 76-22 binder grade would be used. This is done to simplify the mix design process, and to limit the number of binders that must be produced by refineries or delivered to local terminals. In those special cases demanding a binder grade outside those normally available, the binder can be ordered, but the cost might be higher compared to binders within the standard slate.

In those cases where the binder grade must be determined as part of the mix design process, the required final performance grade will depend on five factors:

1. Local climate, dictating the base performance grade
2. Design traffic level
3. Average traffic speed
4. The amount of RAP added to the mix
5. The performance grade of the RAP binder

Details on handling RAP in HMA mix designs are presented in Chapter 9 of this manual; what follows is a brief discussion of the effect of RAP on binder performance grade selection. The essential concept that must be understood is that RAP contains significant amounts of asphalt binder, and this binder has usually undergone significant age hardening. Therefore, adding RAP to an HMA mixture tends to increase the effective grade of the binder in the final mix. For example, an HMA mix made with a PG 64-22 binder and 25% RAP might have an effective binder grade of PG 70-16. If the required grade for the mix is a PG 64-22, a softer binder grade must then be added to this mix—say, a PG 58-28 rather than a PG 64-22, so that the blend of the new binder and RAP binder meets the requirements of a PG 64-22.

The effect of RAP on the final effective binder grade can be handled in three different ways. In the simplest case, if the RAP content is 15% or less, no adjustment is made to the performance grade of the new binder added to the HMA. At higher RAP contents, two approaches can be used in making sure the final effective binder grade meets the given requirements. In the first approach, the desired amount of RAP is selected, and the performance grade of the new binder to be added to the mix is determined so that the final effective performance grade meets the established performance grade requirements. The second approach is to select the performance grade of the new binder and then determine the minimum and maximum amounts of RAP that can be added while still meeting the established performance grade requirements for the final effective binder grade. Either approach can be used with HMA Tools, which gives both the final effective binder grade for a given RAP content and the new binder performance grade, and also the minimum and maximum allowable RAP content for a given new binder performance grade and RAP stockpile or blend of RAP stockpiles. A minimum RAP content requirement can occur when the selected new binder performance grade is softer than the required grade, so that the addition of RAP is needed to effectively stiffen the asphalt binder. For example, a given location might require a PG 70-22 binder, but a PG 64-28 binder is selected because it is expected that the addition of RAP to the mixture will stiffen the final effective binder grade. However, in such a situation, the technician must make sure that enough RAP is actually added to the mix so that the effective binder meets the final grade requirement of PG 70-22.

An important issue related to the effect of RAP on binder grade is the effect of RAP on the variability of various mix properties. In some cases, the maximum amount of RAP that can be added to an HMA mix will be limited by variability, rather than by the final binder performance grade. This issue is discussed in detail in Chapter 9.

The FHWA's software program, LTPPBind, provides information on climate, base performance grade, and performance grade adjustments for traffic and speed. As of September 2008, information on LTPPBind, including a free download of the program, could be found at http://www.fhwa.dot.gov/pavement/ltpp/ltppbind.cfm. One limitation to the current LTPPBind, Version 3.1, is that only fast and slow traffic speeds are addressed, and the specific speeds in kph corresponding to these categories are not given, although it appears that fast traffic corresponds to an average speed of about 70 kph, and slow traffic to a speed of about 35 kph. Performance grade adjustments for very slow traffic are not addressed. Fortunately, recent research published in *NCHRP Report 567: Volumetric Requirements for Superpave Mix Design* has provided a model relating binder performance grade, compaction level (N_{design}), traffic level, traffic speed, and mix composition to rutting. This model was used to develop high-temperature performance grade adjustments, which are given in Table 8-1. These grade adjustments differ only slightly from those given in LTPPBind

Table 8-1. High temperature binder grade adjustments for traffic level and speed.

Design Traffic (MESALs)	*Grade Adjustment for Average Vehicle Speed in kph (mph):* Very Slow	Slow	Fast
	< 25 (< 15)	25 to < 70 (15 to < 45)	≥ 70 (≥ 45)
< 0.3	---	---	---
0.3 to < 3	12	6	---
3 to < 10	18*	13	6
10 to < 30	22*	16*	10
≥ 30	---	21*	15*

** Consider use of polymer-modified binder. If a polymer-modified binder is used, high-temperature grade may be reduced one grade (6 °C), provided rut resistance is verified using suitable performance testing.*

for fast traffic. However, the assumed slow traffic speed in Table 8-1 is somewhat slower than that apparently used in LTPPBind and the adjustments are correspondingly larger. Also, as mentioned above, Table 8-1 includes adjustments for very slow traffic—a speed category not included in LTPPBind v. 3.1. Theoretically, high-temperature performance grades could be reduced for faster traffic at the lowest design traffic level (< 0.3 MESALs). However, construction of such pavements will often be poorly controlled and the use of softer binder grades could result in rutting, shoving, and flushing in many cases. No grade adjustments for very slow speed are given for the highest traffic level because such a combination of traffic speed and volume cannot occur—if the traffic on a road is very slow, the volume cannot be extremely high because the vehicles are not moving at a fast enough speed. This brings up another important point in applying Table 8-1 and similar grade adjustments, such as those given in LTPPBind: the traffic speeds are average speeds for the road, not minimum speeds. Average speeds should be those determined from traffic studies or other objective procedures, not personal judgment. A final difference between Table 8-1 and the traffic and speed adjustments given in LTPPBind is that the adjustments given in Table 8-1 are for binders that are not polymer modified; as explained in the note for the table, the high-temperature performance grade requirement can be reduced by one grade (6°C) if a polymer-modified asphalt binder is used, and if the mix successfully meets appropriate performance testing requirements, as discussed later in this chapter.

A second important high-temperature grade adjustment must be made for temporary construction. This is necessary for two reasons: (1) lack of age hardening and (2) greater potential for extremely hot weather. A pavement carrying 20 million ESALs over 20 years will experience 1 million ESALs per year. During the first 2 to 3 years, such a pavement will undergo significant age hardening, which will greatly reduce the rate of rutting after this initial loading period. A pavement designed to carry 20 million ESALs over 2 years, on the other hand, will carry 10 million ESALs before significant age hardening occurs, and the final 10 million ESALs will be applied after only a modest amount of age hardening. The result will usually be substantial, even catastrophic failure, unless the high-temperature performance grade is adjusted upward. The other factor that must be considered is variability in climate. Occasional extremely hot summers over a 20-year design life are to be expected, but the damage will usually not be severe, given that the amount of traffic traveling over the pavement over a few such summers is limited. Using the same example of a pavement designed for very heavy traffic over 2 years, a single very hot summer would mean that about half of the design traffic would travel over the pavement while in a very soft condition; again, the results could be catastrophic rutting. For this reason, as in the Superpave system, all HMA mixtures should be designed for the projected 20-year traffic level. For example, if a temporary road is to carry 1,500,000 ESALs over 2 years, the mix should be designed not for 1,500,000 ESALs but for 15 million ESALs.

Another question that must sometimes be addressed during binder selection is whether or not to select a polymer-modified asphalt binder. A common rule is that when the two numbers in a performance grade add to a number more than 90, the resulting binder must be modified. For example, most PG 64-22 binders are not modified (64 + 22 = 86), whereas most PG 76-22 binder are (76 + 22 = 98). However, refineries and chemical manufacturers are constantly developing new types of binder modification, and the performance of modified binders is not completely reflected in the current performance grading system. For example, the rut resistance and fatigue resistance exhibited by many commercially available polymer-modified binders is significantly better than non-modified binders of the same grade. For this reason, some states require that polymer-modified asphalt binder be used in certain critical situations. Such specifications often include one or two additional tests, such as elastic recovery, meant to ensure that only certain types of modification be used for these critical applications. Applicable state specifications must be reviewed to determine if such requirements apply. As noted in Table 8-1, consideration should be given to using polymer-modified binders in cases where the grade adjustment exceeds two grades. Furthermore, provided the mixture meets rut resistance requirements as discussed later in this chapter, the high-temperature performance grade can be reduced one level if a modified binder is used. This is because the current performance binder specification does not always adequately address the superior performance of many modified binders. Furthermore, without this adjustment in performance grade requirements, many areas in the southern United States would find it difficult or impossible to obtain suitable binders for HMA mixes intended for pavements subject to very heavy traffic. When selecting modified binders, optimum performance will be ensured if the binder selected is one that has been successfully used in the past under similar conditions or one approved by the state highway agency.

Step 3. Determine Compaction Level

The design compaction level—N_{design}—is a function only of design traffic level. Suggested values for N_{design} as a function of design traffic in million ESALs are listed in Table 8-2. These values are identical to those used in the Superpave method. However, recommended design compaction levels for HMA mixtures are under review and could be modified soon; it is possible that slightly lower N_{design} values could be adopted, to aid in designing mixtures that have higher VMA and are easier to compact in the field.

Table 8-2. Recommended design compaction levels for dense-graded HMA mixtures.

Design Traffic (MESALs)	N_{design}
< 0.3	50
0.3 to < 3	75
3 to < 10	100
10 to < 30	100
≥ 30	125

Step 4. Select Nominal Maximum Aggregate Size

The nominal maximum aggregate size of the aggregate blend for an HMA mixture is most often specified by the owner/agency for a given project. In cases where the aggregate size is not specified, it is determined by the lift thickness during construction. Lift thickness and aggregate size can significantly affect the ease with which a mixture can be compacted in the field and the permeability of the resulting pavement. Brown and associates at the National Center for Asphalt Technology (NCAT) in 2004 published the results of research on this topic in *NCHRP Report 531: Relationship of Air Voids, Lift Thickness, and Permeability in Hot Mix Asphalt Pavements.* The guidelines given here are based on their conclusions and recommendations. The nominal maximum aggregate size should be no more than one-third the lift thickness for fine mixtures, and one-fourth the lift thickness for coarse mixtures. Coarse mixtures are defined as those for which the percent passing is less than the control point for the primary control sieve as listed in Table 8-3; all other mixtures are considered fine graded. All else being equal, smaller aggregate sizes should be preferred for wearing course mixtures and where extra durability is desired; this will help provide a mix that compacts easily, has low permeability, and resists fatigue cracking. Table 8-4 lists recommended NMAS values for different applications of dense-graded HMA. Unless otherwise specified, the smallest possible NMAS from those listed in Table 8-4 should be selected for use in a given mix design.

Table 8-3. Primary control sieve sizes.

Aggregate NMAS (mm)	Primary Control Sieve (mm)	PCS Control Point (% Passing)
4.75	1.18	42
9.5	2.36	47
12.5	2.36	39
19.0	4.75	47
25.0	4.75	40
37.5	9.5	47

Table 8-4. Recommended aggregate nominal maximum aggregate sizes for dense-graded HMA mixtures.

Application	Recommended NMAS, mm	Recommended Lift Thickness, mm	
		Fine-Graded Mixtures	Coarse-Graded Mixtures
Leveling course mixtures	4.75	15 to 25	20 to 25
	9.5	30 to 50	40 to 50
Wearing course mixtures	4.75	15 to 25	20 to 25
	9.5	30 to 50	40 to 50
	12.5	40 to 65	50 to 65
Intermediate course mixtures	19.0	60 to 100	75 to 100
	25.0	75 to 125	100 to 125
Base course mixtures	19.0	60 to 100	75 to 100
	25.0	75 to 125	100 to 125
	37.5	115 to 150	150
Rich base course mixtures	9.5	30 to 50	40 to 50
	12.5	40 to 65	50 to 65

Step 5. Determine Target VMA and Design Air Void Content

One of the unique features of the design method described in this manual for dense-graded mixtures is that the target VMA and air void content—and the resulting target binder content—are determined early on and maintained throughout the mix design procedure. In this way, the proper binder content is ensured, and effort is not wasted evaluating mixtures that do not have the proper VMA and binder content. This also reduces the chances that an error in volumetric calculations or laboratory testing will result in a mix design that does not meet the specified requirements. The allowable VMA range depends only on the aggregate NMAS; as in the Superpave system, minimum and maximum VMA values increase with decreasing aggregate NMAS—1% for each decrease in standard aggregate size. Limits for VMA are given in Table 8-5. A 2% range is specified for allowable VMA. When selecting VMA for a mix design, the target value is in the center of this allowable range. These target values should be used for the initial development of a mix design—for determining the composition of trial mixtures to be evaluated and refined in the laboratory during the mix design process. The design VMA value can be adjusted during the later stages of the mix design process or during construction, in order to further refine the mix or to adjust for field production. Using a target VMA value in the

Table 8-5. VMA requirements for standard dense-graded mixtures.

Aggregate NMAS (mm)	Minimum VMA[a] (%)	Maximum VMA[a] (%)
4.75	16	18
9.5	15	17
12.5	14	16
19.0	13	15
25.0	12	14
37.5	11	13

[a]The specifying agency may increase the minimum and maximum values for VMA by up to 1% to obtain mixtures with increased asphalt binder content, which can improve field compaction, fatigue resistance, and general durability. Care should be taken to ensure that the resulting HMA mixtures maintain adequate rut resistance for their intended application.

center of the allowable range ensures that such adjustments can be made. If a mix design is started at the minimum allowable VMA, adjustments needed later in the mix design process or during field production can be difficult or impossible without lowering the VMA below the specified minimum.

As noted in Table 8-5, the specifying agency can increase the minimum and maximum (and resulting target) VMA values by up to 1% if desired. This will provide additional binder content in the resulting HMA mixtures, which can have several desirable effects—it will tend to produce a mixture that is easier to compact in the field, more fatigue resistant, and, in general, more durable. However, increasing VMA can also decrease rut resistance, so care is needed when increasing minimum VMA requirements. As discussed below, the required dust/binder ratio of 0.8 to 1.6 should not be lowered if VMA requirements are increased beyond those given in Table 8-5, or the resulting mixtures might at times exhibit poor rut resistance. Agencies should in general be wary of simultaneously changing mix design requirements that all tend to reduce rut resistance—these include increasing VMA, decreasing dust/binder ratio, decreasing N_{design}, reducing requirements for FAA, or lower requirements for CAFF. Agencies should also be aware that the only foolproof way of increasing binder content in HMA mixtures is to increase minimum VMA requirements. Reducing N_{design} values will make it easier to design mixtures with higher VMA, but producers will find it easy to adjust their aggregate proportions after such a change in order to maintain binder content at the lowest possible level when economic incentives make such an approach desirable. When considering increasing VMA requirements, it should be remembered that many HMA performance problems are the result of construction problems, especially poor field compaction, rather than improper mix design. If high in-place air void content is the cause of poor durability—raveling and surface cracking—increasing VMA or decreasing N_{design} will not improve field performance unless these changes result in significant improvement in field compaction.

For most surface course and intermediate (binder) course mixes, a design air void content of 4.0% is recommended. However, the design air void content for these mixtures is allowed to vary from 3.5% to 4.5%. Specifying a lower design air void content of 3.5% will result in an increase in binder content of a few tenths of a percent and a mixture that is slightly easier to compact. It will, however, also tend to decrease rut resistance. Increasing the design air void content by 0.5% will have the opposite effect—it will slightly decrease the design binder content and produce a mix that is more difficult to compact, while increasing rut resistance. Rich bottom or base course mixes, as now sometimes used in the design and construction of perpetual pavements, should be designed at a slightly lower air void content of 3.0 to 4.0%. This helps ensure that these mixes have the binder needed for exceptional fatigue life and are also easy to compact to a very low air void content in the field. Because base course mixtures are located deep within the pavement structure, the decrease in rut resistance caused by a lower design air void content is not normally a major concern for these applications.

In HMA Tools, the initial target values for VMA and air void content are selected in the worksheet "General." This worksheet lists the minimum and maximum values for VMA along with the suggested target—the midpoint between the VMA limits. Target VMA and air void content can also be refined in the worksheet "Trial_Blends." As discussed above, it is recommended that (1) such adjustments be made only after evaluating several trial batches and (2) they be kept small—about 0.5% or less. Changing target VMA and air void content during the initial stages of the mix design process can make it difficult to evaluate the effect that changes in aggregate gradation have on these values, making the mix design process longer and more complicated than it needs to be. As discussed above, designing a mix near the minimum or maximum allowable VMA and/or air void content can also make adjustments during field production more difficult.

Step 6. Calculate Target Binder Content

Once the target VMA is selected, calculation of the design binder content is straightforward. The target value for VBE (effective binder content by volume) is calculated by subtracting the design air void content—normally 4%—from the target VMA value. For example, a standard 12.5-mm mixture with a target VMA value of 15% would have a target VBE value of $15 - 4 = 11\%$. The total binder content must also include the amount absorbed by the aggregate. This can be estimated in several ways. A quick approximate estimate, suitable for developing trial batches, is to simply add 1% to the target VBE value. In the example above, this would result in a total target binder content of 12%. A more accurate estimate would be to calculate the volume of water absorbed by the aggregate, divide this by two, and add it to the target VBE value:

$$V_b = VBE + \left(1 - \frac{VMA}{100}\right)\left(\frac{G_{sb}P_{wa}}{2}\right) \tag{8-1}$$

where

V_b = total asphalt content by volume %
VBE = effective asphalt content by volume %
VMA = voids in the mineral aggregate
= V_{be} + air void content
G_{sb} = aggregate bulk specific gravity
P_{wa} = water absorption of the aggregate, weight %

The best approach to estimating absorbed binder and the resulting total binder content is to use past experience. However, this may not always be possible. In any case, it should be remembered that the mixture proportions being determined at this point in the mix design process are only for one or more trial mixtures and that further adjustments will almost always have to be made prior to finalizing the mix design. Therefore, use of estimates in determining total binder content from the target VMA will usually work quite well. In HMA Tools, Equation 8-1 is used to estimate the binder content. As additional trial mixtures are made, the amount of binder absorbed by the aggregate is adjusted according to the values measured during the previous trial mixtures.

The final proportions for the mix design must be given by both percent by volume and weight, so the binder contents calculated above must be converted to percentages by total mix weight. However, this conversion cannot be done until the next two steps in the mix design process are completed and the aggregate proportions determined.

Step 7. Calculate Aggregate Content

The total aggregate content by volume is directly calculated as 100% minus the VMA content. In the example above, the total aggregate volume would be $100 - 15 = 85\%$. Determination of the total aggregate content by weight will depend on the aggregate specific gravity values and the specific blend of aggregates used in each of the trial mixtures, as determined in Step 8 as explained below. As with most other calculations needed during the mix design process, HMA Tools automatically calculates the aggregate content.

Step 8. Proportion Aggregates for Trial Mixtures

Proportioning aggregates for trial mixtures is one of the most important steps in the HMA mix design process. It can also be one of the most complicated. The procedure recommended here sets the binder content at a value that will provide the proper VMA once the design air void content is met. Therefore, proportioning aggregates can be thought of as determining the blend of aggregates that will provide the proper air void content for the mixture. However, because many HMA mix designs can make use of four or more aggregates, determining the right aggregate blend can be difficult and is largely a trial-and-error process.

Engineers and technicians responsible for HMA mix designs should understand that several systems for blending aggregates are very effective. The Asphalt Institute Manuals on mix design—MS-2 and SP-2—provide detailed descriptions of aggregate proportioning methods typically used with the Marshall, Hveem, and Superpave mix design methods. More recently, the Bailey method of aggregate proportioning has become popular among some technicians and engineers. This procedure is based on theoretical principles of particle packing and, although relatively complicated, is unique in that it provides many quantitative rules for modifying aggregate blends to achieve a desired change in VMA. An excellent reference for the Bailey method is Vavrik et al., "Bailey Method for Gradation Selection in HMA Mixture Design," *Transportation Research Board Circular E-C044.* In any case, engineers and technicians who are comfortable with the methods they are using for proportioning aggregates should continue to use them. HMA mix design—particularly determining appropriate aggregate blends—involves science and math, but is also largely an art based on experience and judgment.

The method described below is largely a graphical one and is intentionally simple and flexible, so that it is potentially compatible with the widest possible range of mix design procedures and combination of circumstances. Unfortunately, this also means that applying this procedure efficiently requires some experience—both with the mix design process and with a wide range of materials.

Maximum Density Aggregate Gradation and Fundamentals of Aggregate Blending

For many years, use of the maximum density aggregate gradation has been emphasized in proportioning aggregates for HMA mix design. The maximum density gradation is that which provides the smallest possible volume of space among the aggregate particles—that is, it is the blend providing the lowest possible VMA for a given set of aggregates. Using the maximum density gradation to produce an HMA mix was for many years considered desirable because it would result in a mix with the minimum asphalt binder content and because asphalt binder is much more expensive than aggregate, the resulting mixture would be relatively economical. However, it has become clear that a certain minimum amount of asphalt binder is required in order for an HMA mixture to be workable—easy to place and compact—and also to resist moisture damage, age hardening, and fatigue cracking. Therefore, most HMA designs today use aggregate gradations that vary significantly from maximum density. In fact, achieving the minimum required VMA values can be a problem with some aggregates.

Even though most HMA mixtures do not precisely follow a maximum density gradation, it is often used as a reference when proportioning aggregates. A good estimate of the maximum density aggregate gradation for a given aggregate size can be estimated using the 0.45 power gradation:

$$\%PMD \approx \left(\frac{d}{D}\right)^{0.45} \times 100\% \qquad (8\text{-}2)$$

where

$\%PMD$ = percent passing for maximum density gradation
d = sieve size, mm
D = maximum sieve size for gradation, mm

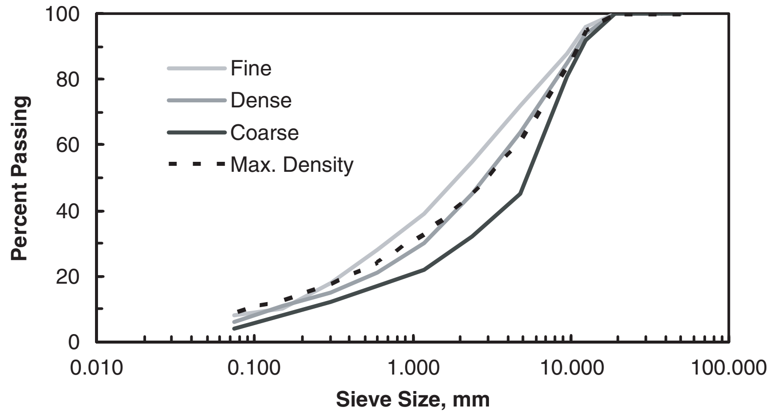

Figure 8-3. Fine, dense, and coarse aggregate gradations for HMA compared to the maximum density gradation.

Figure 8-3 is a plot of three different 12.5-mm NMAS aggregate gradations, with the maximum density gradation calculated using Equation 8-2 included for reference. As shown in this figure, HMA gradations are usually classified as coarse, fine, or dense, depending on where they pass relative to the maximum density gradation. Those falling below the maximum density gradation are called coarse gradations, those passing above are called fine gradations, and those passing near the maximum density gradation are called dense. However, it should be emphasized that, in fact, all three of these gradations are used in producing dense-graded HMA—therefore, a more accurate classification of these gradations would be dense/coarse, dense/fine, and dense/dense, accurately reflecting that all three gradations are close to dense gradations, but some are slightly coarser and some slightly finer than the maximum density gradation.

A concept related to the maximum density gradation, and also useful in analyzing aggregate gradation, is the continuous maximum density gradation. This is calculated using an equation similar to Equation 8-2:

$$P_{CMD}(d_2) \approx \left(\frac{d_2}{d_1}\right)^{0.45} \times P(d_1) \qquad (8\text{-}3)$$

where

$P_{CMD}(d_2)$ = percent passing, continuous maximum density gradation, for sieve size d_2
d_1 = one sieve size larger than d_2
$P(d_1)$ = percent passing sieve d_1

For example, in a selected aggregate gradation the percent passing the 4.75-mm sieve is 84%. The P_{CMD} for the 2.36-mm sieve would be calculated as $(2.36/4.75)^{0.45} \times 84 = 73\%$. The usefulness of the continuous maximum density gradation is that it allows a more realistic evaluation of how closely a given aggregate gradation follows a maximum density gradation compared to the traditional maximum density gradation as calculated using Equation 8-2. The top graph in Figure 8-4 shows a 9.5-mm gradation compared to the standard maximum density gradation as calculated using Equation 8-2. The bottom graph shows the deviation from the continuous maximum density gradation (calculated using Equation 8-3) for this same aggregate. Both figures suggest that the gradation deviates significantly from the maximum density gradation, but the lower graph is much clearer in the way in which this deviation occurs. For example, it is not clear in the top graph that the aggregate gradation in fact follows a maximum density gradation below the 1.18-mm sieve size; this is very clear in the lower graph. Furthermore, the lower plot

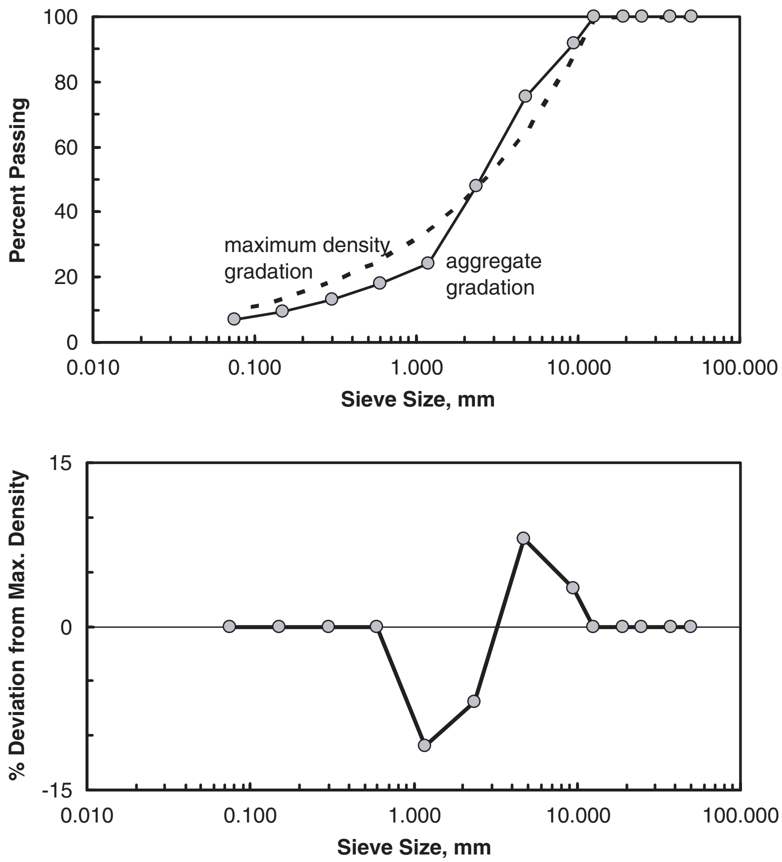

Figure 8-4. Top: 9.5-mm aggregate gradation compared to the maximum density gradation; bottom: deviation from the continuous maximum density gradation for the same 9.5-mm gradation.

exaggerates the deviation from maximum density, so comparing several similar aggregate blends is much easier.

One of the most important—and often the most difficult—parts of the mix design process is adjusting aggregate blends to produce the desired level of air void content and VMA. The lower graph in Figure 8-4, called a continuous maximum density (CMD) plot, is very helpful in this process because, in general, for a given set of aggregate blends the more this plot deviates from zero (the horizontal line through the center of the plot), the greater will be the VMA and air void content. Using the CMD plot to blend aggregates has many advantages:

- It is soundly based in packing theory.
- It is completely flexible—it can be applied to any number of aggregates, any gradation size, and any type of gradation or HMA mix.
- Once set up in a spreadsheet, as in HMA Tools, it is simple to apply; there is no long set of rules and definitions to remember.
- Because of its simplicity and flexibility, the CMD approach can be used along with other procedures.

Figure 8-5 shows how the CMD plot relates to changes in aggregate gradation for a series of 12.5-mm aggregate blends: an SMA aggregate blend; a dense/coarse blend; a dense/dense blend; and a dense/fine blend. The top portion of the figure shows a traditional gradation plot, including the maximum density gradation. The bottom chart in Figure 8-5 shows the CMD plots for these

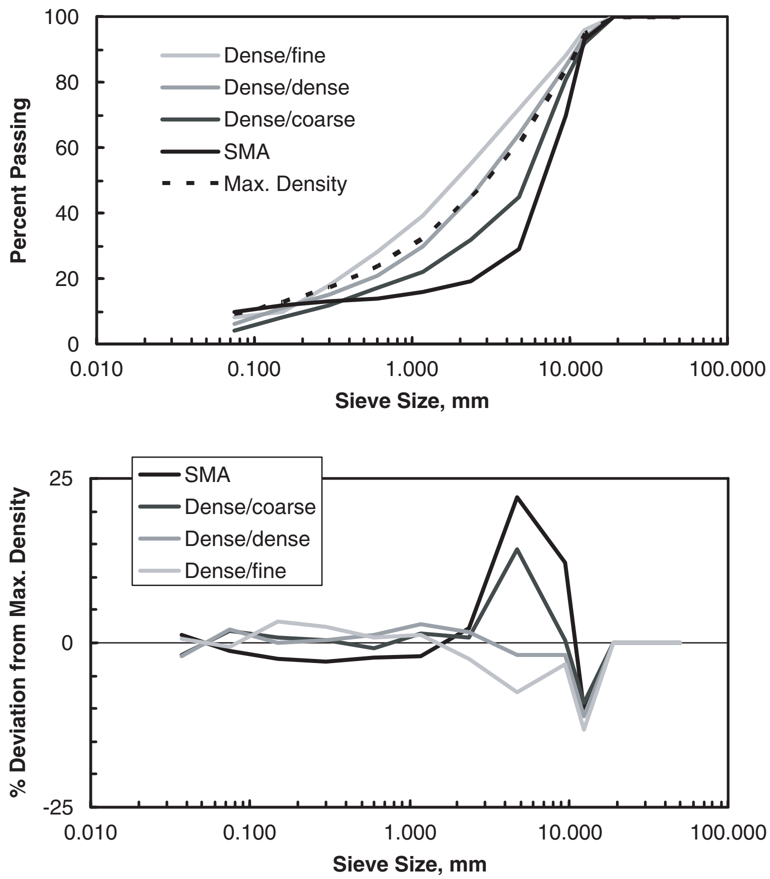

Figure 8-5. Top: four different 12.5-mm gradations; bottom: % deviation from continuous maximum density gradation for these same blends.

same four aggregate blends. One of the most striking features of the CMD plot is that it shows that despite the large differences in the four gradations, the fine aggregate portions of these blends all are fairly close to a maximum density gradation. This does not mean that the fine aggregate portion of these mixtures closely follows the gradation for the traditional maximum density gradation for a 12.5-mm aggregate (the dashed line in the top chart in Figure 8-5). What it means is that the fine aggregate portions of all four blends—considered separately from the coarse portion—follow a maximum density gradation fairly closely. This is a very important concept when adjusting aggregate blends to meet VMA and/or air void requirements. Consider the dense/coarse gradation in Figure 8-5. In the top chart it appears that the fine aggregate portion of this aggregate deviates significantly from the maximum density gradation, and changing this portion of the blend might therefore tend to reduce VMA. However, it is clear from the lower chart that attempting to reduce VMA by changing the fine aggregate portion of this gradation will probably be counter-productive, since it already closely follows a maximum density gradation. If a reduction in VMA is needed for this aggregate, the amount of material between the 2.36- and 9.5-mm sieves must be reduced. In general, the greater the deviation from the zero line on the CMD plot, the greater will be the VMA (and air void content) for the resulting mixture.

In Figure 8-5, it appears that the largest differences in these blends are in the coarse aggregate. This is in fact typical for aggregate blends used in HMA. However, even though the deviations from the maximum density gradation for the fine aggregate portion of many aggregate blends may seem small, such differences can have a significant effect on the air void content and VMA of the resulting HMA mixture.

The specific interpretation of CMD plots such as those shown in Figures 8-4 and 8-5 is as follows. The value on the vertical axis for a given sieve size shows the difference in the percent of material between that sieve size and one sieve size larger for the actual gradation and the continuous maximum density gradation. For example, in Figure 8-5, for the 4.75-mm sieve, the SMA gradation has about 22% more material between the 4.75- and 9.5-mm sieves than the maximum density gradation. This is typical for SMA gradations, which contain very large proportions of coarse aggregates. The dense/coarse gradation, on the other hand, has about 14% more material than the continuous maximum density gradation in this size range. When using CMD plots to blend aggregates during a mix design, the technician should look not only at the amount of the deviation in different size ranges, but also the effect of these deviations on the air void content and VMA of the resulting mixture. The air void content and VMA for some mixtures might be most sensitive to changes in the coarse fraction of the aggregate blend, while other mixtures might be more sensitive to changes in the fine or intermediate portions of the aggregate blend. For this reason, HMA Tools includes, as part of the CMD plot, values for air void content and VMA for each aggregate blend (once they have been determined in laboratory testing). This makes it easy for the technician to determine what changes in the aggregate blends are most important in determining volumetric composition.

Trial Blends for New Mix Designs

When developing an HMA mix design with a new set of aggregates, the general procedure recommended here is very similar to that used in the Superpave method, but with the inclusion of the CMD plot as an additional tool in analyzing the aggregate gradations and resulting volumetric compositions. After performing the initial steps of the mix design as outlined above (including determination of the design VMA, air void content, and binder content), three trial aggregate gradations are prepared: a dense/coarse gradation, a dense/dense gradation, and a dense/fine gradation. As explained later in this chapter, trial batches based on these gradations are prepared in the laboratory, specimens compacted and the VMA, air void content, and effective binder content determined for each. Usually none of the three initial batches will precisely meet all requirements, and one is selected for further refinement. Additional trial batches are prepared and evaluated until all essential mix design criteria are met. It is strongly suggested that during the initial trial batches only the aggregate gradation should be modified, while the asphalt binder content is kept constant. This will make the way changes in the aggregate gradation are affecting VMA and air void content much clearer. Once a trial batch is close to the required composition, the binder content may be slightly adjusted if desired to fine-tune the design. However, engineers and technicians should remember that mix designs will usually be adjusted significantly during the initial stages of field production, so effort spent in unnecessary, minor adjustments to a laboratory mix design will often be wasted.

The HMA Tools spreadsheet has been designed to allow blending of up to eight different aggregates, up to four of which may be recycled asphalt pavement (RAP) material. Gradation data and other properties are entered in the worksheet “Aggregates” for new materials and “RAP_Aggregates” for RAP materials. In the worksheet “Trial_Blends,” the technician enters various proportions for each aggregate or RAP, and the gradation is calculated and plotted on a standard plot and on a CMD plot. Once volumetric test data are available for the trial batches, these plots include VMA and air void values to aid the technician in determining which gradation is most suitable and what sort of adjustments in the gradation (if any) are needed to produce a mixture with the desired properties.

Adjusting Aggregate Gradations When Modifying Existing HMA Designs

In practice, most HMA mix design work involves modifying existing mixtures to meet some new requirement or improve some aspect of the design (such as workability). Therefore, the approach discussed above will usually be the exception, rather than the rule. In modifying an existing mix design, the procedure suggested here is similar to that described above, but somewhat abbreviated. First, the goal of the modification must be clearly understood, primarily in terms of the needed change in VMA. Increased air void content or increased binder content both require increases in VMA. Similarly, if a specification requires an increase in VMA, and the design air void content does not change, an increase in binder content will be needed. Remember, VMA consists of the volume taken up by both asphalt binder and air voids.

Once the required change in VMA has been determined, the aggregate gradation for the existing mix design is plotted, using a traditional gradation plot and the CMD plot. If an increase in VMA is needed, in general, the aggregate gradation must be modified to increase the difference between the CMD plot and the zero line. If a decrease in VMA is needed, the gradation should be modified to decrease this difference. This is shown in Figure 8-6. The heavy, dark line in this example is the aggregate gradation for an existing mix design. Looking at the coarse aggregate portion of the gradation, the lighter lines above this are gradations that would likely increase the VMA for this mixture, since they are further from the zero line. The lighter lines below the existing gradation would probably decrease the VMA, since they are, in general, closer to the zero line on the CMD plot. However, engineers and technicians using this approach should remember that VMA for a given mixture will exhibit different sensitivities to changes in different portions of the aggregate gradation. Some mixtures might be more sensitive to changes in the fine aggregate portion of the gradation, while some might be more sensitive to the coarse aggregate portion of the gradation. Some might be very sensitive to changes in mineral filler content. It should also be noted that the zero line on the CMD plot is only an approximate indicator of the maximum density gradation—the actual position on the CMD plot of the maximum density gradation might vary somewhat from the zero line. This should become apparent when plotting gradations and VMA values for trial mixtures.

It is especially important when modifying existing mix designs to make use of experience with the given aggregates. Often, an engineer or technician who has done previous mix design work with the aggregates at hand will know what changes in the gradation are needed

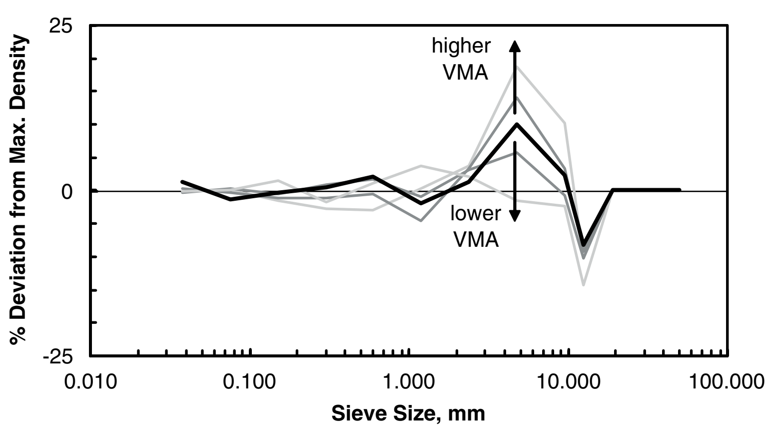

Figure 8-6. Effect of changes in aggregate gradation on VMA as shown on a CMD plot. *Aggregates plotting closer to the zero line will usually have lower VMA values.*

Table 8-6. Control points for 19.0-mm through 37.5-mm aggregate gradations for dense-graded HMA mixtures.

Sieve Size (mm)	*Percent Passing for Nominal Maximum Aggregate Size:*					
	37.5 mm		**25.0 mm**		**19.0 mm**	
	Min.	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
50.0	100					
37.5	90	100	100			
25.0		90	90	100	100	
19.0				90	90	100
12.5						90
9.5						
4.75						
2.36	15	41	19	45	23	49
1.18						
0.600						
0.075	0	6	1	7	2	8

to produce a specific change in VMA or other mix properties. In such cases, the CMD plot can be a useful tool.

When modifying existing mix designs, the HMA Tools spreadsheet is used in a manner very similar to that described above for new mix designs. As before, the needed aggregate and RAP data (if used) are entered in the worksheets "Aggregates," and "RAP_Aggregates." Then, aggregate blend for the existing mix is entered in the worksheet "Trial_Blends," followed by one or two modified aggregate blends. If no information is available concerning the aggregates being used, the general rule described above should be used in developing the new trial aggregate blends—gradations closer to the zero line on the CMD plot will have lower VMA, while those further away will have higher VMA. After this, the design proceeds as before, determining the air void content and VMA for the trial batches and then making further refinements in the aggregate gradation as needed until the desired mix properties are met.

Guidelines for Aggregate Gradations

As with previous HMA mix design methods, there are limits for aggregate gradation for each NMAS; suggested control points for aggregate gradations for dense-graded HMA mixtures are listed in Tables 8-6 and 8-7. It is important to note that, in the system described in this manual,

Table 8-7. Control points for 4.75-mm through 12.5-mm aggregate gradations for dense-graded HMA mixtures.

Sieve Size (mm)	*Percent Passing for Nominal Maximum Aggregate Size:*					
	12.5 mm		**9.5 mm**		**4.75 mm**	
	Min.	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
50.0						
37.5						
25.0						
19.0	100					
12.5	90	100	100		100	
9.5		90	90	100	95	100
4.75				90	90	100
2.36	28	58	32	67		
1.18					30	60
0.600						
0.075	2	10	2	10	6	12

Table 8-8. Coarse aggregate fractured faces requirements.

Design ESALs (million)	Percentage of Particles with at Least One/Two Fractured Faces, for Depth of Pavement Layer[a], mm	
	0 to 100	Below 100
< 0.3	55 / ---	--- / ---
0.3 to < 3	75 / ---	50 / ---
3 to < 10	85 / 80	60 / ---
10 to < 30	95 / 90	80 / 75
30 or more	98 / 98[b]	98/ 98[b]

[a]Depth of pavement layer is measured from pavement surface to surface of pavement layer.
[b]The CAFF requirement for design traffic levels of 30 million ESALs or more may be reduced to 95/95 if experience with local conditions and materials indicate that this would provide HMA mixtures with adequate rut resistance under very heavy traffic.

aggregate control points—with the exception of the maximum aggregate size—are considered guidelines, and not specification requirements. This provides engineers and technicians with additional flexibility in modifying aggregate gradations in order to meet VMA requirements. Differences in aggregate particle shape, angularity, and texture make it impossible to specify one particular gradation that will provide the best performance for all HMA mixtures of a given NMAS. Treating gradation control points with some flexibility helps ensure that engineers and technicians can adequately address these differences and provide HMA mixtures with the proper binder content and air void content needed for good durability.

The gradation plots included in the worksheet "Trial_Blends" in HMA Tools include boundaries showing the control limits for a given mixture. In order for the proper limits to be included in the plot, the proper value for the aggregate NMAS must be entered in the worksheet "General."

Check Aggregate Specification Properties

As in the Superpave method, there are four aggregate specification properties: (1) coarse aggregate fractured faces (CAFF); (2) flat and elongated particles in the coarse aggregate; (3) fine aggregate angularity (FAA); and (4) clay content of the fine aggregate (sand equivalent). A detailed description of these specification properties and the tests used to determine them is given in Chapter 4 of this manual. For the convenience of the readers of this manual, four tables listing the aggregate specification properties as given in Chapter 4 are reproduced below. Table 8-8 lists requirements for coarse aggregate fractured faces; Table 8-9 lists requirements for flat and elongated coarse aggregate particles; Table 8-10 lists requirements for fine aggregate angularity; and Table 8-11 lists requirements for fine aggregate clay content. The values in these tables are very similar to those used in the Superpave method; there are some slight

Table 8-9. Criteria for flat and elongated particles.

Design ESALs (million)	Maximum Percentage of Flat and Elongated Particles at 5:1
< 0.3	---
0.3 to < 3	10
3 to < 10	10
10 to < 30	10
30 or more	10

Criteria are presented as percent flat and elongated particles by mass.

Table 8-10. Fine aggregate angularity requirements.

Design ESALs (million)	Depth of Pavement Layer from Surface, mm	
	0 to 100	Below 100
< 0.3	---[a]	---
0.3 to < 3	40	---
3 to < 10	45[b]	40
10 to < 30	45[b]	45[b]
30 or more	45[b]	45[b]

Criteria are presented as percent air voids in loosely compacted fine aggregate.

[a]Although there is no FAA requirement for design traffic levels below 0.30 million ESALs, consideration should be given to requiring a minimum uncompacted void content of 40 percent for 4.75-mm nominal maximum aggregate size mixes.
[b]The FAA requirement of 45 may be reduced to 43 if experience with local conditions and materials indicate that this would produce HMA mixtures with adequate rut resistance under the given design traffic level.

differences in the requirements for coarse aggregate fractured faces and fine aggregate angularity, intended to make these requirements easier to meet without making any significant sacrifice in performance.

As in the Superpave method, it is intended that aggregate specification properties be applied to the aggregate blend, and not to individual aggregates. An important step in the mix design process is to determine specification property values for aggregate blends, to ensure that the blends will likely meet specification property requirements. For the initial trial batches, the specification properties of the aggregate blends are normally estimated mathematically, by calculating a weighted average for each property. When calculating these weighted averages, care must be taken to consider only that portion of the aggregate tested in a given procedure. For example, CAFF is determined on only that portion of the aggregate retained on the 4.75-mm sieve, so the weighted average CAFF is based on the proportions of this fraction for each aggregate in the blend—not on the overall proportions for each aggregate. An additional complication occurs when RAP is included in the mix design; then, the specification properties of the aggregate in the RAP must also be considered (with the exception of sand equivalent, which is only applied to new aggregates). Estimation of specification properties can be tedious and prone to errors. Fortunately, HMA Tools performs this calculation for the technician. In the worksheets "Aggregates" and "RAP_Aggregates," specification properties are entered for each aggregate. Up to two user-defined properties can also be entered here—these would be aggregate properties specified by the local highway agency, in addition to those given in this manual. In the worksheet "Trial_Blends" the estimated values for all specification properties are then shown.

Aggregate properties for the final mix design should be determined by actual measurement. This is done by preparing the aggregate blend and then sieving out the fraction needed for the particular test and performing the test. Because this can be a time-consuming procedure, the

Table 8-11. Maximum clay content requirements.

Design ESALs (million)	Minimum Sand Equivalency Value
< 0.3	40
0.3 to < 3	40
3 to < 10	45
10 to < 30	45
30 or more	50

Criteria are presented as Sand Equivalent Value.

final mix design should avoid borderline values for aggregate specification properties that, when the blend is actually tested, might fail to meet requirements.

Aggregate Blending: Summary

One of the most important and complicated parts of the HMA mix design process is determining the appropriate aggregate blend to use for a given application. Various procedures are available, including the Bailey method, and several techniques described in mix design manuals published by the Asphalt Institute. Engineers and technicians comfortable with the methods they are currently using for proportioning aggregates for HMA mix designs should continue to use these methods. The procedure given in this manual is based on a few simple concepts relating aggregate blends to VMA.

In most cases, HMA mix designs are not designed from scratch. Instead, existing mix designs are modified by replacing aggregates or the binder or by changing the binder content and VMA. In these cases, the best guide for adjusting the aggregate proportions is the experience of the engineer or technician with the materials being used. When modifying existing mix designs, one or two aggregate blends are developed by modifying the blend used in the existing mix. A trial-and-error approach is then used to refine the aggregate blend until the desired mix properties are achieved. In situations where an entirely new HMA mix design is to be developed, three initial trial blends are developed using dense/coarse, dense/dense, and dense/fine aggregate gradations. The design closest to meeting all requirements is then further refined by making additional trial blends, evaluating their properties, and modifying the aggregate gradation as needed.

An important part of the mix design process is determining the specification properties of the aggregate blends. For initial trial batches, specification properties can be estimated by using mathematical equations and the specification property values for the individual aggregates. This is done automatically in HMA Tools (and many similar spreadsheets and computer programs). However, the specification properties for the final mix design should be verified by actual measurements on the aggregate blend.

Step 9. Calculate Trial Mix Proportions by Weight and Check Dust/Binder Ratio

At this point in the HMA mix design, the amount of air voids, binder, and aggregate has been determined on a volume basis, and up to three different aggregate blends have been developed—on a proportion-by-weight basis. Now, the overall mixture composition in percent by weight must be calculated and the dust/binder ratio checked to make sure it is within the specified values. If desired, the mixture composition by volume can also be determined. The following procedure and equations can be used to calculate mix proportions by weight and related mix properties. First, calculate the overall aggregate bulk specific gravity:

$$G_{sb} = \frac{P_{s1/A} + P_{s2/A} + P_{s3/A} + \cdots}{\left(\frac{P_{s1/A}}{G_{sb1}}\right) + \left(\frac{P_{s2/A}}{G_{sb2}}\right) + \left(\frac{P_{s3/A}}{G_{sb3}}\right) + \cdots} \tag{8-4}$$

where

G_{sb} = overall bulk specific gravity for aggregate blend
$P_{s1/A}$ = volume % of aggregate 1 in aggregate blend
G_{sb1} = bulk specific gravity for aggregate 1
$P_{s2/A}$ = volume % of aggregate 2 in aggregate blend

G_{sb2} = bulk specific gravity for aggregate 2
$P_{s3/A}$ = volume % of aggregate 3 in aggregate blend
G_{sb3} = bulk specific gravity for aggregate 3

As discussed in Step 7, the volume percentage of the aggregate is simply 100% minus the target VMA. The weight percentage of binder and aggregate are then calculated using the following equations:

$$P_b = \frac{V_b G_b}{V_{sb} G_{sb} + V_b G_b} \times 100\% \tag{8-5}$$

$$P_s = \frac{V_{sb} G_{sb}}{V_{sb} G_{sb} + V_b G_b} \times 100\% \tag{8-6}$$

where

P_b = total binder content, % by total mix weight
V_b = total binder content, % by total mix volume
G_b = binder specific gravity
V_{sb} = aggregate content, % by total mix volume
G_{sb} = overall bulk specific gravity of aggregate (Equation 8-4)
P_s = total aggregate content, % by total mix weight

Then, calculate the effective asphalt binder content by weight:

$$P_{be} = \frac{V_{be} G_b}{V_{sb} G_{sb} + V_b G_b} \times 100\% \tag{8-7}$$

where

P_{be} = effective binder content, % by total mix weight
V_{be} = effective binder content, % by total mix volume
G_b = binder specific gravity
V_{sb} = aggregate content, % by total mix volume
G_{sb} = overall bulk specific gravity of aggregate (Equation 8-4)

Calculate the percent by weight of each aggregate:

$$P_{s1} = P_s \left(\frac{P_{s1/A}}{100} \right) \tag{8-8}$$

where

P_{s1} = weight percent (by total mix) of aggregate 1 (or aggregate 2, 3, etc.)
P_s = weight percent (by total mix) of combined aggregate, from Equation 8-6
$P_{s1/A}$ = weight percent (in aggregate blend) of aggregate 1 (or aggregate 2, 3, etc.)

If desired, the volume percent of the aggregates can also be calculated, but the equation is more complicated:

$$V_{sb1} = \frac{P_{s1}\left(100 - P_b\right)}{\left(\frac{P_b}{G_b}\right) + \left(\frac{P_{s1}}{G_{sb1}}\right) + \left(\frac{P_{s2}}{G_{sb2}}\right) + \left(\frac{P_{s3}}{G_{sb3}}\right) + \cdots} \tag{8-9}$$

where

V_{sb1} = volume % of aggregate 1 in total mix
P_{s1} = weight % of aggregate 1 in total mix
P_b = weight % binder in total mix
G_b = binder specific gravity
G_{sb1} = bulk specific gravity for aggregate 1
P_{s2} = volume % of aggregate 2 in aggregate blend
G_{sb2} = bulk specific gravity for aggregate 2
P_{s2} = volume % of aggregate 3 in aggregate blend
G_{sb3} = bulk specific gravity for aggregate 3

Calculate the percent of mineral dust (material finer than 0.075 mm) in the total mixture:

$$P_{0.075} = \frac{P_{0.075/s1}P_{s1} + P_{0.075/s2}P_{s2} + P_{0.075/s3}P_{s3} + \cdots}{100} \tag{8-10}$$

where

$P_{0.075}$ = mineral dust content (material finer than 0.075 mm), percent by total mix weight
$P_{0.075/s1}$ = % passing the 0.075-mm sieve for aggregate 1
P_{s1} = weight percent (by total mix) of aggregate 1
$P_{0.075/s2}$ = % passing the 0.075-mm sieve for aggregate 2
P_{s2} = weight percent (by total mix) of aggregate 2
$P_{0.075/s3}$ = % passing the 0.075-mm sieve for aggregate 3
P_{s3} = weight percent (by total mix) of aggregate 3

Calculate the dust/binder ratio, using the effective asphalt binder content:

$$D/B = \frac{P_{0.075}}{P_{be}} \tag{8-11}$$

where

D/B = dust/binder ratio, calculated using effective binder content
$P_{0.075}$ = mineral dust content, % by total mix weight (Equation 8-10)
P_{be} = effective binder content, % by total mix weight (Equation 8-7)

The required range for dust/binder ratio is 0.8 to 1.6 for all mixtures larger than 4.75-mm NMAS. However, the specifying agency may reduce the requirements to a range of 0.6 to 1.2 if local materials and conditions warrant this change. For 4.75-mm NMAS mixtures, the required dust/binder ratio is 0.9 to 1.2, and this should not be modified. These requirements are similar to those given in the Superpave method, but the required and optional ranges are reversed; in the Superpave method, the required range is 0.6 to 1.2, but agencies can increase the requirement to 0.8 to 1.6. Higher dust/binder ratios are desirable for several reasons. Perhaps most importantly, they help provide stiffness and rut resistance to the HMA. Higher dust/binder ratios also will tend to reduce the permeability of an HMA mixture, improving durability. However, it is possible that in some locations obtaining high dust/binder ratios might be prohibitively expensive, and the nature of the local materials might allow the design of HMA with good performance at lower dust/binder ratios. Because of the beneficial effects of high dust/binder ratios on rut resistance, if VMA requirements are increased above those given in Table 8-5, the dust/binder ratio requirement should not be reduced. Otherwise, the rut resistance of the resulting mixtures might, in some cases, be marginal. Table 8-12 summarizes the requirements for dust/binder ratio.

Table 8-12. Requirements for dust/binder ratio.

Mix Aggregate NMAS, mm	Allowable Range for Dust/Binder Ratio, by Weight
> 4.75	0.8 to 1.6[a]
4.75	0.9 to 2.0

[a]The specifying agency may lower the allowable range for dust/binder ratio to 0.6 to 1.2 if warranted by local conditions and materials. The dust/binder ratio should, however, not be lowered if VMA requirements are increased above the standard values as listed in Table 8-5.

When including RAP in a mixture, the same principles described above are applied. RAP is composed of both binder and aggregate. The weight and volume of binder in the RAP must be added to the weight and volume of new binder added to a mixture. Similarly, the weight and volume of aggregate must be added to the weight and volume of new aggregate added to the mix. As will be discussed in Chapter 9, HMA Tools automatically performs the needed calculations when including RAP in an HMA mix design.

Example Problem 8-1. Calculation of Mix Composition

Table 8-13 presents the results of an example calculation of mixture composition for a trial batch. The mixture is a 12.5-mm NMAS design, with a target air void content of 4% and a target VMA value of 15%. Column 1 describes the various mix components; this includes total binder, absorbed binder, and effective binder—this makes the relationship among these values clear. Column 2 gives the mix composition in percentage by volume, which is determined using the procedure described above. Column 3 lists the bulk specific gravity for the various components, while Column 4 lists apparent specific gravity values for the aggregates. Column 5 lists the aggregate contents as a percentage by weight of the aggregate blend.

Table 8-13. Example calculation of HMA mix composition by weight percentage from volume percentage and specific gravity values.

(1) Mix Component	(2) Percent by Total Mix Volume	(3) Bulk Specific Gravity	(4) Apparent Specific Gravity	(5) Percent by Aggregate Weight	(6) Percent by Total Mix Weight
Air	4.00	---	---	---	0.0
Total Asphalt Binder	11.38	1.025	---	---	4.60
Absorbed Asphalt Binder	-0.40	1.025	---	---	(0.16)
Effective Asphalt Binder	10.98	1.025	---	---	4.44
No. 7 Traprock	19.54	2.971	2.992	24	22.90
Traprock screenings	24.47	2.867	2.893	29	27.67
Manufactured sand	24.46	2.868	2.891	29	27.67
Natural sand	13.74	2.642	2.676	15	14.31
Mineral filler	2.80	2.588	2.629	3	2.86

Note: Calculations may not agree exactly because of rounding.

(continued on next page)

Example Problem 8-1. (Continued)

Column 6 lists the composition of the mix by weight percentage, calculated using Equations 8-4 through 8-11.

The procedure described above is tedious and prone to error if done by hand, so it is normally done with the aid of a spreadsheet designed to perform the calculations needed during the mix design process. It should be emphasized that when developing trial mix designs, calculations such as those given in Table 8-13 are only estimates of the actual mix composition. That is because the actual air void content and the amount of absorbed binder can only be accurately determined by making and testing HMA specimens in the laboratory. The air void content in this example was assumed to be the target value of 4%, even though it would be mostly luck if any of the trial mixtures produced exactly 4% air voids. As described above, the amount of absorbed asphalt is usually only estimated when designing initial trial batches; the actual amount of absorbed asphalt binder might vary significantly from this estimated value. These differences will usually mean that the actual mix composition will differ significantly from the initial estimate as calculated in this example.

Step 10. Evaluate and Refine Trial Mixtures

As discussed previously, when developing mix designs with new aggregates, three initial trial mixes are prepared, representing dense/fine, dense/dense, and dense/coarse aggregate gradations. When modifying an existing mix design, one or two trial batches might be prepared by making slight adjustments in the existing aggregate blend. In either case, the next step in the mix design process is the same: the trial mixtures must be evaluated to determine if any meet the given specifications or, if none meet all criteria, the mix closest to the specifications must be identified and the way it must be modified to produce an acceptable mix must be determined. Table 8-13 showed the results of an initial estimate of volumetric composition for a trial batch. But as described at the end of Step 9, the values in this table are only estimates—the actual composition for this and any other trial mixtures must be determined in the laboratory, by batching, mixing, compacting, and testing laboratory specimens for each of these trial mixtures. The steps involved in this process are described below, using the same trial mix design summarized in Table 8-13.

Calculate Batch Weights

Weights for trial batches are easily calculated from the mix composition by weight percent, as calculated in the example above and shown in Table 8-14. First, the number and size of gyratory specimens must be determined; for normal volumetric analysis, two specimens 150 mm in diameter by 115 mm high are normally required. For moisture resistance testing, as described later in this chapter, six specimens 150 mm in diameter by 95 mm high are needed. Some performance tests require compacted specimens 150 mm in diameter by 165 mm high. The total weight of material needed for a batch is then calculated using the following equation:

$$W_{mix} = G_{mb} V_{spec} N_{spec} \tag{8-12}$$

Example Problem 8-2. Calculating Trial Mix Batch Weights

For example, for the trial mix described in Table 8-13, the bulk specific gravity is estimated to be 2.536. If two 150-mm-diameter by 115-mm-high cylinders are to be prepared, the amount of mixture needed is calculated as $2.536 \times 2{,}439 \times 2 = 12{,}369$ grams. The weight of each component is then calculated by multiplying the total required by weight by the weight percentage of the mix component and dividing by 100%. Continuing with the same example, a table is easily constructed showing the weight percentages and the batch weights for each of the mix components. This is shown in Table 8-14; the weight percentages are the same as those listed in Table 8-13, although carried out to an extra decimal place for greater accuracy in calculating batch weights.

Table 8-14. Example calculation of batch weights for mixture listed in table 8-13.

Mix Component	Percent by Total Mix Weight (P)	Batch Weight, grams (12,369 × P/100)
Air	---	---
Total Asphalt Binder	4.60	569
Absorbed Asphalt Binder	---	---
No. 7 Traprock	22.90	2,832
Traprock screenings	27.67	3,422
Manufactured sand	27.67	3,422
Natural sand	14.31	1,770
Mineral filler	2.86	354
Total	**100.00**	**12,369**

Note: Calculations do not agree exactly because of rounding

where

W_{mix} = total weight of mix in batch, g
G_{mb} = estimated bulk specific gravity of mix
V_{spec} = Volume of specimen, cm^3
= 2,439 for 150-mm diameter by 115-mm high (including 20% extra)
= 2,015 for 150-mm diameter by 95-mm high (including 20% extra)
= 3,499 for 150-mm diameter by 165-mm high (including 20% extra)
N_{spec} = number of specimens, normally two

Batch Aggregates

Although the batch weights given in Table 8-14 could be used to weigh out material for the trial batch directly, this is not recommended because many aggregates tend to segregate during stockpiling, sampling, and handling in the laboratory, so that direct batching of aggregates will often produce specimens with aggregate gradations deviating significantly from the desired target gradation. For this reason, when handling and batching aggregates in the laboratory, they are often broken down and weighed in a number of size fractions. This helps ensure that the aggregate gradation actually used in the specimen is close to the target gradation. The way in which an aggregate is broken down is a matter of judgment and experience. Some engineers or technicians may choose to completely break down aggregates, while some may break down aggregates into only a few size fractions. A typical and fairly conservative approach is to break down aggregates into the following size fractions:

- 37.5 to 50.0 mm
- 25.0 to 37.5 mm
- 19.0 to 25.0 mm
- 12.5 to 19.0 mm
- 9.5 to 4.75 mm
- 2.36 to 4.75 mm
- Passing 2.36 mm

If there is less than 5% within one of these size fractions, it can be combined with an adjacent fraction. Mineral filler is not normally broken down prior to batching. HMA Tools includes the worksheet, "Batch," which calculates batch weights for different levels of aggregate processing. The technician enters which trial batch (out of up to seven) the batching sheet is being prepared for, the number and dimensions of laboratory specimens, and the desired percentage of extra material. The worksheet then provides the appropriate batch weights, including the binder weight and batch weights of each aggregate; coarse aggregates are broken down completely, while fine aggregates are broken down in three different ways—completely, partially (by groups of two sieves), and with no breakdown (that is, a single weight for each fine aggregate). The technician performing the mix design can select among the three different ways of breaking down the fine aggregate in the batching process.

Example Problem 8-3. Breaking Down Aggregates and Calculating Aggregate Batch Weights

An example of aggregate breakdown and batching is shown in Tables 8-15 and 8-16, using the same example problem described in Tables 8-13 and 8-14. Table 8-15 lists the gradations for the coarse and fine aggregates for the example problem. Table 8-16 describes the breakdown and lists the batch weights for each of the four aggregates. The No. 7 traprock is broken down into five size fractions, ranging from the 12.5- to 19.0-mm fraction to the passing 2.36-mm fraction. The screenings are broken down into three fractions: 4.75 to 9.5 mm, 2.36 to 4.75 mm and passing 2.36 mm. The two sands are both broken down into two

Table 8-15. Aggregate gradations for example batching problem.

Sieve Size (mm)	*Weight Percent Passing for Aggregate:*				
	No. 7 Traprock	Traprock Screenings	Manufactured Sand	Natural Sand	Mineral Filler
19.0	100	100	100	100	100
12.5	93	100	100	100	100
9.5	53	100	100	100	100
4.75	32	90	97	99	100
2.36	9	57	78	91	100
1.18	2	34	51	70	100
0.600	1	25	32	53	100
0.300	1	19	14	29	100
0.150	1	14	11	17	92
0.075	1	8	7	9	79

Example Problem 8-3. (Continued)

Table 8-16. Aggregate breakdown and batch weight for example 3.

Size Fraction	Wt. %	Batch Weight (g)
No. 7 Traprock		
12.5 to 19.0 mm	7	198
9.5 to 12. 5 mm	40	1,133
4.75 to 9. 5mm	21	595
2.36 to 4.75 mm	23	651
Passing 2.36 mm	9	255
Total	**100**	**2,832**
Traprock Screenings		
4.75 to 9. 5 mm	10	342
2.36 to 4.75 mm	33	1,129
Passing 2.36 mm	57	1,950
Total	**100**	**3,421**
Manufactured Sand		
2.36 to 9.5 mm	22	753
Passing 2.36 mm	78	2,669
Total	**100**	**3,422**
Natural Sand		
2.36 to 9.5 mm	9	160
Passing 2.36 mm	91	1,611
Total	**100**	**1,771**
Mineral Filler		
Passing 0.075 mm	100	354

fractions: 2.36 to 9.5 mm and passing 2.36 mm. For both sands, the 4.75 to 9.5 fraction contained less than 5%, and this fraction was combined with the 2.36- to 4.75-mm fraction to create a 2.36- to 9.5-mm fraction. The mineral filler does not need to be further broken down, and the batch weight is as given in Table 8-14.

Heat Aggregates and Asphalt Binder

After batching out the aggregates, they are combined in a suitable metal container and heated in an oven to the desired compaction temperature. Asphalt binder is also heated to the same temperature. The binder will be weighed out into the aggregate after it has been thoroughly heated; the amount placed in the oven should be more than enough to provide for the given batch weight (569 g in this example).

It is essential that the aggregates and binders are thoroughly heated to the proper temperature before mixing. For non-modified binders, the mixing and compaction temperatures are calculated on the basis of binder viscosity. The mixing temperature range is that providing a binder viscosity of from 150 to 190 Pa-s, while the compaction temperature range is that providing a binder viscosity of from 250 to 310 Pa-s. For non-modified binders, mixing and compaction temperatures can be estimated using a viscosity-temperature chart, as shown in Figure 8-7. Log viscosity is plotted against temperature, and a curve fit through the data points. The mixing and compaction temperature ranges can then be estimated from the chart as shown. For modified binders, the manufacturer should supply information concerning the mixing and compaction temperature for HMA made with their product. Mixing and compaction temperatures are usually provided by suppliers as part of the specification data provided on the bill of lading for a given binder.

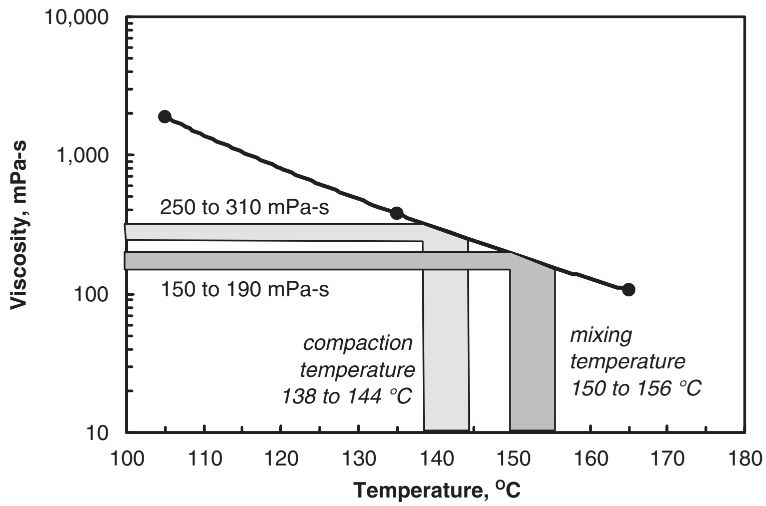

Figure 8-7. Example viscosity-temperature chart showing determination of mixing and compaction temperature ranges for a non-modified binder.

A major research project on mixing and compaction temperatures for HMA was completed as this manual was being completed; the results have been compiled in *NCHRP Report 648: Mixing and Compaction Temperatures of Asphalt Binders in Hot-Mix Asphalt.* Two new promising procedures for determining mixing and compaction temperatures were recommended for further evaluation. The phase angle method involves developing a high-temperature master curve for binder phase angle, determining the frequency where the phase angle is 86 degrees and then applying empirical equations to determine mixing and compaction temperatures. In the steady shear method, viscosity is determined at high shear stresses over a range of temperatures. Viscosity values at a shear stress of 500 MPa are then plotted on a log viscosity versus log temperature chart to determine mixing and compaction temperatures. At the time this manual was being completed, neither method had been accepted as an AASHTO standard, but it is possible than one or both methods could be adopted in the future.

Heating materials prior to compaction will typically take from 2 to 4 hours, but the actual time required to heat aggregates and binders to reach the specified mixing temperature will vary considerably depending on the size and type of oven used, the amount of material being heated, and the properties of the aggregate. The oven should be set to a temperature about 15°C above the mixing temperature range. The actual temperature of the aggregates and binder should be checked prior to mixing and compaction with a properly calibrated electronic thermometer. Mixing bowls, mixing paddles/stirrers, and gyratory compaction molds must also be heated to the same compaction temperature range prior to compacting specimens.

Mix Aggregate and Asphalt Binder

Because the use of 150-mm-diameter specimens requires very large batch sizes, laboratory mixing should be done with a large, heavy-duty mechanical mixer. Mixing should be done quickly and efficiently, so that the materials do not cool significantly before mixing is completed. If the mix is the first one to be prepared that day, the mixer should be "buttered" first. This is important because significant amounts of binder and fine aggregate will stick to the bowl and stirrer during mixing. If the mixer is not buttered first, binder and fines will be removed from the batch, and its composition will not be as designed. The mixer can be buttered either by mixing a batch of HMA or sand asphalt that is then discarded. The composition of these materials is

Figure 8-8. Typical mixer used in preparing HMA specimens in the laboratory.

not important—their only purpose is to coat the mixing bowl and stirrer with binder and fine aggregate. The mixer need only be buttered prior to the first batch of the day. After that, the mixing bowl and stirrer should remain well coated as additional batches are prepared, so that additional buttering is not needed.

The procedure for actual mixing of HMA in the laboratory is as follows. Place the heated bowl on an electronic balance, and zero the balance. Form a depression in the center of the aggregate, and weight out the appropriate amount of hot binder into the aggregate. Place the bowl with aggregate and asphalt on the mixer stand, attach the heated stirring attachment, and begin mixing. Mix just until the aggregate is thoroughly coated with binder—too much mixing can cause the aggregate to break down, changing the aggregate gradation in the specimen. The mix is now ready for short-term oven conditioning, as described below. Figure 8-8 shows a typical mixer used in preparing HMA specimens in the laboratory.

When HMA is produced in a plant, it is not immediately placed and compacted. Often it is held in a silo, placed in a truck, hauled to the site, and then placed and compacted. During this time period the hot aggregate in the mix may absorb significant amounts of asphalt binder, potentially changing the composition and properties of the mix. Short-term oven conditioning of HMA in the laboratory, as described below, is designed to imitate the absorption of binder that occurs during actual production.

Short-Term Oven Conditioning

The procedure for performing short-term oven conditioning is described in AASHTO R 30, Mixture Conditioning of Hot-Mix Asphalt. Immediately after mixing the aggregate and asphalt, place it in a shallow metal pan, spreading it out evenly until the depth is between 25 and 50 mm. Place the mix in a forced-draft oven, pre-heated to within 3°C of the midpoint of the compaction

Figure 8-9. Short-term oven conditioning of HMA mixture in the laboratory.

temperature range for the mixture. Condition the mix for a total time of 2 hours ± 5 minutes, stirring after 1 hour ± 5 minutes. For mixtures having a water absorption value over 2%, the conditioning time should be extended to 4 hours ± 5 minutes, and the mixture should be stirred every hour. Also, if the mix is to be used to prepare specimens for performance testing, the conditioning time should be 4 hours ± 5 minutes at 135°C, regardless of the aggregate absorption. Specimens are compacted immediately after completion of short-term oven conditioning. Figure 8-9 shows HMA mixture spread out in a pan for short-term oven conditioning.

Compact Laboratory Specimens

In this mix design method, specimens are compacted using the Superpave gyratory compactor (SGC), using the standard angle of gyration of 1.25°, and a compaction pressure of 600 kPa, as described in AASHTO T 312, Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the Superpave Gyratory Compactor. It is essential that the SGC is properly maintained and calibrated; engineers and technicians should refer to appropriate specifications and the manufacturer's instructions for information on maintaining and calibrating their device.

Prior to compacting specimens, make sure that the mold, top plate, and base plate have been heated to the compaction temperature for the mix. This generally takes about an hour in an oven set to the compaction temperature. The HMA mix must be short-term conditioned, as described above, prior to compaction. Remove the molds and plates from the oven, place the base plate inside the mold and place a paper disk on the base plate. Then, place the hot mixture in the mold, level, and cover with another paper disk. Place the top plate over the paper disk, and place the mold in the compactor. Set the compactor to the appropriate level of N_{design} (see Table 8-2) and compact the specimen. After compaction is complete, remove the mold from the SGC, and then remove the specimen from the mold. SGCs are normally equipped with a sample press for extruding compacted specimens from molds. Removing the specimens should be done slowly to avoid distorting or even breaking the specimen. Waiting a few minutes after completing compaction to allow the specimen to cool can help prevent damage to the specimen during de-molding. Specimens compacted to high air void levels—about 6% or more—can be even more prone to damage during de-molding and may require additional cooling before removal from the SGC mold.

Figure 8-10. Mold, top and base plates, and paper disks used in compacting specimens with the superpave gyratory compactor.

Remove the paper disks from the top and bottom of the specimen and allow the specimen to cool at room temperature. Handle freshly compacted specimens carefully to avoid damaging them. Specimens must be completely cool prior to performing bulk specific gravity tests, as required for volumetric analysis. Figure 8-10 shows an SGC mold, top and base plates, and the paper disks used in compacting specimens.

Calculate Volumetric Composition of Laboratory Specimens

Chapter 5 of this manual described in detail volumetric analysis of HMA in the laboratory, including the primary tests involved—bulk and maximum theoretical specific gravity of HMA mixtures. Interested readers or those unsure of the details of these tests and the calculations used in volumetric analysis may wish to review Chapter 5. Some of the calculations are similar to those used in estimating the composition of trial batches as presented previously in Step 9. The information presented here is meant only to be a brief review of the major features of volumetric analysis.

Volumetric analysis of compacted HMA mixtures involves two laboratory tests: bulk specific gravity of the compacted HMA mixture and theoretical maximum specific gravity of the loose HMA mixture. As discussed in Chapter 5 of this manual, there are two procedures for determining bulk specific gravity of HMA mixtures:

- AASHTO T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens, and
- AASHTO T 275, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens.

AASHTO T 166 can be used for most HMA mixtures; however, if the absorption of the specimens during AASHTO T 166 is greater than 2.0%, AASHTO T 275 should be used. The procedure for theoretical maximum specific gravity is given in AASHTO T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures.

Bulk specific gravity gives the specific gravity of the compacted specimen, including air voids within the mixture. The theoretical maximum specific gravity is the specific gravity of the mixture at zero air voids; if a laboratory specimen could be compacted to zero air voids, the bulk and theoretical maximum specific gravity values would be equal. One of the most important equations used in volumetric analysis of HMA is Equation 8-13, which relates air void content, bulk specific gravity of the compacted mixture, and maximum theoretical specific gravity:

$$VA = 100\left[1 - \left(\frac{G_{mb}}{G_{mm}}\right)\right] \tag{8-13}$$

where

VA = air void content, volume %
G_{mb} = bulk specific gravity of compacted mixture
G_{mm} = maximum theoretical specific gravity of loose mixture

Various other equations given in Chapter 5 are used to calculate other important properties or volumetric factors of the mixture:

- Total asphalt content by weight (Pb)
- Effective asphalt content by weight (Pbe)
- Absorbed asphalt content by weight (Pba)
- Total asphalt content by volume (VB)
- Effective asphalt content by volume (VBE)
- Absorbed asphalt content by volume (VBA)
- Voids in the mineral aggregate (VMA)
- Voids filled with asphalt (VFA)
- Dust/binder ratio (D/B)
- Apparent film thickness (AFT)

The normal practice in HMA mix design is to determine the bulk specific gravity of two compacted specimens, and then heat these specimens and break them up and use the resulting loose mixture to determine the maximum theoretical specific gravity of the mixture. Alternately, extra loose mixture can be prepared when the specimens are compacted and used in the determination of maximum specific gravity. Actual calculation of air void content, VMA, and other volumetric factors is usually done using a spreadsheet such as HMA Tools. Most SGCs also include spreadsheets for performing these calculations. Values for specified volumetric factors are then compared to the requirements for the mixture. In a complete mix design for new materials, this comparison is made for three trial mixtures made at widely different coarse aggregate contents, to determine the aggregate blend that will provide the proper volumetric composition. In many cases, an existing mix design is being slightly modified, and only one or two trial mixtures will be evaluated.

Example Problem 8-4. Volumetric Analysis of an HMA Mixture

Table 8-17 summarizes a typical volumetric analysis as performed in the evaluation of three trial mixes. The dense/fine mixture in Table 8-17 is the same trial mix used in the examples given in Tables 8-13 through 8-16. The other two trial mixes—the dense/dense and dense/coarse—have been developed using the same binder and same aggregates, but blended in different proportions, as listed in Table 8-18; this table also includes proportions for a fourth trial mix, discussed below. Table 8-17 shows the specific gravity test data and calculations and the results of volumetric analysis for all three trial mixtures. Table 8-17 includes specification limits, and also lists equations that can be used for calculating the various volumetric factors, such as air void content and VMA.

Example Problem 8-4. (Continued)

Table 8-17. Summary of volumetric analysis for example HMA mix design.

Property	Equation	Specification Min.	Specification Max.	Trial Mix 1: Dense/Fine Mix		Trial Mix 2: Dense/Dense Mix		Trial Mix 3: Dense/Coarse Mix	
Bulk Specific Gravity of Compacted Mixture									
Dry mass in air, g	---	---	---	5,221.0	5,190.3	5,135.7	5,392.8	5,321.5	5,175.7
Saturated, surface-dry mass in air	---	---	---	5,241.1	5,211.4	5,153.0	5,414.4	5,343.6	5,212.3
Mass in water	---	---	---	3,160.9	3,140.3	3,172.9	3,324.2	3,228.7	3,137.2
Bulk specific gravity, dry basis	5-1	---	---	2.510	2.506	2.594	2.580	2.516	2.494
Theoretical Maximum Specific Gravity of Loose Mixture									
Dry mass in air, g	---	---	---	2,109.7	2,245.5	2,225.8	2,156.9	2,076.4	2,332.7
Mass in water	---	---	---	1,312.3	1,394.3	1,394.0	1,352.9	1,312.0	1,471.5
Theoretical maximum specific gravity	5-2	---	---	2.646	2.638	2.676	2.683	2.716	2.709
Average	---	---	---	2.642		2.679		2.713	
Volumetric Analysis									
Aggregate bulk specific gravity, dry basis	5-3	---	---	2.846		2.879		2.915	
Air void content, Vol. %	**5-4**	**3.5**	**4.5**	**5.1**		**3.5**		**7.6**	
VMA, Vol. %	**5-11**	**14.0**	**16.0**	**15.9**		**14.2**		**17.9**	
Asphalt content, Wt. %	5-5	---	---	4.60		4.55		4.53	
Effective asphalt content, Wt. %	5-9	---	---	4.44		4.27		4.22	
Effective asphalt content, Vol. %	5-8	---	---	10.9		10.8		10.3	
VFA, Vol. %	5-12	---	---	68.2		75.7		57.4	
Mineral filler (dust) content, Wt. %	8-10	---	---	7.9		6.6		5.1	
Dust/binder ratio	**8-11**	**0.8**	**1.6**	**1.79**		**1.54**		**1.20**	

Table 8-18. Aggregate proportions for trial mixes listed in Table 8-17.

Aggregate	Trial Mix No. 1: Dense/Fine	Trial Mix No. 2: Dense/Dense	Trial Mix No. 3: Dense/Coarse	Trial Mix No. 4: Dense/Fine
No. 7 Traprock	24	45	68	58
Screenings	29	21	12	16
Manufactured sand	29	21	12	16
Natural sand	15	10	5	8
Mineral dust	3	3	3	2

Most often, calculations such as those used in compiling Table 8-17 are done using a spreadsheet developed for this purpose, such as HMA Tools or spreadsheets included with many SGCs. In HMA Tools, specific gravity data for the trial batches are entered in the worksheet "Specific_Gravity"; the necessary calculations are performed and the resulting volumetric composition is summarized in the worksheet "Trial_Blends." Three specified volumetric factors are shown in Table 8-17 and included in most volumetric analyses:

- Air void content
- VMA
- Dust/binder ratio

The air void content in this example has an allowable range of 3.5 to 4.5%. This range has been established for practical purposes, since it is very difficult to match the target air void content of 4.0% exactly. Also, it must be realized that laboratory mix designs almost always must be adjusted during field production, so attempting to exactly meet air void requirements in a laboratory mix design is usually pointless. VMA requirements for dense-graded HMA mixtures are given in Table 8-5; the allowable range for VMA for a 12.5-mm NMAS mixture is from 15.0 to 17.0%. The specified range for dust/binder ratio is 0.8 to 1.6, as given in Table 8-12 previously.

Example Problem 8-5. Adjusting an HMA Trial Mixture

Looking at the three specified volumetric factors in Table 8-17, the dense/dense trial mixture appears to meet all specification requirements—the average air void content of 3.5% is acceptable, as is the average VMA of 14.2%; and the dust/binder ratio of 1.54% is also within the specified range. As long as the workability of this mix is acceptable, it would be an acceptable final mix design. However, the air void content, VMA, and dust/binder ratio values are all close to various limits. It is therefore desirable to adjust this mixture to obtain values closer to the midpoint for air voids, VMA, and dust/binder ratio.

In deciding how to adjust the composition of the fourth trial mix, it is helpful to present the data in Tables 8-17 and 8-18 graphically. Figure 8-11 shows the gradation plot (top) and CMD plot (bottom) for the three initial trial mixes. This figure also includes the fourth trial mix, shown as the dashed line. The plots are

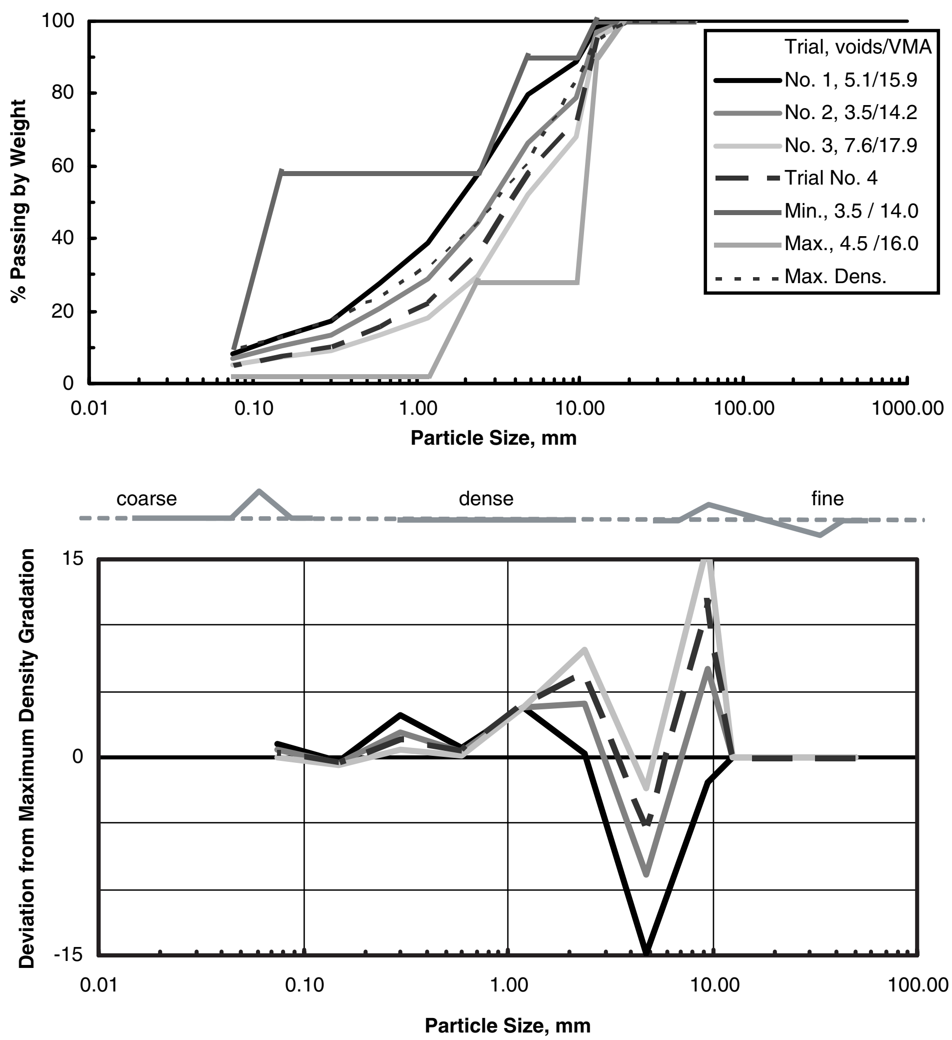

Figure 8-11. Top: gradation plot for example mix design, including fourth trial mix; bottom: CMD plot for example mix design.

Example Problem 8-5. (Continued)

as produced in HMA Tools, though similar plots can be prepared using other spreadsheets or software packages. The legend, given in the top plot, includes the air void content and VMA values for each of the initial three trial mixes, as listed in Table 8-18. It also includes the minimum and maximum for air void content and VMA, given in the legend under "Min." and "Max." As noted above, the second trial mix—the dense/dense mix—meets all requirements for the mix design. However, both the measured air void content (3.5%) and the VMA value of 14.1% are at or very near to the minimum values of 3.5% and 14.0%, respectively. It appears that the air void content and VMA could be increased by either making the gradation slightly finer or slightly coarser. However, because the dust/binder ratio of the dense/fine mix is too high (and only marginal for the dense/dense trial mix), making the fourth trial mix slightly coarser will ensure that the dust/binder ratio remains within allowable limits. The aggregate blend for the fourth trial mix is therefore designed to be intermediate between the dense/dense gradation and the dense/coarse gradation, as is shown in Figure 8-11. It also has slightly less mineral dust (2% rather than 3%) in order to keep the dust/binder ratio near the center portion of the specification.

The next step in the mix design process is to calculate mix proportions and batch weights for the fourth trial mix, in the same way as was done for the initial trial mixtures. Two specimens are then prepared and their bulk and theoretical maximum specific gravity values determined. A second volumetric analysis is performed to determine if this fourth trial mix better meets the specified requirements. The results of specific gravity tests and the resulting volumetric analysis are shown in Table 8-19. The specification properties—air void content, VMA and dust/binder ratio—are well within the specified range. The air void content and VMA are slightly high, but this is desirable since both will tend to drop during field production. Therefore, trial mix four is accepted as the final laboratory mix design.

Table 8-19. Summary of volumetric analysis trial mix four for example HMA mix design.

Property	Specification		Dense/Fine Mix	
	Min.	Max.		
Dry mass in air, g	---	---	5,221.3	5,182.9
Saturated, surface-dry mass in air	---	---	5,237.5	5,206.6
Mass in water	---	---	3,214.6	3,200.4
Bulk specific gravity, dry basis	---	---	2.581	2.583
Dry mass in air, g	---	---	2,115.7	2,268.2
Mass in water	---	---	1,328.6	1,422.7
Theoretical maximum specific gravity	---	---	2.651	2.658
Average	---	---	2.685	
Aggregate bulk specific gravity, dry basis	---	---	2.900	
Air void content, Vol. %	**3.5**	**4.5**	**3.8**	
VMA, Vol. %	**14.0**	**16.0**	**15.0**	
Asphalt content, Wt. %	---	---	4.59	
Effective asphalt content, Wt. %	---	---	4.45	
Effective asphalt content, Vol. %	---	---	11.2	
VFA, Vol. %	---	---	74.5	
Mineral filler (dust) content, Wt. %	---	---	5.0	
Dust/binder ratio	**0.8**	**1.6**	**1.13**	

Adjusting Aggregate Proportions to Meet VMA and Other Volumetric Requirements

The procedure described here for adjusting aggregate proportions to meet the given requirements for VMA and air void content is straightforward. However, a short discussion may help inexperienced technicians and engineers better understand this important topic. First, it should be again emphasized that the procedure presented here is only one of many possible techniques for preparing aggregate blends for trial mixtures prepared during the mix design process. This manual is in large part intended as an instructional tool for the inexperienced; for this reason, the approach given here is relatively simple and flexible and relies on HMA Tools for performing cumbersome calculations. Technicians and engineers who have successfully used other procedures with consistently satisfactory results should continue to use them. Those who try the procedures given here, but think they need some additional tools should look into other procedures; as discussed earlier in this chapter, the Bailey method has recently become a very popular method for blending aggregates to meet volumetric requirements.

The relationship between VMA, air void content, and effective asphalt content must be understood to fully appreciate the procedure given in this manual. VMA is composed of air voids and effective asphalt (a small amount of asphalt binder is absorbed into the aggregate surface). Therefore, if the target VMA is fixed, once the target air void content is met, the effective asphalt content is also met. Therefore, there is no need to simultaneously evaluate air void content and VMA—once the proper air void content is obtained, the VMA level will also meet requirements. HMA Tools makes this calculation for the user, so there is no need to calculate the binder content. However, there is some flexibility in selecting target values for VMA and air void content, which indirectly allows for adjustments in asphalt binder content. Lower VMA values will give less binder, higher VMA values will give more binder. Lower air void contents will provide additional binder at a given VMA value, while higher air void contents will provide less binder at a given VMA value. In the first trial batch in a series, HMA Tools assumes that the amount of absorbed binder is one-half the calculated water absorption (calculated from aggregate bulk and apparent specific gravity values). However, after this first trial batch, HMA Tools compares estimated absorption values with those actually measured in the laboratory and adjusts absorption values in subsequent batches accordingly.

The general rule given here for adjusting aggregate blends to meet VMA requirements is that the closer an aggregate gradation is to a maximum density gradation, the lower will be its VMA. Technicians and engineers should remember that this is only an approximate rule. The maximum density gradation is only approximated by the 0.45 power law; the actual maximum density gradation for a given set of aggregates may deviate significantly from this. Furthermore, some aggregates have unique properties that will affect mixture VMA in unusual ways. For example, relatively soft aggregates can break down during compaction—especially at high gyration levels—making it difficult to reach high VMA values. Other aggregates, with unusually good texture or angular shape, will tend to increase VMA, even when their addition would seem likely to make the aggregate blend denser. The specifications given in this manual for dense-graded HMA may require some mix designs to be adjusted by increasing the mineral filler content. Although a certain amount of mineral filler is necessary for good rut resistance and durability, adding mineral filler to a mix design will normally tend to reduce VMA. Thus, adding mineral filler to a mix design will require adjusting the gradation to provide additional VMA to compensate for the effect of increasing mineral filler. Aggregate blending is one of the most critical aspects of the HMA mix design process, and proficiency requires practice, experience, and judgment.

Conduct Performance Testing as Required

The final stage of laboratory work in an HMA design involves evaluating the performance of the mixture. Chapter 6 of this manual presents a thorough discussion of various factors affecting HMA performance and ways of evaluating this performance through mixture testing and analysis. The following discussion is limited to the practical application of performance testing as part of the routine design of dense-graded HMA mixtures. This involves the evaluation of (1) moisture resistance for all mixtures and (2) evaluation of rut resistance for mixtures designed for traffic levels of 3 million ESALs and higher. As discussed in Chapter 6, more advanced types of performance testing, such as the IDT creep and strength test and fatigue testing, are in general not suitable for use in routine mix design, though they may be useful in research and in developing HMA mixtures for critical or special applications.

Evaluate Moisture Resistance

The moisture resistance of all dense-graded HMA mixtures should be evaluated using AASHTO T 283. Moisture resistance testing is normally performed after a mix design has been developed that meets all requirements for binder grade, mixture composition, and compaction. As also discussed in Chapter 6, in AASHTO T 283, specimens are prepared to an air void content of $7.0 \pm 0.5\%$, then divided into two subsets with approximately equal average air void contents. The tensile strength of one subset is measured dry. The tensile strength of the second subset is measured after conditioning by vacuum saturation followed by a freeze-thaw cycle and a warm-water soak. The ratio of the average tensile strength of the conditioned to unconditioned subsets and a visual assessment of stripping is used to assess moisture sensitivity. A mixture is considered acceptable if the tensile strength ratio is equal to or greater than 80% and there is no visual evidence of stripping in the conditioned test specimens.

There are several ways of improving the performance of mixtures initially failing these requirements for moisture resistance. Small amounts of anti-strip additives can be added to the mixture. Anti-strip suppliers should be contacted concerning recommended products and concentrations for a given HMA mix design. Local hot-mix suppliers may be able to offer suggestions concerning the most effective anti-strip additives for local materials. In many cases, the binder supplier can blend an appropriate anti-strip additive at the necessary concentration directly into the binder. This is a simple but effective approach that requires no special equipment at the plant. One of the most effective and least expensive anti-strip additives is hydrated lime. When used to help prevent moisture damage, hydrated lime should be blended with water to form a slurry and applied directly to the aggregate prior to heating. The typical concentration is 1% hydrated lime by aggregate weight. If used in a laboratory mix design, a slurry composed of 50% hydrated lime and 50% water by weight is prepared and applied to the aggregate prior to heating.

Besides anti-strip additives, the other ways of improving moisture damage are to change the binder, the aggregate, or both materials. Different binders can exhibit a wide range in susceptibility to moisture damage. Aggregates that are most susceptible to moisture damage are those that contain significant amounts of quartz, including many igneous rocks. Eliminating these aggregates from a mix design can, in some cases, significantly improve moisture resistance.

Evaluate Rut Resistance

Before discussing rut resistance testing in detail, it must be noted that the design procedure set forth in this manual has been structured to provide HMA mix designs that will exhibit a high level of rut resistance. The level of reliability against excessive rutting—even without performance testing—ranges from 90 to over 99%, with a typical level of about 95% reliability for design traffic levels of 3 million ESALs and higher. The purpose of rut resistance testing is to increase this level of reliability. For three of the rut resistance tests discussed below—the flow number from the asphalt mixture performance tester (AMPT), the repeated shear at constant height (RSCH) test, and the high-temperature indirect tension (HT-IDT) strength test—the suggested minimum or maximum test values were determined specifically to increase the level of reliability against excessive rutting from about 95 to 98% and higher. It must be emphasized that the reliability achieved through the recommended performance tests is a result of applying both the suggested mix design procedure and the selected performance test together. If the given guidelines for performance test results are applied to mixtures designed following some other procedure, the resulting level of reliability will not necessarily be the same. It might be similar, or it might be lower or higher. It should also be noted that the specified test values have in most cases been selected so that if the procedures given in this manual are followed, most of the resulting HMA mixtures will pass the selected performance test. It is estimated that only about 10 to 20% of properly designed mixtures will fail. Thus, the suggested rutting performance tests not only increase reliability against excessive rutting to a very high level, they do so in a relatively efficient way.

The suggested maximum rut depths for the asphalt pavement analyzer (APA) and the Hamburg Wheel-Track (HWT) tests were taken from specifications already in place in numerous states. In this case, implementation of these performance tests will certainly increase the reliability against excessive rutting, but the specific amount of improvement is unknown as is the percentage of mixes likely to fail the tests. However, because these tests with the stated maximum rut depths have been implemented in several states, it is likely that the increase in reliability and the rejection rate will both be reasonable.

The various rut resistance tests and guidelines are summarized below. The stated minimum or maximum values for each test should be considered guidelines. Although based either on a careful analysis of laboratory and field data or on existing standards, it is quite possible that these values will need to be adjusted by the specifying agency for optimum results in the region. Factors that need to be considered when making such adjustments are climate, the types and grades of binders commonly used in a given locale, aggregates with unusual properties, and typical traffic mixes and traffic levels. For various reasons, some agencies may wish to alter the conditions a test is run under, which will significantly alter the resulting test values and the appropriate specification values. For details on the proper procedures for performing each test, laboratory engineers and technicians should refer to the appropriate standard test method as listed at the end of this chapter. Some additional background on performance testing in general and on these five tests in particular is provided in Chapter 6 of this manual.

The Asphalt Mixture Performance Tester. The asphalt mixture performance tester (AMPT) was initially called the simple performance test system or SPT. Details of the latest equipment specification and test procedure are given in *NCHRP Report 629: Ruggedness Testing of the Dynamic Modulus and Flow Number Tests with the Simple Performance Tester.* Tests are performed on specimens cored and trimmed from large gyratory specimens to final nominal dimensions of 100 mm diameter by 150 mm high. There are three different tests for rut resistance using the AMPT: the dynamic modulus (sometimes referred to as the $|E^*|$ test), the repeated load test (also called the flow number test) and the flow time test. To use the $|E^*|$ test to evaluate rut resistance the $|E^*|$ Implementation Program software must be used. This software was not yet commercially available at the time this manual was written, but should soon be available from AASHTO. In the flow number test, a 600-kPa load is applied to the specimen every second, until the flow point is reached. The flow point represents failure of the specimen, as evidenced by an increasing rate of total permanent strain during the test. Flow number tests are run at the average, 7-day maximum pavement temperature 20 mm below the surface, at 50% reliability as determined using LTPPBind, Version 3.1. Test specimens should be prepared at the expected in-place air void content, typically about 7%. Table 8-20 lists minimum values for flow number determined using the AMPT.

Table 8-20. Recommended minimum flow number requirements.

Traffic Level *Million ESALs*	Minimum Flow Number *Cycles*
< 3	---
3 to < 10	53
10 to < 30	190
≥ 30	740

The flow time test is similar to the flow number test, but a constant load is applied to the specimen and the total deformation is monitored. Test temperature and specimen preparation are identical to those used in the flow number test described previously. It is simply a static creep test and the flow time is the loading time required to initiate tertiary creep, which is the point at which the rate of deformation begins to increase. Recommended minimum flow times are given in Table 8-21.

Table 8-21. Recommended minimum flow time requirements.

Traffic Level *Million ESALs*	Minimum Flow Time *s*
< 3	---
3 to < 10	20
10 to < 30	72
≥ 30	280

The Asphalt Pavement Analyzer. The asphalt pavement analyzer (APA) is growing in popularity among pavement agencies as a test for evaluating the rut resistance of HMA pavements. At the time this manual was written, nine states had specifications for performance testing of HMA pavements using the APA device. The APA test method is available as AASHTO TP 63, which was developed from the test procedure found in Appendix B of *NCHRP Report 508: Accelerated Laboratory Rutting Tests—Evaluation of the Asphalt Pavement Analyzer.* In the APA test, a pressurized hose is placed over a short cylindrical HMA specimen and a wheel is repeatedly passed over the hose. The rut depth is measured after several thousand cycles. Details of the APA test are given in Chapter 6 of this manual. Typical conditions for the APA test—and the ones suggested here for using this procedure as a performance test—are as follows:

- Hose pressure: 100 lb/in^2
- Wheel load: 100 lbf
- Seating cycles: 50
- Test cycles: 8,000
- Specimen size: 75-mm deep by 150-mm diameter
- Specimen air void content: 4.0 ± 1.0%
- Rut depth calculated as the average of three tests of two specimens (six specimens total)

The APA test is most frequently run at 64°C. However, to account for differences in local climate, it is suggested that the APA test be run at the temperature corresponding to the high-temperature binder performance grade specified for the project by the agency for traffic levels of 3 million ESALs or more. Suggested criteria for the APA in terms of maximum rut depth after 8,000 loading and 50 seating cycles are given in Table 8-22. These guidelines are based on values used by the Oklahoma Department of Transportation and are fairly typical for agencies using this test. However, suitable limits for the APA test will depend on the test conditions—if the test conditions are varied, the maximum rut depths may also need to be changed. Furthermore, as with the other performance test guidelines given in this manual, agencies using the APA as a performance test should modify the maximum allowable rut depths given in Table 8-22 if, in their judgment, such modifications are needed to account for unusual conditions or materials.

Table 8-22. Recommended maximum rut depths for the APA test.

Traffic Level *Million ESALs*	Maximum Rut Depth *mm*
< 3	---
3 to < 10	5
10 to < 30	4
≥ 30	3

The Hamburg Wheel-Track Test. The Hamburg wheel-track test (here termed the "Hamburg test") is, like the APA test discussed above, a "torture" test for evaluating the rut resistance or moisture resistance of HMA mixtures, but the procedure given here is meant only to evaluate rut resistance. In the Hamburg test, a 204-mm (8-in)-diameter, 47-mm-wide steel wheel is passed over an HMA slab immersed in a heated water bath. The Hamburg test is not as widely used as the APA, so it is not possible to provide typical test conditions and guidelines. The following test conditions are used by the Texas Department of Transportation:

- Specimen dimensions: 150 mm (6 in) in diameter, 62 ± 2 mm- (2.4 in) thick
- Wheel load: 705 ± 2 N (158 ± 0.5 lb)
- Air void content: 7 ± 1%
- Test temperature: 50 ± 1°C

The requirements for the Texas version of the Hamburg test are given in Table 8-23. As discussed above, developing typical guidelines for the Hamburg test is difficult because the test is not widely used. A detailed procedure for the Hamburg wheel-track test is given in AASHTO T 324. Because this test is not as widely used as some others of this type, agencies wishing to use the Hamburg test as a performance test should consider performing an engineering study to develop appropriate requirements for their local conditions and materials.

Table 8-23. Texas requirements for Hamburg wheel tracking test.

High Temperature Binder Grade	Minimum Passes to 0.5-inch Rut Depth
PG 64 or lower	10,000
PG 70	15,000
PG 76 or higher	20,000

Superpave Shear Tester/Repeated Shear at Constant Height. The Superpave Shear Tester, or SST, can also be used effectively to evaluate the rut resistance of HMA mixtures using the repeated shear at constant height (RSCH) test. Like the flow number test, the RSCH test is a repeated load test, however, the load is applied in shear rather than in compression as in the flow number test. The primary test result is the maximum permanent shear strain (MPSS), which is the total accumulated shear strain at 5,000 loading cycles. However, the SST is a complicated, expensive piece of equipment and the RSCH test can be difficult to run. Therefore, it is not recommended that commercial laboratories, hot-mix producers, and similar organizations purchase SST devices for use in routine mix design work. Either the AMPT or the IDT strength test are far better suited for routine use in HMA mix design and analysis. However, some laboratories in the United States and Canada have SST devices and use them regularly both for research purposes and in the design and analysis of critical or unusual HMA mix designs. As an aid to these laboratories, Table 8-24 gives the recommended maximum allowable values for MPSS. AASHTO has developed a standard procedure for the test, listed under Standard T 320, Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester. The maximum values for MPSS given in Table 8-24 are based on specimens prepared at 3.0 ± 0.5% air void content, as recommended in AASHTO T 320.

Table 8-24. Recommended maximum values for MPSS determined using the SST/RSCH test.

Traffic Level *Million ESALs*	Maximum Value for MPSS *%*
< 3	---
3 to < 10	3.4
10 to < 30	2.1
≥ 30	0.8

Indirect Tensile Strength at High Temperatures. Recently some studies have been done supporting the use of high-temperature indirect tensile (HT-IDT) strength to evaluate the rut resistance of HMA mixtures; a good description of this test (and other performance tests) can be found in "New Simple Performance Tests for Asphalt Mixes" in *Transportation Research Circular E-C068.* Although not as widely used as the other methods given here, it is a very simple inexpensive test that most engineers and technicians are already familiar with. The HT/IDT strength test is performed as described in AASHTO T 283 for unconditioned (dry) specimens, but at a test temperature that is *10°C below* the average, 7-day maximum pavement temperature, 20 mm below the pavement surface at 50% reliability, as determined using LTPPBind, Version 3.1. Unlike AASHTO T 283, in this procedure, specimens should be compacted using the design gyrations, which should produce an air void content close to 4.0%. As described in AASHTO T 283, the specimens are conditioned at the test temperature by placing them in a water bath controlled to within ± 0.5°C of the test temperature for 2 hours ± 10 minutes. The specimens should be wrapped tightly in plastic or placed in a heavy-duty, leak-proof plastic bag prior to conditioning, to prevent them from getting wet. Table 8-25 lists recommended minimum values for HT/IDT strength determined following this protocol.

Table 8-25. Recommended minimum high-temperature indirect tensile strength requirements.

Traffic Level *Million ESALs*	Minimum HT/IDT Strength *kPa*
< 3	---
3 to < 10	270
10 to < 30	380
≥ 30	500

Design Traffic Speed, Depth within the Pavement and Performance Test Requirements

Earlier in this chapter the effect of traffic speed on binder grade was discussed—as design traffic speed decreases, the required high-temperature binder grade increases significantly (see Table 8-1). This is because loading at low speeds will cause much more rutting in a pavement than loading at fast speeds, all else being equal. For this reason, the test requirements given above for the various performance tests should be adjusted if the design traffic speed is slow (25 to < 70 kph or 15 to < 45 mph) or very slow (< 25 kph or < 15 mph). Perhaps the simplest approach to making these adjustments, and the one recommended in this manual, is to adjust the test temperature upwards as design traffic speed decreases. The recommended temperature adjustment is +6°C for slow traffic and +12°C for very slow traffic. However, as with other aspects of performance testing, agencies should use judgment and experience with local conditions and materials when establishing performance test requirements for slow and very slow traffic speeds.

For many applications, performance testing is probably only necessary on material placed within 100 mm of the pavement surface. However, for critical projects, material placed 100 to 200 mm within the pavement surface might also be tested. In such cases, the test temperature should be adjusted to reflect the estimated temperature at the surface of the material as placed within the pavement structure, as determined using LTPPBind. For example, a base course placed 100 mm below the pavement surface will have an estimated critical high pavement temperature 7.7°C lower than material at the pavement surface; the test temperature for this material would then be reduced by 7.7°C.

Adjusting Mix Designs to Improve Rut Resistance

As mentioned earlier in this section, the rut resistance tests and recommended minimum and maximum values for test results have been selected so that most dense-graded HMA designs developed following the procedures given in this manual will meet the requirements, and no additional laboratory work will be needed. However, some mix designs will fail to meet requirements for rut resistance. In such cases, the test results should first be checked to make sure there were no errors in either the procedures used or in the calculation of the test results. If no errors are found, and the test results are close to meeting the requirements, the test can be repeated. In this case, the results of both tests should be averaged and compared to the test criteria. If the mix still fails to meet the requirements for rut resistance testing, the mix design will have to be modified. Rut resistance of an HMA mix design can be improved as follows:

- Increase the binder high-temperature grade.
- If the binder is not modified, consider using a polymer-modified binder of the same grade or one high-temperature grade lower.
- If the binder is polymer-modified, try a different type of modified binder.
- Increase the amount of mineral filler in the mix, adjusting the aggregate gradation if necessary to maintain adequate VMA.
- Decrease the design VMA value, if possible, by adjusting the aggregate gradation.
- Replace part or all of the aggregate (fine or coarse or both) with a material or materials having improved angularity.

If a different asphalt binder is used in the mix, the volumetric composition should not change. However, if other aspects of the mix design are changed, the volumetric composition might change significantly, which will require further refinement of the mix prior to further rut resistance testing.

Step 11. Compile Mix Design Report

The final step in preparing an HMA mix design is compiling a report documenting the mix design. In many states, standard forms must be filled out by hot-mix producers and submitted to the appropriate state agency or office for approval. In some cases, engineers or technicians may wish to develop their own mix design reports, for internal purposes or for use on private jobs. In such cases, the following information should be included in the report:

- The organization that performed the mix design.
- The name of the technician or engineer responsible for developing the mix design.

- The date the mix design was completed.
- The name of the client for which the mix design was developed.
- The name of the project for which the mix design was developed (if applicable).
- General mix design information, including the type of mix (surface course, intermediate course, base course), the nominal maximum aggregate size, the design traffic level, the N_{design} value, and any special requirements.
- Complete aggregate information, including for each aggregate the producer, the size designation of the aggregate, gradation, specific gravity, and all applicable specification properties.
- Binder information, including the binder performance grade and the name of the supplier.
- Composition of the mixture, including the design air void content, the design VMA, the design VBE, the mineral filler content, the target dust/binder ratio, and the estimated unit weight for the mix
- Brief comments on the workability of the mix.
- The results of moisture resistance testing.
- The results of rut resistance testing, if applicable (generally for mixtures designed for traffic levels of 3 million ESALs and over).

The spreadsheet, HMA Tools, can generate a comprehensive mix design report containing all of this information as well as additional information on the results of trial mixtures evaluated during the mix design process. This report might be useful to some engineers and technicians for internal purposes and might also serve as a template for those wishing to develop their own customized mix design report.

Bibliography

AASHTO Standards

M 320, Performance-Graded Asphalt Binder
M 323, Standard Specification for Superpave Volumetric Mix Design
R 30, Mixture Conditioning of Hot-Mix Asphalt (HMA)
R 35, Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt (HMA)
T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens
T 209, Theoretical Maximum Specification Gravity and Density of Bituminous Paving Mixtures
T 269, Percent Air Voids in Compacted Dense and Open Asphalt Mixtures
T 275, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens
T 283, Resistance of Compacted Asphalt Mixture to Moisture-Induced Damage
T 312, Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the Superpave Gyratory Compactor
T 320, Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester.
T 324, Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA)
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CHAPTER 9

Reclaimed Asphalt Pavement

The asphalt paving industry is a leader in the use of recycled products. HMA pavement is the most recycled material in the world. In addition to recycled asphalt pavement (RAP), other recycled materials are routinely used in HMA mixtures including

- Foundry sand
- Glass
- Roofing shingles
- Slag
- Tire rubber

Recommendations for designing HMA with recycled materials other than RAP are not included in this manual. In areas where they are used, HMA producers have worked closely with agencies to modify mixture design and production methods to effectively use recycled products. Engineers and technicians considering the use of such products in HMA should obtain specific guidance from agencies with substantial experience in their use. The references at the end of this chapter include several publications addressing various recycled products used regionally in HMA.

RAP, on the other hand, is a recycled product that is used extensively in HMA throughout the United States. RAP is old asphalt pavement that has been removed from a road by milling or full depth removal. With appropriate mixture design and production considerations, RAP can be reused in HMA to produce mixtures meeting normal specification requirements. The use of RAP in HMA eliminates the need to dispose of old asphalt pavements and conserves asphalt binders and aggregates. This results in significant production cost savings and benefits to society.

RAP has been effectively used in HMA since the mid 1970s. Advances in HMA plants and processing equipment make it possible to consider mixtures with RAP contents of 50% or more. Experience has shown that, when properly designed and constructed, HMA mixtures with RAP will perform as well as mixtures produced from all new materials. The remainder of this chapter presents recommended methods for designing HMA mixtures with RAP. These methods are based primarily on those contained in *NCHRP Report 452: Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual.*

General Mixture Design Considerations for RAP

The design of HMA containing RAP assumes that the mixture is produced using a process that results in complete mixing of the RAP with the new binder and aggregate. When complete mixing occurs, the total binder content of the mixture includes both the binder contained in the RAP and the amount of new binder added. In this situation, the properties of the binder in the mixture are the blended properties of the new binder and the RAP binder. Additionally, the gradation and other properties of the aggregates are those for the combined blend of the new and RAP aggregates. An important consideration in the production of HMA containing RAP is ensuring that adequate mixing of the new and RAP materials occurs.

The design of HMA containing RAP requires substantial testing to characterize the properties of the RAP—testing that is not needed in mix designs that do not contain RAP. To effectively design and produce any mixture with RAP, the following properties of the RAP must be determined:

- RAP binder content
- RAP aggregate gradation
- Specific gravity of the RAP aggregate
- Angularity of the coarse aggregate particles in the RAP
- Flat and elongated particles for the coarse aggregate in the RAP
- Angularity of the fine aggregate in the RAP

To determine these properties, the binder must be removed from the RAP using either an ignition oven or solvent extraction. The ignition oven is preferred when correction factors can be reasonably estimated and it is known that the properties of local aggregates are not altered substantially by exposure to the high temperatures in the ignition oven.

When the RAP is limited to a low percentage of the new mixture, 15% or less (as a percentage of the aggregate blend), it is not necessary to determine the properties of the RAP binder. Small amounts of the aged RAP binder have little effect on the properties of the mixture. When higher percentages of RAP are used, the stiffness of the RAP binder must be considered. This can be done in several ways. Some agencies simply require that mixtures containing RAP use a softer binder than normally used in their region. A more precise approach involves using a blending chart or a spreadsheet to estimate the grade of the binder formed by blending the RAP binder and the new binder. When a blending chart (or an appropriately designed spreadsheet such as HMA Tools) is used, samples of the RAP binder must be obtained for testing through the use of solvent extraction and subsequent recovery of binder. The recommended blending analysis considers the high, intermediate, and low pavement temperature properties of both the RAP binder and the new binder. The objective of the analysis is to obtain a blended binder meeting the performance grade requirements for the environment where the pavement will be constructed.

The range of RAP that can be feasibly added to a mixture is an important consideration in the design of HMA with RAP. The minimum RAP content depends on the capability of plant equipment to accurately and consistently add the RAP. For most plants, this amount is on the order of 10 percent. The maximum amount of RAP that can be added to a mixture depends on several factors including

- Amount of RAP available
- Specification limits
- Capability of the hot-mix plant to dry, heat, and effectively mix the RAP material
- Gradation of the RAP aggregate, particularly the amount of material passing the 0.075-mm sieve
- Variability of the RAP
- Properties of the RAP binder and available new binders

Often the maximum RAP content is governed by the amount of material in the RAP passing the 0.075-mm sieve and the variability of the RAP. In recent years, equipment to size (or fractionate) RAP has been developed to overcome these limitations. This equipment can separate RAP into various sizes for more effective use in HMA mixtures. Sizing the RAP reduces variability in the gradation and binder content and provides the flexibility to use specific components of the available RAP in different mixtures. A consequence of this practice is that the design of mixtures incorporating RAP may require the consideration of multiple RAP stockpiles each with a specific aggregate gradation and binder content.

Overview of the Mixture Design Process with RAP

Figure 9-1 is a flowchart for designing HMA mixtures with RAP. In the volumetric design and performance analysis, RAP stockpiles are treated much the same as new material stockpiles. RAP stockpiles, however, require additional testing to characterize the properties and the variability of the RAP material.

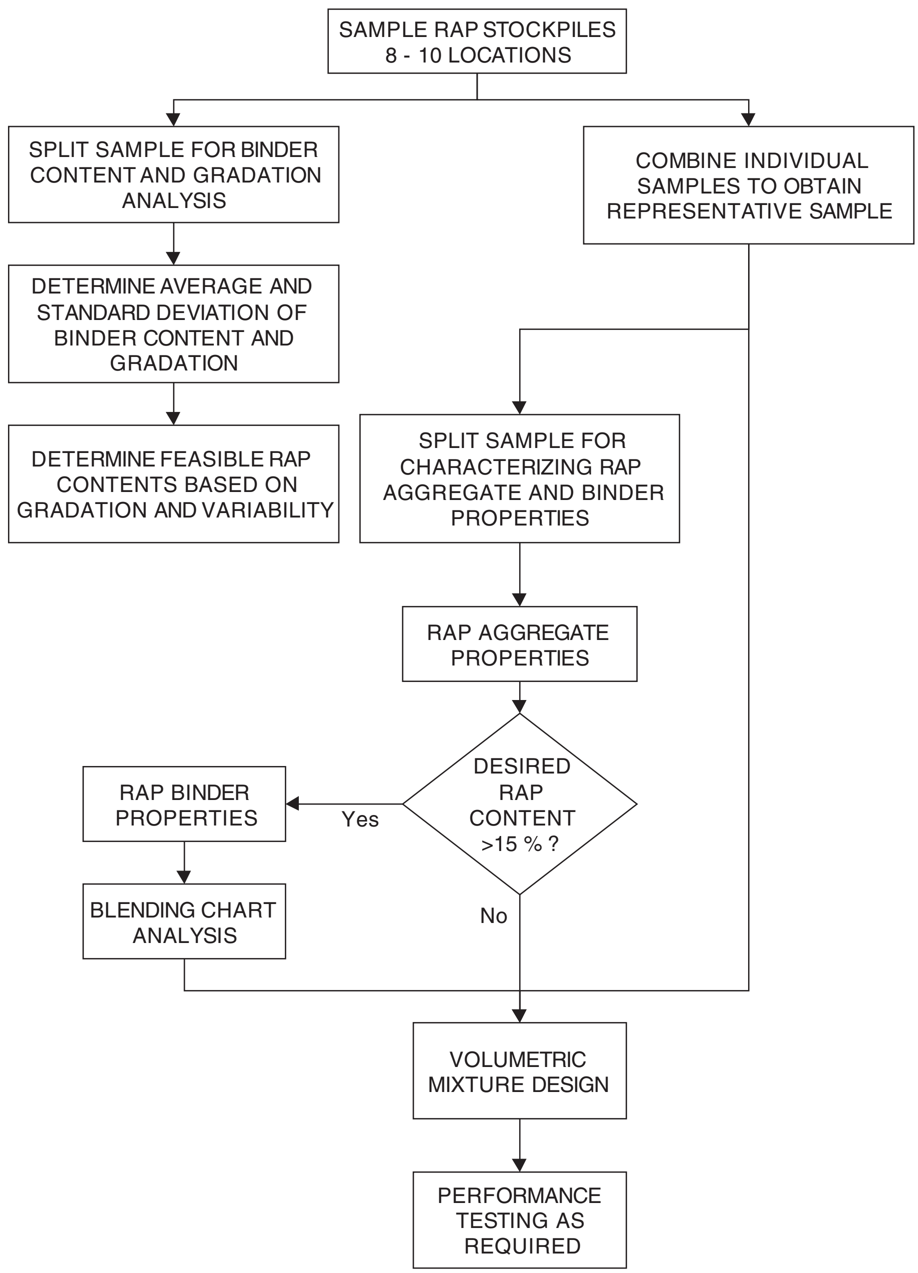

Figure 9-1. Overview of HMA mixture design with RAP.

When RAP is used in HMA, the first step in the mixture design process is to obtain samples of the RAP. Samples should be taken from random locations in each RAP pile. A portion of each sample is then used to determine the average and standard deviation of the binder content and gradation. This information is used to estimate feasible RAP contents that will satisfy gradation and variability requirements. The remaining portion of the RAP samples for each stockpile are combined to provide (1) a representative sample for determining RAP binder and aggregate properties and (2) material for preparing specimens for the volumetric design and performance analysis.

If the desired RAP content exceeds 15% or a limit specified by the agency, the properties of the RAP binder are analyzed and used to create a blending chart for the new and RAP binder. This blending chart can be used to determine the maximum RAP content of a specified new binder to ensure the blended binder meets the specified grade. Or it can be used to determine the grade of new binder needed for the desired RAP content to ensure that the blended binder meets the specified grade. If more than one RAP stockpile will be used in a mixture, the binder analysis is performed using a blended sample of the RAP stockpiles.

The results of the variability and RAP binder analyses are used to select the final RAP content and performance grade of the new binder that will be used in the mixture. The volumetric design and performance analysis then proceed in the same manner as for a mixture produced with all new materials. The only difference is the binder content in the RAP stockpiles must be accounted for when computing binder contents and preparing specimens.

A Note on General Methods of Handling RAP

This manual assumes that RAP will be collected and stored in discrete stockpiles, with each stockpile representing a single RAP source or several very similar and carefully handled RAP sources. There are several good reasons for handling RAP in this way. Perhaps most importantly, it ensures that each stockpile is relatively uniform in character and will not vary too much as the stockpile is depleted during production. It makes characterizing the RAP simpler and more accurate, since the variability in the RAP source is minimized. Some agencies require that RAP be handled in this way and place restrictions on the size of RAP stockpiles that can be used at HMA plants.

However, there are other ways to handle RAP in HMA plants. One approach gaining popularity is to fully process the RAP—to sieve and blend it into carefully controlled size fractions which are then stored and handled in much the same way as aggregates. This approach has many advantages, but because it is relatively new it is not directly addressed in this manual. Many aspects of the mix design process, such as determining binder grades for mix designs containing RAP, will be essentially the same regardless of how the RAP is handled. One important aspect of the mix design process that will depend on how RAP materials are handled at the plant is the effect of RAP variability on the allowable RAP content. Producers using highly processed RAP and other alternative approaches to handling RAP should rely on their experience and judgment in determining the maximum amount of RAP that can be used in HMA mix designs without unacceptably increasing production variability.

Using HMA Tools to Design HMA Mixes with RAP

Many of the equations used in the design of HMA mixes containing RAP are complex, and the calculations required to properly complete such designs can be tedious and prone to error. The HMA Tools spreadsheet has been constructed to simplify the incorporation of RAP into dense-graded HMA mix designs. Technicians and engineers need not be overly concerned with the mathematics involved in incorporating RAP into HMA mix designs, but technicians and engineers should understand the principles involved. The information in this chapter, therefore, focuses on these principles—not the mathematical details. Readers interested in the technical details underlying the information in this chapter should refer to the Commentary on the HMA Mix Design Manual. Frequent reference is also made to how the HMA Tools spreadsheet is used in the mix design process; the examples have been constructed around HMA Tools.

In compiling this chapter, it is assumed that the reader has reviewed Chapter 8 on dense-graded HMA mix designs and understands both the concepts involved in the mix design process and the use of most of the functions in the HMA Tools spreadsheet. Readers unfamiliar with either of these topics should take the time to review Chapter 8.

RAP Sampling

Samples of RAP can be obtained from several locations, including the roadway by coring, trucks hauling milled material for stockpiling, stockpiles prior to processing, and stockpiles after processing. Milling and processing operations break down some of the aggregate in the RAP, producing finer gradations with increased percentages passing the 0.075-mm (#200) sieve. Since a representative sample of RAP will be used directly in the mixture design process, it is critical that RAP samples obtained for HMA mixture design be representative of the RAP that will be used in the mixture. For this reason, the location recommended for sampling RAP is from stockpiles of the RAP after final processing.

Sampling RAP from a stockpile is similar to sampling aggregate from a stockpile, except crusts form on RAP stockpiles that must be removed before samples can be taken. General guidance for stockpile sampling is provided in AASHTO T 2. As discussed in AASHTO T 2, segregation is a major concern when sampling from stockpiles. The preferred approach for stockpile sampling is to use a loader or other power equipment to obtain smaller sampling piles from selected locations within the main stockpile. The RAP sample for each location is then taken from the smaller sampling pile.

RAP stockpiles should be sampled from at least five locations distributed throughout the pile. A larger number of samples—up to 20 or more—is desirable, since this will allow a more accurate characterization of the RAP and permit the highest possible RAP content in the final mix design(s). The sample size depends on the number of mixture designs that will be developed using the RAP in the stockpile. At each sampling location, obtain 10 kg (22 lb) of RAP for each mixture design that will be prepared, plus 5 kg (11 lb) for characterization of the RAP. For example, if three mixtures (a base, intermediate, and surface) will be designed using the same RAP, then obtain 35 kg (77 lb) of RAP at each sampling location. This sample size will provide sufficient material to characterize the properties and variability of the RAP stockpile and provide a representative sample for use in mixture design.

There are two objectives for the RAP sampling. The first is to determine the average and variability (in terms of standard deviation) of the binder content and aggregate gradation in the RAP in the stockpile. For this analysis, a 5 kg (11 lb) sub-sample should be split from the sample taken at each stockpile location using the methods described in AASHTO T 248. This sub-sample will then be tested to determine the average and standard deviation of the binder content and aggregate gradation within the RAP stockpile. As discussed in the next section, variability is an important consideration in the selection of feasible RAP contents that can be considered in design. The second objective of the sampling is to obtain representative materials for the mixture design. A representative sample is formed by combining the remaining portion of the samples from each sampling location. From this representative RAP sample, aggregate properties and, if required, binder properties will be determined and specimens for volumetric and performance analysis

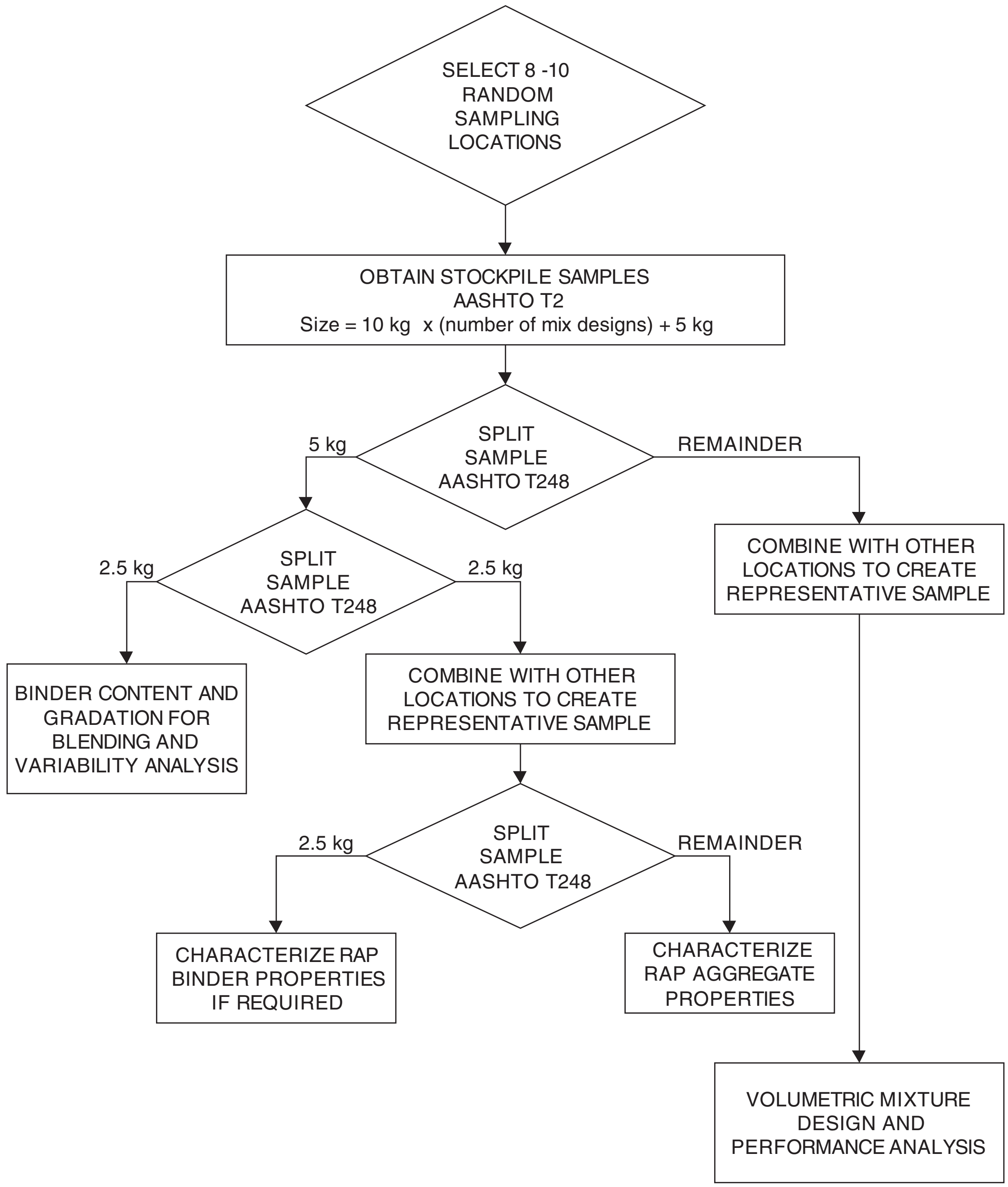

Figure 9-2. Flow chart for recommended sampling of RAP stockpiles.

prepared. Figure 9-2 is a general flowchart for sampling a RAP stockpile for use in HMA mixture design.

Blending and Variability

RAP variability is an important consideration in the design of HMA incorporating RAP. Many agency specifications require mixtures with RAP to be produced to the same production tolerances as mixtures made with all new materials. If highly variable RAP is used, then the HMA may not meet production tolerances, resulting in lost production time, a penalty, or, in extreme cases, the need to remove and replace the mixture. The amount of RAP that can be added without exceeding specification limits depends on the limits themselves, the variability of the RAP, the variability of similar mixtures produced without RAP, and the consistency of the equipment adding the RAP. Unfortunately, the calculation of maximum RAP content based on a variability analysis is quite complicated and often some of the information required is uncertain or even unknown. For these reasons, the approach recommended in this manual is somewhat simplified, but based soundly on the basic statistical theory involved. It will provide engineers and technicians with reasonable estimates of the amount of RAP that can be incorporated into a mix design without unacceptable increases in production variability. Two different methods can be used to determine the maximum allowable RAP content based on variability: a graphical approach and the HMA Tools spreadsheet. The same statistics are used in both cases, but HMA Tools is a more precise approach that will often allow for somewhat higher RAP contents.

Because many state highway agencies already have specifications in place establishing allowable RAP contents in HMA mixtures, HMA Tools allows any RAP content in the design process. If desired, HMA Tools can be used to perform the statistical analysis of RAP stockpiles and to determine the maximum allowable RAP content. However, this is not required. Thus, the user can enter any desired RAP content when using HMA Tools to develop mix designs.

RAP Binder Content and Aggregate Gradation: Laboratory Procedures

The average and standard deviation of the binder content and aggregate gradation in the RAP stockpiles are properties that must be measured to effectively design HMA with RAP. The 5 kg sub-samples split from the sample taken at each sampling location are used for this analysis. If reasonable estimates of the ignition oven correction factors for local aggregates can be made, then the RAP binder content can be determined using an ignition oven, AASHTO T 308. The gradation of the RAP aggregates is then determined using AASHTO T 30 after application of the aggregate correction factors as described in AASHTO T 308. If correction factors for local aggregates are unknown or highly variable, then the RAP binder content must be determined by solvent extraction, AASHTO T 164. The gradation of the extracted aggregate is then determined using AASHTO T 30. If desired, ignition oven correction factors can be established by performing both analyses on split samples from at least three locations in the stockpile.

Determining Combined Gradation and Binder Content

The computation of blends for mixtures incorporating RAP is a little different than that for mixtures made with all new stockpiles. When RAP is used, the RAP material that is added includes both the RAP aggregate and the RAP binder. Since gradation data are based on the weight of aggregate, and binder contents are based on the total weight, the stockpile percentages must be adjusted for combined gradation analysis based on the amount of binder contained in the RAP. In using HMA Tools to determine the composition of HMA mixes containing RAP, data on the aggregate gradations—including the gradation of aggregate contained in the RAP—is entered in the worksheet "RAP_Aggregates." Aggregate bulk and apparent specific gravity data is also entered here, along with the asphalt binder content and specific gravity. Data for up to four RAP stockpiles can be entered. General information for the mix—most importantly the target VMA and air void content—are entered in the worksheet "General." The actual composition of the blend is entered in the worksheet "Trial_Blends," which then lists the combined gradation and various other data for the mix. The example below illustrates the use of HMA Tools in calculating the composition of an HMA mixture containing RAP.

Example Problem 9-1. Gradation and Binder Content Analysis for an HMA Mixture Containing RAP

A 9.5-mm NMAS mixture will be produced using three new aggregate stockpiles, hydrated lime, and two RAP stockpiles. Gradation and binder content data for each stockpile is given in Table 9-1. The stockpiles will be combined in the following proportions: 29% of Aggregate 1, 40% of Aggregate 2, 10% of Aggregate 3, 1% lime, 10% Coarse RAP, and 10% Fine RAP. The target VMA should be 16.0% and the target air void content 4.0%. The specific gravity value for both the new binder and the RAP binders is 1.030. For this combination of stockpiles, compute the combined gradation, the binder provided by the RAP, and the amount of new binder required.

Table 9-1. Stockpile materials for example 1.

Property		Agg. 1	Agg. 2	Agg. 3	Hydrated Lime	Coarse RAP	Fine RAP
	Sieve Size, mm						
Gradation, % Passing	19.0	100	100	100	100	100	100
	12.5	100	100	100	100	100	100
	9.5	91	100	100	100	94	100
	4.75	19	98	90	100	34	91
	2.36	6	61	52	100	25	65
	1.18	5	37	31	100	22	46
	0.600	4	24	20	100	20	34
	0.300	4	16	14	100	16	25
	0.150	3	8	10	96	12	19
	0.075	2.9	3.6	8.4	89	10.4	15.7
Agg. Bulk Spec. Grav.		2.610	2.627	2.619	2.602	2.624	2.614
Agg. App. Spec. Grav.		2.628	2.651	2.645	2.675	2.638	2.637
Binder Content, %		---	---	---	---	3.1	4.5

Solution

The general mix information is entered in the worksheet "General"; this must include the target VMA of 16% and the target air void content of 4%. The gradation information and specific gravity values for the aggregates are entered in the worksheet "Aggregates" and for the RAP in the worksheet "RAP_Aggregates." Asphalt binder content and other information for the RAP is also entered in this worksheet. The composition of the blend, that is, the weight percentage of each aggregate and the RAP materials, is entered in the worksheet "Trial_Blends" as Trial No. 1. Make sure to enter the target VMA and target air void content in this worksheet. The total binder content for the mix is given in cell F43 as 5.46%. Of this amount, 0.75% is from the RAP (cell F143) and 4.72% from the new binder (cell F145). The combined gradation for this example problem is given in Table 9-2.

Table 9-2. Combined gradation for the proposed HMA mixture for example 1.

Sieve Size, mm	Percent Passing
19	100
12.5	100
9.5	97
4.75	67
2.36	41
1.18	27
0.60	19
0.30	14
0.15	9
0.075	6.6

Limiting Variability in HMA Mixes Containing RAP

Because the variability of the RAP properties in a given stockpile can be quite large, it is important to estimate this variability and make sure that the addition of the RAP to an HMA mixture will not cause unacceptable increases in production variability. Controlling variability in mixes containing RAP involves three steps:

1. Sampling RAP stockpiles, as described previously.
2. Calculating the standard deviation for aggregate percent passing and binder content for all RAP stockpiles.
3. Estimating the maximum amount of RAP that can be added to the mix without exceeding allowable production variability.

The variability of a mixture of several components, such as HMA, depends on the variability of the components, the proportions of the components, the precision of the blending, and the mean value of the components. Calculation of the standard deviation for aggregate gradations (percent passing) and asphalt content for HMA containing RAP can be quite complicated; as with other aspects of developing RAP mix designs, the details are not presented here but are included in the Commentary. The mean and standard deviation values for aggregate gradation and asphalt binder content are calculated in HMA Tools in the worksheet “RAP_Variability.” Data for aggregate gradation and binder content are entered here for up to four RAP stockpiles. Data for up to 30 specimens can be entered for each of these four stockpiles. If no more than 15% RAP is to be used in a mix design, there is no need to perform a variability analysis of the RAP stockpiles used in the mix design.

The standard deviation values calculated in the worksheet “RAP_Variability” are only estimates of the true values. There is a 50% chance that the true standard deviation for a given RAP stockpile will be higher than the calculated value. There is a relatively small chance that the true standard deviation will be much, much higher than the estimated value. Because the standard deviation values calculated for the RAP stockpiles are only estimates, the values used by HMA Tools in determining the maximum allowable RAP content is an upper confidence limit, rather than the calculated value. In cell B6, the reliability (confidence) level for this estimate is entered; a value of 80% is suggested. The more samples used in calculating the standard deviation, the more accurate the estimate will be and the lower the value of upper confidence limit for the standard deviation. Therefore, it is suggested that at least five samples be used for calculating the mean and standard deviation for a RAP stockpile. Larger numbers of samples—up to 30—will provide greater accuracy and will normally allow greater percentages of RAP to be used in the mix design.

In using HMA Tools to perform a variability analysis of RAP stockpiles, gradation data for up to 30 samples for the first RAP stockpile are entered in cells B19:AE31; asphalt content is entered in cells B33:AE33. Gradation data for up to three more RAP stockpiles are entered in cell ranges immediately below this. The approximate proportions to which the stockpiles will be blended must be entered in cells B9:B12. The estimated maximum allowable RAP content will appear in cell B14 when data entry is complete and the calculation is completed. Because up to 15% RAP can be used in any HMA design without performing a variability analysis, the maximum allowable RAP content will never go below 15%. Similarly, since handling large amounts of RAP during HMA production is often difficult for practical reasons, HMA Tools limits the maximum allowable RAP to 50%. If needed, the average percent passing and standard deviation for percent passing for the first RAP stockpile can be read in cells AH19:AH31 and AI19:AI31, respectively. The average and standard deviation of the asphalt binder content

Example Problem 9-2. Calculation of Mean, Standard Deviation, and Maximum Allowable RAP Content for a Single RAP Stockpile

A 30,000-ton RAP stockpile was constructed using millings from several projects. Table 9-3 summarizes the results of binder content and gradation tests on 10 random samples from the stockpile. Calculate the mean, the standard deviation, and the 80% upper confidence limit of the standard deviation for percent passing. Also, using HMA Tools determine the maximum allowable RAP content for this RAP stockpile.

Table 9-3. Results of binder content and gradation tests on RAP stockpile for example 2.

Property	Sieve Size, mm	Sample Number 1	2	3	4	5	6	7	8	9	10
Gradation, % Passing	**19.0**	100	100	100	100	100	100	100	100	100	100
	12.5	98	100	100	99	99	100	100	100	98	98
	9.5	91	98	100	94	97	97	95	93	94	94
	4.75	67	77	75	71	73	78	75	69	70	72
	2.36	53	59	55	54	58	59	57	50	52	53
	1.18	39	44	48	43	49	46	45	41	39	41
	0.600	32	38	37	35	39	36	37	33	32	33
	0.300	22	27	25	23	26	23	25	21	22	22
	0.150	14	17	16	15	16	14	15	14	13	15
	0.075	10.7	12.2	11.9	10.7	12.9	10.3	11.9	10.5	9.8	10.8
Asphalt Content, %		4.0	4.5	4.7	4.4	5.1	4.6	4.6	4.3	4.6	4.8

Solution

The values for percent passing and asphalt binder content given in Table 9-3 are entered in cells B19:K31 and B33:K33 in the worksheet "RAP_Variability." The reliability level in cell B6 should be the default value of 80%. Only one RAP stockpile is being used, so 100 is entered in cell B9, and cells B10:B12 are left blank. After calculation (press F9 to make sure HMA_Tools performs the needed calculations), the mean values are given in cells AH19:AH31 and AH33, the standard deviation values are given in cells AI19:AI31 and AI33, and the values for the upper confidence limit for standard deviation are given in cells AJ19:AJ31 and AJ33. Values for average, standard deviation, and the upper confidence limit for standard deviation are listed in Table 9-4. The maximum allowable RAP content of 42% appears in cell B19.

Table 9-4. Computed averages and standard deviations for the RAP stockpile for example 2.

Property	Sieve Size, mm	Average	Standard Deviation	Upper Confidence Limit for Std. Dev.
Gradation, % Passing:	**19.0**	100.9	0.00	0.00
	12.5	99.2	0.92	1.19
	9.5	95.3	2.67	3.45
	4.75	72.7	3.56	4.61
	2.36	55.0	3.13	4.04
	1.18	43.5	3.54	4.57
	0.600	35.2	2.57	3.33
	0.300	23.6	2.01	2.60
	0.150	14.9	1.20	1.55
	0.075	11.2	0.99	1.28
Asphalt Binder Content, Wt. %	---	4.56	0.295	0.382

appear in cells AH33 and AI33, respectively. The average and standard deviation of the percent passing and binder content for up to three additional RAP stockpiles appear immediately below these cells.

Determining Maximum Allowable RAP Content Based on Variability Using a Graphical Approach

Instead of using HMA Tools, a graphical approach can be used to determine the maximum allowable RAP content in an HMA mix design. Figures 9-3 through 9-6 are design charts for estimating the maximum allowable RAP content for an HMA mix design, based on the variability in gradation and asphalt binder content of the RAP. Figure 9-3 gives the maximum RAP content based on the standard deviation for aggregate percent passing for a single RAP stockpile. Figure 9-4 gives estimated maximum RAP content based on the standard deviation for asphalt binder content for a single RAP stockpile. Figure 9-5 gives estimated maximum RAP content based on the average standard deviation for aggregate percent passing for a blend of RAP stockpiles, while Figure 9-6 gives estimated maximum RAP content based on the average

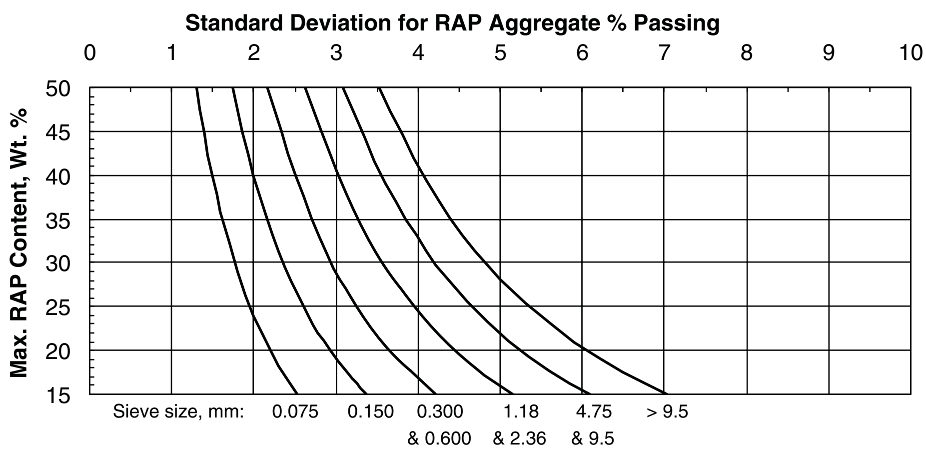

Figure 9-3. Maximum RAP content as a function of standard deviation for aggregate percent passing. *For n = 5 Samples from a single RAP stockpile.*

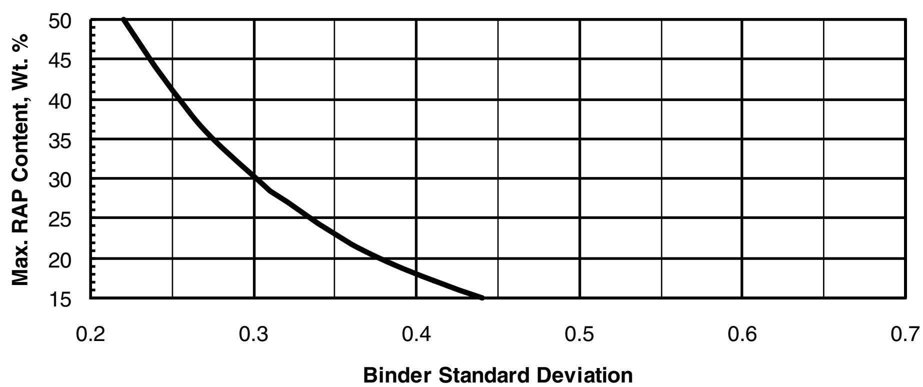

Figure 9-4. Maximum RAP content as a function of standard deviation for asphalt binder content. *For n = 5 Samples from a single RAP stockpile.*

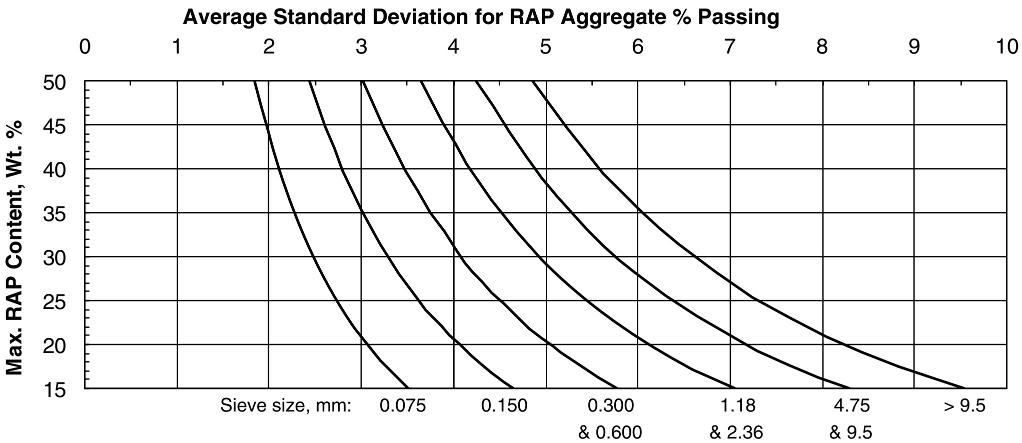

Figure 9-5. Maximum RAP content as a function of average standard deviation for aggregate percent passing. *For n = 5 Samples from a blend of RAP stockpiles, and no stockpile making up more than 70% of the RAP blend.*

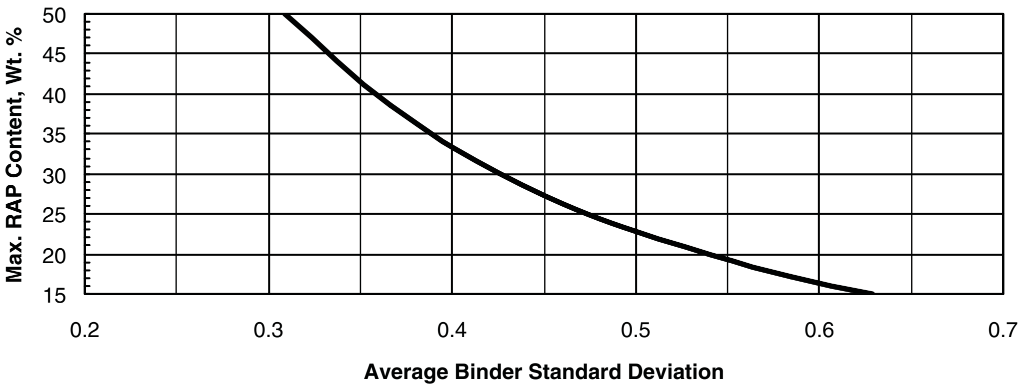

Figure 9-6. Maximum RAP content as a function of average standard deviation for asphalt binder content. *For n = 5 Samples from a blend of RAP stockpiles, and no stockpile making up more than 70% of the RAP blend.*

standard deviation for asphalt binder content for a blend of RAP stockpiles. Figures 9-5 and 9-6 are different from 9-3 and 9-4 because, when several RAP stockpiles are blended, the variability in the resulting blend will tend to be significantly lower than the variability in the individual stockpiles. Figures 9-5 and 9-6 are based on the assumption that no RAP stockpile in the RAP blend will make up more than 70% of the RAP blend; if this assumption is not correct, Figures 9-3 and 9-4 should be used with the standard deviation values for the stockpile making up most of the RAP blend. All four charts are based on statistics calculated from five independent samples; they cannot be used for smaller sample sizes. These charts can be used for statistics calculated using more than five samples, but doing so will tend to underestimate the amount of RAP that can be used in the mix design.

To use these charts, the maximum allowable RAP must be determined for each sieve size for which the percent passing is less than 100%. The maximum allowable RAP must also be determined for the asphalt binder content. The maximum allowable RAP content is then the lowest of all these individual values. The procedure is probably best illustrated with an example problem.

Example Problem 9-3. Determination of Maximum Allowable RAP Content Based on Variability Analysis Using the Graphical Approach

Using the standard deviation values for aggregate percent passing and asphalt binder content calculated for Example 2, estimate the maximum allowable RAP content based on variability using Figures 9-3 and 9-4.

Solution

Table 9-5 lists the average percent passing and standard deviation for percent passing for the RAP stockpile first introduced in Example 2. This also shows the average and standard deviation for asphalt binder content. The last column in Table 9-5 shows the maximum allowable RAP content based on variability for each individual sieve and asphalt binder content, as determined using Figures 9-3 and 9-4. The values range from 30 to 50%; the lowest value is 30% (for the 1.18-mm sieve and the asphalt binder content); therefore, the overall maximum allowable RAP content based on variability is 30%. Note that this percentage is significantly less than the 42% found in Example 2 for the same standard deviation values. The value determined using the graphical approach is lower because it is based on a sample size of n = 5. As mentioned above, using Figures 9-3 through 9-6 for cases where samples sizes larger than 5 are used to calculate standard deviation values will provide lower estimates of maximum allowable RAP content than would be found using more accurate methods such as the HMA Tools spreadsheet.

Table 9-5. Standard deviation values and estimated maximum allowable RAP content for example 3.

Property	Sieve Size, mm	Average	Standard Deviation	Maximum Allowable RAP Content %
Gradation, % Passing:	**19.0**	100.9	0.00	50
	12.5	99.2	0.92	50
	9.5	95.3	2.67	50
	4.75	72.7	3.56	40
	2.36	55.0	3.13	38
	1.18	43.5	3.54	**30**
	0.600	35.2	2.57	38
	0.300	23.6	2.01	50
	0.150	14.9	1.20	50
	0.075	11.2	0.99	50
Asphalt Binder Content, Wt. %	---	4.56	0.295	**30**
Maximum Allowable RAP Content for Stockpile:				30

Maximum RAP Content, Variability and Binder Properties

The methods described above for estimating maximum allowable RAP content are based only on variability analysis—the maximum values for RAP content determined in this way only provide an estimate of how much RAP can be used in a mix design without significantly increasing production variability. These maximum values do not address the equally important issue of how the RAP content will affect the final binder grade in the HMA mix. The asphalt binder contained in RAP is likely much harder than new asphalt binders used in HMA mix designs and, when significant amounts of RAP are added to a mix, the binder from the RAP will blend with the new asphalt binder added to the mix to produce a blended binder that can be substantially harder than the new binder added to the HMA mixture. For this reason, the amount of RAP that can be added to a mixture can be limited not only by variability, but also by the blended binder grade. The issue of determining blended binder grades in HMA mix designs containing RAP is discussed later in this chapter.

RAP Aggregate Properties

In addition to gradation, bulk specific gravity, angularity, and particle shape are properties of the RAP aggregate that are needed to effectively design HMA with RAP. These properties are measured on a representative sample of the RAP stockpile that is formed by combining samples from each of the stockpile locations.

Some tests must be performed on bare aggregate after removing the binder from the RAP. When experience with local aggregates indicates that they are not substantially altered by exposure to the high temperature in the ignition oven, AASHTO T 308 is the preferred method for removing the binder from the RAP aggregate for testing. Otherwise solvent extraction, AASHTO T 164, must be used. If the ignition oven is used, the aggregates must be cooled to room temperature before further handling and testing. If solvent extraction is used, the aggregates must be thoroughly dried in an oven and cooled to room temperature before further handling and testing. When the ignition oven is used, some aggregates may exhibit significant breakdown as a result of the high temperatures used in this procedure; this breakdown can alter the resulting gradation.

The representative sample should be large enough to provide sufficient material for all of the tests being performed; Table 9-6 lists the required sample sizes for laboratory tests normally performed on RAP aggregate. If the nominal maximum aggregate size in the RAP is 12.5 mm or less, a 4 kg sample will be sufficient. If the nominal maximum aggregate size in the RAP is 19.0 mm or more, a 10 kg sample will be required.

RAP Aggregate Bulk Specific Gravity

The bulk specific gravity of each aggregate stockpile used in an HMA mixture is needed for the computation of the voids in the mineral aggregate (VMA). Two methods can be used to determine the bulk specific gravity of the RAP aggregate. The first is to estimate the bulk specific gravity of the RAP aggregate from the RAP binder content, the maximum specific gravity of the RAP, and estimates of the binder absorption in the RAP and the specific gravity of the RAP binder. The second is to measure the bulk specific gravity of the coarse and fine fraction of the

Table 9-6. Sample size for RAP aggregate tests.

Property	Method	Fraction	Sample Size, kg 12.5-mm NMAS	Sample Size, kg 19.0-mm NMAS
Specific Gravity of Coarse Aggregate	AASHTO T 85	+2.36 mm	2	3
Specific Gravity of Fine Aggregate	AASHTO T 84	-2.36 mm	1	1
Coarse Aggregate Fractured Faces	ASTM D 5821	+4.75 mm	0.5	1.5
Fine Aggregate Angularity	AASHTO T 304 Method A	-2.36 mm	0.5	0.5
Flat and Elongated Particles	ASTM D 4791	+4.75 mm	2	5

RAP aggregate after removing the binder with the ignition oven or solvent extraction. Details of these approaches are discussed below.

Estimating RAP Aggregate Bulk Specific Gravity

In this approach, the maximum specific gravity of the RAP is measured in accordance with AASHTO T 209. The maximum specific gravity is measured on a sample split from the representative sample formed for the RAP aggregate and binder analysis. The measured maximum specific gravity, the average RAP binder content from the variability analysis, and an estimate of the RAP binder specific gravity are then used to calculate the effective specific gravity of the RAP aggregate using Equation 9-1.

$$G_{se} = \frac{(100 - P_b)}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)} \tag{9-1}$$

where

G_{se} = effective specific gravity of the RAP aggregate
G_{mm} = maximum specific gravity of the RAP measured by AASHTO T 209
P_b = RAP binder content, wt %
G_b = estimated specific gravity of the RAP binder

The bulk specific gravity of the RAP aggregate can then be estimated from Equation 9-2, which is a rearranged version of the equation used in volumetric analysis to compute asphalt absorption.

$$G_{sb} = \frac{G_{se}}{\left(\frac{P_{ba} G_{se}}{100 \times G_b}\right) + 1} \tag{9-2}$$

where

G_{sb} = estimated bulk specific gravity of the RAP aggregate
G_{se} = effective specific gravity of the RAP aggregate from Equation 9-1
P_{ba} = estimated binder absorption for the RAP, wt % of aggregate
G_b = estimated specific gravity of the RAP binder

The overall error associated with this analysis is difficult to quantify. It depends on (1) the precision of the maximum specific gravity measurement and (2) the accuracy of the RAP binder content measurement and the estimated RAP binder absorption and specific gravity. As shown in the analysis below, the accuracy of the RAP binder content in turn depends on the accuracy of the correction factor used to analyze the ignition oven data.

The single-operator precision of the maximum specific gravity test, AASHTO T 209, is 0.011 when the dry-back procedure is not required and 0.018 when it is. These are somewhat better than the single-operator precision of the aggregate bulk specific gravity tests which are 0.032 for fine aggregate (AASHTO T 84) and 0.025 for coarse aggregate (AASHTO T 85). The potential error associated with estimating the specific gravity of the RAP binder is small. For a typical mixture it is only ± 0.002 for a ±0.010 error in the bulk specific gravity of the binder. Potential errors associated with errors in the RAP binder content or the RAP binder absorption are significantly larger. These errors are shown in Figure 9-7 for RAP having a maximum specific gravity of 2.500, a total binder content of 4%, and binder absorption of 0.5%. In this case, underestimating the absorbed binder by 0.3% results in an overestimation of the bulk specific gravity of the RAP

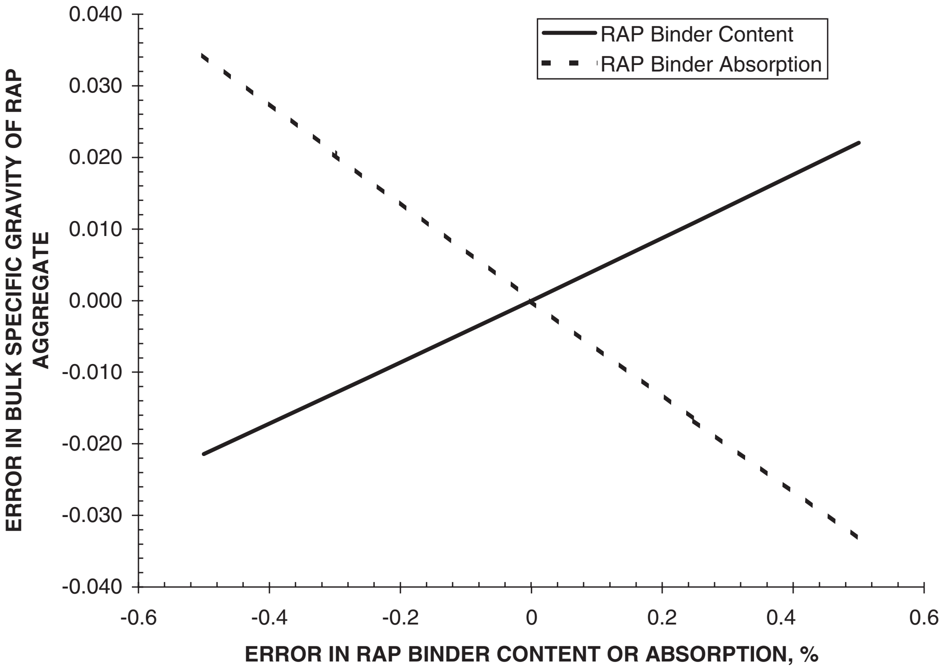

Figure 9-7. Potential errors in bulk specific gravity of the RAP aggregate for errors in RAP binder content and binder absorption.

aggregate of 0.020. Underestimating the total binder content of the RAP by 0.5% results in an underestimation of the bulk specific gravity of the RAP aggregate of 0.021.

Thus the accuracy of estimating the RAP aggregate specific gravity from the maximum specific gravity and binder content of the RAP depends mostly on the accuracy of the estimated correction factor used to determine binder content with the ignition oven and the accuracy of the assumed binder absorption. The correction factor for the ignition oven should not be in error by more than 0.3% and the assumed binder content should not be in error by more than 0.2% to obtain estimated RAP aggregate specific gravity values with an accuracy similar to those measured in AASHTO T 84 and T 85. As discussed earlier, correction factors for the ignition oven can be established by performing both the ignition oven and solvent extraction analyses on split samples from at least three locations in the RAP stockpile.

Measuring RAP Aggregate Specific Gravity

If a reasonable estimate of the binder absorption for the RAP is not available, the specific gravity of the RAP aggregate can be measured after removing the RAP binder using an ignition oven or solvent extraction. The specific gravities of the coarse and fine fractions of the RAP aggregate are measured in accordance with AASHTO T 85 and AASHTO T 84, respectively.

HMA Tools and RAP Aggregate Specific Gravity

HMA Tools is designed so that either approach discussed above can be used to estimate RAP aggregate specific gravity values. If the specific gravity values are to be estimated from maximum theoretical specific gravity, binder content, and related information, the data are entered in cells C6:F9 in the worksheet "RAP_Aggregates." If actual measured values for RAP aggregate specific gravity are used, these are entered in cells C11:F14. The calculated values for bulk and apparent specific gravity for each of up to four RAP stockpiles then appear in cells C16:F17. If data for both methods are entered in the worksheet, HMA Tools will use the measured aggregate specific gravity values in estimating the RAP specific gravity values. The estimated water absorption for each RAP stockpile appears in cells C18:F18.

RAP Aggregate Specification Properties

The specification properties for the RAP aggregates are determined in the same manner as for new aggregates, except that the sand equivalent test, AASHTO T 176, is not performed. The sand equivalent test measures the amount of fine clay particles contained in the aggregate. It is an indicator of how well binder will coat the fine aggregate. This test is not needed for RAP material because the RAP aggregate is already coated with binder. Additionally, some of the fine aggregate is lost when the ignition oven or solvent extraction is used to remove the RAP binder.

The coarse fraction of the RAP aggregate is tested for fractured faces ("crush count") in accordance with ASTM D 5821 and flat and elongated particles in accordance with ASTM D 4791. The fine fraction is tested for angularity in accordance with Method A of AASHTO T 304. Remember that the aggregate consensus properties apply to the blend of all aggregates in the mixture and not the individual stockpiles. Specification properties for the coarse aggregate blend can be computed from stockpile properties based on the proportion of each stockpile used in the HMA. The fine aggregate angularities of the stockpiles can be used to estimate the fine aggregate angularity of the blend. However, because different particle shapes may pack differently when combined, the fine aggregate angularity test, Method A of AASHTO T 304, should be conducted on the final blend of fine aggregates used in the mixture—both from RAP stockpiles and the new fine aggregates used in the mix design.

When using HMA Tools to perform a mix design, coarse aggregate specification properties are input in cells C20:F24 of the worksheet "RAP_Aggregates." The data for fractured faces and flat and elongated particles are entered in this range. Up to two user-defined aggregate specification properties can also be entered at the bottom of this cell range. Fine aggregate angularity values for RAP materials are entered in cells C26:F26 of the worksheet "RAP_Aggregates." As with the coarse aggregate fraction of the RAP, up to two user-defined fine aggregate specification properties can be entered in cells C28:F29. HMA Tools will use values entered in this worksheet to estimate specification properties for the final aggregate blend. These values appear in the worksheet "Trial_Blends" and are also given in the worksheet "Report."

RAP Binder Properties

When the RAP is limited to a low percentage of the new mixture, 15% or less, it is not necessary to determine the properties of the RAP binder. Small amounts of the aged RAP binder have little effect on the properties of the mixture. When higher percentages of RAP are used, the properties of the RAP binder must be considered through the use of blending charts as described in this section. Some agency specifications adjust the new binder grade to account for the presence of RAP. These adjustments are based on analyses similar to those presented in this section using typical properties of local RAP binders and available new binders.

Extraction and Recovery to Determine RAP Binder Properties

To determine the RAP binder properties, the binder from the RAP must be extracted by solvent extraction, then recovered from the solvent. Historically AASHTO T 164 has been used for solvent extraction and the Abson recovery method, AASHTO T 170, has been used to recover the binder for testing. There is general consensus among asphalt technologists that this combination of tests alters the properties of the recovered binder. During the Strategic Highway Research Program (SHRP), a new method for extracting and recovering binders that does not alter the properties of the recovered binder was developed. This method has been standardized as AASHTO T 319 and is the extraction and recovery method recommended for RAP binder analysis.

When using AASHTO T 319 to extract and recover RAP binder for testing, it is extremely important to have the appropriate sample size. At least 50 g of recovered RAP binder are needed to perform the testing required to develop a blending chart. AASHTO T 319 becomes inefficient if the amount of binder recovered exceeds 60 g. Fortunately, the binder content of the RAP was determined from the stockpile variability analysis, and this binder content can be used to determine the size of the sample needed for extraction and recovery. Table 9-7 lists recommended RAP sample sizes based on the binder content of RAP. This table is based on a target of 55 g of recovered binder.

Table 9-7. Recommended sample size for RAP binder extraction and recovery.

RAP Binder Content, %	Recommended Sample Size, g
3.0	1833
3.2	1719
3.4	1618
3.6	1528
3.8	1447
4.0	1375
4.2	1310
4.4	1250
4.6	1196
4.8	1146
5.0	1100
5.2	1058
5.4	1019
5.6	982
5.8	948
6.0	917

Recovered RAP Binder Testing

Figure 9-8 is a flowchart of the RAP binder testing required to prepare a blending chart for a specific RAP binder. These are the same tests used for the performance grading of asphalt binders

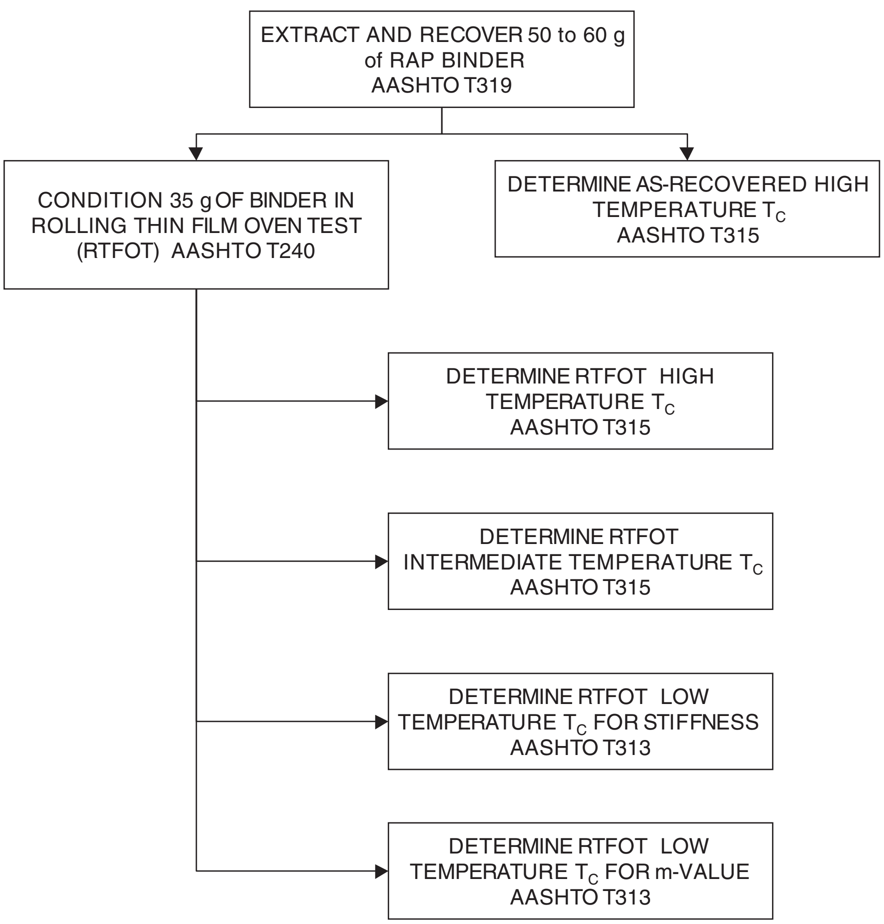

Figure 9-8. Flowchart for RAP binder testing.

Table 9-8. Criteria for determining critical temperatures.

Critical Temperature	Criteria	Sample
High Pavement Temperature	G*/sinδ =1.00 kPa	As Recovered
	G*/sinδ = 2.20 kPa	RTFOT Aged
Intermediate Pavement Temperature	G*sinδ = 5000 kPa	RTFOT Aged
Low Pavement Temperature	S = 300 MPa	RTFOT Aged
	m = 0.300	RTFOT Aged

as discussed in Chapter 3. Laboratories equipped for performance grading of asphalt binders can perform the testing required to characterize RAP binders for blending chart analysis.

The RAP binder testing involves determining the critical high, intermediate, and low pavement temperatures, T_C, for the recovered RAP binder. The critical temperature is the temperature where the properties of the RAP binder meet the specification requirements contained in the performance graded binder specification, AASHTO M 320. These requirements are summarized in Table 9-8. For high and low pavement temperature conditions, two criteria must be considered. The critical temperature for the high pavement temperature condition is the lower of the two, while that for the low pavement temperature condition is the higher of the two.

This testing, also known as determining the "true" or "continuous" grade of the binder, is performed in the same manner as grading a new binder, except the intermediate and low temperature properties are measured on residue from the rolling thin film oven test (RTFOT), AASHTO T 240, instead of residue from the pressure aging vessel (PAV), AASHTO R 28. Research conducted in NCHRP Project 9-12 concluded that the AASHTO M 320 properties of a blend of RAP and new binder could be accurately estimated from the as-recovered and RTFOT aged RAP binder without the need for performing PAV aging of the recovered RAP binder. This finding significantly reduces the testing time and the amount of recovered binder needed for the testing.

Using HMA Tools to Grade Asphalt Binders Recovered From RAP

The calculations involved in grading binders recovered from RAP materials can be tedious to apply and are not included in this manual. The interested reader can find a detailed procedure for performing these and related calculations in Appendix A of AASHTO M 323. HMA Tools can be used not only as an aid in grading RAP binders, but also in performing all binder grade calculations typically needed when designing HMA mixtures containing RAP.

To use HMA Tools to perform grading calculations for binders recovered from RAP, binder test data are entered in the worksheet "RAP_Binders." The final binder grades are given in cells D73, H73, L73, and P73 for each of up to four RAP stockpiles. The continuous grade for high, intermediate, and low temperatures are given immediately below in cells D75:D77, H75:H77, L75:L77 and P75:P77.

Because continuous grading information is needed for new binders used in a RAP mix design in order to determine the grade of the blended binder in the HMA mixture, grading data for new binders must also be entered in HMA Tools when RAP designs are being prepared (unless no more than 15% RAP is being used, in which case binder grading information is not required). Binder grading data for new binders used in a mix design are entered in the worksheet "Binders." As with the worksheet "RAP_Binders," HMA Tools performs all needed grading calculations, and the final grading information appears at the bottom of the worksheet.

Example Problem 9-4. Critical Temperatures for RAP Binder

A sample of RAP binder was extracted, recovered, and tested to provide data for a blending chart analysis. Table 9-9 presents results of tests on the RAP binder as-recovered and after RTFOT conditioning. Determine the critical high, intermediate, and low temperatures for the RAP binder.

Table 9-9. Test results for extracted RAP binder for example 3.

Critical Temperature	Condition	Property	Method	Temperature	Test Result
High	As-Recovered	G*/sinδ	AASHTO T 315	88 °C	1.57 kPa
				94 °C	0.76 kPa
	RTFOT	G*/sinδ	AASHTO T 315	88 °C	4.06 kPa
				94 °C	2.14 kPa
Intermediate	RTFOT	G*/sinδ	AASHTO T 315	25 °C	6,414 kPa
				28 °C	4,701 kPa
Low	RTFOT	S	AASHTO T 313	-18 °C	440 MPa
				-12 °C	229 MPa
				-6 °C	120 MPa
	RTFOT	m	AASHTO T 313	-18 °C	0.220
				-12 °C	0.281
				-6 °C	0.336

Solution

The binder test data from Table 9-9 are entered in the worksheet "RAP_Binders" in the appropriate cells in columns D and E. The continuous critical high temperature is given as 91.7°C in cell E75. The continuous critical intermediate temperature is 27.4°C and is given in cell E76. The continuous critical low temperature is –9.8°C, and is given in cell E77. The final binder grade for the recovered binder is PG 88-16, given in cell E73. Note that HMA Tools reports the binder grade by including the intermediate temperature grade in parentheses: PG 88-(28)-16. This is done because some agencies have requirements for intermediate temperature grade, and reporting the grade in this way makes it easy to determine if such requirements are met.

Blending Charts For RAP Binders

Linear blending charts are used to estimate the properties of blends of new binder and RAP binder. Figure 9-9 is a schematic of a linear blending chart. Separate blending charts for high, intermediate, and low temperature properties are used. Equation 9-3 describes the line in Figure 9-9.

$$T_c(Blend) = T_c(New) + \frac{\%RAPB}{100}\left(T_c(RAP) - T_C(New)\right) \tag{9-3}$$

where

$T_c(Blend)$ = critical temperature for the blend of RAP and new binder
$T_c(New)$ = critical temperature for the new binder
$T_c(RAP)$ = critical temperature for the RAP binder
$\%RAPB$ = percentage of total binder content obtained from the RAP, wt %

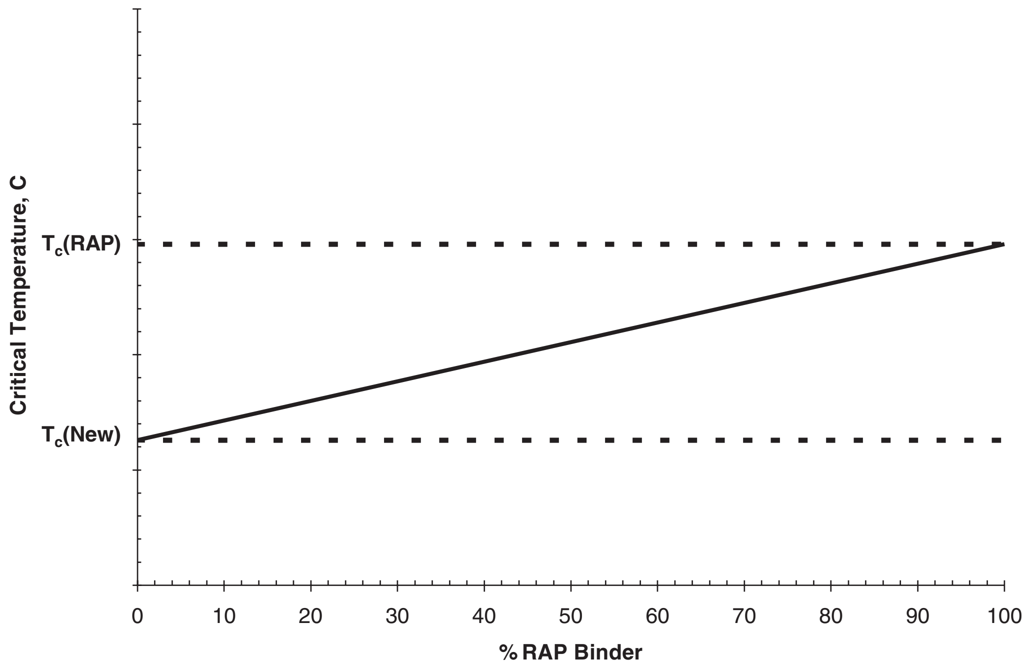

Figure 9-9. Linear blending chart.

The blending charts are based on the percentage of the total binder in the mixture that is from the RAP. As discussed earlier, the RAP added to a mixture includes both RAP aggregate and RAP binder. Because computer spreadsheets are now almost universally used to perform the calculations needed to perform HMA mix designs, including the potentially tedious calculations required when the design includes more than 15% RAP, it is not necessary to construct blending charts as part of the mix design process. If a blending chart is needed for an HMA mix design containing RAP, a detailed procedure can be found in AASHTO M 323.

There are two different approaches to designing HMA mixtures with RAP so that the resulting blended binder meets the required grade. The first is to establish the RAP content, and then determine the performance grade of the new binder needed to make sure the blended binder in the final mix is the proper grade. The second approach is to select the new binder grade, and then determine the required RAP content to ensure that the blended binder is the proper grade. A surprising twist on this second approach is that the required RAP content can be a maximum or a minimum. A maximum RAP content can occur when a relatively stiff new binder is used, so that addition of too much RAP will make the blended binder too stiff to meet the performance grade requirements at intermediate and/or low temperatures. A minimum RAP content can occur when a relatively soft new binder is used, in which case enough RAP must be added to provide sufficient stiffening to the blended binder so that high-temperature grade requirements are met. In some cases, there may be both minimum and maximum RAP content requirements.

HMA Tools provides solutions for both types of RAP content calculation. Once appropriate binder grade information is entered along with the mix composition (aggregate blend data and volumetric data), HMA Tools will calculate the resulting blended binder grade and also estimate the minimum and maximum RAP contents based on binder grade requirements. If RAP from more than one stockpile is used in the mix design, HMA Tools assumes that the relative proportions of material in the RAP blend will remain constant as the overall RAP content varies. Two example problems below show how HMA Tools can be used in performing both types of binder grade calculations when designing HMA mixtures containing RAP.

Example Problem 9-5. Selection of a New Binder Grade for an HMA Mix Design Containing RAP

The RAP binder from Example 4 will be used in an HMA mixture. The RAP will be added to the mixture at 30% of the aggregate blend by weight. The RAP has a binder content of 4.4%, and the HMA mixture has a binder content of 5.2%. Determine the new binder grade that should be used so that the blended binder meets the specification requirements for PG 64-22.

Solution

Information on the required binder grade is entered in the worksheet "Blended_Binders" in cells C5:C8—performance grade 64-22, high temperature grade 64°C, intermediate temperature grade 25°C and low temperature grade –22°C. The binder content of 5.20% is entered in cell C11, and an absorption value of 0.0 is entered in cell C12. Since only one RAP is being used, 100 is entered in cell C16, and cells C17:C19 are left blank. In cell C22 the RAP binder content of 4.40% is entered, with an absorption value of 0.0 entered in cell D22. Since only one RAP stockpile is being used, cells C23:D25 are left blank.

The RAP binder being used is the same as already entered in the worksheet "RAP_Binders," so the grading information (in the form of critical temperatures) appears in cells H30:J30. These values are then copied into cells C30:E30. Again, if more RAP stockpiles were used, additional grading information could be entered in cells C31:E33, but only one RAP is being used in this example so these cells are left blank. The RAP content of 30% by weight of total RAP plus aggregate is entered in cell C70.

The required new binder grade of PG 58-28 is reported in the worksheet "Blended_Binders" in cell C80. Note that the required critical temperatures for the new binder are given in cells C76:C78 as follows: high critical temperature—57.7°C, intermediate critical temperature—22.9°C, and low critical temperature—15.3°C. As in many other locations, HMA Tools reports the required grade by including the intermediate temperature grading of 19°C in parentheses, so cell C80 actually reads "58-(19)-28." The worksheet "Blended_Binders" is designed to only provide one binder in this cell, although several others would typically meet the requirements. The grade reported in cell C80 represents that with the lowest high temperature grade and the highest low temperature grade. For instance, in this example a PG 64-28 binder would also meet the grade requirements. When determining whether or not performance grades other than that listed in cell C80 would meet the requirements for an HMA design containing RAP, the engineer or technician must make certain that not only the high and low critical temperature requirements are met, but also that the intermediate critical temperature requirement (cell C77) is met. In this example, a PG 64-28 binder would have a maximum intermediate critical temperature of 22°C and would therefore meet the specified grading requirements. However, a PG 70-28 binder has a maximum intermediate critical temperature of 25°C, which is greater than the calculated maximum intermediate critical temperature of 22.9°C and therefore should not be used in this mix design.

Example Problem 9-6. Calculating the Maximum Allowable RAP Content Given an HMA Design with a Specific New Asphalt Binder

The RAP binder from Example 4 will be used in an HMA mixture. A PG 64-22 binder with excellent low temperature characteristics is available, and the producer would like to use this binder in mixtures with RAP because of limited binder storage at the plant. The RAP has a binder content of 4.4%, and the HMA mixture has a binder content of 5.2%. Determine the amount of RAP that can be used with this binder to meet the specification requirements for PG 70-22. The critical temperatures for the new binder, based on historical data provided by the supplier, are T_c(High) = 66.0 °C, T_c(Int) = 19.3 °C, and T_c(Low) = –17.3 °C.

Solution

Much of the required information for this problem is the same as Example 5 and is entered in HMA Tools in the same way. The required continuous grade temperatures for a PG 70-22 are T_c(High) = 70.0 °C, T_c(Int) = 28.0 °C, and T_c(Low) = –22.0 °C; these values are different from those for the PG 64-22 required in the previous example and should be entered in cells C6:C8. The critical temperatures for the new binder (PG 64-22) are entered in cells C44:C45 and C47.

The minimum and maximum RAP contents, as a weight percentage of the total RAP plus aggregate weight, are given in cells C64 and C65, respectively: 33.5% minimum and 38.1% maximum. For the actual mix design, a value of 35% RAP would be a good choice. Note that for RAP contents of 15% and lower, the effects of RAP on binder grading need not be considered. Therefore, if HMA Tools calculates that less than 15% RAP should be used in a mix because of binder grading considerations, a value of 15% is reported in cell C64. In a similar way, RAP contents above 50% are discouraged because of practical difficulties in handling such a quantity of RAP in many plants, so that the highest value HMA Tools will report in cell C65 is 50%, regardless of the value calculated on the basis of binder grading.

Handling RAP Materials in the Laboratory

RAP materials must be handled differently than new aggregates when preparing specimens for volumetric design and performance analysis. First, the weight of the binder contained in the RAP must be accounted for during laboratory batching. Second, the RAP must be heated gently during the preparation of laboratory specimens to avoid changing the properties of the RAP binder. Recommended methods for handing RAP are discussed below.

Laboratory Batching

A supply of RAP for the preparation of laboratory specimens is obtained by combining the main portion of the individual samples for each stockpile and thoroughly mixing the combined material. Material for each RAP stockpile should be kept separate. Prior to batching, the RAP should be dried in an oven at 60 °C (140 °F) to remove field moisture. Laboratory batching is based on the weight of the aggregate, while the stockpile percentages used to produce the HMA at the hot-mix plant are based on total weight. When the HMA is composed of all new aggregates, the stockpile percentages can be used directly. However, when RAP stockpiles are used, the weight of the binder contained in the RAP must be accounted for and stockpile percentages based on the total weight of aggregate must be computed and used for laboratory batching. The required analysis was discussed earlier in the Blending and Variability section of this chapter. Example 1 illustrated the required computations.

When preparing laboratory specimens, new aggregate stockpiles should be separated into the following size fractions:

- Passing 37.5 mm—Retained 25.0 mm
- Passing 25.0 mm—Retained 19.0 mm
- Passing 19.0 mm—Retained 12.5 mm
- Passing 12.5 mm—Retained 9.5 mm
- Passing 9.5 mm—Retained 4.75 mm
- Passing 4.75 mm—Retained 2.36 mm
- Passing 2.36 mm

Because of the relatively large amount of fine material in most RAP, it is not necessary to separate the RAP into size fractions for laboratory batching. The amount of each size fraction of new aggregate needed for a laboratory batch is computed directly from the proportions based on the total weight of aggregate. The amount of each RAP stockpile needed for a laboratory batch must be increased to account for the weight of binder in the RAP using Equation 9-4:

$$M_{RAP} = \frac{M_{RAPAGG}}{\left(100 - Pb_{RAP}\right)} \times 100 \tag{9-4}$$

where

M_{RAP} = mass of RAP required for the laboratory batch, g
M_{RAPAGG} = mass of RAP aggregate required for the laboratory batch, g
Pb_{RAP} = RAP binder content, wt %

The mass of binder provided by the RAP is then

$$M_{RAPBINDER} = \left(\frac{Pb_{RAP}}{100}\right) \times M_{RAP} \tag{9-5}$$

where

$M_{RAPBINDER}$ = mass of RAP binder in the laboratory batch, g
M_{RAP} = mass of RAP in the laboratory batch, g
Pb_{RAP} = RAP binder content, wt %

HMA Tools can be used to perform these and all other required calculations for determining batch weights of mix designs containing RAP. Before determining batch weights for a mix design, all of the pertinent mix design data should be entered on various worksheets, including "General," "Binders," "RAP_Binders," "Aggregates," "RAP_Aggregates," and "Trial_Batches." Other data should be entered in the worksheet "Batch." For cylindrical specimens, the diameter and height of up to three different-sized specimens are entered in cells F3:F5 and G3:G5, respectively. The numbers of specimens needed for each of these three different cylinder sizes are entered in cells I3:I5. The dimensions and number of specimens for up to two differently shaped beam-shaped specimens are entered in a similar way in cells F8:I9. If desired, an extra amount of loose mix can be entered in cell K3 (in grams)—this might be material needed for theoretical maximum specific gravity determinations. In addition to this loose mix, an extra percentage of material can be specified in cell O7. This represents a contingency, to ensure that enough material is available for the required specimens; 5 or 10% should normally be entered here. The user must also make sure to specify the trial batch number for the batch calculations in cell O1; this would be a number from 1 to 7, corresponding to the trial batch numbers in the worksheet "Trial_Batch."

HMA Tools will calculate batch weights for each aggregate and RAP material, providing a complete breakdown for coarse material (retained on the 2.36-mm sieve) and three different breakdowns for fine aggregate: complete, partial (in two-sieve fractions), and with no breakdown. HMA Tools also calculates the amount of new binder required—the amount of binder contributed by the RAP is automatically accounted for. The worksheet "Batch" is conveniently designed to print out a one page report that can be used to weigh out materials in the laboratory. The last example in this chapter shows how HMA Tools is used to calculate batch weights for mix designs containing RAP.

Example Problem 9-7. Laboratory Batching for an HMA Mix Design Containing RAP

Laboratory specimens of the mixture from Example 1 will be prepared. The aggregate gradations were given previously in Table 9-1. The mixture will be produced using three new aggregate stockpiles, hydrated lime, and two RAP stockpiles. The stockpiles will be combined in the following proportions: 29% of Aggregate 1, 40% of Aggregate 2, 10% of Aggregate 3, 1% lime, 10% Coarse RAP, and 10% Fine RAP. The target VMA for the mix is 16.0% and the target air void content is 4.0%. Determine batch weights for making two 150-mm-diameter by 100-mm-tall cylindrical specimens with 2,000 grams of extra material for performing a theoretical maximum specific gravity test. Allow 5% extra material for contingency. The fine aggregate fraction (passing the 2.36-mm sieve) will not be further broken down, and the RAP will not be broken down at all. Calculate weights for this trial batch.

Solution

The aggregate and RAP data are entered into the worksheets "Aggregates" and "RAP_Aggregates" as for Example Problem 1. In the worksheet "Batch," 1 should be entered for batch number in cell O1. Dimensions for the cylindrical specimen are entered in cells F3:G3, and 2 is entered for the number of specimens in cell I3. The amount of loose mix (2,000 g) is entered in cell K3, and 5% is entered in cell O7 for the desired amount of extra mix. After pressing shift+F9 to calculate, the batch weights appear on the lower portion of the spreadsheet. Table 9-10 summarizes the batch weights calculated by ***HMA Tools*** for this example.

Table 9-10. Summary of batch weights for example 7.

Material	Total wt., g	Fraction Mm	Batch wt, g
Aggregate 1		12.5 – 9.5	267
		9.5 – 4.75	2,136
		4.75 – 2.36	386
		- 2.36	179
Aggregate 2	4,091	9.5 – 4.75	82
		4.75 – 2.36	1,514
		-2.36	2,495
Aggregate 3	1,023	9.5 – 4.75	102
		4.75 – 2.36	389
		-2.36	532
Lime	102	All	102
CRAP	1,022	All	1,022
FRAP	1,023	All	1,023
New Binder	506	All	506
Total	10,735		

Heating RAP

Exercise care when preparing laboratory specimens with RAP to avoid changing the properties of the RAP binder. The RAP should be heated for the shortest time possible to reach the mixing temperature. Heat the RAP for no more than 2 hours in a separate oven set to 110°C (230°F). Higher temperatures and longer heating times have been shown to change the properties of some RAP. The new aggregates should be heated from 10 to 20 °C above the mixing temperature before combining with the RAP and new binder. The mixture should be mixed, short-term aged, and compacted in the usual manner.

Bibliography

AASHTO Standards

M 320, Standard Specification for Performance-Graded Asphalt Binder
R 29, Grading or Verifying the Performance Grade of an Asphalt Binder
T 2, Standard Method of Test for Sampling Aggregates
T 248, Standard Method of Test for Reducing Samples of Aggregate to Testing Size
T 308, Standard Method of Test for Determining the Asphalt Binder Content of Hot-Mix Asphalt (HMA) by the Ignition Method
T 30, Standard Method of Test for Mechanical Analysis of Extracted Aggregates
T 164, Standard Method of Test for Quantitative Extraction of Bitumen for Bituminous Paving Mixtures
T 209, Standard Method of Test for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
T 84, Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate
T 85, Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate
T 176, Standard Method of Test for Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test
T 319, Standard Method of Test for Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures
T 240, Standard Method of Test for Effect of Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)
T 315, Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
T 313, Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)

Other Standards

ASTM D 5821, Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate
ASTM D 4791, Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate

Other Publications

McDaniel, R., and R. M. Anderson (2001) *NCHRP Report 452: Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual,* TRB, National Research Council, Washington, DC, 58 pp.
McDaniel, R. S., et al. (2000) *NCHRP Web-Only Document 30: Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Contractor's Final Report,* TRB, National Research Council, Washington, D.C., October, 461 pp.
NAPA (2007) *NAPA Quality Improvement Series 124: Designing HMA Mixtures with High RAP Content,* NAPA, Lanham, MD
NAPA (1996) *Recycling Hot Mix Asphalt Pavement,* Information Series 123, NAPA, Lanham, MD
NAPA (2000) Recycling Practices for HMA, *Special Report 187,* NAPA, Lanham, MD, 2000.
FHWA (1998) "User Guidelines for Waste and Byproduct Materials in Pavement Construction" *FHWA-RD-97-148,* FHWA, Washington, DC.

CHAPTER 10

Design of Gap-Graded HMA Mixtures

Gap-graded HMA (GGHMA) consists of two parts: a coarse aggregate skeleton and a mortar. The coarse aggregate skeleton consists of crushed coarse aggregate particles; these make up about 70 to 80% of the total aggregate blend. The mortar consists of asphalt binder, fine aggregate, and mineral filler and fills the voids in the coarse aggregate skeleton.

Stone matrix asphalt (SMA), one particular type of GGHMA, is widely known for its durability and rut resistance. SMA has been used in Europe for over 30 years. The first U.S. project that used this high-performance HMA was constructed in 1991. Since then, the use of SMA has steadily increased within the United States. The GGHMA discussed in this chapter is similar to SMA in many ways, but there are some differences, so to avoid confusion or arguments over whether or not the mix design presented in this chapter is truly an "SMA," the term GGHMA is used instead.

In Europe, SMA mixtures have primarily been designed by recipes. It was not until 1994 that a formalized mix design procedure was available in the United States This mix design procedure was developed by a Technical Working Group established by the FHWA. This procedure was based on the Marshall mix design method, since this was the method used to design SMA in Europe. In 1994, the National Center for Asphalt Technology (NCAT) began a 4-year study to develop a mix design system to design SMA using the concepts and methods of the Superpave mix design system. Results from this research project were published in 1998, and, along with more recent experience and research, they are the basis for the GGHMA mix design system described in this chapter.

The philosophy of GGHMA mix design is not complicated. The first principle is that a gap-graded blend of aggregate is needed so that the coarse particles will have stone-on-stone contact. The second principle is that the voids within the coarse aggregate skeleton must be filled with fine aggregate, mineral filler, and asphalt binder. In order to provide increased durability, GGHMA has a relatively high asphalt binder content. This leads to the third principle of GGHMA mix design, which is that the aggregate must have a high VMA value—typically 18 to 20%. This relatively high asphalt binder content can result in an increased potential for draindown if not properly taken into account. Draindown can be a common problem in GGHMA; it occurs when the asphalt binder and fine aggregate separate from the coarse aggregate during storage, transport, or placement. The fourth and final principle of GGHMA mix design is that small amounts of stabilizing additives, such as mineral fibers or cellulose fibers, are usually needed to prevent draindown. The sections below describe in detail how to design GGHMA to achieve the unique properties and excellent performance for which this mix type is known.

Overview of GGHMA Mix Design Procedure

The mix design procedure for GGHMA contains five primary steps (Figure 10-1). First, suitable materials must be selected to compose the GGHMA. Materials needed include coarse aggregates, fine aggregates, mineral fillers, asphalt binder and stabilizing additives. The second step is to blend

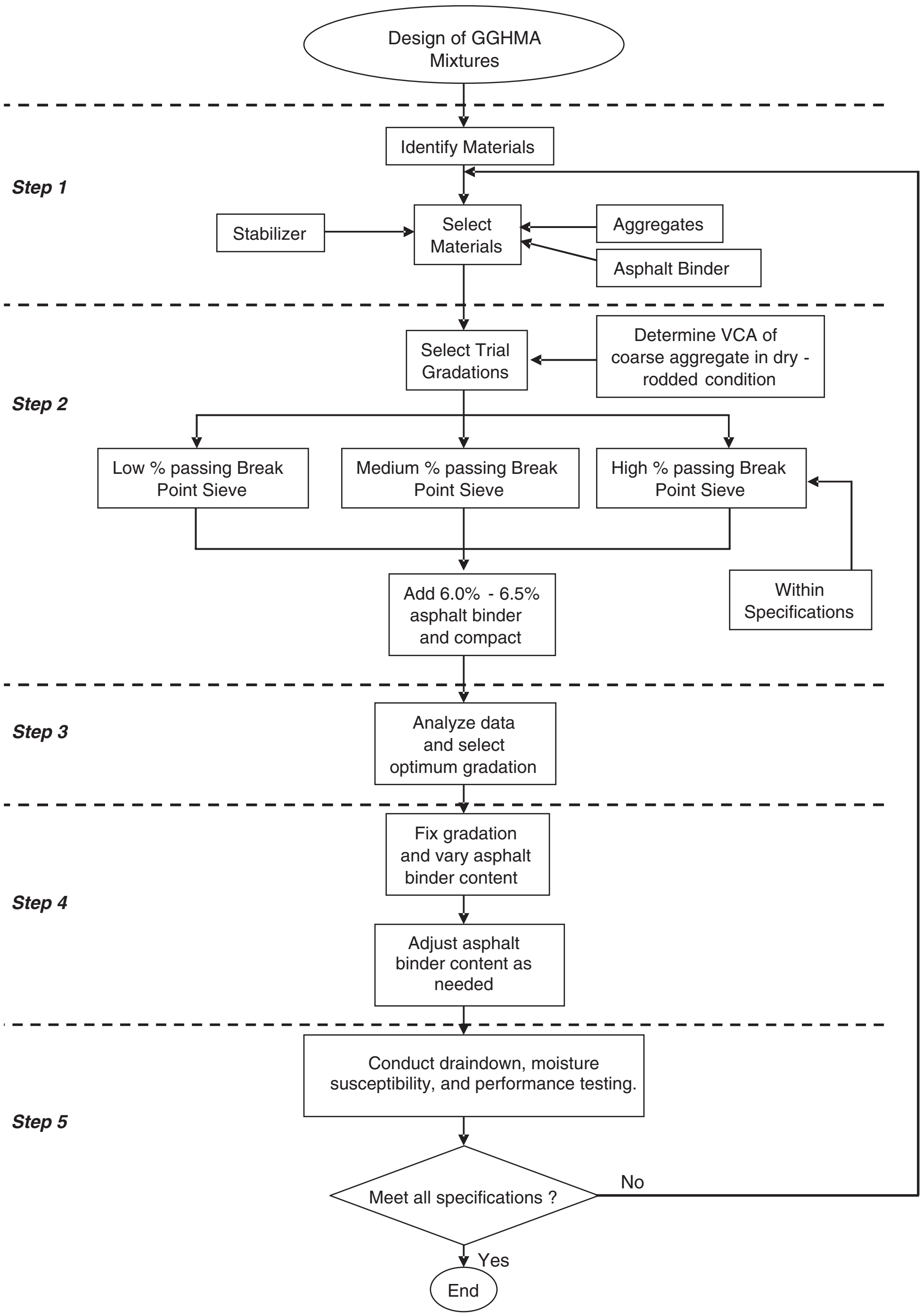

Figure 10-1. Flow diagram illustrating GGHMA mix design methodology.

three trial gradations. For each trial gradation, asphalt binder is added and the mixture compacted. After each trial gradation has been compacted, mixture volumetric data for each trial mixture is evaluated as the third step in order to select the best gradation. The fourth step is to fix the selected gradation and compact mixtures with varying asphalt binder contents. The asphalt binder content that produces 4% air voids is selected as optimum asphalt binder content. The final step in the mix design procedure is to evaluate the moisture susceptibility, draindown sensitivity, and rut resistance of the designed mixture.

The standards and overall procedure given in this chapter closely follow those given in two AASHTO standards: M 325 and R 46. These in turn are based on research on SMA mix designs performed by Brown and Cooley as described in *NCHRP Report 425.* Although developed for SMA mixtures, these guidelines can be effectively applied to GGHMA. The following sections describe each step in the design of a GGHMA mixture in detail.

Step 1—Materials Selection

As with most HMA mixes, suitable coarse aggregate, fine aggregate, and asphalt binder must be selected for GGHMA. However, two additional materials are also typically needed for GGHMA designs: commercial mineral filler and stabilizing additives. Aggregates used for GGHMA should be angular, cubical, and roughly textured. These properties help ensure that the aggregate particles composing the stone skeleton cannot slide past one another. Angular, cubical, and textured aggregate particles will lock together, providing a rut-resistant pavement layer. Figure 10-2 illustrates desirable aggregates for GGHMA.

Asphalt binders used in GGHMA mixtures should perform at high, intermediate, and low temperatures. Chapter 3 provided a detailed discussion on asphalt binders. It should be stated, however, that most GGHMA mixes have been designed with polymer-modified binders. Polymer-modified binders are not required, but are generally used to (1) reduce the potential for draindown and (2) improve durability.

Mineral fillers help fill the voids within the coarse aggregate skeleton. Many types of materials have been used as mineral filler, including marble dust, limestone dust, agricultural lime, and fly ash. For agencies that require the use of hydrated lime to reduce the potential of moisture damage, hydrated lime can be considered a portion of the mineral filler. Because of the large percentage of coarse aggregate in GGHMA blends, natural crushed aggregate stockpiles do not generally have sufficient materials passing the 0.075-mm sieve to help fill the voids of the stone skeleton, hence the need for mineral fillers. Without the use of mineral fillers to fill the voids, GGHMA mixes would be very permeable.

Figure 10-2. Illustration of desirable aggregate shape and angularity.

The primary purpose for stabilizing additives is to reduce the potential for draindown. When added to stiffen an asphalt binder, polymer modifiers can be considered a stabilizing additive. Likewise, mineral fillers can also be considered a stabilizing additive, since these small particles help "soak up" the asphalt binder. However, the most effective stabilizing additive is a fiber. Several types of fiber have been used in GGHMA with cellulose and mineral fiber being the most common. Generally, cellulose fibers are added at 0.3% of the total mix mass and mineral fibers are added at 0.4%.

The following sections provide requirements for the various materials used to fabricate GGHMA. These requirements are provided for guidance to agencies not having experience with these types of mixtures. Some agencies have used other test methods and criteria with success.

Coarse Aggregates

As described previously, the success of a GGHMA pavement depends heavily on the existence of stone-on-stone contact. Therefore, in addition to angularity, shape, and texture, the toughness and durability of the coarse aggregates must be such that they will not degrade during production, construction, and service. Table 10-1 presents coarse aggregate requirements for GGHMA mixtures. The Los Angeles Abrasion and Soundness tests should be required for individual stockpiles while the Flat or Elongated and Uncompacted Voids tests should be required for the total aggregate blend.

Fine Aggregates

The role of fine aggregates in GGHMA is to help fill the voids between coarse aggregate particles. Therefore, the primary requirements for fine aggregates in GGHMA are to ensure a durable and angular material. Requirements for fine aggregates in GGHMA are provided in Table 10-2. The Uncompacted Voids and Sand Equivalency tests should be required for the total aggregate blend, while the soundness test should be applied to individual fine aggregate stockpiles.

Asphalt Binder

Asphalt binders should be performance graded, in accordance with the requirements of AASHTO M320-04, to satisfy the climate and traffic loading conditions at the site of the GGHMA project. Guidelines described in Chapter 8 for selecting binder performance grades for dense-graded HMA also apply to GGHMA, with the exception that the high-temperature performance grade

Table 10-1. Coarse aggregate quality requirements.

Test	Method	Minimum	Maximum
Los Angeles Abrasion, % Loss	AASHTO T96	-	30[a]
Flat and Elongated, 5 to 1 Ratio, %	ASTM D 4791	-	10[b]
Soundness (5 Cycles), %	AASHTO T104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Fractured Faces, %	AASHTO D 5821		
One face		98[c]	-
Two faces		98[c]	-

[a]Aggregates with L.A. Abrasion loss values up to 50 have been successfully used to produce GGHMA mixtures. However, when the L.A. Abrasion exceeds approximately 30, excessive breakdown may occur in the laboratory compaction process or during in-place compaction.

[b]Flat and elongated criteria apply to the design aggregate blend.

[c]The CAFF requirement for design traffic levels of 30 million ESALs or more may be reduced to 95/95 if experience with local conditions and materials indicate that this would provide HMA mixtures with adequate rut resistance under very heavy traffic.

Table 10-2. Fine aggregate quality requirements.

Test	Method	Minimum	Maximum
Soundness (5 Cycles), %	AASHTO T 104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Uncompacted Voids	AASHTO T 304, Method A	45[a]	-
Sand Equivalency	AASHTO T176	50	-

[a]The FAA requirement of 45 may be reduced to 43 if experience with local conditions and materials indicate that this would produce HMA mixtures with adequate rut resistance under the given design traffic level.

should be no less than 76 and the binder should be polymer modified. This ensures that GGHMA mixes will exhibit the exceptional rut resistance that pavement engineers expect from this mix type. Note that, for some applications, GGHMA mixes might require binders with high-temperature performance grades exceeding PG 76. The required binder grade for a GGHMA mix should be determined following the procedure given in Chapter 8; if the high-temperature performance grade is a PG 76 or less, then a PG 76 binder should be used. If the resulting high-temperature PG grade is above a PG 76, then the higher PG grade should be used in the GGHMA mix design.

Mineral Fillers

Mineral fillers used for GGHMA should be finely divided mineral matter such as crushed fines, agricultural limes, or fly ash. Figure 10-3 illustrates some typically used mineral fillers. The mineral filler should be free from organic impurities. It is recommended that mineral fillers with modified Rigden voids (sometimes called Dry Compaction Test) higher than 50% not be used in GGHMA. Rigden voids is in some ways a similar test to that used to evaluate fine aggregate

Figure 10-3. Typical mineral fillers used in GGHMA.

angularity by measuring uncompacted voids. However, the Rigden voids test is smaller in scale, and the mineral filler is compacted with a small drop hammer prior to determining the void content. Mineral fillers with very high Rigden voids can sometimes cause excessive stiffening in SMA mixtures. The equipment and test method for conducting the Dry Compaction Test can be found in the National Asphalt Pavement Association's Information Series 127, "Evaluation of Baghouse Fines for Hot Mix Asphalt."

Other requirements for mineral fillers can be found in AASHTO M-17, "Mineral Fillers for Bituminous Paving Mixtures." However, the gradation requirements stated in AASHTO M-17 should only be used for guidance. The important gradation is that of the designed GGHMA and not the mineral filler.

Stabilizing Additives

Stabilizing additives are needed in GGHMA to prevent the draining of mortar from the coarse aggregate skeleton during storage, transportation, and placement. Stabilizing additives such as cellulose fiber, mineral fiber, and polymers have been used with success to minimize draindown potential. Other types of fibers have been used with success; however, the most common types are cellulose and mineral fibers. When using a polymer as a stabilizer, the amount of polymer added should be that amount necessary to meet the performance grade of the asphalt binder.

Step 2—Trial Gradations

As with any HMA, specified aggregate gradations should be based on aggregate volume and not aggregate mass. However, for most conventional HMA mixtures (dense-graded), the specific gravities of the different aggregate stockpiles are close enough to make a gradation based on mass percentages similar to that based on volumetric percentages. With GGHMA, the specific gravities of the different aggregate components are not always similar. This is especially true when commercial fillers are used in GGHMA. Therefore, the gradation bands presented in Table 10-3 are based on % passing by volume. Similar to those for dense-graded mixes, GGHMA gradation bands are described by the nominal maximum aggregate size (NMAS) of the gradation. The following section provides guidance in the form of an example problem on how to blend aggregate components based on volumes to meet the gradation bands in Table 10-3. However, if the bulk specific gravities of the different stockpiles (including mineral filler) used to compose the aggregate blend vary by 0.02 or less, gradations based on mass percentages can be used.

Table 10-3. Stone matrix asphalt gradation specification bands (Percent Passing by Volume).

Sieve Size, mm	19-mm NMAS		12.5-mm NMAS		9.5-mm NMAS	
	Min.	Max.	Min.	Max.	Min.	Max.
25.0	100	100				
19.0	90	100	100	100		
12.5	50	88	90	100	100	100
9.5	25	60	26	88	70	95
4.75	20	28	20	35	30	50
2.36	16	24	16	24	20	30
1.18	--	--	--	--	--	21
0.6	--	--	--	--	--	18
0.3	--	--	--	--	--	15
0.075	8.0	11.0	8.0	11.0	8.0	12.0

Note: NMAS – Nominal Maximum Aggregate Size – one sieve size larger than the first sieve that retains more than 10%.

Example Problem 10-1. Blending Aggregates to Meet GGHMA Gradation Requirements

Washed Sieve Analyses

As with any blending problem, the first step is to perform washed sieve analyses based on mass for the various stockpiles to be used in the GGHMA mixture, following procedures described in AASHTO T 27. For this example, a 19.0-mm GGHMA mixture is to be blended. Table 10-4 provides the results of washed gradation tests performed on four stockpiles, which are to be blended for this example problem. Also needed to determine aggregate gradations based on volume are the bulk specific gravities (G_{sb}) of the different stockpiles. Table 10-4 also provides the G_{sb} values for each stockpile. Notice that the G_{sb} values differ by more than 0.02 in Table 10-4.

Determine Percent Mass Retained

After performing the washed sieve analyses, the percent mass retained on each sieve for the different stockpiles is determined. For a given sieve, this is done by subtracting the percent passing the given sieve from the percent passing the next larger sieve. For example, using Aggregate C in Table 10-4, the % mass retained on the 4.75-mm sieve would be calculated as

% Retained on 4.75-mm Sieve = 84.6 – 48.9 = 35.7%

Table 10-4. Results of gradation and specific gravity tests for stockpiles to be used in example GGHMA mix design.

Sieve, mm	Stockpile and Percent Passing Based on Mass, %			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	100.0	100.0	100.0	100.0
19.0	95.0	100.0	100.0	100.0
12.5	66.0	71.0	97.4	100.0
9.5	43.0	46.0	84.6	100.0
4.75	9.0	6.0	48.9	100.0
2.36	5.0	4.0	27.8	100.0
1.18	2.0	4.0	16.6	100.0
0.60	2.0	3.0	10.7	100.0
0.30	2.0	3.0	7.6	100.0
0.075	1.0	1.5	4.6	72.5
G_{sb}	2.616	2.734	2.736	2.401

Table 10-5. Percent by mass retained on each sieve.

Sieve, mm	Percent Mass Retained per Sieve			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	0.0	0.0	0.0	0.0
19.0	5.0	0.0	0.0	0.0
12.5	29.0	30.0	2.6	0.0
9.5	23.0	24.0	12.8	0.0
4.75	34.0	40.0	35.7	0.0
2.36	4.0	2.0	21.1	0.0
1.18	3.0	0.0	11.2	0.0
0.60	0.0	1.0	5.9	0.0
0.30	0.0	0.0	3.1	0.0
0.075	1.0	1.5	3.0	27.5
-0.075	1.0	1.5	4.6	27.5
Total, Σ	100	100	100	100

Example Problem 10-1. (Continued)

where

84.6 = percent mass passing 9.5-mm sieve (Table 10-4)
48.9 = percent mass passing 4.75-mm sieve (Table 10-4)
35.7 = percent mass retained on 4.75-mm sieve

Table 10-5 presents the values of percent mass retained for all sieves and stockpiles. Note that a row has been added to reflect that material finer than the 0.075-mm (–0.075) sieve is included.

Calculate Percent Mass Retained

In this calculation a simple assumption is made, "Assume the Mass of Each Aggregate Stockpile is 100 grams." Using this assumption allows for the mass that would be retained of each size fraction for each stockpile to be determined and can be shown to be equal to the numbers shown in Table 10-5.

Convert Percent Mass Retained to Volume per Sieve

In this step of developing an SMA gradation, the values for percent mass retained determined previously are converted to volumes per sieve. To make this conversion, the bulk specific gravity of the individual stockpiles is needed. The volume of aggregate retained on each sieve for each stockpile can be determined from the following equation:

$$V_{agg} = \frac{M_{agg}}{G_{sb}\gamma_w}$$

where

V_{agg} = volume of aggregate retained on a given sieve, cm^3
M_{agg} = mass of aggregate retained on a given sieve, g
γ_w = unit weight of water (1.0 g/cm^3)

The following calculation demonstrates how the volume is calculated for the aggregate retained on the 4.75-mm sieve of Aggregate C.

$$Volume = \frac{35.7g}{2.736 \times 1.0\, g/cm^3} 13.05\, cm^3$$

where

35.7 g = mass of Aggregate C retained on 4.75-mm sieve (Table 10-5)
2.736 = bulk specific gravity of Aggregate C (Table 10-4)
1.0 g/cm^3 = unit weight of water (γ_w)
13.05 cm^3 = volume of Aggregate C retained on 4.75-mm sieve

The volumes retained on all sieves for each of the four stockpiles are provided in Table 10-6.

(continued on next page)

Example Problem 10-1. (Continued)

Table 10-6. Volumes of aggregate retained on each sieve.

Sieve, mm	Volume of Aggregate Retained per Sieve, cm^3			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	0.00	0.00	0.00	0.00
19.0	1.91	0.00	0.00	0.00
12.5	11.09	10.97	0.95	0.00
9.5	8.79	8.78	4.68	0.00
4.75	13.00	14.63	13.05	0.00
2.36	1.53	0.73	7.71	0.00
1.18	1.15	0.00	4.09	0.00
0.60	0.00	0.37	2.16	0.00
0.30	0.00	0.00	1.13	0.00
0.075	0.38	0.55	1.10	11.45
-0.075	0.38	0.55	1.68	30.20

Blend Stockpiles

The values provided in Table 10-7 are used to blend the different stockpiles to meet the desired gradation based on volumes. This process is identical to blending stockpiles by mass and is a trial and error process. To perform the blending, select the estimated percentages of the different stockpiles to be used. For this example, the following percentages will be evaluated first:

Stockpile	% Blend
Aggregate A	30
Aggregate B	30
Aggregate C	30
Mineral filler	10

The percent of each stockpile in the blend is multiplied by the volume retained on a given sieve for each stockpile to determine the total volume retained on

Table 10-7. Total volumes retained per sieve.

Sieve, mm	Volume Retained per Sieve, cm^3
25.0	0.00
19.0	0.57
12.5	6.90
9.5	6.67
4.75	12.20
2.36	2.99
1.18	1.57
0.60	0.76
0.30	0.34
0.075	1.75
-0.075	3.80
Total Volume, Σ	37.55

Example Problem 10-1. (Continued)

that sieve. Using the 4.75-mm sieve as an example, the total volume retained on the 4.75-mm sieve would be calculated as follows:

$$\textit{Total Volume Retained on 4.7 - mm sieve} = (0.30 \times 13.00) + (0.30 \times 14.63) +$$
$$(0.30 \times 13.05) + (0.10 \times 0.00) = 12.20\ cm^3$$

where 0.30, 0.30, 0.30 and 0.10 are the percentages by mass of each aggregate in blend expressed as decimals; and 13.00, 14.63, 13.05, and 0.00 are the % volume retained on 4.75-mm sieve for each stockpile (Table 10-6).

This calculation is performed for each of the sieves in the gradation. Table 10-7 presents the total volume retained for each of the sieves in the gradation.

Now, based on the total volume retained per sieve and the summed total volume of the blended aggregates, the percent retained per sieve by volume can be determined for the blend. This is accomplished for a given sieve by dividing the volume retained on that sieve by the total volume of the blend. The following equation illustrates this calculation for the 4.75-mm sieve.

$$\textit{\% Volume Retained on 4.7-mm Sieve} = 12.20 \times 100/37.55 = 32.50\%$$

where

12.20 = volume retained on 4.75-mm sieve (Table 10-7)
37.55 = summed total volume of blend (Table 10-7)
32.50 = percent volume of blend retained on 4.75-mm sieve

Table 10-8 provides the percents retained based on volumes for each of the sieves and converts this to percent volume passing.

Using the % retained per sieve based on volume, the % passing by volume for the gradation can be determined similar to the method used with gradations based on mass. Determine the cumulative % retained for each sieve and then subtract from 100. Now, the blended gradation is compared to the required gradation band

Table 10-8. Percent passing based on volumes.

Sieve, mm	Percent Retained Per Sieve	Cumulative Percent Retained	Percent Passing by Volume
25.0	0.0	0.0	100.0
19.0	1.5	1.5	98.5
12.5	18.4	19.9	80.1
9.5	17.8	37.7	62.3
4.75	32.5	70.1	29.9
2.36	8.0	78.1	21.9
1.18	4.2	82.3	17.7
0.60	2.0	84.3	15.7
0.30	0.9	85.2	14.8
0.075	4.7	89.9	10.1
-0.075	10.1	100.0	---

(continued on next page)

Example Problem 10-1. (Continued)

Table 10-9. Comparison of gradation blend based on volume to specified gradation band.

Sieve, mm	Gradation Band Requirements	Blend Percent Passing
25.0	100	100
19.0	90-100	98.5
12.5	50-88	80.1
9.5	25-60	62.3*
4.75	20-28	29.9*
2.36	16-24	21.9
1.18	---	17.7
0.60	---	15.7
0.30	---	14.8
0.075	8-10	10.1

*** Does not meet requirements**

(also based on volume) provided in Table 10-3. Table 10-9 compares the gradation band for a 19.0-mm NMAS GGHMA to the gradation shown in Table 10-8.

Based on Table 10-9, the blended gradation did not meet the specified gradation band for a 19.0-mm nominal maximum aggregate size GGHMA. Therefore, different blending percentages for the various stockpiles are needed. Below are the percentages of the four stockpiles used for the second trial.

Stockpile	% Blend
Aggregate A	40
Aggregate B	41
Aggregate C	10
Mineral Filler	9

Table 10-10 presents the blended gradation of the four aggregates for the second trial. The second trial blend percentages were used along with the values of Table 10-6 to determine the percent passing by volume for the blend.

Table 10-10. Percents passing based on volumes.

Sieve, mm	Percent Retained Per Sieve by Volume	Cumulative Percent Retained by Volume	Percent Passing by Volume	Percent Passing by Mass (For Comparison)	Gradation Band by Volume
25.0	0.0	0.0	100.0	100.0	100
19.0	2.0	2.0	98.0	98.0	90-100
12.5	24.0	26.0	74.0	74.3	50-88
9.5	20.1	46.1	53.9	53.5	25-60
4.75	33.2	79.3	21.7	20.0	20-28
2.36	4.5	83.7	16.3	15.4	16-24
1.18	2.3	86.0	14.0	13.1	---
0.60	1.0	87.0	13.0	12.1	---
0.30	0.3	87.3	12.7	11.8	---
0.075	4.0	91.3	8.7	8.0	8-11
-0.075	8.7	100.0	---	---	---

Example Problem 10-1. (Continued)

Based on Table 10-10, the following percentages produce a gradation based on volume, which meets the 19.0-mm nominal maximum aggregate size gradation band for GGHMA.

Stockpile	% Blend by Mass
Aggregate A	40
Aggregate B	41
Aggregate C	10
Mineral Filler	9

Selection of Trial Gradations

When designing GGHMA mixtures, the initial trial gradations should be selected to be within the master specification range shown in Table 10-3. To design a GGHMA mixture it is recommended that at least three trial gradations be initially evaluated. It is suggested that the three trial gradations fall along and in the middle of the coarse and fine limits of the gradation range. These trial gradations are obtained by adjusting the amount of fine and coarse aggregates in each blend. The percent passing the 0.075-mm sieve should be approximately 10 percent for each trial gradation.

Determination of VCA in the Coarse Aggregate Fraction

For best performance, the GGHMA mixtures must have a coarse aggregate skeleton with stone-on-stone contact. The coarse aggregate fraction is not defined by a particular sieve size but rather is that portion of the total aggregate blend retained on the breakpoint sieve. The breakpoint sieve is defined as the finest (smallest) sieve to retain at least 10% of the aggregate gradation (Figure 10-4).

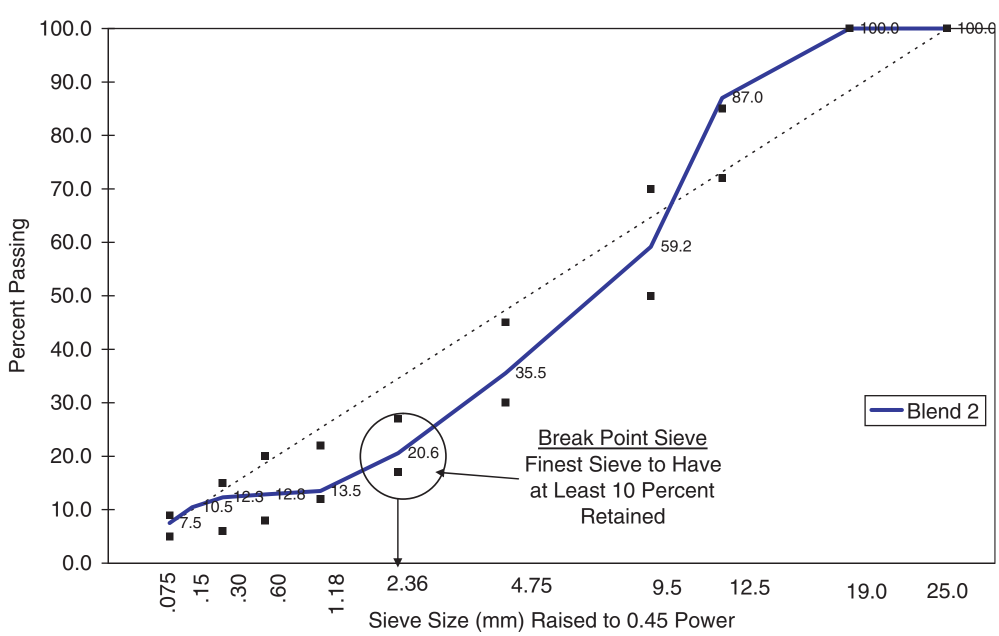

Figure 10-4. Definition of breakpoint sieve.

Figure 10-5. Method of determining VCA in dry-rodded condition.

The method of measuring the existence of stone-on-stone contact is called the voids in coarse aggregate (VCA) method. The concept is quite simple and practical. The first step is to determine the VCA of the coarse aggregate fraction only (material larger than breakpoint sieve) in a dry-rodded condition (VCA_{DRC}) (Figure 10-5). AASHTO T 19, "Bulk Density ("Unit Weight") and Voids in Aggregate," is used to compact the aggregate. Then, using Equation 10-1, the VCA_{DRC} can be calculated. The VCA_{DRC} is nothing more than the volume between the coarse aggregate particles after compaction in accordance with AASHTO T 19. When asphalt binder is added and the GGHMA is compacted, the VCA will again be calculated (VCA_{MIX}). This calculation for VCA_{MIX} also calculates the volume between the coarse aggregate particles. As long as the volume between the coarse aggregate particles is less in the compacted GGHMA (VCA_{MIX}) than the coarse aggregate only (VCA_{DRC}), then the GGHMA is deemed to have stone-on-stone contact and the aggregate structure is acceptable. This means that the GGHMA mixture has been compacted more than the dry-rodded condition of the aggregate; therefore, the coarse aggregate particles within the compacted mixture must be compacted closer than the dry-rodded condition and stone-on-stone contact exists.

$$VCA_{DRC} = \frac{G_{ca}\gamma_w - \gamma_s}{G_{ca}\gamma_w} * 100 \qquad (10\text{-}1)$$

where

VCA_{DRC} = voids in coarse aggregate in dry-rodded condition
γ_s = unit weight of the coarse aggregate fraction in the dry-rodded condition (kg/m^3),
γ_w = unit weight of water (998 kg/m^3), and
G_{ca} = bulk specific gravity of the coarse aggregate fraction

Table 10-11. Minimum asphalt binder content requirements for aggregates with varying bulk specific gravities.

Combined Aggregate Bulk Specific Gravity	Minimum Asphalt Content Based on Mass, %
2.40	6.8
2.45	6.7
2.50	6.6
2.55	6.5
2.60	6.3
2.65	6.2
2.70	6.1
2.75	6.0
2.80	5.9
2.85	5.8
2.90	5.7
2.95	5.6
3.00	5.5

Selection of Target Asphalt Content

The minimum desired asphalt binder content for GGHMA mixtures is presented in Table 10-11. This table illustrates that the minimum asphalt binder content is based on the combined bulk specific gravity of the aggregates used in the mix. These minimum asphalt binder contents are provided to ensure enough volume of asphalt binder exists in the GGHMA mix to provide a desirable mortar and, thus, a durable mixture. It is recommended that the mixture be designed at 0.3% above the minimum values given in Table 10-11 to allow for adjustments during plant production without falling below the minimum requirement. For example, for a GGHMA mixture to be made with an aggregate blend having a combined bulk specific gravity of 2.75, the minimum asphalt content is 6.0% by mass, and the target asphalt content would be $6.0 + 0.3 = 6.3\%$ by mass. The minimum binder content values given in Table 10-11 have been calculated so that, in most cases, the resulting mixes will meet the suggested minimum VMA of 17.0% at 4.0% air voids for GGHMA mixtures.

Sample Preparation

As with the laboratory design of any HMA, the aggregates to be used in GGHMA should be dried to a constant mass and separated by dry-sieving into individual size fractions. The following size fractions are recommended:

- 37.5 mm to 25.0 mm
- 25.0 mm to 19.0 mm
- 19.0 mm to 12.5 mm
- 12.5 mm to 9.5 mm
- 9.5 mm to 4.75 mm
- 4.75 mm to 2.36 mm
- Passing 2.36 mm (for 25.0-, 19.0-, 12.5-, and 9.5-mm NMAS gradations)
- 2.36 mm to 1.18 mm (for 4.75-mm NMAS gradations)
- Passing 1.18 mm (for 4.75-mm NMAS gradations)

After separating the aggregates into individual size fractions, they should be recombined at the proper percentages based on the gradation blend being evaluated.

The mixing and compaction temperatures are determined in accordance with AASHTO T 245, Section 3.3.1. Mixing temperature will be the temperature needed to produce an asphalt binder viscosity of 170±20 cSt. Compaction temperature will be the temperature required to provide an asphalt binder viscosity of 280±30 cSt. However, although these temperatures work for neat asphalt binders, the selected temperatures may need to be changed for polymer-modified asphalt binders. The asphalt binder supplier's guidelines for mixing and compaction temperatures should be used.

When preparing GGHMA in the laboratory, a mechanical mixing apparatus should be utilized. The gap-grading and use of stabilizing additives make GGHMA very difficult to hand-mix. Aggregate batches and asphalt binder are heated to a temperature not exceeding 28°C above the temperature established for mixing temperature. The heated aggregate batch is placed in the mechanical mixing container. Asphalt binder and any stabilizing additive are placed in the container at the required masses. Mix the aggregate, asphalt binder, and stabilizing additives rapidly until thoroughly coated. Mixing times for GGHMA should be slightly longer than for dense-graded HMA to ensure a good distribution of the stabilizing additives. After mixing, the GGHMA mixture should be short-term aged in accordance with AASHTO R 30, Mixture Conditioning of Hot Mix Asphalt. For aggregate blends having water absorption less than 2% the mixture should be aged for 2 hours. If the water absorption of the blend is 2% or higher, the mixture should be aged for 4 hours.

Number of Samples

Twelve samples are initially required—four samples for each of the three trial gradations. Each sample is mixed with the trial asphalt binder content and three of the four samples for each trial gradation are compacted. All 12 samples should be short-term aged. The remaining sample of each trial gradation is used to determine the theoretical maximum specific gravity (G_{mm}) according to AASHTO T 209.

Sample Compaction

Specimens should be compacted at the established compaction temperature after laboratory short-term aging in accordance with AASHTO R 30. Laboratory samples of GGHMA are compacted using 75 gyrations of the Superpave gyratory compactor (SGC). Some agencies have used 100 gyrations with success; however, GGHMA is relatively easy to compact in the laboratory and exceeding 100 gyrations can cause aggregate breakdown and may result in an unacceptably low asphalt content.

Step 3—Selection of Optimum Gradation

After the samples have been compacted and allowed to cool, they are tested to determine the bulk specific gravity, G_{mb} (AASHTO T166). Using G_{mb}, G_{mm}, and the bulk specific gravity of the coarse aggregate fraction (G_{ca}), the percent air voids (VTM), voids in mineral aggregate (VMA), and VCA_{MIX} are calculated. The VTM, VMA, and VCA_{MIX} are calculated as shown below:

$$VTM = 100 * \left(\frac{1 - G_{mb}}{G_{mm}} \right) \tag{10-2}$$

$$VMA = 100 - \left(\frac{G_{mb} * P_s}{G_{sb}} \right) \tag{10-3}$$

$$VCA_{MIX} = 100 - \left(\frac{G_{mb} * P_{ca}}{G_{ca}} \right) \tag{10-4}$$

Table 10-12. GGHMA mixture specification for SGC compacted designs.

Property	Requirement
Asphalt Binder, %	Table 10-12
Air Void Content, %	4.0 ± 0.5
VMA, %	17 min.
VCA_{MIX}, %	Less than VCA_{DRC}
Tensile Strength Ratio	0.80 min.
Draindown at Production Temperature, %	0.30 max

where

P_s = percent of aggregate in the mixture
P_{ca} = percent of coarse aggregate in the mixture
G_{sb} = combined bulk specific gravity of the total aggregate
G_{ca} = bulk specific gravity of the coarse aggregate (coarser than break point sieve)

Once the VTM, VMA, and VCA_{MIX} are determined, each trial blend mixture is compared to the GGHMA mixture requirements. Table 10-12 presents the requirements for GGHMA designs. The trial blend mix that meets or exceeds the minimum VMA requirement, has an air void content between 3.5 and 4.5%, and has a VCA_{MIX} less than VCA_{DRC} is selected as the design gradation. If none meet these requirements, additional aggregate blends should be evaluated. If one of the trial blends is very close to meeting these requirements, with the air void content and VMA just outside their acceptable ranges, an adjustment in the binder content might provide an acceptable mix design, as discussed below.

Step 4—Refine Design Asphalt Binder Content

Once the design gradation of the mixture is chosen, it may be necessary to raise or lower the asphalt binder content to obtain the proper amount of air voids in the mixture. In this case, additional samples are prepared using the selected gradation and varying the asphalt binder content. The optimum asphalt binder content is chosen to produce 4.0% air voids in the mixture; because of typical error in volumetric analysis, air void contents within ± 0.5% of this target are acceptable. The optimum asphalt binder content should meet the minimum asphalt content requirements in Table 10-11.

The number of samples needed for this portion of the procedure is again twelve, with three compacted and one uncompacted sample at each of three asphalt binder contents. The mixture properties are again determined, and the optimum asphalt binder content is selected. The designed GGHMA mixture at optimum asphalt content selected should have properties meeting the criteria shown in Table 10-12. If these criteria are not met, the mixture must be modified so that all criteria are met.

Step 5—Conduct Performance Testing

Performance testing of GGHMA mixtures consists of three tests: (1) evaluation of moisture susceptibility; (2) evaluation of draindown sensitivity; and (3) evaluation of rut resistance.

Evaluation of Moisture Susceptibility

Moisture susceptibility of the selected mixture is determined using AASHTO T 283. One minor change to AASHTO T 283 is that GGHMA samples should be compacted to 6±1% air voids instead of 7±1%. This air void content approximates the recommended higher level of compaction in the field of 94 to 95% of G_{mm}. The mixture should have a minimum tensile strength ratio of 80%.

Evaluation of Draindown Sensitivity

Draindown sensitivity of the selected mixture is determined in accordance with AASHTO T 305. Draindown sensitivity is determined at the anticipated plant production temperature and should not exceed 0.3%.

Evaluation of Rut Resistance

The final step in the design of a GGHMA mixture is the evaluation of rut resistance, also called performance testing. Chapter 6 presents a general discussion of performance evaluation of HMA mixtures and discusses specific tests. Chapter 8 provides a more practical discussion of how, during the HMA mix design process, rut resistance is evaluated using one from among the following tests: (1) the flow number test; (2) the flow time test; (3) the asphalt pavement analyzer (APA) test; (4) the Hamburg wheel-track test; (5) the repeated shear at constant height (RSCH) test as performed using the Superpave shear tester (SST); or the high-temperature indirect tension (IDT) strength test. Detailed discussions of these tests can be found in Chapters 6 and 8. A summary of performance testing requirements and their application to GGHMA mixtures is given below.

The design procedures set forth in this manual—including that for GGHMA—are structured to provide HMA mix designs that exhibit a high level of rut resistance. The level of reliability against excessive rutting—even without performance testing—ranges from 90 to over 99%, with a typical level of about 95% reliability for design traffic levels of 3 million ESALs and higher. The purpose of rut resistance testing is to increase this level of reliability. For three of the rut resistance tests recommended in this manual—the flow number from the asphalt mixture performance test (AMPT), the repeated shear at constant height (RSCH) test, and the high-temperature indirect tension (HT-IDT) strength test—the suggested minimum or maximum test values were determined specifically to increase the level of reliability against excessive rutting from about 95 to 98% and higher. It must be emphasized that the reliability achieved through the recommended performance tests is a result of applying both the suggested mix design procedure and the selected performance test together. If the given guidelines for performance test results are applied to mixtures designed following some other procedure, the resulting level of reliability will not necessarily be the same. It might be similar, or it might be lower or higher. It should also be noted that the specified test values have, in most cases, been selected so that if the procedures given in this manual are followed, most of the resulting HMA mixtures will pass the selected performance test. It is estimated that only about 10 to 20% of properly designed mixtures will fail. Thus, the suggested rutting performance tests not only increase reliability against excessive rutting to a very high level, they do so in a relatively efficient way.

The suggested maximum rut depths for the asphalt pavement analyzer (APA) and the Hamburg wheel-track (HWT) tests were taken from specifications already in place in a number of states using these tests. In this case, implementation of these performance tests will certainly increase the reliability against excessive rutting, but the specific amount of improvement is unknown as is the percentage of mixes likely to fail the tests. However, because these tests with the stated maximum rut depths have been implemented in several states, it is likely that the increase in reliability and the rejection rate will both be reasonable.

Guidelines for interpreting the various rut resistance tests are given in Tables 8-21 through 8-25 in Chapter 8. As an example, Table 10-13 gives recommended minimum values for flow number as determined using the AMPT. This test was initially called the simple performance test or SPT. Details of the latest equipment specification and test procedure are given in AASHTO TP 79-09.

Table 10-13. Recommended minimum flow number requirements.

Traffic Level *Million ESALs*	Minimum Flow Number
< 3	---
3 to < 10	200
10 to < 30	320
≥ 30	580

Tests are performed on specimens cored and trimmed from large gyratory specimens to final nominal dimensions of 100 mm diameter by 150 mm high. In the flow number test, a 600 kPa load is applied to the specimen every second, until the flow point is reached, representing failure of the specimen as seen in an increasing rate of total permanent strain during the test. Flow number tests are run at the average, 7-day maximum pavement temperature 20 mm below the surface, at 50% reliability as determined using LTPPBind version 3.1. Specimens should be prepared at the expected average air void content at the time of construction, typically 7.0±0.5%. Because GGHMA mixtures are high-performance materials, usually intended for very demanding applications, it is likely that the required test values used for these mixes will be those for the highest design traffic levels. For example, most GGHMA mixtures should probably meet or exceed a flow number of 580—representing the minimum flow number for design traffic levels of 30 million ESALs and higher—when tested using the flow number test as described in Chapter 8.

As noted in Chapter 8, the minimum flow number values given in Table 10-13 are for fast traffic; it is suggested that, to account for the greater damage associated with slow traffic, the test temperature be increased for slower traffic speeds. The specifics of such adjustments are given in Chapter 8. Although the suggested required test values given in Table 10-13 and in the related tables presented in Chapter 8 are based either on a careful analysis of laboratory and field data, or on existing standards, it is quite possible that they will need to be adjusted by the specifying agency for optimum results for GGHMA mixtures in its region. Factors that need to be considered when making such adjustments are climate, the types and grades of binders commonly used in a given locale, aggregates with unusual properties, and typical traffic mixes and traffic levels. For various reasons, some agencies might wish to alter the conditions a test is run under, which will significantly alter the resulting test values and the appropriate specification values. For details on the proper procedures for performing each test, engineers and technicians should refer to the appropriate standard test method.

Trouble Shooting GGHMA Mix Designs

If the designer cannot produce a mixture that meets all requirements, remedial action will be necessary. Some suggestions to improve mixture properties are provided below.

Air Voids

The amount of air voids in the mixture can be controlled by the asphalt binder content. However, a problem occurs when low voids exist at the minimum asphalt binder content. Lowering the asphalt binder content below the minimum to achieve the proper amount of air voids violates the required minimum asphalt binder content (Table 10-11). Instead, the aggregate gradation should be modified to increase the VMA so that additional asphalt binder can be added without decreasing the voids below an acceptable level.

Voids in the Mineral Aggregate

The VMA may be raised by decreasing the percentage of aggregate passing the breakpoint sieve or by decreasing the percentage passing the 0.075-mm sieve. Changing aggregate sources or stockpiles may also be required to solve the problem.

Voids in the Coarse Aggregate

If the VCA_{MIX} is higher than that in the dry-rodded condition (VCA_{DRC}) then the mixture gradation must be modified. This is typically accomplished by decreasing the percent passing the breakpoint sieve.

Moisture Susceptibility

If the mixture fails to meet the moisture susceptibility requirements, lime or liquid anti-strip additives can be used. If these measures prove ineffective, the aggregate source, asphalt binder source, or both can be changed to obtain better aggregate and asphalt binder compatibility.

Draindown Sensitivity

Problems with draindown sensitivity can be remedied by increasing the amount of stabilizing additive or by selecting a different stabilizing additive. Fibers have been shown to be very effective in reducing draindown.

Rut Resistance

As mentioned earlier in this section, the rut resistance tests and recommended minimum or maximum values for test results have been selected so that most GGHMA designs developed following the procedures given in this manual will meet the requirements, and no additional laboratory work will be needed. However, some mix designs may fail to meet requirements for rut resistance. In such cases, the test results should first be checked to make sure there were no errors in either the procedures used or in the calculation of the test results. If no errors are found, and the test results are close to meeting the requirements, the test can be repeated. In this case, the results of both tests should be averaged and compared to the test criteria. If the mix still fails to meet the requirements for rut resistance testing, the mix design will have to be modified. The rut resistance of a GGHMA mix design can be improved in the following ways:

- Increase the binder high-temperature grade.
- If the binder is not modified, consider using a polymer-modified binder of the same grade or one high-temperature grade lower.
- If the binder is polymer modified, try a different type of modified binder.
- Increase the amount of mineral filler in the mix, adjusting the aggregate gradation if necessary to maintain adequate VMA.
- Decrease the design VMA value, if possible, by adjusting the aggregate gradation.
- Replace part or all of the aggregate (fine or coarse or both) with a material or materials having improved angularity.

If a different asphalt binder is used in the mix, the volumetric composition should not change. However, if other aspects of the mix design are changed, the volumetric composition might change significantly, which will require further refinement of the mix prior to further rut resistance testing.

Bibliography

AASHTO Standards

M 17, Mineral Filler for Bituminous Paving Mixtures
M 320, Performance-Graded Asphalt Binder
M 325, Standard Specification for Designing Stone Matrix Asphalt (SMA)
R 46, Standard Practice for Designing Stone Matrix Asphalt (SMA)
R 30, Mixture Conditioning of Hot-Mix Asphalt
T 19, Bulk Density ("Unit Weight") and Voids in Aggregate
T 96, Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.
T 104, Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens
T 176, Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalency Test.

T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures.
T 245, Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.
T 283, Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage.
T 304, Uncompacted Void Content of Fine Aggregate
T 305, Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures.
T 326, Uncompacted Void Content of Coarse Aggregate (As Influenced by Particle Shape, Surface Texture and Grading).
TP 79-09, Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)

ASTM Standards

C 612, Mineral Fiber Block and Board Insulation
D 4791, Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate

Other Publications

Bonaquist, R. F. (2008) *NCHRP Report 629: Ruggedness Testing of the Dynamic Modulus and Flow Number Tests with the Simple Performance Tester,* Transportation Research Board, National Research Council, Washington, DC, 130 pp.
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NAPA (1999) *Designing and Constructing SMA Mixtures—State of the Practice (QIP-122),* Lanham, MD, 43 pp.
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CHAPTER 11

Design of Open-Graded Mixtures

Open-graded friction course (OGFC) is a specialty HMA that uses an extremely open aggregate gradation to improve frictional resistance, reduce splash and spray, improve nighttime visibility, reduce hydroplaning, or reduce pavement noise levels. OGFC is specifically designed to have a large percentage of a single size coarse aggregate with a low percentage of fine aggregates and a very low percentage of materials finer than 0.075 mm (dust or mineral filler). A relatively single size coarse aggregate combined with a low amount of fine aggregate and dust provides for a much more open aggregate gradation compared to other HMA mix types.

OGFCs were first developed during the 1940s through experimentation with plant-mix seal coats. Even though these plant-mix seal coats provided excellent frictional properties, their use spread slowly because they required a different mix design method and special construction considerations than typically used HMA. It was not until the 1970s, when the FHWA published a formalized mix design procedure that the use of OGFCs began to increase. This procedure entailed an aggregate gradation requirement, a surface capacity of coarse aggregate, determination of fine aggregate content, determination of optimum mixing temperature, and determination of the resistance of the designed mixture to moisture.

During the 1970s and 1980s, some agencies observed performance problems when using OGFCs. Primarily, the problems were raveling and delamination. These distresses were caused by problems associated with mix design, material specification, and construction. The primary issue involved mix temperature during construction. The gradations associated with OGFCs were much coarser than typical dense-graded mixes. Additionally, unmodified asphalt binders were used with these OGFCs at that time. Because of the open nature of the aggregate gradation, there were problems with the asphalt binder draining from the coarse aggregate during transport. To combat these draindown problems, the mix temperature was reduced during production. This reduction in temperature resulted in two problems. First, the internal moisture within the aggregates was not removed and, second, the OGFC did not bond with the existing pavement when placed. These two issues resulted in the raveling and delamination problems frequently encountered with OGFCs during the 1970s and 1980s.

During the 1990s, major improvements were made in HMA specifications and design methods. Additionally, new technologies were adopted from Europe for stone matrix asphalt (SMA), gap-graded hot mix asphalt (GGHMA), and OGFC. These improvements and ideas have resulted in new methods for designing and constructing OGFC, notably the inclusion of polymer-modified asphalt binders and stabilizing materials, which have significantly improved the resulting mixtures and the overall performance of OGFCs. Stability additives, such as mineral filler now help prevent draindown and allow higher mixing temperatures. This allows better bonding between the OGFC and the underlying layer through the use of a proper tack coat, reducing the potential for delamination. Additionally, the higher mixing temperatures help to dry the aggregate more completely, which reduces the potential for moisture damage and raveling. The use of polymer-modified binders in OGFCs has increased the durability of these mixes. All of these improvements have led to increased OGFC service life.

Within the overall category of OGFC, there are two predominant types used in the United States, which can be generically called permeable friction courses (PFCs) and asphalt concrete friction courses (ACFCs). PFCs are an OGFC that are specifically designed to have high air void contents, typically in the range of 18 to 22%, which helps remove water from the pavement surface during a rain event. PFCs have been referred to as porous European mixes in the United States and are effective in improving frictional resistance, reducing splash and spray, improving nighttime visibility, reducing hydroplaning, and reducing pavement noise levels. The term ACFC is applied to OGFC mixes that are not specifically designed for removing water from the pavement surface. Some agencies within the United States use ACFCs as a wearing surface to simply improve frictional resistance and reduce tire/pavement noise levels. These agencies typically include 8 to 9% of rubber modified asphalt binders within ACFCs. Though ACFCs are designed to have relatively high air void contents (10 to 15%), they are not specifically designed to remove large volumes of water from the pavement surface. Of the two OGFC categories, ACFCs are likely more effective at reducing pavement noise levels. However, some agencies have become concerned about the durability of ACFCs, particularly when their air void contents approach 10%. PFCs have become the more common type of OGFC in the United States, and the remainder of this chapter deals specifically with the design of this mixture type.

Overview of PFC Mix Design Procedure

The design of PFC mixtures is similar to the design of SMA and GGHMA in that PFC should have stone-on-stone contact and low potential for draindown. However, because of the past problems dealing with durability, there is a laboratory test designed specifically to evaluate the potential durability problems of PFC mixtures. *NCHRP Report 640* provides much useful information on the mix design, construction, and maintenance of permeable friction courses.

The design of PFCs consists of five primary steps (Figure 11-1). The first step is to select suitable materials. Materials needing selection include coarse aggregates, fine aggregates, asphalt binder, and stabilizing additives. Step 2 includes blending three trial gradations using the selected aggregate stockpiles. For each trial gradation, asphalt binder is added and the mixture compacted. The third step in the mix design procedure entails evaluating the three compacted trial blends in order to select the design gradation. Next, the selected design gradation is fixed and the asphalt binder content is varied. The resulting mixtures are evaluated in order to select optimum asphalt binder. Finally, the design gradation at optimum asphalt binder content is evaluated for moisture susceptibility. This manual does not include provisions for performance testing of PFC mixtures, because there is limited experience at present in performing and interpreting rut resistance tests on this type of material.

The information given in this chapter is largely taken from *Design Construction and Performance of New-Generation Open-Graded Friction Courses, NCAT Report 00-01,* by Mallic, Kandhal, Cooley, and Watson. This report is an excellent reference for technicians and engineers involved in the design of OGFC mixtures.

Step 1—OGFC Materials Selection

The first step in the PFC mix design procedure is to select suitable materials: coarse aggregates, fine aggregates, asphalt binder, and stabilizing additives. Aggregates used in PFC should be cubical, angular, and roughly textured. The stability and strength of PFCs are derived from the stone

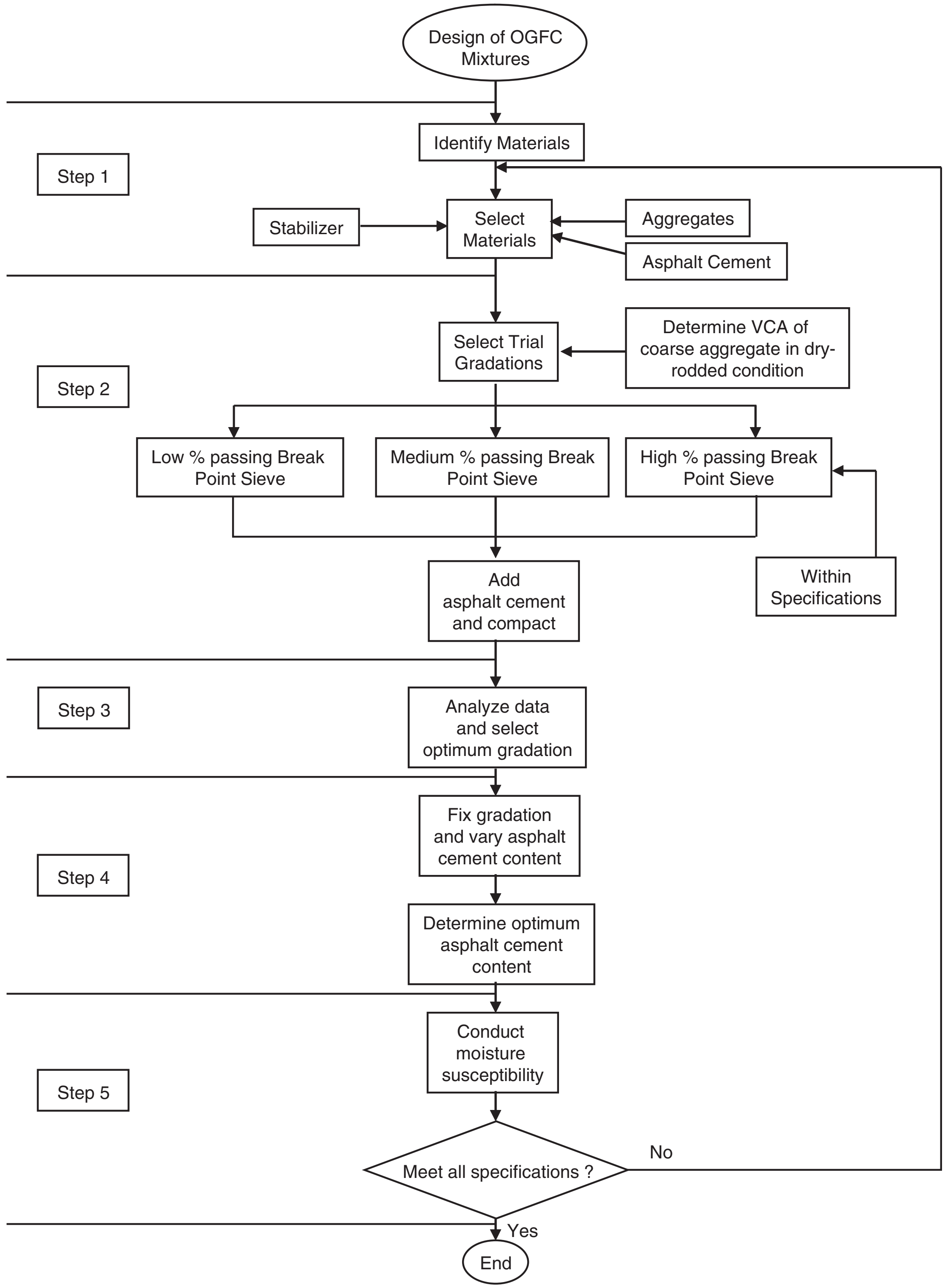

Figure 11-1. Flow diagram illustrating PFC mix design methodology.

skeleton and, therefore, the shape and angularity should be such that the aggregates will not slide past each other. Angular, cubical, and textured aggregate particles will lock together providing a stable layer of PFC.

Because of the open-graded aggregate structure, the aggregate surface area of PFCs is very low. Like GGHMA and SMA, PFC mixes are required to have a relatively high asphalt binder content. Therefore, the aggregates are coated with a thick film of asphalt binder and the properties of the asphalt binder are important to the performance of PFC. The asphalt binder must be very stiff at high temperatures to resist the abrading action of traffic; however, they should also perform at intermediate and low temperatures. Modified binders are not necessarily required; however, experience indicates better and longer service when modified binders are used.

Because of the high asphalt binder content and low aggregate surface area, PFC mixes have a high potential for draindown problems. In order to combat the draindown problems, stabilizing additives are used. The most common stabilizing additive is fiber. Asphalt binder modifiers that stiffen the asphalt binder can also be a considered a stabilizing additive. However, fibers are more effective at reducing draindown potential.

The following sections provide requirements for the various materials used in PFC mixes. These requirements are provided for guidance to agencies not having experience with these types of mixtures. Some agencies have successfully used other test methods and criteria for specifying materials.

Coarse Aggregate

The success of a PFC pavement depends heavily on the existence of particle-on-particle contact. Therefore, in addition to particle shape, angularity, and texture, the toughness and durability of the coarse aggregates must be such that they will not degrade during production, construction, and service. Table 11-1 presents coarse aggregate requirements for PFC mixtures.

Fine Aggregate

The role of fine aggregates in a PFC is to assist the coarse aggregate particles in maintaining stability. However, the fine aggregates must also resist the effects of weathering. Therefore, the primary requirements for fine aggregates within a PFC are to ensure a durable and angular material. Requirements for fine aggregates for use in PFCs are provided in Table 11-2.

Asphalt Binder

Asphalt binders should meet the performance grade requirements of AASHTO M 320-04. Chapter 8 discussed binder selection for dense-graded HMA mixtures in detail; much of this

Table 11-1. Coarse aggregate quality requirements for PFC.

Test	Method	Spec. Minimum	Spec. Maximum
Los Angeles Abrasion, % Loss	AASHTO T 96	-	[a]
Flat or Elongated, %	ASTM D 4791		
2 to 1		-	50
Soundness (5 Cycles), %	AASHTO T 104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Uncompacted Voids	AASHTO T 326 Method A	45	-

[a]Aggregates with L.A. Abrasion loss values up to 50 have been successfully used to produce OGFC mixtures. However, when the L.A. Abrasion exceeds approximately 30, excessive breakdown may occur in the laboratory compaction process or during in-place compaction.

Table 11-2. Fine aggregate quality requirements for PFC.

Test	Method	Spec. Minimum	Spec. Maximum
Soundness (5Cycles), %	AASHTO T 104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Uncompacted Voids	AASHTO T 304, Method A	45	-
Sand Equivalency	AASHTO T 176	50	-

discussion also applies to PFC mixtures. However, because of the high binder content and open-graded aggregate in PFC mixtures, a stiff asphalt binder is needed to ensure a durable mixture. For pavements with design traffic levels of 10 million ESALs and higher, the high-temperature performance grade should be increased by two grades (12°C) over that which would normally be used for the given conditions. For lower design traffic levels, the high-temperature performance grade should be increased at least one grade (6°C). The use of polymer modified binders or asphalt rubber binders is strongly indicated for PFC mixtures.

Stabilizing Additives

Stabilizing additives are needed within PFC to prevent the draindown of asphalt binder from the coarse aggregate during transportation and placement. Stabilizing additives, such as cellulose fiber, mineral fiber, and polymers, have been used successfully to minimize draindown potential. When using polymer or rubber as a stabilizer, the amount of additive added should be that amount necessary to meet the specified performance grade of the asphalt binder.

Cellulose fibers are typically added to a PFC mixture at a dosage rate of 0.3% by total mixture mass. Requirements for cellulose fibers are presented in Table 11-3. Mineral fibers are typically added at a dosage rate of 0.4% of total mixture mass. Requirements for mineral fibers are provided in Table 11-4. Experience has shown that fibers are the best draindown inhibitor.

Step 2—Trial Gradations

As with any HMA, specified aggregate gradations should be based on aggregate volume and not aggregate mass. However, for most PFC mixtures, the specific gravities of the different aggregate stockpiles are close enough to make the gradations based on mass percentages similar to that based on volumetric percentages. The specified PFC gradation bands presented in Table 11-5 are based on % passing by volume.

Selection of Trial Gradations

The initial trial gradations must be selected to be within the master specification ranges presented in Table 11-5 and illustrated in Figures 11-2 through 11-4. It is recommended that at least three trial gradations be initially evaluated. These three trial gradations should fall along and in the middle of the coarse and fine limits of the gradation range. These trial gradations are obtained by adjusting the amount of fine and coarse aggregates in each blend.

Determination of VCA in the Coarse Aggregate Fraction

For best performance, the PFC mixture must have a coarse aggregate skeleton with stone-on-stone contact. The coarse aggregate fraction of the blend is that portion of the total aggregate

Table 11-3. Cellulose fiber requirements.

Property	Requirement
Sieve Analysis	
Method A – Alpine Sieve[1] Analysis	
Fiber Length	6-mm (0.25 in.) Maximum
Passing 0.150-mm (No. 100 sieve)	70±10%
Method B – Mesh Screen[2] Analysis	
Fiber Length	6-mm (0.25 in.) Maximum
Passing 0.850-mm (No. 20) sieve	85±10%
0.425-mm (No. 40) sieve	65±10%
0.160-mm (No. 140) sieve	30±10%
Ash Content[3]	18±5% non-volatiles
pH[4]	7.5±1.0%
Oil Absorption[5]	5.0±1.0% (times fiber mass)
Moisture Content[6]	Less than 5% (by mass)

[1]Method A – Alpine Sieve Analysis. This test is performed using an Alpine Air jet Sieve (type 200LS). A representative 5-g sample of fiber is sieved for 14 minutes at a controlled vacuum of 75 kPa (11 psi) of water. The portion remaining on the screen is weighed.
[2]Medthod B – Mesh Screen Analysis. This test is performed using standard 0.850, 0.425, 0.250, 0.180, 0.150, and 0.106-mm sieves, nylon brushes, and a shaker. A representative 10 gram sample of fiber is sieved, using a shaker and two nylon brushes on each screen. The amount retained on each sieve is weighed and the percentage passing calculated. Repeatability of this method is suspect and needs to be verified.
[3]Ash Content. A representative 2-3 gram sample of fiber is placed in a tared crucible and heated between 595 and 650°C (1100 and 1200°F) for not less than 2 hours. The crucible and ash are cooled in a desiccator and weighed.
[4]pH Test. Five grams of fiber are added to 100 ml of distilled water, stirred, and let sit for 30 minutes. The pH is determined with a probe calibrated with pH 7.0 buffer.
[5]Oil Absorption Test. Five grams of fiber are accurately weighed and suspended in an excess of mineral spirits for not less than 5 minutes to ensure total saturation. It is then placed in a screen mesh strainer (approximately 0.5 mm^2 opening size) and shaken on a wrist action shaker for 10 minutes [approximately 32-mm (1¼ in) motion at 240 shakes per minute]. The shaken mass is then transferred without touching to a tared container and weighed. Results are reported as the amount (number of times its own weight) the fibers are able to absorb.
[6]Moisture Content. Ten grams of fiber are weighed and placed in a 121°C (250°F) forced air oven for 2 hours. The sample is then re-weighed immediately upon removal from the oven.

Table 11-4. Mineral fiber quality requirements.

Property	Requirement
Size Analysis	
Fiber Length[1]	6-mm (0.25 in.) Maximum mean test value
Thickness[2]	0.005-mm (0.0002 in.) Maximum mean test value
Shot Content[3]	
Passing 0.250-mm (No. 60) sieve	90±5%
Passing 0.005-mm (No.230) sieve	70±10%

[1]The fiber length is determined according to the Bauer McNett fractionation.
[2]The fiber diameter is determined by measuring at least 200 fibers in a phase contrast microscope.
[3]Shot content is a measure of non-fibrous material. The shot content is determined on vibrating sieves. Two sieves, 0.250 and 0.063 are typically utilized. For additional information see ASTM C612.

Table 11-5. PFC gradation specification bands.

Sieve Size, mm	9.5-mm PFC	12.5-mm PFC	19-mm PFC
Grading Requirements	*% Passing*		
25mm			100
19mm		100	85-100
12.5mm	100	80-100	55-70
9.5mm	85-100	35-60	---
4.75mm	20-30	10-25	10-25
2.36mm	5-15	5-10	5-10
75µm	0-4	0-4	0-4

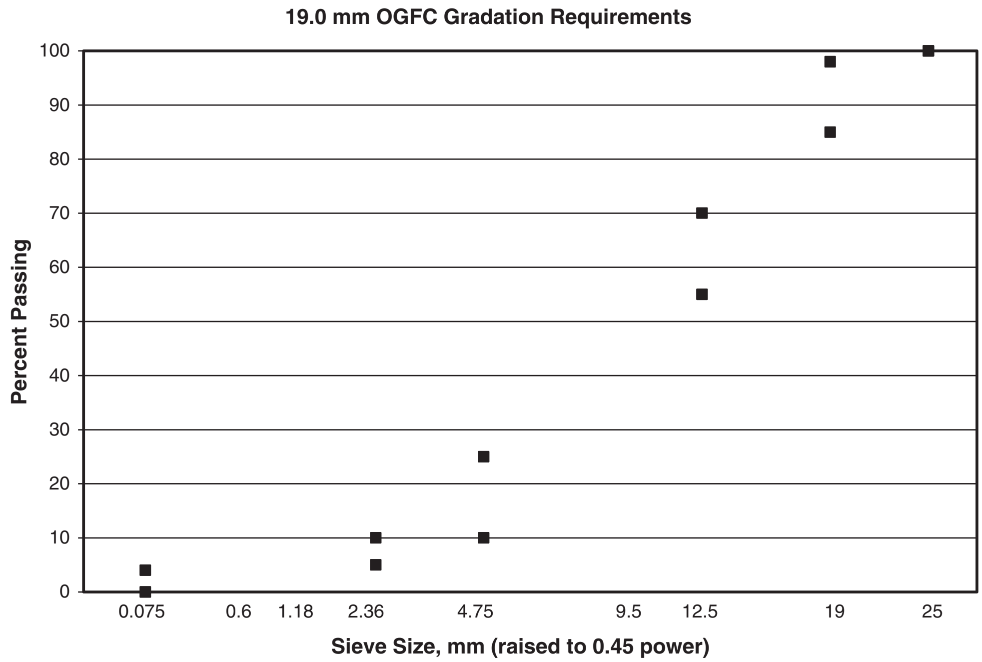

Figure 11-2. 19.0-mm PFC gradation requirements.

retained on the breakpoint sieve. The breakpoint sieve is defined as the finest (smallest) sieve to retain 10 percent of the aggregate gradation. The voids in coarse aggregate for the coarse aggregate fraction (VCA_{DRC}) are determined using AASHTO T 19. When the dry-rodded density of the coarse aggregate fraction has been determined, the VCA_{DRC} for the fraction can be calculated using the following equation:

$$VCA_{DRC} = \frac{G_{ca}\gamma_w - \gamma_s}{G_{ca}\gamma_w} * 100 \qquad (11\text{-}1)$$

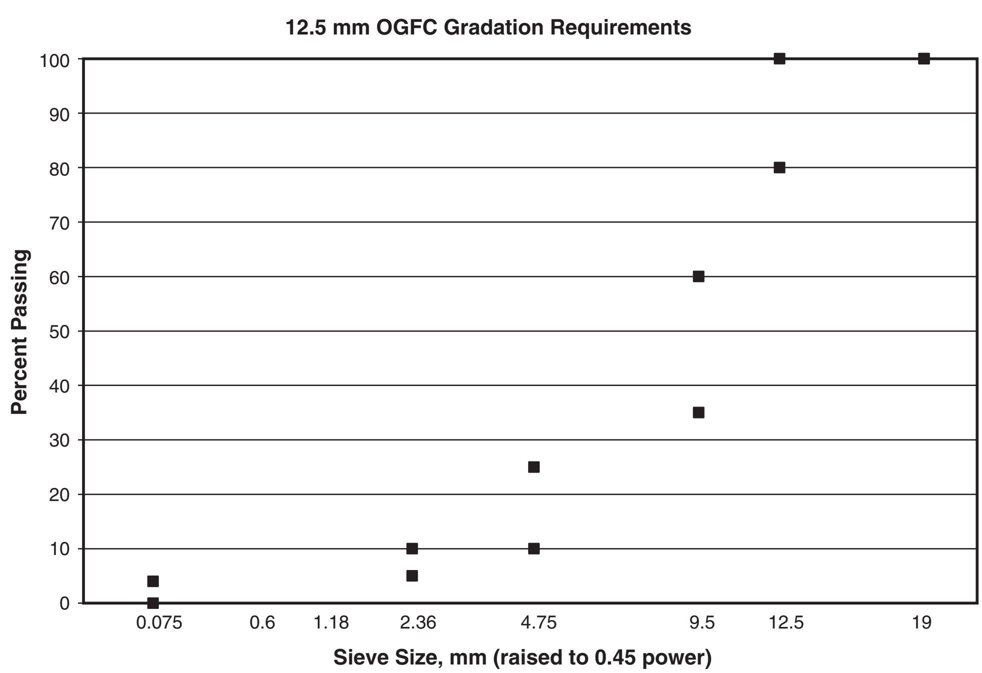

Figure 11-3. 12.5-mm PFC gradation requirements.

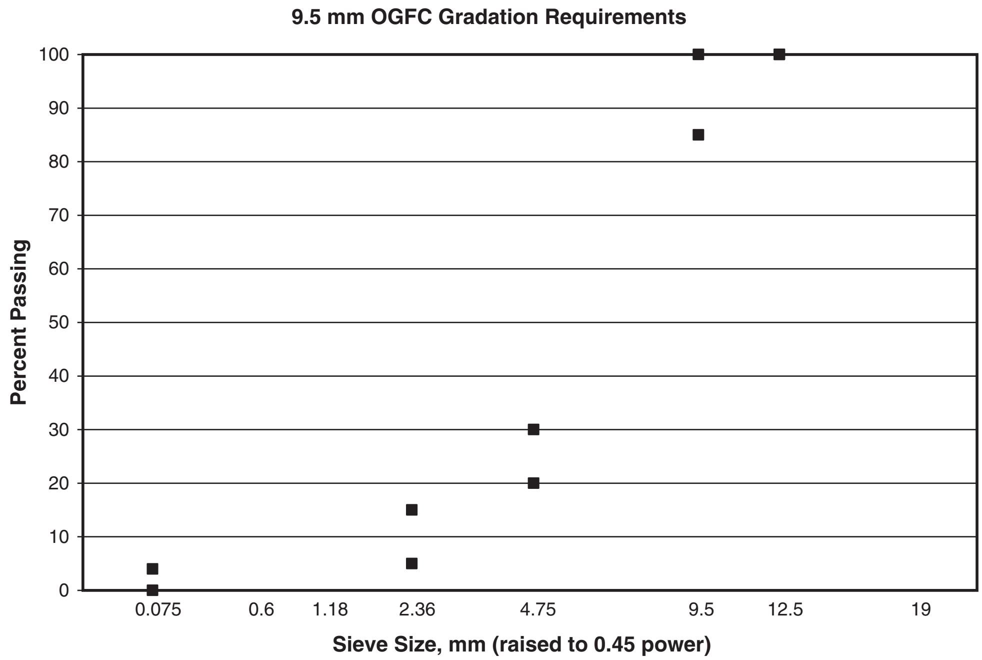

Figure 11-4. 9.5-mm PFC gradation requirements.

where

VCA_{DRC} = voids in coarse aggregate in dry-rodded condition
γ_s = unit weight of the coarse aggregate fraction in the dry-rodded condition (kg/m^3),
γ_w = unit weight of water (998 kg/m^3), and
G_{ca} = bulk specific gravity of the coarse aggregate

The results from this test are compared to the VCA in the compacted PFC mixture (VCA_{MIX}). Similar to GGHMA and SMA, when the VCA_{MIX} is equal to or less than the VCA_{DRC}, stone-on-stone contact exists.

Selection of Trial Asphalt Binder Content

The minimum desired asphalt binder content for PFC mixtures is presented in Table 11-6. Table 11-6 illustrates that the minimum asphalt binder content for PFCs is based on the combined bulk specific gravity of the aggregates used in the mix. These minimum asphalt binder contents are provided to ensure a sufficient volume of asphalt binder in the PFC mix. It is recommended that the mixture be designed at some amount over the minimum to allow for adjustments during plant production without falling below the minimum requirement. As a starting point for trial asphalt binder contents of PFCs, for aggregates with combined bulk specific gravities less than or equal to 2.75, an asphalt binder content between 6 and 6.5% should be selected. If the combined bulk specific gravity of the coarse aggregate exceeds 2.75, the trial asphalt binder content can be reduced slightly.

Sample Preparation

As with the design of any HMA, the aggregates to be used in the mixture should be dried to a constant mass and separated by dry-sieving into individual size fractions. The following size fractions are recommended:

- 19.0 to 12.5 mm
- 12.5 to 9.5 mm

Table 11-6. Minimum asphalt content requirements for aggregates with varying bulk specific gravities—PFCs.

Combined Aggregate Bulk Specific Gravity	Minimum Asphalt Content Based on Mass, %
2.40	6.8
2.45	6.7
2.50	6.6
2.55	6.5
2.60	6.3
2.65	6.2
2.70	6.1
2.75	6.0
2.80	5.9
2.85	5.8
2.90	5.7
2.95	5.6
3.00	5.5

- 9.5 to 4.75 mm
- 4.75 to 2.36 mm
- Passing 2.36 mm

After separating the aggregates into individual size fractions, they should be recombined at the proper percentages based on the gradation blend being used.

The mixing and compaction temperatures are determined in accordance with AASHTO T 245, Section 3.3.1. Mixing temperature will be the temperature needed to produce an asphalt binder viscosity of 170±20 cSt. Compaction temperature will be the temperature required to provide an asphalt binder viscosity of 280±30 cSt. However, although these temperatures work for neat asphalt binders, the selected temperatures may need to be changed for polymer-modified asphalt binders. The asphalt binder supplier's guidelines for mixing and compaction temperatures should be used.

When preparing PFC in the laboratory, a mechanical mixing apparatus should be used. Aggregate batches and asphalt binder are heated to a temperature no more than 28°C greater than the temperature established for mixing. The heated aggregate batch is placed in the mechanical mixing container. Asphalt binder and any stabilizing additive are placed in the container at the required masses. Mix the aggregate, asphalt binder, and stabilizing additives rapidly until thoroughly coated. Mixing times for PFC should be slightly longer than for conventional mixtures to ensure that the stabilizing additives are thoroughly dispersed within the mixture. After mixing, the PFC mixture should be short-term aged in accordance with AASHTO R 30. For aggregate blends having combined water absorption values less than 2%, the mixture should be aged for 2 hours. If the water absorption of the aggregate blend is 2% or more, the mixture should be aged for 4 hours.

Number of Samples

Typically, 12 samples are initially required: four samples for each three trial gradations. Each sample is mixed with the trial asphalt binder content and three of the four samples for each trial gradation are compacted. The remaining sample of each trial gradation is used to determine the theoretical maximum density according to AASHTO T 209.

Sample Compaction

Specimens should be compacted at the established compaction temperature after laboratory short-term aging. Laboratory samples of PFC are compacted using 50 SGC gyrations. More than 50 gyrations should not be used; PFC is relatively easy to compact in the laboratory and exceeding this compactive effort can cause excessive aggregate breakdown.

Step 3—Selection of Optimum Gradation

After the samples have been compacted, extruded, and allowed to cool, they are tested to determine their bulk specific gravity, G_{mb}, using dimensional analysis. Dimensional analysis entails calculating the volume of the sample by obtaining four height measurements with a calibrated caliper, with each measurement being 90 degrees apart. The area of the specimen is then multiplied by the average height to obtain the sample volume:

$$V = h \times \pi \left(\frac{D}{2} \right)^2 \tag{11-2}$$

where

V = specimen volume, in^3
h = specimen height, in
D = specimen diameter, in

Then G_{mb} is determined by dividing the dry mass of the sample by the sample volume. Uncompacted samples are used to determine the theoretical maximum density, G_{mm} (AASHTO T 209). Using G_{mb}, G_{mm}, and G_{ca}, percent air voids, or voids in the total mixture (VTM) and VCA_{MIX} are calculated. The VTM and VCA_{MIX} are calculated by equations 3 and 4 below.

$$VTM = 100 * \left(\frac{1 - G_{mb}}{G_{mm}} \right) \tag{11-3}$$

$$VCA_{MIX} = 100 - \left(\frac{G_{mb} * P_{ca}}{G_{ca}} \right) \tag{11-4}$$

where

P_{ca} = percent of coarse aggregate in the mixture
G_{mb} = combined bulk specific gravity of the total aggregate
G_{ca} = bulk specific gravity of the coarse aggregate

Once VTM and VCA_{MIX} are determined, each trial blend mixture is compared to the PFC mixture requirements, which are presented in Table 11-7. The trial blend with the highest air void content that meets the 18% minimum and exhibits stone-on-stone contact is considered the design gradation. The Cantabro Abrasion test or draindown test may be required in order to select the design gradation.

Step 4—Selection of Optimum Asphalt Binder Content

Once the design gradation has been selected, it is necessary to evaluate various asphalt binder contents in order to select the optimum binder content. Additional samples are prepared using the design gradation and at least three asphalt binder contents. Eighteen samples are needed for this procedure. This provides for three compacted (for G_{mb} and Cantabro Abrasion Loss) and

Table 11-7. PFC mixture specification for SGC compacted designs.

Property	Requirement
Asphalt Binder, %	Table 10-6
Air Void Content, % [1]	18 to 22
Cantabro Loss %	15 max.
VCA_{MIX}%	Less than VCA_{DRC}
Tensile Strength Ratio	0.70 min.
Draindown at Production Temperature, %	0.30 max

[1] Air void requirements are provided for PFC mixes but not ACFC mixes.

three uncompacted samples (one for determination of theoretical maximum density and two for draindown testing) at each of the three asphalt binder contents. Optimum asphalt binder content is selected as the binder content that meets all of the requirements of Table 11-7.

Cantabro Abrasion Loss Test

The Cantabro Abrasion test is used as a durability indicator during the design of PFC mixtures. In this test, three PFC specimens compacted with 50 SGC gyrations are used to evaluate the durability of a PFC mixture at a given asphalt binder content. To begin the test, the mass of each specimen is weighed to the nearest 0.1 gram. A single test specimen is then placed in the Los Angeles Abrasion drum without the charge of steel spheres. The Los Angeles Abrasion machine is operated for 300 gyrations at a speed of 30 to 33 rpm and a test temperature of 25±5°C. After 300 gyrations, the test specimen is removed from the drum and its mass determined to the nearest 0.1 gram. The percentage of abrasion loss is calculated as follows:

$$PL = \frac{(P_1 - P_2)}{P_2}100 \tag{11-5}$$

where

PL = percent loss
P_1 = mass of specimen prior to test, gram
P_2 = mass of specimen after 300 gyrations, gram

The test is repeated for the remaining two specimens. The average results from the three specimens are reported as the Cantabro Abrasion Loss. Resistance to abrasion generally improves with an increase in asphalt binder content or the use of a stiffer asphalt binder. Figure 11-5 illustrates a sample after the Cantabro Abrasion Loss test.

Draindown Sensitivity

The draindown sensitivity of the selected mixture is determined in accordance with AASHTO T 305 except that a 2.36-mm wire mesh basket should be used. Draindown testing is conducted at a temperature of 15°C higher than the anticipated production temperature.

Permeability Testing

A laboratory permeability test is conducted on the selected PFC mixture. Laboratory permeability values greater than 100 m/day are recommended. Permeability of asphalt concrete mixtures can be measured using the provisional standard ASTM PS 129, Measurement of Permeability of Bituminous Paving Mixtures Using a Flexible Wall Permeameter.

Figure 11-5. Illustration of sample after Cantabro Abrasion test.

Step 5—Moisture Susceptibility

Moisture susceptibility of the selected mixture is determined using the modified Lottman method in accordance with AASHTO T 283 with one freeze-thaw cycle. The AASHTO T 283 method should be modified as follows: (1) PFC specimens should be compacted with SGC 50 gyrations; (2) no specific air void content level is required; (3) a vacuum of 26 inches Hg is applied for 10 minutes to saturate the compacted specimens, with no specific saturation level required; and (4) the specimens are kept submerged in water during the freeze-thaw cycle.

Trouble Shooting PFC Mix Designs

If the designer cannot produce a mixture that meets all requirements, remedial action will be necessary. Some suggestions to improve mixture properties are provided below.

Air Voids

The amount of air voids in the mixture can be controlled by the asphalt binder content. However, lowering the asphalt binder content below the minimum to achieve a proper amount of air voids violates the required minimum asphalt binder content (Table 11-6). Instead, the aggregate gradation must be modified to increase the space for additional asphalt binder without decreasing the voids below an acceptable level. Decreasing the percent passing the breakpoint sieve will generally increase the air void content at a given asphalt binder content.

Voids in the Coarse Aggregate

If the VCA_{mix} is higher than that in the dry-rodded condition (VCA_{DRC}), then the mixture gradation must be modified. This is typically done by decreasing the % passing the breakpoint sieve.

Cantabro Abrasion Loss

If the Cantabro Abrasion loss is greater than 15%, then either more asphalt binder or a binder with a greater high-temperature stiffness is needed.

Moisture Susceptibility

If the mixture fails to meet the moisture susceptibility requirements, lime or liquid anti-strip additives can be used. If these measures prove ineffective, the aggregate source or asphalt binder source can be changed to obtain better aggregate/asphalt binder compatibility.

Draindown Sensitivity

Problems with draindown sensitivity can be remedied by increasing the amount of stabilizing additive or by selecting a different stabilizing additive. Fibers have been shown to be very effective in reducing draindown.

Bibliography

AASHTO Standards

AASHTO M 320, Performance-Graded Asphalt Binder
AASHTO R 30, Mixture Conditioning of Hot-Mix Asphalt
AASHTO T 19, Bulk Density ("Unit Weight") and Voids in Aggregate
AASHTO T 96, Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
AASHTO T 104, Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
AASHTO T 176, Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalency Test
AASHTO T 209, Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
AASHTO T 245, Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus
AASHTO T 283, Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage
AASHTO T 304, Uncompacted Void Content of Fine Aggregate
AASHTO T 305, Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures
AASHTO T 326, Uncompacted Void Content of Coarse Aggregate (As Influenced by Particle Shape, Surface Texture and Grading)

Other Standards

ASTM C 612, Mineral Fiber Block and Board Insulation
ASTM D 4791, Flat Particles, Elongated Particles or Flat and Elongated Particles in Coarse Aggregate
ASTM PS 129, Measurement of Permeability of Bituminous Paving Mixtures Using a Flexible Wall Permeameter

Other Publications

Cooley, L. A., et al. (2009) *NCHRP Report 640: Construction and Maintenance Practices for Permeable Friction Courses,* TRB, National Research Council, Washington, DC, 90 pp.
Mallick, R. B., et al. (2001) *Design Construction and Performance of New-Generation Open-Graded Friction Courses, NCAT Report 00-01.* NCAT. Auburn University. Auburn, AL.
NAPA (2002) *Open-Graded Asphalt Friction Courses: Design, Construction and Maintenance (IS-115),* Lanham, MD, 22 pp.

CHAPTER 12

Field Adjustments and Quality Assurance of HMA Mixtures

Previous chapters of this manual deal with various parts of the HMA mix design process, including laboratory tests, material specifications, and calculations. The goal of these activities is to produce high-quality HMA at the plant—HMA that meets all the requirements of the end user. As discussed below, mix designs developed in the laboratory almost always must be adjusted for field production. Some agencies use the term “mix verification” for this important action. Once a mix design has been adjusted and verified, quality control procedures performed by plant personnel help to ensure that the mix properties stay within established limits, so that the agency will accept all of the production at full price. This chapter describes procedures for adjusting laboratory mix designs for field production and typical quality control procedures for HMA production. The information presented here is only an introduction to HMA quality control; engineers and technicians responsible for quality control should further their knowledge through training classes offered by their local highway agency, the FHWA or by other organizations such as The Asphalt Institute, the National Center for Asphalt Technology (NCAT), and the National Asphalt Paving Association (NAPA).

Adjusting Laboratory HMA Mix Designs for Plant Production

Most of this manual deals with developing HMA designs in the laboratory. Eventually, such mix designs will be used in actual pavement construction and must be produced at a hot mix plant—a much different environment than a laboratory. Engineers and technicians responsible for the design and production of HMA mixtures must understand that HMA designs will almost always have to be adjusted during field production. These adjustments are necessary for several reasons. One of the most common differences between laboratory mix designs and HMA produced in a plant is that the large amount of aggregate handling—transport from stockpiles to the aggregate bins, batching from the bins to the aggregate belt, heating in the drum or hot bin, mixing, and so forth—generates significant amounts of mineral filler. Also, mineral dust content in coarse aggregates can be significant, but is often—but should not be—ignored during mix design, which will also cause an increase in mineral filler content in plant-produced HMA compared to the design aggregate blend. This increase in mineral filler typically results in a decrease in air void content and VMA during field production. These changes can be large enough to negatively affect the performance or result in the HMA composition falling outside of the specification limits.

Aggregate Gradation and Mineral Filler Content in HMA Mix Designs and During Plant Production

The increase in mineral filler from laboratory design to plant production varies considerably, depending on the type of aggregate used, the aggregate gradation, and the type of HMA plant.

Very hard aggregates will tend to produce less dust during handling, and the increase in mineral filler may be less than 0.5%. Softer aggregates may produce an increase in mineral filler of 2% or more. Analysis of a large number of HMA mixes placed at the test track at the National Center for Asphalt Technology showed an average increase in mineral filler during production of 2.2% for 12.5-mm dense-graded HMA, 2.8% for 19-mm dense-graded HMA, and 3.5% for 12.5-mm SMA mixes. In the NCAT test track mixes, it appears that the increase in mineral filler during plant production increases with an increasing proportion of coarse aggregate in the HMA.

Table 12-1 gives typical values for the amount of mineral dust generated during plant production of HMA mixtures. These are only estimates, based on a limited amount of data on HMA field production. However, this table should provide technicians and engineers with some idea of how much aggregate gradations will change during production. In general, softer aggregates and coarser aggregate gradations will result in larger amounts of mineral dust being generated during plant production.

A related problem is the potential use at the plant of mineral fines recycled from the dust collection system. Unfortunately, the effect of fines recycling varies significantly from plant to plant and depends, in part, on the type of collection system used and also upon how the plant uses recycled fines, if at all. If a wet scrubber is used to collect fines, these are normally wasted and not returned to the mix. Therefore, the aggregate gradation in the mix produced in such a plant might have fewer fines than suggested by the aggregate stockpile gradations. If a baghouse is used to collect fines at an HMA plant, they may or may not be returned to the mix. To further complicate matters, the nature of fines collected in any given baghouse will vary depending upon the condition of the filter bags—clean bags are inefficient and allow significant amounts of very small aggregate particles to escape. As the bags become coated with dust and organic material, they will capture a much greater proportion of these very fine particles. Cyclone collectors and similar dry dust collection systems that do not use cloth filters will tend not to collect extremely fine material; material collected by these systems tends to be no finer than about 150 μm.

If a plant does use a significant amount of recycled fines in their mixes, the technician developing mix designs should include a typical amount of this material in laboratory trial mixes. Reasonable efforts should be made to ensure that the recycled fines used in mix design work are representative of what normally goes into the mix at the plant.

The use of ignition ovens to determine asphalt content and aggregate gradation from plant produced mixes can also contribute to differences between mix design and apparent aggregate

Table 12-1. Typical amounts of mineral dust generated during HMA plant production for different aggregates and gradations.

Abrasion Resistance Level	L.A. Abrasion Loss *Wt. %*	Examples	% Retained on 2.36-mm sieve in theoretical aggregate blend: < 25	25 to 35	> 35
Good	< 20%	Dense igneous rocks such as basalt, diabase and gabbro ("trap rock")	2.5	1.5	1.0
Moderate	20 to 35%	Good quality igneous rocks of moderate density such as granite, syenite, diorite; good quality dolomites, limestones and dolomitic limestones; most sandstones, graywackes, slags, and crushed gravels	3.5	2.5	2.0
Poor	> 35%	Soft limestones, sandstones, graywackes and granite	4.5	3.5	3.0

gradation. This can occur when aggregates break down during the excessive heating that occurs in the ignition oven procedure. Aggregates containing significant amounts of limestone are especially prone to this problem. However, it must be emphasized that discrepancies caused by the use of the ignition oven are not real—these differences are not present in the actual mix as produced, but only in the aggregate left over after the ignition oven procedure. Care must therefore be used in interpreting aggregate gradation information gathered using material that has been through the ignition oven.

Asphalt Binder Hardening

Another source of difference between plant and laboratory mixes is the consistency of the asphalt binder. When developing HMA mix designs in the laboratory, asphalt binder is often handled in small containers and kept at high temperatures for varying periods of time. This can lead to significant but highly variable amounts of age hardening in the binder. Asphalt binder tanks, pumps, and piping at a hot mix plant are enclosed systems that tend to minimize age hardening in the binder. On the other hand, mixing in the pugmill or drum and subsequent storage of the mix in a silo can cause significant amounts of age hardening in the mixture prior to transport and placement. Such differences in binder or mixture age hardening can cause differences in air void content and other volumetric properties when comparing laboratory mix designs with plant-produced HMA. Laboratory engineers and technicians should use care in handling asphalt to ensure that unintentional age hardening is minimized during the mix design process.

Differences Between HMA Mixing in the Laboratory and in the Plant

Another source of differences between laboratory mix designs and plant-produced mixtures is the vast difference in the type and size of mixers used. Any HMA mixer will collect asphalt binder and mineral fines on the surfaces of the mixing container and stirrer blades. Because of the small size of laboratory mixers, the surface area of the container and blades is much larger relative to the volume of HMA, compared to plant mixers. This problem can be addressed by "buttering" the laboratory mixer before performing actual mix design work. Buttering is done by mixing a preliminary batch of relatively rich HMA and discarding this material before doing any mix design work. This preliminary mixture should use the same aggregate and asphalt binder used in the HMA mix design. Another problem related to mixers is differences in aggregate breakdown between laboratory- and plant-produced HMA. Large aggregate particles can be fractured in mixers, by being caught between the mixing blades and the container, or simply by the vigorous nature of the mixing process. The degree of breakdown can vary dramatically among different types of laboratory mixers, among different HMA plants, and between laboratory- and plant-produced mix. There is little that can be done either to predict these differences or to minimize them. To maintain the consistency of HMA mix design and verification work, all laboratories within a given organization (or among cooperating organizations) should use identical laboratory mixers with the same mixing blades.

The discussion above should make it clear that there are many different sources of potentially large differences in composition between laboratory mix designs and plant produced HMA. Therefore, engineers and technicians responsible for HMA mix designs and for taking these mix designs from the laboratory to the field should always remember that an HMA mix design is only a starting point for developing the plant job mix formula (JMF). Significant adjustments in the mix design will almost always be needed when producing HMA for the first time based on a laboratory mix design. Such adjustments are not only acceptable, they are usually necessary to producing a good-quality mix.

Minimizing Differences Between HMA Mix Designs and Plant-Produced Mix

Technicians can take several measures when developing an HMA mix design to ensure that the design can be easily adjusted for plant production. The number of aggregates used in an HMA design should be kept as low as possible, while still meeting all pertinent requirements. Also, a reasonable effort should be made to use aggregate gradations in the laboratory that are representative of average stockpile values. As discussed in Chapter 4, this is normally done by breaking down aggregates into fractions and recombining them to create blends closely matching average gradation values provided by the HMA plant. The design VMA for a laboratory mix design should fall near the middle of the allowable range. For example, if the specified range for VMA for the given mix type is 14.0 to 16.0%, the laboratory design should have a VMA value close to 15.0%. This allows some room for field adjustments without producing a mixture that is out of specification. For the same reason, the dust/binder ratio should be kept away from the allowable minimum and maximum values. As mentioned above, care should be taken in handling asphalt binder in the laboratory to minimize age hardening. Some technicians may wish to adjust the laboratory mix design to account for additional mineral filler generated during plant processing of the aggregate. The amount of fines generated during plant production is best estimated by plant personnel familiar with the materials being used in the mix design. If this information is not available from the plant, the information given in Table 12-1 (presented earlier) can be used to estimate the extra mineral dust generated during plant production of HMA mixes. To use the information in Table 12-1 to make adjustments in the aggregate gradation, the percent passing all sieves smaller than the NMAS, including the 75μm sieve, should be increased by the same amount as the anticipated difference in mineral filler. This approach assumes all of the extra mineral dust is produced from aggregate abrasion between the NMAS and the next smaller sieve size. Although not entirely accurate, this approach is a good first estimate of what happens during HMA field production.

Mix Composition Adjustments During Plant Production

The discussion above focused on (1) developing mix designs that, as much as possible, reflect actual production at the plant and (2) designing mixes that will be relatively easy to adjust at the plant during production. As emphasized many times before in this manual, laboratory designs for HMA mixtures are only starting points for the actual plant mix design used during construction. The plant operator will need to adjust the laboratory mix design to provide an HMA mix that is workable and meets all requirements of the pertinent specification. Although this manual is intended for engineers and technicians responsible for laboratory mix designs and related activities, it is important that these professionals understand the adjustments usually made when taking a laboratory mix design into field production.

By far the most common reason for adjusting a mix design during initial plant production is to adjust the air void content and VMA obtained during testing of laboratory-compacted specimens. Air void content and VMA are directly related at a given asphalt content, so if the air void content decreases, VMA will also decrease unless the asphalt content is changed. As discussed above, most often, an increase in the amount of mineral filler during plant production will cause a decrease in air void content and VMA. Many plant operators will address this problem by decreasing asphalt content slightly, which will tend to increase air void content. However, the end result of this adjustment will be a decrease in VMA. Furthermore, decreasing asphalt binder content below the laboratory design value can decrease pavement performance. It is therefore suggested that low air void content and VMA be corrected during field production by adjusting aggregate proportions. For fine/dense-graded HMA, low air void contents can be increased by increasing the proportion of fine aggregate. For coarse/dense-graded HMA, air void content can be increased by increasing the proportion of coarse aggregate. For mixtures near the maximum density gradation, air void content can be increased by increasing the proportion of fine or coarse aggregate. The rules given above are general and, in practice, there are many exceptions; plant operators should use their experience in adjusting aggregate proportions during field production to obtain the target air void content and VMA. The general mix design procedure described in Chapter 8 of this manual should also provide information on how aggregate gradation changes will affect air void content and VMA, which can be used as a guide when making adjustments during field production. It should be emphasized that, if a reasonable attempt is made in the laboratory design to include the proper amount of mineral filler, this will minimize the amount of adjustment needed during field production.

Quality Control of HMA

AASHTO Standard Practice R 10, Definition of Terms Related to Quality and Statistics As Used in Highway Construction, defines quality assurance (QA) as "(1) all those planned and systematic actions necessary to provide confidence that a product or facility will perform satisfactorily in service; or (2) making sure the quality of a product is what it should be." For HMA production and pavement construction, the overall quality assurance process generally includes (1) quality control, (2) acceptance, and (3) independent assurance, which are defined in AASHTO R 10 as follows:

- Quality Control—The system used by a contractor to monitor, assess, and adjust their production or placement processes to ensure that the final product will meet the specified level of quality.
- Acceptance—The process whereby all factors used by the agency (i.e., sampling, testing, and inspection) are evaluated to determine the degree of compliance with contract requirements and to determine the corresponding value for a given product.
- Independent Assurance—Activities that are an unbiased and independent evaluation of all the sampling and testing (or inspection) procedures used in the quality assurance program.

Obviously, quality control is essential to any business, including HMA production. Quality control is needed to ensure that the HMA produced at a given plant meets all of the specification requirements of the customer. An effective quality control program will also help lower costs and increase profits, because it will result in more efficient production.

Many state highway agencies now use quality acceptance specifications when selecting and monitoring HMA producers and contractors for pavement construction. In a quality acceptance specification, the HMA producer is responsible for controlling HMA quality, while the highway agency is responsible for material acceptance. In HMA production, quality control refers to all actions and analyses performed by the plant to ensure that the material it produces is of good quality—practically speaking, that it meets all specifications and reasonable user expectations. Material acceptance is the procedure used by the state agency to determine if the HMA is acceptable, and if so, how much the plant will be paid for the material. The precise sampling and testing procedures used in product acceptance and the calculations to be followed in deciding on acceptance or rejection and payment factors are outlined in the agency's acceptance plan. Within the past 20 years, more and more states have implemented statistically based acceptance plans. These plans typically require random sampling of HMA, testing of multiple samples from a selected amount of HMA (often called a *lot*), and use of statistical calculations to determine product acceptance or rejection and payment amounts. It is critical to understand that these two operations—quality control and product acceptance—are completely separate and must not be confused. Although a state highway agency can and should review an HMA plant's quality control records to verify that it is in fact following a good plan, quality control test results should not be used for acceptance. Ideally, the effectiveness of quality control operations at an HMA plant and the accuracy of product acceptance decisions made by a state agency should be regularly evaluated by an independent organization. This third part of a quality acceptance specification is independent assurance, as defined above.

An important concept in both quality control and acceptance plans is that of quality characteristics. A quality characteristic is simply something about a product that is measured as an indication of its quality. Both HMA quality control and acceptance revolve around various quality characteristics: asphalt content, mineral filler content, aggregate gradation, in-place air void content, and so forth. Tests used to control the quality of HMA, and especially those used by agencies in making acceptance decisions, should be those most clearly related to pavement performance, but must also be fairly quick and easy to accurately measure.

Both quality control and acceptance activities require an understanding of basic statistics—calculation of average, standard deviation, mean, and other quantities. Quality control technicians must also be able to properly perform material sampling and testing and to then use such data to construct and use quality control charts to help keep a plant running smoothly and producing material that is within specifications. This has made the work of HMA technicians more difficult, but also more important to the success of plant operations. The sections below present an overview of the essential parts of quality control and acceptance plans. These discussions must be general, since the requirements for HMA quality control and acceptance vary significantly from state to state. HMA plant technicians should make sure they are familiar with specifications in their state and should take advantage of any training programs offered by their state highway agency or asphalt paving association on HMA quality assurance and related topics.

Variability, Mean and Standard Deviation

When engineers and technicians discuss a set of test results, the two most important characteristics of the data are the central tendency and variability. All real data sets exhibit variability, that is, the test results are not all exactly the same but vary within a certain range. The overall value of a data set is usually measured by calculating the mean value, often called the average. The mean is calculated by adding all the numbers in a data set and dividing by the total number of measurements. Variability in a set of numbers is usually measured by calculating the standard deviation:

$$s = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \overline{x})^2}{n-1}} \qquad (12\text{-}1)$$

Where

s is the standard deviation
x_i is the value for the ith test
$\overline{x}$ is the mean value for all the tests
n is the total number of test results.

Equation 12-1 is used for data sets with less than about 30 results; for larger data sets, n is used in place of $(n-1)$ in calculating standard deviation. Although Equation 12-1 may look complicated, most of the time standard deviation calculations are done using a calculator or spreadsheet. Calculators and spreadsheets will usually give the operator the choice of using n or $(n-1)$ in the standard deviation calculation; engineers and technicians need to make sure that $(n-1)$ is used when doing calculations for small data sets.

Variability in HMA Production, Sampling, and Testing

In designing and implementing a quality control program for HMA, it is critical to understand that asphalt concrete is, by nature, a highly variable material. Furthermore, the procedures we use to test aggregate, asphalt binder, and HMA are also variable. Even the procedures we use to sample materials at the plant and on the roadway cause variability in test data. Engineers and technicians must always consider these three sources of variability: materials, sampling, and testing.

In a quality control program, the variability we should be most concerned with is the inherent variability in the material, for example, the range in the asphalt content for a particular HMA mix during a specific project. However, a substantial percentage of the variability in test data on pavement construction projects will be from test variability. Table 12-2 summarizes precision statements for different tests commonly used in HMA quality control plans. Precision statements are calculated using large data sets gathered from different laboratories on identical or nearly identical sets of specimens. They are usually given in terms of both standard deviation and the d2s precision. “d2s” stands for “difference, two standard deviations.” In practical terms, the d2s precision represents the maximum expected difference between two test results on the same material. Precision statements also provide information on single-operator precision and multi-laboratory precision. Single-operator precision represents the variability expected for a given test when performed by the same technician in the same laboratory over and over again. The single-operator precision is useful in evaluating the quality of data being produced by a given technician. Laboratory managers can occasionally give technicians blind samples of the same materials, to determine if the test results are within the single-operator d2s limit. If not, it is likely the technician is performing the test improperly. The multilaboratory precision is probably more important to engineers and technicians involved in the paving industry. Multilaboratory precision represents the variability expected for a given test when it is performed repeatedly on the same material by a large number of different laboratories. The multilaboratory d2s is very useful when comparing test results produced by two different laboratories. For example, based on the data in Table 12-2,

Table 12-2. Single operator and multilaboratory precision for test results commonly used in HMA quality control plans.

Test Procedure	Single Operator		Multilaboratory	
	Std. Dev.	d2s	Std. Dev.	d2s
Aggregate gradation, percent passing				
Coarse aggregate (CA)*	0.27 to 2.25	0.8 to 6.4	0.35 to 2.82	1.0 to 8.0
Fine aggregate (FA)*	0.14 to 0.83	0.4 o 2.4	0.23 to 1.41	0.6 to 4.0
Mineral Filler (in CA/ in FA)	0.10/0.15	0.28/0.43	0.22/0.29	0.62/0.82
Asphalt content, weight %				
Ignition oven	0.04	0.11	0.06	0.17
Quantitative extraction**	0.19 to 0.30	0.54 to 0.85	0.29 to 0.37	0.82 to 1.05
Maximum theoretical specific gravity	0.0040	0.011	0.0064	0.019
Bulk specific gravity, SSD	0.0124	0.035	0.0269	0.076
Bulk specific gravity, Paraffin-coated	0.028	0.079	0.034	0.095
Air void content, Vol. %***	0.5	1.5	1.1	3.0
Effective asphalt content, Vol. %***	0.3	0.9	0.6	1.6
Voids in mineral aggregate, Vol. %***	0.5	1.5	1.1	3.1
Voids filled with asphalt, Vol. %***	2.2	6.2	4.5	12.8
Dust/asphalt ratio, by weight***	0.05	0.13	0.09	0.25

* Lower values are for very high and/or very low percent passing; higher values are for percent passing values close to 50%.

** Value depends on method used.

*** Typical values, estimated from data on aggregate gradation, aggregate and mixture specific gravity and asphalt content using ignition oven. Values estimated using standard deviations for quantitative extraction vary slightly from these values.

Table 12-3. Effect of sample size on test precision.

Sample Size, n	$\frac{1}{\sqrt{n}}$	Relative Precision
1	1.000	1.0 ×
2	0.707	1.4 ×
3	0.577	1.7 ×
4	0.500	2.0 ×
5	0.447	2.2 ×
6	0.408	2.4 ×
7	0.378	2.6 ×
8	0.354	2.8 ×
9	0.333	3.0 ×
16	0.250	4.0 ×
25	0.224	5.0 ×

air void content data on the same material from two different laboratories should not be considered suspect unless they differ by more than 3 to 4% for a single specimen.

The relatively large values for multilaboratory precision for some of the tests in Table 12-2 may surprise and concern some engineers and technicians. Although test methods with better precision would improve quality control and acceptance procedures for asphalt concrete pavements, until such methods are developed, the only way to improve the precision of these and other test methods is to increase the number of tests performed on a given material. The number of tests performed on a given HMA lot or section of pavement is usually called the sample size n; the precision of any test performed on such a pavement increases with the square root of the sample size, $\sqrt{n}$. This is why many quality control and acceptance plans for HMA construction divide a lot into subsections called *sublots* and require testing of one sample from each sublot. Such an approach improves the precision of the measurements and makes the quality control and acceptance plan more reliable. Table 12-3 gives values of $1/\sqrt{n}$ for different sample sizes n, along with relative precision—a higher value of relative precision indicates an improved level of precision. Increasing the sample size from 1 to 4 doubles the precision of a measurement; increasing the sample size to 9 triples the precision. To increase the precision by a factor of 4, 16 replicate measurements are needed. To estimate the d2s precision when replicate measurements are made, or when more than one sample is taken from a given lot of material, the d2s precision from Table 12-2 (or from some other source) should be divided by the square root of the sample size, $\sqrt{n}$, which is given as the "relative precision" in Table 12-3.

As an example of how precision statements and sample size can be used in paving technology, consider a situation where a highway agency and contractor disagree on air void values. The agency's acceptance tests show an average air void content of 10.7% for a day's production of pavement, using an average of $n = 4$ measurements. The contractor's tests on companion samples gave an average of only 8.5%, using the same sample size. A penalty is applied if the air void content for a given lot exceeds 10%. The contractor believes the agency's tests are in error. Is the difference between these measurements surprising? Is further investigation needed to determine whether or not the air void content for this day's production is out of specification?

According to Table 12-2, the d2s precision for air void content is 3 to 4%. For a sample size of $n = 4$, the relative precision would be cut in half, reducing the d2s precision to 1.5 to 2.0%. The difference in the measurements is $10.7 - 8.5 = 2.2\%$, which is greater than the calculated d2s precision. Therefore, the difference in these test values is too large and should be investigated.

Control Charts

Control charts are one of the most important parts of a good quality control program. Control charts are simply a way of using a graph to show how important test results are changing over time at a given HMA plant. An experienced technician or engineer can look at a set of control charts and determine if a plant is operating smoothly and producing acceptable material or if there is a production problem that requires investigation and perhaps adjustment. Control charts are not just used at HMA plants, but form an essential part of quality control plans in most manufacturing industries.

Several types of control charts are commonly used in HMA production. One of the simplest is made by plotting test results as a function of time. Such a control chart is shown in Figure 12-1, which shows asphalt content as a function of date and time. In this case, the target asphalt content is 4.8%. The specification requires that individual asphalt content measurements fall within ±0.7% of the target. To make the control chart more useful, both the target and the upper and lower specification limits are shown on this control chart. Note that on or about June 24, many of the

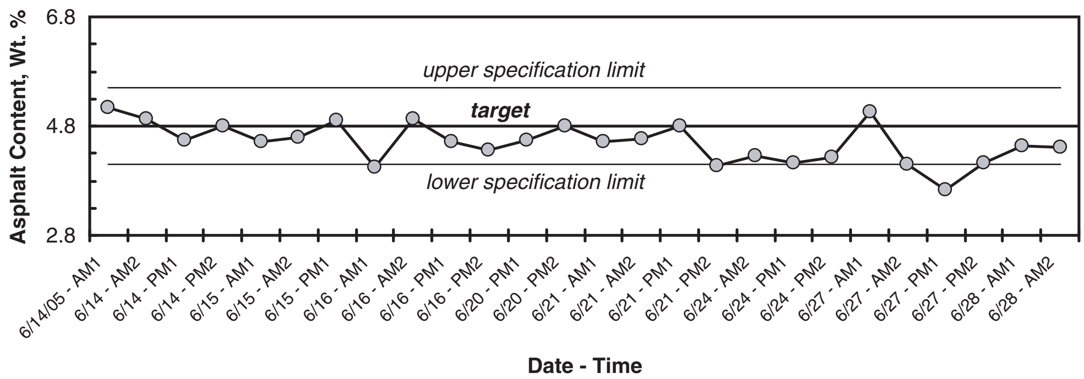

Figure 12-1. Simple control chart showing single measurements of asphalt content.

asphalt contents are near the specification minimum, and on the morning of June 27, one of the measurements is below the specification minimum.

Figure 12-1 is useful, but has several drawbacks—it only shows single measurements, and the high level of variation among single test results makes overall trends hard to see. Figure 12-2 is a control chart of the same data, but here running averages are plotted instead of single measurements. The specification requirement shown in this plot is that the average asphalt content for the lot must be within ±0.4% of the target. Because the specification in this case is based on the average of $n = 5$ samples, the running average was calculated using the five latest asphalt content tests. Because this method reduces much of the test variability seen in Figure 12-1, Figure 12-2 is much smoother. It is clear from Figure 12-2 that the average asphalt content is slowly decreasing. Figure 12-2 is also useful because it gives the technician an idea of whether or not lot averages will meet the specification requirement.

In actual practice, lots in acceptance plans for pavement construction are most often based on 1 day's production. For this reason, Figure 12-2 gives only an approximate indication of whether or not the lot average asphalt content will meet specification requirements. A better indication of whether or not the requirement for lot average asphalt content is being met can be gained by plotting average asphalt content as a function of the date of production, as shown in Figure 12-3. This plot shows the steady decrease in asphalt content seen in Figure 12-2, but also suggests that production for the last 2 days is out of specification. Although Figure 12-3 will better reflect the results of acceptance based on daily production, it should be remembered that such control charts are based on the plants quality control data—acceptance test results may be different, because of

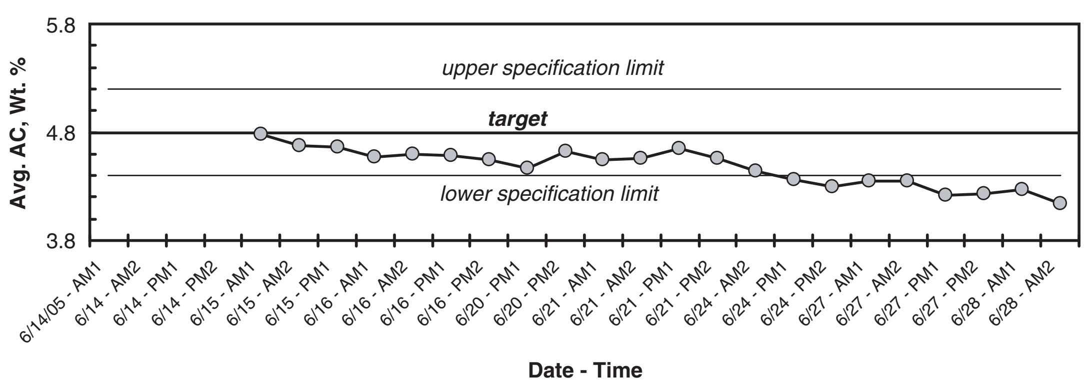

Figure 12-2. Control chart for asphalt content using running average of five measurements.

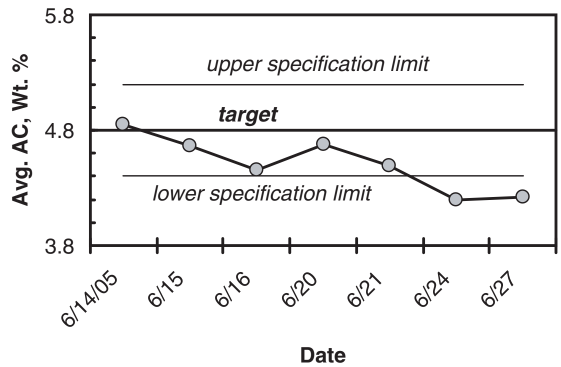

Figure 12-3. Control chart for average asphalt content, as a function of production day.

differences in location of samples and sampling methods and lab-to-lab variability in test results. Also, using daily averages to construct control charts will mean that the charts cannot be constructed until test data for each day are complete and have been tabulated. This will delay creation of the control chart and identification and correction of any problems that occur.

Plotting the specification limits on control charts can be useful, but HMA production can usually be better controlled by using statistical control limits. Sometimes these limits are based on three times the average standard deviation. Because the Greek symbol σ ("sigma") is used to represent the actual standard deviation of a large group, these kinds of control limits are sometimes called three sigma (3σ) limits. These limits are used because the chances of a given test result falling outside these boundaries are very small—about one in one thousand—so a test result near or outside of these control limits should be investigated immediately. A similar chart often used in quality control for HMA plants uses the average range ($\bar{R}$ or R-bar) instead of the standard deviation to calculate control limits; the target value is the overall average, rather than the center of the specification limits. Figure 12-4 is an example of such a chart, constructed for the daily average percent passing the 0.075-mm sieve. In this case, the overall average range for percent passing the 0.075-mm sieve for this plant was estimated to be 1.15%, and the overall average was 4.2%. The control limits are then calculated using the following formulas:

$$UCL = \bar{X} + \left(A_2 \times \bar{R}\right) \tag{12-2}$$

$$LCL = \bar{X} - \left(A_2 \times \bar{R}\right) \tag{12-3}$$

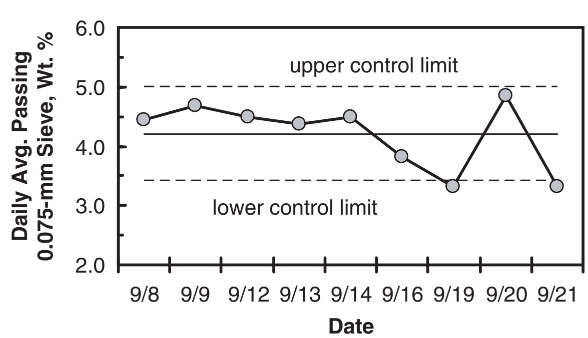

Figure 12-4. Statistical control chart for average percent passing the no. 200 sieve.

Table 12-4. Factors for computing control limits for control charts.

Sample Size n	A_2	D_3	D_4
2	1.88	0.00	3.27
3	1.02	0.00	2.58
4	0.73	0.00	2.28
5	0.58	0.00	2.12
6	0.48	0.00	2.00
7	0.42	0.08	1.92

where

UCL = upper control limit
LCL = lower control limit
$\overline{X}$ = overall average or X-bar = 4.2% for Figure 12-4
$\overline{R}$ = overall range or R-bar = 1.35% for Figure 12-4
A_2 = a factor that depends on sample size n (given in Table 12-4); in this example $n = 5$ and $A_2 = 0.58$

The average mineral filler contents in Figure 12-4 are within the control limits and close to the target until September 19. At this point, the values become much more variable, going below the lower control limit on September 19 and 21, and near the upper control limit on September 20. Clearly, there is a serious problem with variability in the mineral filler at this plant starting on September 19, and the situation should be investigated to determine the source of the problem. Because specification limits for average percent passing the 0.075-mm sieve are often ±2.0%, the data shown in Figure 12-4 would probably be within the specification and would not represent a payment penalty. If the specification limits of 2.2 to 6.2% were used as the limits in Figure 12-4, it might not be clear that there is a problem in the plant operation until some data exceed the specification limits, incurring a payment penalty. The advantage of using statistical control limits is that potential problems in production can often be identified and corrected before material exceeds specification limits.

An important question in the construction of statistical control charts is what values to use for the overall average and range. In many cases, a well-run plant will continuously collect and analyze information on variability in asphalt content, aggregate gradation, laboratory air void content, and other important aspects of HMA production. The plant operator or technician in charge of quality control should then have values for overall average and range. If not, Equation 12-1 can be used to calculate overall standard deviation for a given property for a given mix type from the available data. However, for reasonably accurate estimates of overall average and range, data from at least 30 production days are needed. Production data can be used once data from about 20 days are available, although the results will not be completely reliable.

When calculating data for use in control charts, it should be emphasized that production variability will change over time for several reasons: employee turnover, equipment wear, changes in plant layout, improvements in employee training, and so forth. Therefore, overall average and range values should be recalculated regularly, using the last 30 to 50 data points available for a given test. Typical overall standard deviations for different HMA test properties are listed in Tables 12-5 and 12-6; these values have been gathered and reported in research studies listed at the end of this chapter. Expected values for $\overline{R}$ (overall range) can be estimated from values given in this table by dividing the standard deviation by the factor A_2 found in Table 12-4; remember

Table 12-5. Typical overall standard deviation values for aggregate gradation.

Sieve Size	Typical Range for Overall Standard Deviation
19 mm	1.5 to 4.5%
12.5 mm	2.5 to 5.0%
9.5 mm	2.5 to 5.0%
4.75 mm	2.5 to 5.0%
2.36 mm	2.5 to 4.0%
1.18 mm	2.5 to 4.0%
0.60 mm	2.0 to 3.5%
0.30 mm	1.0 to 2.0%
0.15 mm	1.0 to 2.0%
0.075 mm	0.6 to 1.0%

Table 12-6. Typical overall standard deviation values for asphalt content, air void content, VMA and VFA.

Property	Typical Range of Value for Overall Standard Deviation
Asphalt content	0.15 to 0.30%
Air void content, from field cores	1.3 to 1.5%
Laboratory air void content	0.9%
VMA	0.9%
VFA	4.0%

that the value of A_2 and the resulting estimate of $\overline{R}$ will depend on the sample size n—the larger the sample size, the larger the value of $\overline{R}$ for a given standard deviation.

All of the control charts shown above are plots of measured values or of averages of measured values. In another type of control chart, values of the range (as determined in each lot's or day's production) are plotted as a function of time of sampling. In Figure 12-5, the same data used in Figure 12-4 were used to calculate range values for each production day. The control limit in this case is calculated by multiplying $\overline{R}$ (1.35%) by a factor that depends on sample size:

$$LCL = D_3 \times \overline{R} = 0.0 \times 1.35 = 0.0 \tag{12-4}$$

$$UCL = D_4 \times \overline{R} = 2.12 \times 1.35 = 2.86 \tag{12-5}$$

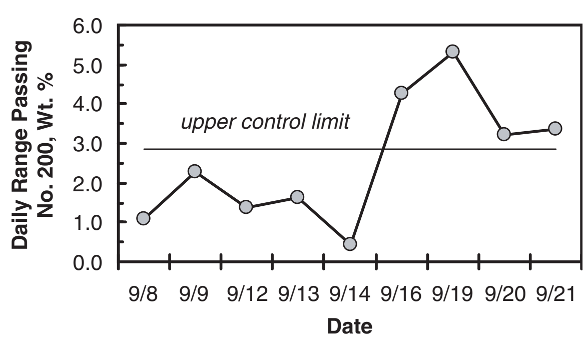

Figure 12-5. Range control chart for percent passing the no. 200 sieve, plotted by date.

where

LCL = lower control limit = 0.0 for Figure 12-5
UCL = upper control limit = 2.86% for Figure 12-5
D_3, D_4 = factors for computing control limits for standard deviation control charts; see Table 12-4
$\overline{R}$ = overall range = 1.35% for Figure 12-5

Figure 12-5 makes it clear what the problem is with the mineral filler data—the variability suddenly increases starting on September 16. After this date, most of the standard deviations are above the upper control limit, a situation that must be investigated and corrected. Note that there is no lower control limit in Figure 12-5. This is because the factor D_3 for calculating the LCL (see Table 12-4) is zero for a sample size of 6 or less and need not be plotted. Even for $n = 7$, the value of D_3 is 0.08, which will still result in very low values for the LCL relative to the UCL.

Control charts are an important part of quality control at HMA plants. They are useful tools for adjusting plant production to help ensure that specifications are met, work stoppages minimized, and payment penalties avoided. The number and types of charts used at a given HMA plant will depend on the specifications that the plant must usually meet and on the preferences of the technicians and engineers responsible for running the plant. As a minimum, it is suggested that control charts should be kept for asphalt content, percent passing the No. 200 sieve, percent passing the No. 8 sieve, air void content of laboratory-compacted specimens, and VMA of laboratory-compacted specimens. Technicians responsible for plant quality control may also be required to keep control charts for in-place air void content and pavement thickness. For each set of test data, it is suggested that plots be made for individual measurements, running averages, running standard deviation, daily average, and daily standard deviation. The number used for calculating running averages and standard deviations should be equal to the number of sublots generally used in the local agency's acceptance plan—usually $n = 4$ or $n = 5$.

There are many rules for interpreting statistical control charts. A few simple rules for when to investigate a potential problem will help technicians and engineers use these tools effectively:

- One or more points outside UCL or LCL
- Seven or more points on one side of target for $\overline{x}$ charts
- A gradual increase or decrease in the value of $\overline{x}$
- A gradual increase in standard deviation
- A sudden shift in $\overline{x}$ or standard deviation
- Daily or weekly variations in $\overline{x}$

When one of these potential problems is seen in a control chart, the following procedure is suggested for investigating the reason for the unusual data:

1. Check to make sure that the data were correctly recorded.
2. Check the calculation of the test data.
3. Interview the technician or engineer responsible for taking the sample, to determine if there was anything unusual about the sample.
4. Interview the technician or engineer responsible for performing the test, to determine if anything unusual occurred when the test was performed.
5. If the data appear to be reliable—if there were no errors in sampling, testing, calculating, and recording the data—investigate those parts of the HMA plant or production process that could affect the test result. This might include valves, meters, scales, screens, aggregate stockpiles, and so forth.

6. If a reason for the unusual data is found, it is called an "assignable cause." The problem should be corrected as soon as possible.
7. If no problems are found in the plant or process, the unusual test data are determined to be the result of "chance cause," meaning that it was likely the result of normal variability in materials, sampling, or testing.

Technicians and engineers responsible for controlling an HMA plant should be careful not to over-control. Over-control occurs when adjustments are made in the HMA plant that are not needed. This might sound harmless, but over-control is a serious problem that can lead to HMA being out of specification, resulting in work delays and penalties. Consider the following example. An inexperienced plant operator who does not fully understand control charts sees that a few values for aggregate passing the No. 8 sieve are a few percent above the target, so the operator decreases the fine aggregate in the aggregate blend by 2%. The next day, the operator notes that the fine aggregate is now running about 3% low, so the operator increases the fine aggregate by the same amount. By the end of the day, the operator notes that the fine aggregate is 5% high, and the operator now decreases the setting by 4%. The next morning, the new plant operator realizes that the plant has produced nearly a full day's production that is out of specification because of highly variable aggregate gradation, including the mineral filler content. In reality, the variability that the operator first saw was normal, and no adjustment in the process was needed. When the operator made the first "tweak" in the fine aggregate, this operator was only making the situation worse, by adding an additional source of variability to the process. Each additional adjustment only made the situation worse. Adjustments in the plant process should not be made unless truly unusual data are observed and, as discussed above, an assignable cause is found.

A second important aspect of quality assurance involves continuous quality improvement. As discussed earlier in this chapter, a certain amount of variability in quality control data is normal—it is the result of variation in sampling, test procedures, and HMA plant operations. However, this does not mean that plant personnel need to be satisfied with a given level of variability. Continuous quality improvement is the process of identifying and reducing or eliminating sources of variability. This might mean training laboratory technicians to reduce variability in sampling and testing. Purchasing new or improved test equipment can also decrease testing variability. High variability in aggregate gradation might be caused by improper stockpiling—proper training of plant personnel responsible for aggregate handling will result in more uniform stockpiles and reduced variability in aggregate gradation. If mechanical parts in the plant are wearing out regularly, causing spikes in production variability when they fail, they can be replaced regularly before they fail. Although a good quality control program might seem an unnecessary cost, it is now widely accepted in many industries that proper quality control will save more money than it costs.

Acceptance Testing During HMA Production

As mentioned above, acceptance decisions in HMA production are the responsibility of the highway agency. In order to keep costs low and to ensure that agency decisions can be made quickly, acceptance testing is usually more limited than quality control testing performed by contractors. Tests included in acceptance plans might include asphalt content, percent of aggregate passing the 0.075-mm sieve, percent of aggregate passing the 2.36-mm sieve (or some other intermediate sieve size), in-place air void content, pavement smoothness, and pavement thickness. Pavement smoothness is usually measured using a profilometer or similar device. Sampling for in-place air void content and pavement thickness must be done from the finished pavement, either by sawing slabs or removing cores from the pavement. HMA samples for asphalt content and aggregate gradation can be cores taken from the finished pavement, loose mix taken from behind the paver, or loose mix taken from the truck at the plant site. It is essential that the sampling plan clearly describe the location of the sample, the size of the sample, and the procedure to be used in taking the sample. Samples from the actual pavement—either cores or loose mix taken from behind the paver—are preferred for acceptance tests because they more closely represent the final product than do samples taken from the truck or other locations within the plant. HMA plant technicians and engineers should study their state agency's acceptance plans, so that they can understand where problems might occur that might affect test results and acceptance decisions. For example, inexperienced inspectors might use poor techniques in sampling HMA, leading to highly variable test results and rejection of good-quality HMA.

Acceptance sampling and testing may be done by employees of the highway agency or by an independent engineering firm or testing laboratory. Sometimes, because of reductions in state government staffing or budgets, contractors may be required to perform acceptance sampling and testing. As a general principle, such acceptance tests should be kept separate from quality control tests. Acceptance testing, regardless of who is responsible for performing it, should be verified regularly by an independent third party, through inspections, interviews, and comparison of test data on samples taken from the same pavement section.

Sampling Materials for Quality Control and Acceptance Testing

It is critical in both quality control and acceptance testing that the samples tested are representative of what is being produced or of the materials used in production. AASHTO publishes three standard methods for sampling aggregate, asphalt binders, and HMA:

- T 2, Sampling of Aggregates
- T 40, Sampling Bituminous Materials
- T 168, Sampling Bituminous Paving Mixtures

Engineers and technicians responsible for collecting samples for quality control and acceptance testing should follow these procedures or appropriate procedures provided by their state highway agency. Often large samples of material taken during production must be reduced in size for testing purposes. Again, appropriate procedures for sample splitting, as published by AASHTO or the local state highway agency, should be followed. This is especially important for aggregates, where segregation of samples during handling and transport is a common problem. AASHTO T 248, Reducing Samples of Aggregate to Testing Size, describes appropriate procedures for reducing large samples of aggregate for testing purposes. Although there is no AASHTO standard for reducing large samples of binder to testing size, careless handling of binder in the laboratory can also cause problems during mix design, quality control testing, and acceptance testing. The length of time and number of times asphalt binder is heated in the laboratory should be kept to a minimum. This means that if a large sample of binder, such as a quart or gallon, is to be divided for testing and mix design, the sample should be heated just until fluid and then poured into a range of smaller containers. These smaller containers are then used for testing and mix design. Large containers of binder should not be kept in an oven for an extended period of time and should not be repeatedly reheated for testing and mix design. Such extended or repeated heating will harden the binder, yielding erroneous binder test results and potential inconsistencies in HMA mix designs. As with asphalt binder (and for similar reasons), HMA mixture samples should not be heated repeatedly or for extended periods of time. Also, when HMA samples are split for testing, care should be taken to make sure that asphalt binder and fines are not lost on containers and laboratory tools. Some HMA mixtures are prone to segregation, and special care is needed in handling such mixtures.

Many apparent problems in quality control and acceptance testing during HMA production are the result of improper sampling practices or segregation during sample handling. Following the appropriate AASHTO procedures, or those provided by the local highway agency, will minimize such problems. When specific procedures for sampling and sample size reduction are not available, reasonable care and common sense will go far to eliminate potential problems.

Equally important to proper sampling technique is developing and following an appropriate sampling plan. A sampling plan describes when and where to take samples for testing and also what type of sample should be taken and how large the sample should be. Samples should be taken randomly, meaning that the exact time or location should vary in unpredictable ways. This prevents production personnel from anticipating when and where samples will be taken and changing their practices to make certain that the results of quality control or acceptance testing are favorable. Random number tables are often used to generate the times and locations of sampling in HMA quality control and acceptance sampling plans.

Many HMA acceptance plans are stratified random sampling plans. Such plans divide an amount of material to be tested into lots and sublots. Samples are taken from each sublot within a lot and tested, and the results are used to calculate average values, standard deviation, and other statistics for the entire lot. These statistics are then used by the agency to decide whether the lot should be accepted or rejected or if the lot should be accepted at reduced payment. As an example, consider the main features of a typical stratified random sampling plan for HMA acceptance:

- HMA samples weighing between 2 and 3 kg will be taken behind the paver, before compaction. Samples will be stored in clean, unlined, tightly sealed metal cans.
- For purposes of sampling, a lot is normally defined as 1,000 tonnes of hot mix. Each such lot will be divided into five equal sublots of 200 tonnes each.
- One HMA sample will be taken from each sublot. The location of this sample, both along and across the roadway, will be determined using a set of two random numbers, one representing the location of the sample along the roadway, and the other representing the location across the pavement.
- Samples should not be taken within 2 feet of obstructions to paving.

The resulting sampling of a typical lot might look something like that sketched in Figure 12-6. The coordinates (x, y) for each sample are determined from a random number table.

The example given above is greatly simplified; real specifications must provide detailed directions for a wide range of situations, so that there is no confusion about when and where to take samples for acceptance testing. For example, poor weather may result in lots smaller than the normal 1,000 tonnes described above; an effective specification must explain clearly how to deal with such situations. Technicians and engineers responsible for sampling should make sure they are familiar with the specification that describes the sampling plan that they should be following and

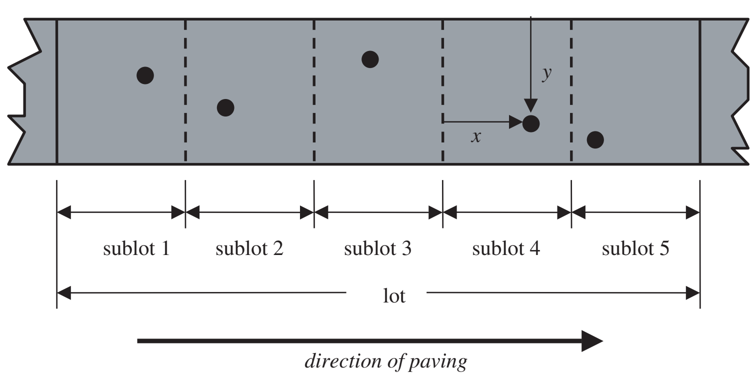

Figure 12-6. Example of a stratified random sampling plan for HMA.

should also understand the importance of obtaining representative samples for both quality control and acceptance testing. AASHTO provides detailed guidance for developing acceptance plans in R 9, Acceptance Sampling Plans for Highway Construction.

Quality Control Plans

AASHTO R 10, Definition of Terms for Specification and Procedures, gives the following definition of a quality control plan:

> A detailed description of the type and frequency of inspection, sampling, and testing deemed necessary to measure and control the various properties governed by agency specifications. This document is submitted to the agency for approval by the contractor during the preconstruction conference.

Although quality control plans are meant to be documents used by the HMA supplier or contractor to help ensure that their product meets the customer's needs and will thus be accepted at full payment, many highway agencies will require the supplier to submit a quality control plan for approval. This gives the highway agency additional confidence that the HMA supplier will be able to meet the required specification. The highway agency may list the items to be included in the quality control plan. A typical quality control plan for an HMA supplier might include the following items, as a minimum:

1. Quality Control Organization Chart
   1.1. Names of personnel responsible for quality control
   1.2. Area of responsibility of each individual
   1.3. List of outside agencies, such as testing laboratories, involved in quality control and a description of services provided by each
2. Testing Plan with Action Points
   2.1. List of all tests to be performed
   2.2. Frequency of testing
   2.3. List of action points for initiating corrective procedures
   2.4. Recording method to be used to document corrective procedures
3. Materials Storage and Handling
   3.1. Aggregate and RAP stockpiles
   3.2. Cold-feed systems for aggregates and/or RAP
   3.3. Additives or modifiers
   3.4. Asphalt binder and other liquid storage tanks
   3.5. Surge and storage silos for HMA
   3.6. All measuring and conveying devices, including calibration procedures for each
   3.7. Description of procedure for loading haul vehicles

Engineers and technicians responsible for developing quality control plans for HMA production should follow guidelines provided by their local highway agencies. They should also keep in mind that, to be effective, a quality control plan should be as simple as possible, while addressing all important items and activities that affect the quality of HMA produced at the plant.

Bibliography

AASHTO Standards

R 9, Acceptance Sampling Plans for Highway Construction
R 10, Definition of Terms for Specification and Procedures
T 2, Sampling of Aggregates
T 40, Sampling Bituminous Materials
T 168, Sampling Bituminous Paving Mixtures

Other Standards

ASTM D 3665, Random Sampling of Construction Materials

"Section 409—Superpave Mixture Design, Standard and RPS Construction of Plant-Mixed HMA Courses" in *Publication 408,* Harrisburg, PA: The Pennsylvania Department of Transportation, 2007.

Other Publications

Burati, J. L., Jr., and C. S. Hughes (1990) *Highway Materials Engineering, Module I: Materials Control and Acceptance—Quality Assurance,* National Highway Institute Course No. 13123, February.

Hot-Mix Asphalt Paving Handbook. (1991) AC150/5370-14, Appendix I; UN-13 (CEMP-ET), J. Sherocman, Consultant, AASHTO, FAA, FHWA, NAPA, USACE, American Public Works Association, National Association of County Engineers, July 31, 218 pp.

NCAT (1996) *Hot-Mix Asphalt Materials, Mixture Design and Construction,* NAPA, Lanham, MD, 585 pp.

Commentary to the Mix Design Manual for Hot Mix Asphalt

This Commentary, a companion to the Manual compiled during NCHRP Project 9-33, includes large amounts of supporting technical information and detailed references that, although important, would distract the user's attention from the essential information in the Manual. The information presented herein will also help engineers and researchers in the future to edit, update, and refine the Manual.

The Commentary is organized in chapters, with each chapter in the Commentary corresponding exactly with a chapter in the Manual. Many of the chapters in the Commentary contain only general introductory information that can be found in many different texts and references on construction materials and pavements; in such cases, the chapters are brief and do not list specific technical supporting information. Furthermore, information given in the Manual that is well supported, clearly referenced, or both, is generally not included in the Commentary. Technical information in the Commentary has been presented as concisely as possible, its primary purpose being to provide a record for those wishing to thoroughly evaluate and revise the Manual in the future.

It is not necessary to read the Commentary in order to understand or use the information in the Manual. The Commentary is intended for researchers and engineers seeking a deeper understanding of some of the more complex technical underpinnings of the Manual.

CHAPTER 1

Introduction

This chapter serves only as an introduction to the Manual and contains no critical information that requires additional supporting details or justification.

CHAPTER 2

Background

This chapter of the Manual presents general background information on construction materials and flexible pavements. It is intended for engineers and technicians with little background in these subjects. The information presented is not controversial and can be found in many other references, including introductory texts on construction materials and pavements. Therefore, further supporting details and justification are not needed for the information given in this chapter.

CHAPTER 3

Asphalt Binders

This chapter of the Manual presents information on asphalt binders. This includes background information of a more detailed nature than that provided in Chapter 2, such as brief discussions of asphalt refining, temperature sensitivity, and age hardening. Performance grading of asphalt binders is described in some detail, including discussion of the various test methods involved and presentation of some of the pertinent specifications in tabular format. The chapter concludes with a few paragraphs giving practical guidelines for the selection of binders during the mix design process.

Figure 3-1 is a plot of dynamic shear modulus ($|G^*|$) as a function of temperature at 10 rad/s for a typical asphalt binder. This figure represents actual data for a PG 64-22 binder.

Table 3-1 presents details of the current standard for performance-graded asphalt binders, as described in AASHTO M320. Table 3-1 includes low temperature grading solely based on the bending beam rheometer (BBR); the direct tension test is not used for this purpose. AASHTO M320 contains different tables for low temperature grading with the BBR and the direct tension test; the direct tension grading is omitted in this chapter in the interest of brevity and because grading using the BBR is much more common.

CHAPTER 4

Aggregates

This chapter of the Manual is an introduction to construction aggregates and includes detailed information on particle size analysis, definition of nominal maximum aggregate size, and a description of how to perform a sieve analysis. This chapter of the Manual discusses the different types of aggregate gradation, such as dense-graded and gap-graded aggregate blends, and includes a table giving specifications for the various AASHTO aggregate gradations. This chapter also presents information on aggregate specific gravity and absorption and what were formerly called the Superpave "consensus" properties: coarse aggregate fractured faces, fine aggregate angularity, flat and elongated particles, and the sand equivalent test. However, this chapter points out that because these tests are now generally accepted by the pavement engineering community and are supported by substantial experience, these properties no longer represent the "consensus" of an expert panel and so should be referred to simply as "specification" properties rather than "consensus" properties. Chapter 4 of the Manual concludes with discussions of aggregate toughness as measured by the Los Angeles Abrasion test, aggregate soundness tests, and tests for deleterious materials.

All of the critical tables given in Chapter 4 are based on those found in existing AASHTO standards, as listed in Table 1. In two cases—requirements for coarse aggregate fractured faces (CAFF) and fine aggregate angularity (FAA)—the requirements have been modified slightly from those given in existing standards, as described in the notes to the table. These modifications are based in part on the recommendations of *NCHRP Report 539* (1). In this report, it is suggested that there is no need for minimum CAFF values exceeding 95%. However, the minimum value in the Manual for the highest design traffic level is 98%, with the option of a further reduction to 95% if experience with local conditions and materials warrant such a reduction. This approach represents a compromise between the recommendations of *NCHRP Report 539* and the reluctance of many engineers to reduce minimum CAFF values to 95% without further experience with HMA mixtures produced with coarse aggregates exhibiting lower values for fractured faces.

The equations given in Chapter 4 are also taken directly from various AASHTO standards. Equations 4-1 through 4-3, dealing with an example calculation of aggregate gradation, are based on AASHTO T 27. Equations 4-5 through 4-7, dealing with aggregate specific gravity and absorption are based on AASHTO T 84 (fine aggregate) and T 85 (coarse aggregate).

Table 1. Sources for critical tables in chapter 4 of the mix design manual.

Table No.		Source Standard
Table. 4-1.	Minimum Test Sample Size for Sieve Analysis of Aggregate as a Function of Nominal Maximum Aggregate Size.	AASHTO T 2
Table 4-3.	Standard Sizes of Coarse Aggregates for Road and Bridge Construction as Adapted from AASHTO M 43.	AASHTO M 43
Table 4-4.	Standard Sizes of Fine Aggregates for Bituminous Paving Mixtures as Adapted from AASHTO M 29.	AASHTO M 29
Table 4-6.	Coarse Aggregate Fractured Faces Requirements.	AASHTO M 323[a]
Table 4-7.	Fine Aggregate Angularity Requirements.	AASHTO M 323[b]
Table 4-8.	Criteria for Flat and Elongated Particles.	AASHTO M 323
Table 4-9.	Clay Content Requirements.	AASHTO M 323

[a]Minimum required values for coarse aggregate fractured faces given in Table 4-6 differ slightly from those in M 323; for design traffic levels of 30 million ESALs or more, the minimum required value is 98% for particles with both one and two fractured faces, rather than 100% as given in M323. Furthermore, this value may be further reduced to 95% if experience with local conditions and materials suggests that the lower value would provide mixtures with adequate rut resistance under very heavy traffic. These changes are largely based on recommendations made in *NCHRP Report 539* (*1*).

[b]Minimum required values for fine aggregate angularity given in Table 4-7 differ slightly from those in M 323; for mixtures placed within 100 mm of the pavement surface subject to design traffic levels of 3 million ESALs or higher, or for mixtures placed 100 mm or deeper from the pavement surface subject to design traffic levels of 30 million ESALs or more, the required FAA value may be reduced from 45% to 43% if experience with local conditions and materials suggests that this will produce mixtures with adequate rut resistance. These changes are largely based on recommendations made in *NCHRP Report 539* (*1*).

CHAPTER 5

Mixture Volumetric Composition

Chapter 5 of the *Manual* discusses the volumetric composition of HMA mixtures. The chapter includes a significant amount of introductory material, including the definitions of many terms related to HMA composition and various relationships between HMA compositional factors such as voids in mineral aggregate (VMA) and air voids and pavement performance. Much of the second half of the chapter is devoted to a detailed description of volumetric analysis of HMA mixtures, including numerous equations and example calculations. The primary source for the terminology and equations given in this chapter is AASHTO R 35, Superpave Volumetric Design for Hot-Mix Asphalt. In some cases, the Asphalt Institute's MS-2 and SP-2 manuals were also used as references since these manuals are also referenced in AASHTO R 35 (*2, 3*).

The critical information in Chapter 5 is the various equations presented for calculating various factors of HMA volumetric composition, such as air void content, VMA, and effective binder content. There are many different ways of calculating these factors, and many different forms of what are, in many cases, identical mathematical relationships. Furthermore, all of the equations given in Chapter 5 can be derived from fundamental physical relationships among volume fraction, mass fraction, specific gravity, density, and absorption. These relationships—and the resulting mathematical equations—are often represented through the use of a phase diagram. Although a very useful concept, the phase diagram approach to volumetric analysis has not been included in the Manual because it was believed that its interpretation would be too challenging for many technicians and some engineers. Table 2 lists sources for the various equations presented in Chapter 5; again, it should be noted that all of these equations can be derived from the physical relationships involved, but it is useful to show other references using the same or similar equations in discussion of HMA volumetric composition and analysis.

Table 2. Sources for equations in chapter 5.

Equation No.	For Calculation of	Source
5-1	Bulk specific gravity of compacted specimen	AASHTO T 166
5-2	Maximum specific gravity of loose mixture	AASHTO T 209
5-3	Bulk specific gravity of aggregate blend	AASHTO R 35; TAI SP-2, MS-2
5-4	Air void content of compacted specimen,% by mixture volume	AASHTO R 35, T 269
5-5	Total asphalt binder content of mixture,% by mixture mass	By definition
5-6	Total asphalt binder content of mixture,% by mixture volume	By definition
5-7	Absorbed asphalt binder content,% by mixture volume	By Definition
5-8	Effective asphalt binder content,% by mixture volume	By Definition
5-9	Effective asphalt binder content,% by mixture mass	By Definition
5-10	Absorbed asphalt binder content,% by mixture mass	By Definition
5-11	Voids in mineral aggregate,% by mixture volume	AASHTO R 35
5-12	Voids filled with asphalt,% by volume	AASHTO R 35
5-13	Apparent film thickness	*NCHRP Report 567* (4)
5-14	Aggregate specific surface (method 1)	*NCHRP Report 567* (4)
5-15	Aggregate specific surface (method 2)	*NCHRP Report 567* (4)

CHAPTER 6

Evaluating the Performance of Asphalt Concrete Mixtures

Chapter 6 discusses various means of evaluating the potential performance of HMA mixtures. It includes discussions of the relationships between mixture composition and performance and binder test properties and performance. The chapter also gives detailed practical information on various test methods being used to characterize HMA mixture performance. The last part of the chapter discusses how the Mechanistic-Empirical Pavement Design Guide (MEPDG) may be used to develop predictions of HMA pavement performance and includes specific guidelines for the number and types of tests required as input when using the higher MEPDG design levels.

Much of the most important information provided in Chapter 6 is not in the form of tables, figures, and equations, but is descriptive information on various tests for characterizing HMA mixture performance. Table 3 lists references for the various performance tests discussed in the *Manual.* The references listed include those mentioned in the Manual and one or two additional references that will provide interested readers with more detailed information on the procedures.

References for Table 3

5. Bonaquist, R., D. W. Christensen, and W. Stump, *NCHRP Report 513: Simple Performance Tester for Superpave Mix Design: First Article Development and Evaluation,* Washington, DC: Transportation Research Board, 2003, 54 pp.
6. Bonaquist, R., *NCHRP Report 629: Ruggedness Testing of the Dynamic Modulus and Flow Number Tests with the Simple Performance Tester,* Washington, DC: Transportation Research Board, 2008, 39 pp.

Table 3. References for performance tests discussed in chapter 6 of the mix design manual.

Performance Test	Used to Evaluate	References
Asphalt Mixture Performance Test System (AMPT)	Rut resistance, dynamic modulus	5, 6
Superpave shear tester, repeated shear at constant height test	Rut resistance	AASHTO T 320, 7, 8
High-temperature IDT strength test	Rut resistance	9, 10, 11
Asphalt pavement analyzer (APA)	Rut resistance	AASHTO TP 63, 12
Hamburg wheel-tracking test	Rut resistance and/or moisture resistance	AASHTO T 324
Flexural fatigue test	Fatigue resistance	AASHTO T 321, 13
Low temperature IDT creep and strength tests	Resistance to thermal/low temperature cracking	AASHTO T 322, 14, 15
Modified Lottman procedure	Resistance to moisture-induced damage	AASHTO T 283
Short- and long-term oven conditioning	Age-hardening	AASHTO R 30

7. Harvey, J., C. Monismith and J. Sousa, "An Investigation of Field- and Laboratory-Compacted Asphalt-Rubber, SMA, Recycled and Conventional Asphalt-Concrete Mixes Using SHRP Project A-003A Equipment," *Journal of the Association of Asphalt Paving Technologists,* Vol. 63, 1994, pp. 511–548.
8. Sousa, J., "Asphalt-Aggregate Mix Design Using the Repetitive Simple Shear Test (Constant Height)," *Journal of the Association of Asphalt Paving Technologists,* Vol. 63, 1994, pp. 298–333.
9. Christensen, D. W., R. Bonaquist, and D. P. Jack, *Evaluation of Triaxial Strength as a Simple Test for Asphalt Concrete Rut Resistance,* Final Report to the Pennsylvania Department of Transportation, Report No. FHWA-PA-2000-010+97-04 (19), PTI Report 2K26, University Park: The Pennsylvania Transportation Institute, August 2000, 80 pp.
10. Christensen, D. W., et al., "Indirect Tension Strength as a Simple Performance Test," *New Simple Performance Tests for Asphalt Mixes,* Transportation Research Circular E-C068, http://gulliver.trb.org/publications/circulars/ec068.pdf, Washington, D.C.: Transportation Research Board, 2004, pp. 44–57.
11. Zaniewski, J. P., and G. Srinivasan, *Evaluation of Indirect Tensile Strength to Identify Asphalt Concrete Rutting Potential,* Morgantown, WV: West Virginia University, Department of Civil and Environmental Engineering, May 2004, 65 pp.
12. Kandhal, P. S., and L. A. Cooley, *NCHRP Report 508: Accelerated Laboratory Rutting Tests: Evaluation of the Asphalt Pavement Analyzer,* Washington, DC: Transportation Research Board, 2003, 73 pp.
13. Tayebali, A. et al., "Mix and Mode-of-Loading Effects on Fatigue Response of Asphalt-Aggregate Mixes," *Journal of the Association of Asphalt Paving Technologists,* Vol. 63, 1994, pp. 118–143.
14. Hiltunen, D. R. and R. Roque, "A Mechanics-Based Prediction Model for Thermal Cracking of Asphaltic Concrete Pavements," *Journal of the Association of Asphalt Paving Technologists,* Vol. 63, 1994, pp. 81–113.
15. Roque, R., D. R. Hiltunen and W. G. Buttlar, "Thermal Cracking Performance and Design of Mixtures Using Superpave," *Journal of the Association of Asphalt Paving Technologists,* Vol. 64, 1995, pp. 718–733.

Table 6-1 of the Manual summarizes the effects of various HMA characteristics on performance, and is reproduced here as Table 4 for the convenience of the reader. In the Manual, two NCHRP reports are cited as the major basis for this table: *NCHRP Report 539* and *NCHRP Report 567* (*1, 4*).

Table 4. Effect of mixture composition of performance—table 6-1 in the mix design manual.

Typical Effects of Increasing Given Factor within Normal Specification Limits
While Other Factors Are Held Constant within Normal Specification Limits
"↑" indicates improved performance; "↓" indicates reduced performance

Component	Factor	Resistance to Rutting and Permanent Deformation	Resistance to Fatigue Cracking	Resistance to Low Temperature Cracking	Durability/ Resistance to Penetration by Water and Air	Resistance to Moisture Damage
Asphalt Binder	Increasing High Temperature Binder Grade	↑↑↑				
	Increasing Low Temperature Binder Grade			↓↓↓		
	Increasing Intermediate Temperature Binder Stiffness		↑↓			
Aggregates	Increasing Aggregate Angularity	↑↑				
	Increasing Proportion of Flat and Elongated Particles					
	Increasing Nominal Maximum Aggregate Size		↓	↓	↓	
	Increasing Mineral Filler Content and/or Dust/Binder Ratio	↑↑			↑	
	Increasing Clay Content					↓
Volumetric Properties	Increasing Design Compaction Level	↑↑	↑↑			
	Increasing Design Air Voids	↑↑				
	Increasing Design VMA and/or Design Binder Content	↓↓	↑	↓		
	Increasing Field Air Voids	↓↓	↓↓	↓	↓↓↓	↓↓

Table 5 gives a more complete list of references supporting the information given in Table 6-1 of the Manual, including several notes explaining complex relationships between HMA characteristics and performance. For convenience, references for Table 4 appear below the table and are given again at the end of the Commentary. In many cases, the relative strength of these relationships—indicated by the number of arrows in Table 6-1—is to some degree a matter of engineering judgment.

Table 6-2 in the Manual, which lists performance-related test properties used in specifying asphalt binders in AASHTO M 320, is directly based on M 320 itself. However, as noted above, the use of $|G^*| \sin \delta$ to limit susceptibility to fatigue damage is controversial and at this writing much effort is underway to develop a more effective means of specifying the fatigue-related properties of asphalt binders.

Table 5. References for table 6-1 in the manual, on the effect of mixture composition of performance.

Component	Factor	Resistance to Rutting and Permanent Deformation	Resistance to Fatigue Cracking	Resistance to Low Temperature Cracking	Durability/ Resistance to Penetration by Water and Air	Resistance to Moisture Damage
Asphalt Binder	Increasing High Temperature Binder Grade	M 320, 16, 17, 18				
	Increasing Low Temperature Binder Grade			M 320, 19		
	Increasing Intermediate Temperature Binder Stiffness		M 320, Note 1			
Aggregates	Increasing Aggregate Angularity	M 323, 1, 20, 21				
	Increasing Proportion of Flat and Elongated Particles					
	Increasing Nominal Maximum Aggregate Size		Notes 2, 3	Notes 2, 3	Note 4	
	Increasing Mineral Filler Content and/or Dust/Binder Ratio	4, 16			4	
	Increasing Clay Content					M 323, 20
Volumetric Properties	Increasing Design Compaction Level	M 323, 16	M 323, 16			
	Increasing Design Air Voids	16				
	Increasing Design VMA and/or Design Binder Content	16, 17, 18	16, 22, 13, 24	20		
	Increasing Field Air Voids	16, 17, 18	17, 23, 24	20	16, 25, 26	Note 5

[1]Most research suggests that the fatigue resistance of an HMA mixture shows a complex relationship with its stiffness and indirectlywith binder stiffness; for thin pavements, fatigue resistance decreases with increasing modulus, whereas for thick pavements, fatigue resistance increases with increasing modulus (see references 7 and 8).

[2]In current and previous HMA mix design systems, VMA and binder content typically decrease with increasing aggregate NMAS. In the interest of clarity, these two factors have been separated, and the effect of design VMA and binder content on mixture performance are listed separately in this table. However, it should be kept in mind that increasing aggregate NMAS will usually decrease VMA and binder content, which in turn will affect mixture performance in various ways. Decreasing aggregate NMAS will, in general, increase VMA and binder content, which will also affect mixture performance.

[3]Although there is currently little research linking HMA fatigue properties and resistance or low temperature cracking to aggregate NMAS, the strength properties and fatigue resistance of most particulate composites like HMA increase with decreasing particle size because the size of flaws and magnitude of internal stress concentrations tend to decrease with decreasing particle size. This is the reason for the thoroughly documented increase in compressive strength with decreasing aggregate size in portland cement concrete mixtures. It is highly likely that similar relationships exist for HMA mixtures.

[4]As discussed in Note 2 above, increasing aggregate NMAS will tend to result in an overall increase in the number of large flaws in an HMA mixture. This is partly a result of occasional poor bonding at the asphalt-aggregate interface. Such large flaws will tend to result in a significant increase in permeability to both air and water, reducing the durability of HMA mixtures made with large-sized aggregates.

[5]The current test procedure used widely to evaluate the resistance of HMA to moisture damage, AASHTO T 283, is performed at a constant air void content of 7 ± 1%. Therefore, little information concerning the effect of air void content on moisture resistance is available. However significant research shows permeability increases with increasing air voids, so it should be expected that as air voids and permeability increase, resistance to moisture damage will decrease.

Table 6. Recommended changes to high temperature performance grade to account for traffic volume and speed—table 6-3 in the mix design manual.

Design Traffic (MESALs)	*Grade Adjustment for Average Vehicle Speed in kph (mph):* Very Slow	Slow	Fast
	< 25 (< 15)	25 to < 70 (15 to < 45)	≥ 70 (≥ 45)
< 0.3	---	---	---
0.3 to < 3	12	6	---
3 to < 10	18*	13	6
10 to < 30	22*	16*	10
≥ 30	---	21*	15*

* *Consider use of polymer-modified binder. If a polymer-modified binder is used, high temperature grade may be reduced one grade (6 °C), provided rut resistance is verified using suitable performance testing.*

Table 6-3 in the Manual lists recommended high-temperature performance grade adjustments to account for traffic volume and speed. These grade adjustments should be applied to the base high-temperature binder grade in order to ensure that the resulting HMA mixture will have adequate resistance to rutting for the given traffic conditions. In AASHTO M 320, these adjustments are given without explanation. In LTPPBind Version 3.1, the adjustments given follow directly from the rational approach taken in the development of the software—that is, the adjustments are based on predicted damage under different traffic levels and traffic speeds. However, the problem encountered with LTPPBind Version 3.1 is that the traffic speeds are limited to fast and slow—there is no adjustment for very slow traffic. Furthermore, it is not clear what range in average traffic speeds were assumed in the calculation of grade adjustments. Table 6-3 in the Manual (reproduced as Table 6) is presented to provide grade adjustments for the full range of traffic speeds, and with a full rational derivation, as given below.

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26. Huang, B., et al., "Fundamentals of Permeability in Asphalt Mixtures," *Journal of the Association of Asphalt Paving Technologists,* Vol. 68, 1999, pp. 479–496.

Estimating grade adjustments such as those shown in Table 6 is somewhat complicated, given that three different factors must be considered: traffic volume, traffic speed, and design compaction. It must be remembered that when designing HMA mixtures following this method (or the Superpave system), the design compaction level changes along with traffic level, so the properties of the mix, including rut resistance, will change significantly. This will affect the required binder grade. One other piece of information is needed to develop binder grade adjustments: the typical change in binder $|G^*|/\sin\delta$ values with temperature. Recent research performed for the Airfield Asphalt Pavement Technology Program (AAPTP) Project 4-2 described the calculation of high-temperature binder grade adjustments in detail; the development given here closely follows that given in the Final Report for AAPTP Project 4-2 (*27*).

The effect on rut resistance of differences in mixture properties can be estimated using the resistivity-rutting model initially developed during NCHRP Projects 9-25 and 9-31, and further refined as part of NCHRP Project 9-33 and AAPTP Project 4-2 (*4, 16, 27*). The most recent version of the resistivity/rutting equation gives allowable traffic as a function of mixture composition, compaction, and air voids:

$$TR = 9.85 \times 10^{-5} \left(\mathrm{P}NK_s\right)^{1.373} V_{QC}^{1.5185} V_{IP}^{-1.4727} M \tag{1}$$

where

TR = million ESALs to an average rut depth of 7.2 mm (50% confidence level)
= million ESALs to a maximum rut depth of 12 mm (95% confidence level)

P = resistivity, s/nm

$$= \frac{\left(|G^*|/\sin\delta\right) S_a^2 G_a^2}{49 VMA^3}$$

$|G^*|/\sin\delta$ = Estimated *aged* performance grading parameter at high temperatures, determined at 10 rad/s and at the yearly, 7-day average maximum pavement temperature at 20 mm below the pavement surface, as determined using LTPPBind, Version 3.1 (units of Pa); aged value can be estimated by multiplying the RRTFOT value by 4.0 for long-term projects (10 to 20 year design life), and by 2.5 for short-term projects of 1 to 2 years.

S_a = specific surface of aggregate in mixture, m^2/kg
≅ the sum of the percent passing the 75, 150, and 300 micron sieves, divided by 5.0
≅ 2.05 + (0.623 × percent passing the 75 micron sieve)

G_a = the bulk specific gravity of the aggregate blend
VMA = voids in the mineral aggregate for the mixture, volume%, as determined during QA testing
N = design gyrations
K_s = speed correction
= $(v/70)^{0.8}$, where v is the average traffic speed in km/hr
V_{QC} = air void content, volume%, determined during QA testing at design gyrations
V_{IP} = air void content, volume%, in-place
M = 7.13 for mixtures containing typical polymer-modified binders, 1.00 otherwise

The equation for resistivity can be inserted into Equation 1 and the results simplified to give an alternate form for allowable traffic:

$$TR = 4.71 \times 10^{-7} \left(|G^*|/\sin\delta\right)^{1.373} S_a^{2.746} G_a^{2.746} VMA^{-4.119} N_{eq}^{1.373} K_s^{1.373} V_{QC}^{1.5185} V_{IP}^{-1.4727} M \tag{2}$$

In order to develop high-temperature binder grade adjustments, Equation 2 must be manipulated into a form that allows the direct calculation of the temperature adjustment needed to offset a specified change in a given property or combination of properties:

$$\frac{TR_2}{TR_1} = \left[\frac{\left(|G^*|/\sin\delta\right)_2}{\left(|G^*|/\sin\delta\right)_1}\right]^{1.373} \left(\frac{S_{a2}}{S_{a1}}\right)^{2.746} \left(\frac{G_{a2}}{G_{a1}}\right)^{2.746} \left(\frac{VMA_2}{VMA_1}\right)^{-4.199} \left(\frac{N_2}{N_1}\right)^{1.373} \left(\frac{v_{s2}}{v_{s1}}\right)^{1.098} \left(\frac{V_{QC2}}{V_{QC1}}\right)^{1.5158} \left(\frac{V_{IP2}}{V_{IP1}}\right)^{-1.4727} \left(\frac{M_2}{M_1}\right) \tag{3}$$

In Equation 3, subscripts 1 and 2 refer to two different sets of conditions—binder $|G^*|/\sin\delta$, aggregate surface area, mix VMA, design gyrations, and so forth. Because we are only interested in changes in three of the properties included in Equation 3 ($|G^*|/\sin\delta$, N, and v), Equation 3 can be simplified by removing the other variables:

$$\frac{TR_2}{TR_1} = \left[\frac{\left(|G^*|/\sin\delta\right)_2}{\left(|G^*|/\sin\delta\right)_1}\right]^{1.373} \left(\frac{N_2}{N_1}\right)^{1.373} \left(\frac{v_{s2}}{v_{s1}}\right)^{1.098} \tag{4}$$

An analysis was performed on a set of nine different binders from various accelerated pavement tests. Eight of the binders were from projects included in development of the AMPT: the FHWA ALF rutting test; MnRoad; and Westrack (*28, 29, 30*). One binder tested was a PG 64-22 used in NCHRP Projects 9-25 and 9-31 (*4*). These binders were chosen for this analysis because they have been included in well-known studies, and their flow properties have been thoroughly documented. As shown in Figure 1, the relationship between temperature and modulus ($|G^*|/\sin\delta$ in this case) is exponential:

$$\frac{\left(|G^*|/\sin\delta\right)_1}{\left(|G^*|/\sin\delta\right)_2} = \exp\left[A\left(T_1 - T_2\right)\right] \tag{5}$$

The value of constant A in Equation 5 varies somewhat among the binders included in Figure 1, but is typically very close to –0.135, as shown in Figure 9. Equation 5 can be substituted

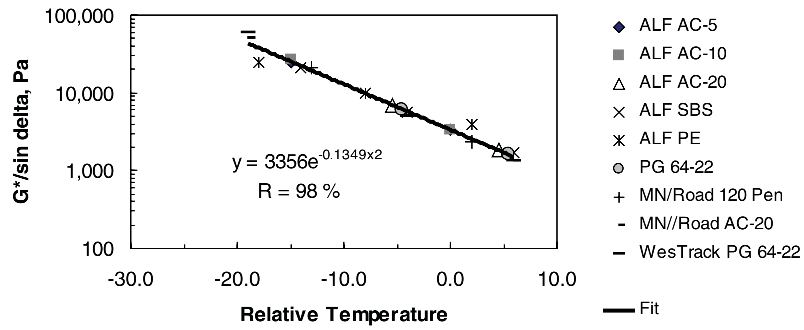

Figure 1. Temperature dependence of nine asphalt binders relative to the performance grading temperature.

into Equation 4 and rearranged, giving the following relationship between binder grade adjustment ΔT, traffic level, design gyrations, and traffic speed:

$$\Delta T = T_2 - T_1 = 7.41 \ln\left[\left(\frac{TR_2}{TR_1}\right)^{0.728}\left(\frac{N_1}{N_2}\right)\left(\frac{v_{s1}}{N_{s2}}\right)^{0.800}\right] \tag{6}$$

Equation 6 can then be used to estimate the high-temperature binder grade adjustments given in Table 6, keeping in mind the design gyration levels for various traffic levels: 50 gyrations for less than 0.3 million ESALs, 75 gyrations for 0.3 million to less than 10 million ESALs, 100 gyrations for 10 million to less than 30 million ESALs, and 125 gyrations for traffic levels of 30 million ESALs or more. The traffic level used to calculate the grade adjustments was the highest in the given range, and 100 million for traffic levels of 30 million ESALs or more. Traffic speeds used in the calculations were 70 kph for fast traffic, 25 kph for slow traffic, and 10 kph for very slow traffic.

Table 6-4, in the Manual (included here as Table 7) summarizes the performance testing recommended for routine, dense-graded HMA mix designs. As in the Superpave system, moisture resistance testing is required for all mix designs. The only other performance testing normally required for any routine mix design is rut-resistance testing. This is because of the high level of complexity and cost for performing tests to characterize resistance to fatigue cracking and thermal cracking. This table is largely based on engineering judgment. It is assumed that the reliability for HMA mixtures intended for pavements at lower traffic levels—below 3 million ESALs—does not need to be as high as that for mixtures intended for higher traffic levels, and so rut resistance testing is not warranted for these cases.

Table 6-5 in the Manual summarizes mixture properties used as input in the MEPDG. This table is straightforward and is based directly on the MEPDG User Manual (*31*). Table 6-6 in the Manual, reproduced as Table 8 below, summarizes the effect of changes in mixture composition and high-temperature binder grade on MEPDG performance predictions. Like Table 6-5, this is based on information contained in the MEPDG User Manual (*31*).

Table 7. Recommended performance tests for HMA mixtures made with conventional materials including most modified binders—table 6-4 in the mix design manual.

Property	Recommended Test	Design Traffic Levels for Which Property Should be Evaluated
Moisture Sensitivity	AASHTO T 283	All
Permanent Deformation	Flow Number or Dynamic Modulus, NCHRP 9-29 PT 01	3 Million ESAL and greater
Fatigue Cracking	None	NA
Thermal Cracking	None	NA

Table 8. Summary of effect of mixture composition on performance predictions—table 6-6 in the mix design manual.

HMA Property	Rutting	Thermal Cracking	Alligator Cracking HMA ≥ 5 in	Alligator Cracking HMA < 3 in	Longitudinal Cracking
High Temperature Binder Grade	**Increase to improve**		Increase to improve	Decrease to improve	Decrease to improve
Low Temperature Binder Grade		**Decrease to improve**			
Design VMA	Decrease to improve		**Increase to improve**	**Increase to improve**	**Increase to improve**
Design VFA		Increase to improve			
Filler Content	Increase to improve				
In-Place Air Voids	Decrease to improve	Decrease to improve	**Decrease to improve**	**Decrease to improve**	**Decrease to improve**

CHAPTER 7

Selection of Asphalt Concrete Mix Type

Chapter 7 provides engineers and technicians with guidance on the selection of HMA mix types for different applications. In most cases, when an engineer or technician is performing a mix design, the mix type will be specified by the owner or agency requesting the mix design. However, this may not always be the case—especially for private paving work, where the owner may not have any idea what type of mix is best suited for her or his particular application. Furthermore, because selection of mix type is a direct function of where within a pavement the mix is located, Chapter 7 also discusses the topic of pavement structure in some detail.

The primary reference for Chapter 7 is a publication of the National Asphalt Pavement Association (NAPA): *HMA Pavement Mix Type Selection Guide* (*33*). Some additional information is given on lift thickness, based on *NCHRP Report 531* (*32*), and on pavement structure and mix type selection for perpetual pavements, based on *TRB Circular 503* (*34*).

The figures and tables presented in Chapter 7 are not highly technical in nature, instead presenting, for the most part, general knowledge concerning pavement types and pavement structure. Figure 7-1 presents the different types of pavement structures incorporating HMA in new construction. Figure 7-2 is similar, but presents different types of pavement structures using HMA that result from pavement maintenance operations.

The traffic levels listed in Table 7-1 are defined in both AASHTO M 323 and R 35. However, the descriptions of typical traffic and road types for the different traffic levels occurs only in AASHTO R 35. Table 7-2, giving recommended lift thicknesses for different mix types and NMAS, is taken directly from *NCHRP Report 531* (*32*), while Table 7-3 is a summary of information taken from NAPA's publication *HMA Pavement Mix Type Selection Guide* (*33*).

CHAPTER 8

Design of Dense-Graded HMA Mixtures

Chapter 8 is probably the most important chapter in the *Manual.* It describes in detail the recommended procedure for designing dense-graded HMA mixtures. Much of the material presented here also appears in other chapters—it is repeated in Chapter 8 for the convenience of the reader, and so that Chapter 8 can be used as a stand-alone document for designing dense-graded HMA mixtures. For example, the tables on aggregate specifications also appear in Chapter 4.

Many of the tables that appear in Chapter 8 are either identical or nearly identical to tables used in the Superpave system, as described in AASHTO standards M 323 and R 35 and the Asphalt Institute's SP-2 manual. Many of these tables have been only slightly modified, based upon the results of various recent research projects. Some of the tables, such as those providing guidelines for interpreting various performance tests, do not appear in standards and publications dealing with Superpave. As described below, in some cases development of these tables was simply a matter of presenting typical current practice. However, for some of the performance tests developing meaningful guidelines for interpreting results was a complex task.

Chapter 8 presents a brief history of HMA mix design methods, in order to provide inexperienced technicians and engineers with some background. Of particular interest is the information provided on the Superpave system, which is quite similar to the mix design method presented in the Manual. After the background section, Chapter 8 presents a short summary of the proposed mix design procedure, followed by a detailed, step-by-step description. This includes numerous example problems with solutions.

An important feature in Chapter 8 of the Manual is the frequent references to HMA Tools, which is a Microsoft Excel spreadsheet application designed to accompany the Manual. HMA Tools is a powerful spreadsheet which can perform virtually all of the calculations needed when performing an HMA mix design. The examples given in Chapter 8 generally refer to HMA Tools. As pointed out in the Manual, it is not necessary to use HMA Tools to perform mix designs according to the recommended method, but if other software is used, it will usually be necessary to update the various specifications and limits to reflect those given in the *Manual.*

Table 8-1 in the Manual shows recommended grade adjustments for traffic level and speed. This table is identical to Table 6-3, presented and discussed in detail previously in the *Commentary* as Table 6. Readers should refer to this discussion for the derivation of the grade adjustments shown in Table 8-1.

Table 8-2 lists compaction effort as a function of design traffic level. The values for design gyrations—N_{design}—are identical to what appears in AASHTO R 35. However, values for $N_{initial}$ and N_{max} have been eliminated from the table. $N_{initial}$ and N_{max} requirements have been eliminated on the basis of work done during NCHRP Project 9-9(1)(*35*); as documented in *NCHRP Report 573,* two of the major conclusions of this project were that neither $N_{initial}$ nor $N_{maximum}$ correlated with rutting observed in an extensive field experiment and are not required in designing HMA mixtures (*35*). *NCHRP Report 573* also suggested new gyration levels, as listed in Table 9 below. *NCHRP Report 573* recommends two sets of compaction levels—one for binders with a high temperature grade of 76 and greater (or for binders used in mixes placed more than 100 mm from the pavement surface), and one set for other binders. Furthermore, the overall level of compaction is lower than is currently suggested in R 35 (*35*). The recommended compaction levels are based on matching densification as it occurs in the gyratory compactor and as it occurs under traffic loading. It should be pointed out that the correlations reported in *NCHRP Report 573* between densification during laboratory compaction and under traffic loading are not strong—34 and 37% for the two different compactors used in the study (*35*). Furthermore, this study does not address the strong effect laboratory compaction has on rut resistance, as noted in *NCHRP Report 567* (*4*). In the procedure given in the *Manual,* the current compaction levels are maintained for several reasons. First, HMA mixtures made with binders of grade PG 76-XX and higher are in general polymer modified and usually intended for pavements subjected to very heavy traffic loading—major urban highways which demand the highest levels of reliability against rutting. The significant reduction in N_{design} recommended in *NCHRP Report 573* could reduce the rut resistance of such HMA designs to an unacceptable level. A second factor to consider when evaluating the use of a different set of compaction levels for what, in effect, are mostly polymer-modified binders is the probable adoption of the MSCR test to grade asphalt binders at high temperatures. Use of this test might result in changes in binder grade selection that in combination with a change in compaction levels could yield inadequate performance for mixtures that should exhibit outstanding levels of performance. A third consideration is that there has of yet been little time to validate the findings of *NCHRP Report 573.* Additional time is needed for industry input and further independent evaluation of the proposed compaction levels prior to implementation.

Table 8-3 lists the primary control sieve (PCS) size for different NMAS, along with the PZC control point, which is the % passing above which an aggregate gradation is considered a "coarse" gradation and below which it is considered a "fine" gradation. Table 8-3 is nearly identical to Table 4 from AASHTO M 323-5, with the exception that Table 8-3 includes information on 4.75 mm NMAS gradations, while Table 4 in M 323-5 does not.

Table 8-4 in the Manual (Table 10 below) lists recommended NMAS for different types of dense-graded HMA mixtures, along with recommended lift thicknesses. The values for NMAS follow directly from the recommendations of Chapter 7 on mix type selection—specifically, Table 7-3, which lists recommended mix types and NMAS values for different traffic levels. Table 7-3 in turn was based on the recommendations given in NAPA's publication *HMA Pavement Mix Type Selection Guide, IS 128* (*33*). Lift thickness values are based on recommendations given

Table 9. Compaction effort as a function of design traffic level as recommended in *NCHRP Report 573* and as given in AASHTO R 35 and the mix design manual (*35*).

	From *NCHRP Report 573*:		
20-Year Design Traffic, Million ESALs	N_{design} for Binders < PG 76-XX	N_{design} for Binders ≥ PG 76-XX or for Binders Used in Mixes Placed > 100 mm from Pavement Surface	N_{design} in AASHTO R 35 and in the Mix Design Manual
< 0.30	50	NA	50
0.30 to < 3.0	65	50	75
3.0 to < 10	80	65	100
10 to < 30	80	65	100
> 30	100	80	125

Table 10. Recommended aggregate nominal maximum aggregate sizes (NMAS) for dense-graded HMA mixtures—table 8-4 in the mix design manual.

Application	Recommended NMAS, mm	Recommended Lift Thickness, mm	
		Fine-Graded Mixtures	Coarse-Graded Mixtures
Leveling course mixtures	4.75	15 to 25	20 to 25
	9.5	30 to 50	40 to 50
Wearing course mixtures	4.75	15 to 25	20 to 25
	9.5	30 to 50	40 to 50
	12.5	40 to 65	50 to 65
Intermediate course mixtures	19.0	60 to 100	75 to 100
	25.0	75 to 125	100 to 125
Base course mixtures	19.0	60 to 100	75 to 100
	25.0	75 to 125	100 to 125
	37.5	115 to 150	150
Rich base course mixtures	9.5	30 to 50	40 to 50
	12.5	40 to 65	50 to 65

in *NCHRP Report 531:* lift thickness values 3 to 5 times NMAS for fine-graded mixtures, and 4 to 5 times NMAS for coarse-graded mixtures.

Table 8-5 in the Manual lists maximum and minimum VMA values as a function of aggregate NMAS. Minimum VMA values are the same as those specified in AASHTO M 323-04. However, there is a note to the table allowing agencies to increase minimum VMA values by up to 1.0%, in order to improve field compaction, fatigue resistance, and durability. The note also contains a caution that if VMA is increased, care should be taken to ensure that the resulting mix maintains adequate rut resistance. This note has been included to address the concern of many agencies that HMA mixtures designed according to existing Superpave methods often exhibit durability problems—raveling, surface cracking, and moisture damage. Many agencies have already increased minimum VMA values for Superpave mixes in order to address these perceived problems. Allowing an increase of up to 1.0% in minimum VMA addresses the concerns of agencies that have experienced durability problems in Superpave mixes, but allows those agencies that have not seen such problems to maintain minimum VMA values at the current levels specified in M 323-04.

Maximum VMA values in the Manual are held to 2% above the minimum values. VFA is no longer specified. One of the reasons for eliminating the requirements for minimum and maximum VFA is that the relationship among VMA, VFA, and design air voids is complex and makes simultaneous control of all three difficult and confusing. The requirements currently given in Table 6 of M 323 in fact require some effort to interpret precisely; specification of minimum and maximum VFA, in combination with the specified design air void content of 4.0%, establish an alternate set of minimum and maximum VMA values, since $VMA = 4.0/(1\text{-}VFA/100)$. The implied VMA values calculated in this way are either equal to or less than the stated VMA values (allowing for rounding errors), and so the stated minimum VMA values are not ambiguous. However, specifying a minimum VMA and a design air void content also implies a minimum VFA, since $VFA = (VMA\text{-}4.0)/VMA \times 100\%$. In this case, the implied minimum VFA values are often greater than those specifically listed in Table 6 of M 323-04. The conservative interpretation would be that the highest of the two alternate sets of minimum VFA values applies, but the standard as written is somewhat ambiguous. Table 11 lists the minimum VMA values and minimum and maximum VFA values specified in Table 6 of M 323-04, along with the minimum VFA calculated from the specified minimum VMA, and the calculated maximum VMA values calculated from the maximum VFA. The approach used in the Manual—simply specifying a minimum and maximum VMA—is simpler and avoids the ambiguity inherent in trying to simultaneously control

Table 11. Specified minimum VMA values and implied minimum and maximum VMA values calculated from VFA values specified in table 6 of AASHTO M 323-04.

NMAS, mm	Design Traffic Level, Million ESALs	Minimum VMA, %	Minimum VFA, %	Maximum VFA, %	Calculated Minimum VFA, %	Calculated Maximum VMA, %
4.75	< 3.0	16.0	70	80	75.0	20.0
4.75	≥ 3.0	16.0	75	78	75.0	18.2
9.5	< 3.0	15.0	65	78	73.3	18.2
9.5	≥ 3.0	15.0	73	76	73.3	16.7
12.5	All	14.0	65	75	71.4	16.0
19	All	13.0	65	75	69.2	16.0
25	≥ 0.3	12.0	65	75	66.7	16.0
25	< 0.3	12.0	67	75	66.7	16.0
37.5	All	11.0	64	75	63.6	16.0

air voids, VMA, and VFA. For the smaller NMAS values and higher traffic levels, the approach in the Manual results in similar ranges for VMA. For the larger NMAS value, the approach in the Manual is somewhat more restrictive, since M 323-04 in effect specifies maximum VMA values 3 to 4% higher than the minimum for these aggregates. However, in reality, because of the high cost of asphalt binders most mix designers select VMA values very close to the minimum values specified in M 323-04. Therefore, the difference in the two approaches is probably negligible in practice.

Tables 8-6 and 8-7 in the Manual list aggregate control points for aggregate blends of different NMAS values. These tables contain values identical to those given in Table 3 of AASHTO M 323-04. However, in the Manual, the aggregate control points are given as suggested limits, and not specified limits as is done in M 323-04. This change was made because it provides the mix designer with much greater flexibility in obtaining specified VMA values, and virtually all evidence relating HMA performance to composition suggests that it is much more important to control VMA than to control details of aggregate gradation. For instance, none of the models discussed in Chapter 6 relating HMA composition to rut resistance and fatigue resistance contain factors related to aggregate gradation as predictor variables (*16, 17, 18, 22, 23, 24*). Although *NCHRP Report 405* states that aggregate gradation effects rut resistance and fatigue resistance, these statements are made without support—and in fact made with the admission that evaluating the effect of aggregate gradation on HMA performance was outside the scope of the project (*20*).

Aggregate specification properties—coarse aggregate fractured faces, flat and elongated particles, fine aggregate angularity, and clay content—are given in Tables 8-8 through 8-11. These tables are identical to Tables 4-6 through 4-9 given in Chapter 4. As discussed in the Commentary section dealing with Chapter 4, these tables are very similar to the corresponding tables in M 323-04. The reader should refer to the Commentary section on Chapter 4 for a discussion of these tables.

Table 8-12 in the Manual lists requirements for dust/binder ratio; it is reproduced here as Table 12. The requirements given for 4.75 mm NMAS mixes are identical to those given in AASHTO M 323-04. The requirements for other mixes—allowable dust/binder ratios in the range of 0.8 to 1.6, with an option of lowering this range to 0.6 to 1.2—are slightly higher than those in M 323-04. In M 323-04, the specified range for mixes other than 4.75 mm NMAS is from 0.6 to 1.2, with an option of raising this range to 0.8 to 1.6. The requirements in the *Manual* are therefore similar, but encourage slightly higher dust/binder ratios. There are two reasons for this increase. The first is that research performed during NCHRP Projects 9-25 and 9-31 showed a

Table 12. Requirements for dust/binder ratio—table 8-12 in the mix design manual.

Mix Aggregate NMAS, mm	Allowable Range for Dust/Binder Ratio, by Weight
> 4.75	0.8 to 1.6[A]
4.75	0.9 to 2.0

[A]The specifying agency may lower the allowable range for dust/binder ratio to 0.6 to 1.2 if warranted by local conditions and materials. The dust/binder ratio should however not be lowered if VMA requirements are increased above the standard values as listed in Table 8-5.

strong relationship between permeability and aggregate surface area—as aggregate surface area increases, permeability tends to decrease, all else being equal (*13*). Therefore, specifying slightly higher dust/binder ratios should result in mixes with lower permeability to air and water and improved durability. The second reason for encouraging slightly higher dust/binder ratios in HMA mixes is that the design method given in the Manual attempts to encourage slightly higher VMA and asphalt binder contents in order to improve the durability of the resulting mixtures. One example of how this is done is the VMA requirements described above, which include an option of increasing the minimum VMA values by up to 1% to improve field compaction, fatigue resistance, and durability. It was found in NCHRP Projects 9-25 and 9-31 that rut resistance of HMA mixes tends to increase as aggregate specific surface increases relative to VMA. Since the *Manual* encourages higher VMA values, higher dust/binder values are also encouraged in order to maintain or improve rut resistance compared to mixes designed according to the Superpave system. The extremely premature rutting of many of the mixtures placed at the WesTrack facility was attributed in part to high VMA and relatively low dust/binder ratios (*36*). Promoting an increase in dust/binder ratio will help to prevent such failures in the future.

Table 8-20 in the Manual (reproduced here as Table 13) lists recommended minimum values for flow numbers as a function of design traffic level. The values in this table are based on a relationship between flow number values and maximum allowable traffic level estimated using the resistivity/rutting model given earlier as Equation 1. Data used in developing this relationship was collected by the FHWA in one of their field trailers, for nine different projects in New England, New York State, Nebraska, North Carolina, Minnesota, and Wisconsin. The mix composition, binder $|G^*|/\sin\delta$ values, flow number values, and related data appear in Table 14 of this report. Data forwarded by the FHWA included tests of both laboratory-prepared mixes having different binder contents and field produced mix. A meaningful relationship between flow number and calculated maximum traffic could only be developed using the laboratory mixes. The reason this relationship did not hold up for the field-produced mixes is not clear, but it is possibly due to differences in age hardening during production, transport, and sample storage. The design air void content was known precisely only for mixes which used the design binder content. For the other two mixes for each project—one above and one below the design binder content—the design air void content was estimated using the following equation (*3*):

$$V_{adb} = V_{ad} - 2.5\left(P_b - P_{bd}\right) \tag{7}$$

where

V_{adb} = Estimated design air voids at some binder content P_b
V_{ad} = Design air voids at design binder content P_{bd}

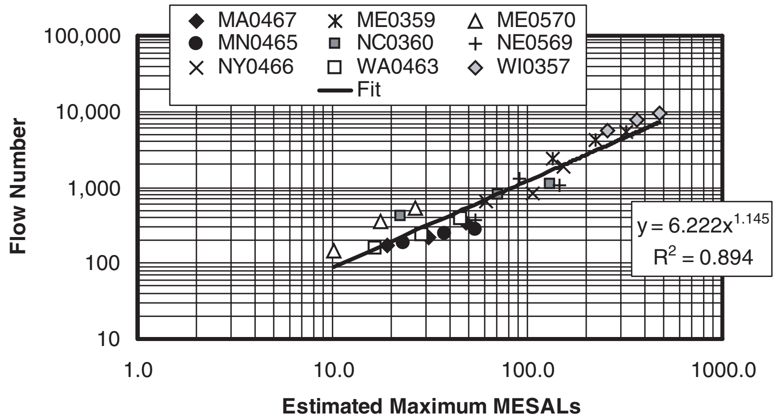

Figure 2. Relationship between flow number and estimated maximum traffic from FHWA field data on six projects.

Another complication in calculating the allowable traffic values shown in Figure 2 and Table 14 is that, in most cases, it is not clear whether the binders used in the nine mixes was polymer modified or not—as seen in Equation 1, there is a factor M that is applied to mixes made using polymer modified binders to account for the superior rut resistance of these materials compared to non-modified binders of the same high-temperature grade. Recent surveys of asphalt binder producers suggests that, at the time these mixes were produced (mostly in 2004) approximately 90% of PG 64-28 binders and PG 70-22 binders were modified (*37, 38*). The PG 70-28 binder used for the MN0465 project was clearly modified, based on the rheological behavior of the binder. From this information and the observed relationship between binder flow properties and mixture flow number, it was assumed that five of the binders used in the nine projects were polymer modified. The four projects in which non-modified binders were assumed to be used in the mixes were NC0360, WI0357, NE0569, and WA0463. Analysis of the data using this assumption provided good results, but it appeared that the value of M used in Equation 1 (7.13) was somewhat too high. The best correlation between estimated maximum traffic and flow number was found when the value of M was assumed to be 3.0. The lower value of M might be because the modified binders in the nine projects contained only relatively small amounts of polymer, or possibly because much of the testing was performed at intermediate temperatures (mostly 38 or 45 °C), where the effect of polymer modification on rut resistance may not be as large as it is at higher temperatures.

The values of flow number appearing in Table 13 were calculated from the regression equation given in Figure 2 using traffic levels at the midpoint of the given design traffic range. For a range of 3 to 10 million ESALs, a value of 6.5 million ESALs was used to estimate a minimum flow number of 53. For the 10 to 30 million ESAL range, a value of 20 million ESALs was used to calculate the minimum flow number of 190. For traffic above 30 million ESALs, a traffic level of 65 million ESALs was used to calculate the required flow number value of 740. Selecting these values, rather than the maximum for each range was done to provide some insurance against excessive rutting, while avoiding being too restrictive, which might result in having a large number of mixes fail the performance test and needing to be redesigned.

Table 13. Recommended minimum flow number requirements (Table 8-20 in the Mix Design Manual).

Traffic Level *Million ESALs*	**Minimum Flow Number** *Cycles*
< 3	---
3 to < 10	53
10 to < 30	190
≥ 30	740

Table 15 lists recommended minimum flow time values as given in the Manual (Table 8-21 in the Manual). These flow time values were calculated by developing a regression equation relating flow time and flow number, as shown in Figure 3. This plot includes data from five projects included in the NCHRP Project 9-19 database (*39*). The data used in the plot are shown in Table 16. The flow time values in Table 15 and the flow number values given in Table 13 are intended to be, for all practical purposes, equivalent.

Table 14. Results of AMPT tests and related properties used in calculation of flow number limits (*39*).

Project	ID	Binder Grade PG-	P_b *Wt.%*	Voids as Tested *Vol.%*	Design Voids *Vol.%*	N_{design} *Gyr.*	Design VMA *Vol.%*	Agg. Spec. Surface *M^2/kg*	Agg. G_s	Temp. *℃*	$\|G^*\|/\sin\delta$ *Pa*	M	Allowable Traffic *MESALs*	Flow No.
MA0467	4.6-1	64-28	4.6	6.9	5.8	100	17.0	3.96	2.681	45.2	38,073	3.00	48.4	346
MA0467	5.1-1	64-28	5.1	7.2	4.5	100	16.9	3.96	2.681	45.1	38,073	3.00	31.4	216
MA0467	5.6-1	64-28	5.6	7.2	3.3	100	16.9	3.96	2.681	45.0	38,073	3.00	19.1	171
ME0570	5.4-1	64-28	5.4	4.9	5.8	75	15.3	5.28	2.560	54.3	10,265	3.00	26.6	541
ME0570	5.9-1	64-28	5.9	5.0	4.5	75	15.3	5.28	2.560	54.1	10,265	3.00	17.6	351
ME0570	6.4-1	64-28	6.4	5.0	3.3	75	15.6	5.28	2.560	54.0	10,265	3.00	10.2	148
NC0360	4.5-1	70-22	4.5	8.0	3.9	100	13.9	6.32	2.599	45.0	73,343	1.00	131.1	1121
NC0360	5.0-1	70-22	5.0	7.9	2.6	100	14.0	6.32	2.599	45.1	73,343	1.00	70.5	801
NC0360	5.5-1	70-22	5.5	8.2	1.4	100	14.3	6.32	2.599	45.2	73,343	1.00	22.4	411
NY0466	4.5-1	64-28	4.5	7.1	5.5	100	13.5	4.24	2.618	44.9	44,891	3.00	153.2	1911
NY0466	5.0-1	64-28	5.0	7.0	4.2	100	13.5	4.24	2.618	44.9	44,891	3.00	106.4	841
NY0466	5.5-1	64-28	5.5	7.6	3.0	100	13.2	4.24	2.618	44.9	44,891	3.00	61.1	666
WI0357	4.4-1	64-22	4.4	7.1	6.7	100	15.2	3.64	2.725	31.3	326,035	1.00	478.5	9416
WI0357	4.9-1	64-22	4.9	7.0	5.4	100	15.1	3.64	2.725	31.8	326,035	1.00	364.2	7896
WI0357	5.4-2	64-22	5.4	6.9	4.2	100	15.0	3.64	2.725	31.6	326,035	1.00	255.8	5653
ME0359	5.3 - 1	64-28	5.3	4.7	5.8	75	14.5	3.78	2.599	37.3	96,540	3.00	322.1	5472
ME0359	5.8-2	64-28	5.8	4.8	4.5	75	14.3	3.78	2.599	37.6	96,540	3.00	224.7	4177
ME0359	6.3 - 2	64-28	6.3	4.8	3.3	75	14.4	3.78	2.599	37.5	96,540	3.00	135.1	2356
MN0465	4.8-1	70-28	4.8	7.9	6.0	100	15.5	3.78	2.686	44.7	38,036	3.00	53.7	283
MN0465	5.3-1	70-28	5.3	8.0	4.7	100	15.4	3.78	2.686	44.8	38,036	3.00	37.4	251
MN0465	5.8-1	70-28	5.8	8.0	3.5	100	15.4	3.78	2.686	44.8	38,036	3.00	23.1	186
NE0569	5.0-1	64-28	5.0	7.5	5.0	96	15.8	6.06	2.596	37.8	92,875	1.00	145.0	1083
NE0569	5.5-4	64-28	5.5	7.9	3.7	96	15.6	6.06	2.596	38.0	92,875	1.00	91.3	1294
NE0569	6.0-5	64-28	6.0	7.1	2.5	96	15.8	6.06	2.596	37.7	92,875	1.00	53.7	374
WA0463	5.5-1	64-22	5.5	7.9	4.9	100	14.8	5.26	2.717	44.8	41,895	1.00	45.7	378
WA0463	6.0-2	64-22	6.0	8.2	3.6	100	14.7	5.26	2.717	44.9	41,895	1.00	28.8	239
WA0463	6.5-1	64-22	6.5	7.9	2.4	100	14.5	5.26	2.717	45.0	41,895	1.00	16.5	156

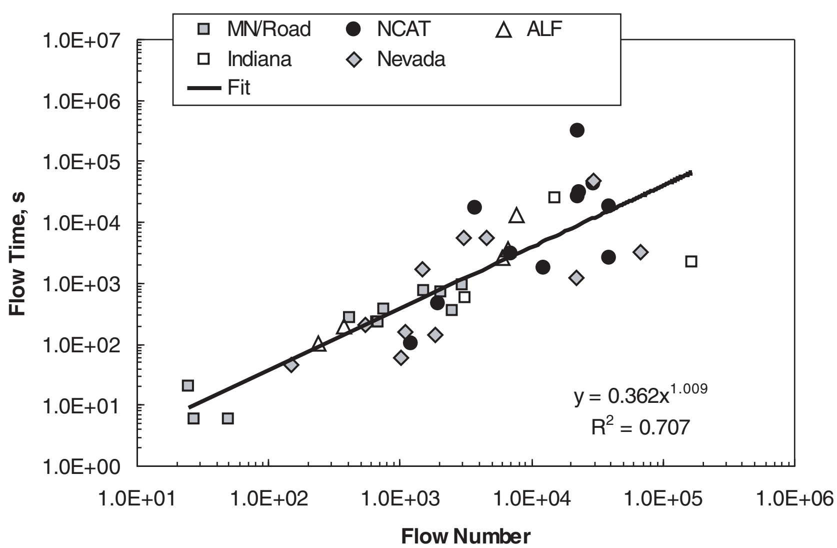

Figure 3. Plot of flow time as a function of flow number for five projects from the NCHRP project 9-19 database (39).

Table 15. Recommended minimum flow time requirements—table 8-21 in the mix design manual.

Traffic Level *Million ESALs*	Minimum Flow Time *s*
< 3	---
3 to < 10	20
10 to < 30	72
≥ 30	280

Table 16. Data from NCHRP project 9-19 used to correlate flow number and flow time (*39*).

Project	Phase	Section	Con. Stress *lb/in²*	Dev. Stress *lb/in²*	Temp. *°C*	Voids *Vol.%*	Flow No.	Flow Time *s*
MN/Road	1	Cell 16	0	207	37.8	**7.7**	2,041	730
MN/Road	1	Cell 17	0	207	37.8	**8.0**	2,482	360
MN/Road	1	Cell 18	0	207	37.8	**5.9**	2,991	935
MN/Road	1	Cell 20	0	207	37.8	**6.1**	659	236
MN/Road	1	Cell 22	0	207	37.8	**6.9**	1,511	770
MN/Road	2	Cell 01	0	173	37.8	**6.5**	683	226
MN/Road	2	Cell 01	0	173	54.4	**6.8**	27	6
MN/Road	2	Cell 03	0	173	37.8	**6.3**	416	267
MN/Road	2	Cell 03	0	173	54.4	**6.6**	49	6
MN/Road	2	Cell 04	0	173	37.8	**6.5**	753	371
MN/Road	2	Cell 04	0	173	54.4	**6.8**	24	21
NCAT	2	E06	0	173	37.8	**6.9**	12,289	1,807
NCAT	2	E06	0	173	54.4	**7.1**	1,926	455
NCAT	2	N02	0	173	37.8	**5.3**	23,181	31,953
NCAT	2	N02	0	173	54.4	**5.5**	6,860	3,024
NCAT	2	N03	0	173	37.8	**5.5**	22,563	324,161
NCAT	2	N03	0	173	54.4	**6.6**	1,211	102
NCAT	2	N05	0	173	54.4	**6.1**	3,682	17,009
NCAT	2	N07	0	173	54.4	**5.8**	22,203	26,677
NCAT	2	N11	0	173	54.4	**6.2**	39,000	2,576
NCAT	2	N12	0	173	37.8	**5.2**	29,605	43,763
NCAT	2	N12	0	173	54.4	**5.6**	39,000	18,230
ALF	2	Cell 05	0	138	54.4	**9.2**	241	104
ALF	2	Cell 07	0	69	54.4	**10.6**	7,761	13,208
ALF	2	Cell 07	0	138	54.4	**10.4**	6,028	2,611
ALF	2	Cell 08	0	138	54.4	**9.3**	6,651	3,542
ALF	2	Cell 09	0	138	54.4	**7.3**	376	202
Indiana	2	4-A 64-28	0	173	54.4	**7.7**	165,931	2,251
Indiana	2	4-B 64-28	0	173	54.4	**3.7**	15,060	25,151
Indiana	2	6-B 64-16 I	0	173	54.4	**6.4**	3,159	589
Nevada	2	HV 64-22b	0	173	37.8	**5.8**	29,497	47,851
Nevada	2	HV 64-22b	0	173	`54.4	**5.9**	1,494	1,676
Nevada	2	HV 64-22T	0	173	54.4	**6.8**	541	205
Nevada	2	HV AC 20-P B	0	173	54.4	**7.7**	1,009	61
Nevada	2	HV AC 20-P T	0	173	37.8	**6.2**	3,071	5,603
Nevada	2	HV AC 20-P T	0	173	54.4	**6.2**	1,858	144
Nevada	2	SP 64-22 B	0	173	54.4	**1.8**	4,531	5,649
Nevada	2	SP 64-22 T	0	173	37.8	**5.8**	22,050	1,210
Nevada	2	SP 64-22 T	0	173	54.4	**5.8**	149	47
Nevada	2	SP AC-20P B	0	173	54.4	**1.8**	67,000	3,228
Nevada	2	SP AC-20P T	0	173	54.4	**5.7**	1,087	159

Table 17. Recommended maximum rut depths for the APA test—table 8-22 in the mix design manual.

Traffic Level *Million ESALs*	Maximum Rut Depth *Mm*
< 3	---
3 to < 10	5
10 to < 30	4
≥ 30	3

Table 17 (Table 8-22 in the Manual) lists recommended minimum rut depths for the Accelerated Pavement Analyzer (APA) test. At the time of the writing of the Manual and Commentary, there was not a large amount of data on the use of the APA as a performance test. In *NCHRP Report 508,* the results of research evaluating the APA were reported (*40*). It was found that although there were reasonable correlations between rut depths determined with the APA and those observed in the field on a project-by-project basis, an overall relationship for multiple projects could not be developed. As a result, *NCHRP Report 508* does not provide specific guidelines for interpreting the results of the APA test (*40*). For the purposes of providing such guidelines in the *Manual,* test procedures and requirements for several states were reviewed (*41, 42, 43*). The most common conditions, as reported by Mr. Chad Hawkins at the 2006 APA User Group Meeting, for running the APA have been reported as a 100 lbf load applied through the hose inflated to a pressure of 100 lb/in²; the rut depth is measured after 8,000 loading cycles (*43*). The most common test temperature is 64°C. These are the conditions used in Oklahoma—the values in

Table 16 are, in fact, the same as those used in Oklahoma's APA specification, although Oklahoma includes requirements at lower traffic levels (*41*). North Carolina's specification uses somewhat higher maximum rut depth values, but its standard specifies a load of 120 lbf and a hose pressure of 120 lb/in^2 (*42*). According to LTTPBind Version 3.1, the typical high-temperature PG binder grade in Oklahoma is very close to 64.0, so the test temperature in their APA standard corresponds to the high temperature binder grade at 98% reliability. This is the basis for the suggested test temperature for the APA equal to the high-temperature binder performance grade specified by the local state highway agency for traffic levels of 3 million ESALs or less.

Establishing guidelines for interpreting the results of the Hamburg wheel tracking test is difficult because the test is not widely used (*43*). The Manual gives test conditions and minimum passes to a half-inch rut depth as specified by the State of Texas (*44*). Test conditions for the Hamburg test as specified by the Texas Department of Transportation are as follows (*44*):

- Specimen dimensions: 150 mm (6 in.) in diameter, 62 ± 2 mm (2.4 in.) thick
- Wheel load: 705 ± 2 N (158 ± 0.5 lb)
- Air void content: 7 ± 1%
- Test temperature: 50 ± 1°C

Test requirements used in Texas—in terms of minimum passes to a 0.5-inch rut depth—are given in Table 18 (Table 8-23 in the Manual). The Manual states that agencies wishing to use the Hamburg device as a performance test should consider doing an engineering study to develop appropriate requirements for their local conditions and materials.

Suggested test values for both maximum permanent shear strain (MPSS) determined using the repeated shear at constant height (RSCH) test and strength measured using the high-temperature indirect tension (HT/IDT) test were calculated in a manner similar to that used to develop suggested requirements for the flow number and flow time test. Data from NCHRP Projects 9-25/9-31 were used in combination with the rutting-resistivity model (Equation 1) to estimate maximum allowable traffic, which was then compared to test values for MPSS and IDT strength (*4, 27*). For both sets of test data, the air void content varied from about 2 to about 6%, with an average of 4.0%. The protocol for the HT/IDT test is to compact specimens using the design gyrations (N_{design}) which is what was done for the 9-25/9-31 tests. However, the protocol for RSCH/MPSS (AASHTO T 320) is to prepare specimens at an air void content of 3.0 ± 0.5%. This requires an adjustment to the MPSS values reported in 9-25/9-31, which was made by developing a relationship between MPSS and allowable traffic at the design voids, calculating the allowable traffic at 3% voids, and then adjusting the MPSS value according to the difference between the estimated allowable traffic values at the design voids and at 3% air voids. As discussed in the analysis of flow number values, in order to avoid rejection of a large number of mixes by performance testing, the traffic level used in estimating the values for both HT/IDT strength and MPSS was the midpoint of the traffic levels for the 3 to 10 million ESALs and 10 to 30 million ESAL ranges; for traffic levels above 30 million ESALs, a value of 65 million ESALs was used to estimate the minimum IDT and maximum MPSS.

Table 18. Texas requirements for Hamburg wheel tracking test—table 8-23 in the mix design manual (*44*).

High-Temperature Binder Grade	Minimum Passes to 0.5-inch Rut Depth
PG 64 or lower	10,000
PG 70	15,000
PG 76 or higher	20,000

A final adjustment to HT/IDT strength is needed because the suggested protocol—testing at a loading rate of 50 mm/min at 10°C below the average 7-day maximum pavement temperature, rather than at 3.75 mm/min at 20°C below the critical high pavement temperature—gives IDT strength values about 10% higher than the original protocol, as used in 9-25/9-31. The relationship between IDT strength using the two protocols is shown in Figure 4, reproduced from a letter report prepared for PennDOT as part of a small research project investigating the IDT strength test (*45*). Figures 5 and 6, respectively, show the relationships between MPSS and IDT strength and allowable ESALs determined from the 9-25/9-31 data. Tables 19 and 20 show recommended values for maximum MPSS and minimum IDT strength as determined from the relationships shown in the two figures. The data on which this analysis is based are summarized in Table 21 (*4*).

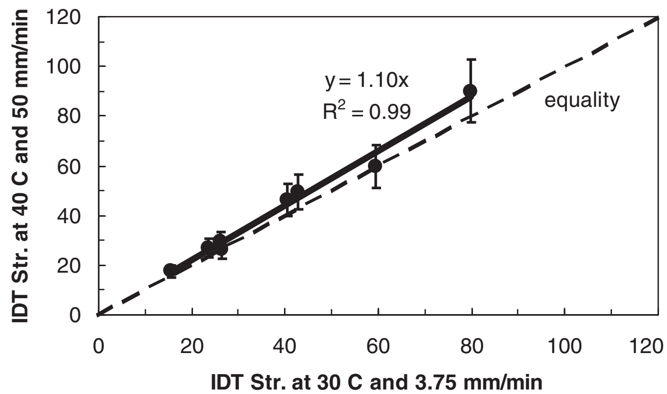

Figure 4. Comparison of IDT strengths from the two procedures (45).

Table 19. Recommended maximum values for MPSS determined using the SST/RSCH test—table 8-24 in the mix design manual.

Traffic Level *Million ESALs*	Maximum Value for MPSS %
< 3	---
3 to < 10	3.4
10 to < 30	2.1
≥ 30	0.8

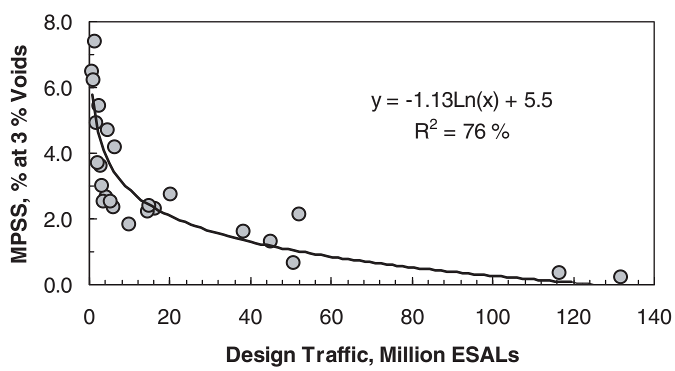

Figure 5. Plot of MPSS vs. allowable million ESALs for NCHRP 9-25/9-31 data.

Table 20. Recommended minimum high-temperature indirect tensile strength requirements—table 8-25 in the mix design manual.

Traffic Level *Million ESALs*	Minimum HT/IDT Strength *kPa*
< 3	---
3 to < 10	270
10 to < 30	380
≥ 30	500

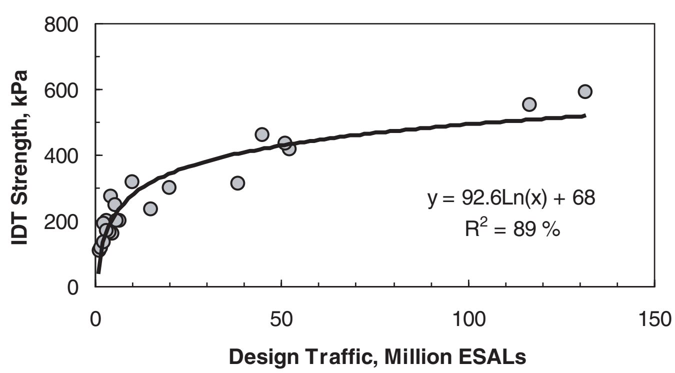

Figure 6. Plot of IDT strength vs. estimated allowable million ESALs for NCHRP 9-25/9-31 data.

Table 21. Data used in developing correlations between maximum permanent shear strain, high-temperature IDT strength and estimated allowable traffic (*4*).

Aggregate, NMAS, Gradation	Binder Grade	Temp. ℃	N_{design} Gyrations	Agg. G_s	Agg. Spec. Surface m^2/kg	Binder $\|G^*\|/\sin\delta$ Pa	VTM Vol.%	VMA Vol.%	MPSS @ 3% Voids %	HT/IDT Strength lb/in^2	Traffic @ 7% Voids MESALs
KY limestone, 19 mm, coarse	PG 64-22	54	100	2660	4.32	14,500	4.40	15.10	149.6	4.75	5.59
KY limestone, 19 mm, coarse	PG 64-22	54	50	2660	4.32	14,500	3.30	17.40	114.1	7.97	0.78
KY limestone, 19 mm, dense	PG 64-22	54	100	2648	4.75	14,500	3.60	12.10	226.8	3.92	13.14
KY limestone, 19 mm, dense	PG 64-22	54	50	2648	4.75	14,500	4.00	13.60	182.0	4.83	3.68
Crushed gravel, 19 mm, coarse	PG 64-22	54	100	2566	3.76	14,500	3.20	14.00	202.0	3.62	2.91
Crushed gravel, 19 mm, coarse	PG 64-22	54	75	2566	3.76	14,500	4.20	14.90	192.1	3.71	2.29
Crushed gravel, 19 mm, dense	PG 64-22	54	100	2575	4.32	14,500	4.40	12.80	316.9	1.83	10.09
Crushed gravel, 19 mm, dense	PG 64-22	54	75	2575	4.32	14,500	3.40	13.00	273.5	2.64	4.31
VA limestone, 9.5 mm, coarse	PG 64-22	54	100	2671	4.40	14,500	4.30	15.80	160.0	4.71	4.76
VA limestone, 9.5 mm, coarse	PG 64-22	54	50	2671	4.40	14,500	3.50	18.20		6.46	0.75
VA limestone, 9.5 mm, coarse	PG 76-16	54	100	2671	4.40	66,200	4.30	15.80	314.8	1.61	38.29
VA limestone, 9.5 mm, coarse	PG 58-28	54	100	2671	4.40	4,880	4.30	15.80	108.6	6.21	1.07
VA limestone, 9.5 mm, fine	PG 64-22	54	100	2659	6.21	14,500	4.50	18.70	199.6	4.19	6.48
VA limestone, 9.5 mm, fine	PG 64-22	54	50	2659	6.21	14,500	3.90	20.30		7.39	1.44
VA limestone, 9.5 mm, fine	PG 76-16	54	100	2659	6.21	66,200	4.50	18.70	416.8	2.12	52.14
Granite, 12.5 mm, dense	PG 64-22	54	125	2632	4.79	14,500	4.20	13.10		2.33	16.37
Granite, 12.5 mm, dense	PG 76-16	54	125	2632	4.79	66,200	4.20	13.10	589.6	0.24	131.71
Granite, 12.5 mm, dense	PG 58-28	54	125	2632	4.79	4,880	4.20	13.10	164.8	2.54	3.67
Granite, 12.5 mm, fine	PG 64-22	54	125	2631	6.15	14,500	3.50	14.90		2.20	14.49
Granite, 12.5 mm, fine	PG 76-16	54	125	2631	6.15	66,200	3.50	14.90	550.6	0.33	116.59
Granite, 12.5 mm, fine	PG 58-28	54	125	2631	6.15	4,880	3.50	14.90	171.4	3.00	3.25
VA limestone, 9.5 mm, coarse	PG 64-22	60	100	2671	4.40	6,920	4.30	15.80	116.2	4.90	1.72
VA limestone, 9.5 mm, coarse	PG 76-16	60	100	2671	4.40	33,100	4.30	15.80	235.5	2.39	14.78
VA limestone, 9.5 mm, fine	PG 64-22	60	100	2659	6.21	6,920	4.50	18.70	133.1	5.42	2.35
VA limestone, 9.5 mm, fine	PG 76-16	60	100	2659	6.21	33,100	4.50	18.70	300.6	2.73	20.13
Granite, 12.5 mm, dense	PG 64-22	60	125	2632	4.79	6,920	4.20	13.10	198.2	2.36	5.93
Granite, 12.5 mm, dense	PG 76-16	60	125	2632	4.79	33,100	4.20	13.10	436.1	0.65	50.85
Granite, 12.5 mm, fine	PG 64-22	60	125	2631	6.15	6,920	3.50	14.90	247.2	2.51	5.25
Granite, 12.5 mm, fine	PG 76-16	60	125	2631	6.15	33,100	3.50	14.90	460.8	1.30	45.01

The final critical part of the mix design procedure as described in the Manual is adjustment of the performance test temperature to account for slow traffic speeds. This is necessary because as traffic speed decreases, permanent deformation can significantly increase. Three approaches are possible to account for this effect: (1) increase or decrease required performance test value as traffic speed decreases; (2) decrease test loading rate as traffic speed decreases; or (3) increase test temperature as traffic speed decreases. There is not enough data at this time to use the first approach and it would also be a relatively complicated approach. The second approach is not feasible, since the loading rate on some of the proposed test methods is fixed and is not normally varied. Therefore, the third approach is suggested. The *Manual* suggests increasing the test temperature 6°C for slow traffic and 12°C for very slow traffic. These adjustments were calculated directly from Equation 6 given above, using a traffic speed of 70 kph for fast traffic, 25 kph for slow traffic and 10 kph for very slow traffic.

CHAPTER 9

Reclaimed Asphalt Pavement

Chapter 9 of the Manual deals with incorporation of reclaimed asphalt pavement (RAP) into HMA mix designs. This is a complicated topic potentially involving numerous calculations:

1. Calculation of the blended aggregate gradation, including contribution from the RAP.
2. Calculation of the binder content, again including contribution from the RAP.
3. Calculation of the blended binder grade, based upon both the new binder added and the binder contributed from the RAP.
4. Calculation of the required new binder grade needed to achieve a specified binder grade, given a certain RAP content and a binder grade for the RAP binder.
5. Calculation of the minimum and maximum RAP that can be used in a mix, given a new binder grade and a grade for the RAP binder.
6. Estimation of the variability in aggregate gradation and binder content, given a job mix formula (JMF) containing a certain amount of RAP.
7. Estimation of the maximum amount of RAP that can be used in a mix without exceeding typical limits on variability of production, given variability in the RAP stockpiles being used.

The mathematics of some of these calculations—in particular the variability calculations—can be challenging. For this reason, HMA Tools has been designed to perform these calculations, and the Manual in general simply discusses how to use this spreadsheet to perform the needed calculations during the mix design process. This avoids having to show and document some involved equations. The Commentary for Chapter 9 therefore involves showing and describing the equations used in HMA Tools to perform these RAP calculations, along with any associated assumptions and/or simplifications.

Chapter 9 is structured for the most part as a series of example problems of increasing complexity. The section below, on critical tables, figures, and equations therefore is mostly organized on the basis of these examples, presenting and describing the critical information used in solving each example problem. Calculations involving RAP binder properties are based on Appendix A of AASHTO M 323 and are not documented in detail here.

Much of what is contained in Chapter 9 of the Manual has been based on *NCHRP Report 452 (46)*. This is an excellent reference for technicians and laboratory engineers responsible for the design and/or analysis of HMA mix designs containing RAP.

Example 1. Gradation and Binder Content Analysis for an HMA Mixture Containing RAP

The computation of blends for mixtures incorporating RAP is a little different than that for mixtures made with all new stockpiles. When RAP is used, the RAP material that is added includes both the RAP aggregate and the RAP binder. Since gradation data are based on the weight of aggregate, and binder contents are based on the total weight, the stockpile percentages must be adjusted for combined gradation analysis based on the amount of binder contained in the RAP. The binder included in the RAP is computed from Equation 8:

$$wb_{RAP_i} = ps_{RAP_i} \times \frac{Pb_{RAP_i}}{100} \tag{8}$$

where

wb_{RAP_i} = weight of RAP binder from RAP stockpile i, wt%
ps_{RAP_i} = percentage of RAP stockpile i in the total blend,%
Pb_{RAP_i} = binder content of RAP stockpile i, wt%

The total weight of binder contributed by all RAP stockpiles is the sum of the weight contributed by each RAP stockpile and is computed from Equation 9:

$$wb_{RAPTotal} = \sum_{i=1}^{j} wb_{RAP_i} \tag{9}$$

where

$wb_{RAP_{Total}}$ = total weight of RAP binder from all RAP stockpiles, weight%
wb_{RAP_i} = weight of RAP binder from RAP stockpile i, wt%
j = total number of RAP stockpiles

The total binder content for the mix is simply calculated by adding the weight of binder contributed by all RAP stockpiles to the weight of new binder added.

For gradation analysis, the percentage of each stockpile based on the total weight of aggregate is needed. The percentage of each new aggregate stockpile based on the total weight of aggregate is given by Equation 10:

$$p_{new_k} = \frac{ps_{new_k}}{\left(100 - wb_{RAPTotal}\right)} \times 100\% \tag{10}$$

where

P_{new_k} = percentage of new aggregate k, weight% of total aggregate
ps_{new_k} = percentage of new aggregate stockpile k in the total blend, weight%
$wb_{RAP_{Total}}$ = total weight of RAP binder from all RAP stockpiles, weight%

The percentage of each RAP aggregate based on the total weight of aggregate is given by Equation 11:

$$p_{RAP_i} = \left[\frac{ps_{RAP_i} - wb_{RAP_i}}{\left(100 - wb_{RAPTotal}\right)}\right] \times 100\% \tag{11}$$

where

p_{RAP_i} = percentage of RAP aggregate i, weight% of total aggregate
ps_{RAP_i} = percentage of RAP stockpile i in the total blend, weight%
wb_{RAP_i} = weight of RAP binder from RAP stockpile i, weight%
$wb_{RAP_{Total}}$ = total weight of RAP binder from all RAP stockpiles, weight%

For each sieve, the gradation of the blend of the stockpiles is then computed using the percentage of each stockpile based on the total weight of aggregate using Equation 12:

$$tpp = \sum_{k=1}^{n} \left(\frac{p_{new_k}}{100} \times pp_k \right) + \sum_{i=1}^{j} \left(\frac{p_{RAP_i}}{100} \times pp_i \right) \tag{12}$$

where

tpp = total percent passing a given sieve, weight% of total aggregate
P_{new_k} = percentage of new aggregate k, weight% of total aggregate
p_{RAP_i} = percentage of RAP aggregate i, weight% of total aggregate
pp_k = percent passing a given sieve for new aggregate k, weight%
pp_i = percent passing a given sieve for RAP aggregate i, weight%
j = number of RAP stockpiles
n = number of new stockpiles

Example 2. Calculation of Mean, Standard Deviation, and Maximum Allowable RAP Content for a Single RAP Stockpile

The mean is calculated using Equation 13 (*47*):

$$\bar{X} = \sum_{i=1}^{n} \frac{X_i}{n} \tag{13}$$

where

$\bar{X}$ = stockpile average
X_i = result for location i
n = total number of locations tested

The standard deviation is calculated using Equation 14 (*47*):

$$s = \sqrt{\frac{\sum_{i=1}^{n} \left(X_i - \bar{X}\right)^2}{n-1}} \tag{14}$$

where

s = standard deviation
$\bar{X}$ = stockpile average
X_i = result for location i
n = total number of locations tested

Derivation of the procedure for calculation of the maximum allowable RAP content is as follows. ASTM D 4460 gives equations for calculating standard deviation values for quantities determined from calculations involving two other values. From these equations, the following formula for calculating the standard deviation of a blend of two materials can be derived:

$$\sigma_m = \sqrt{\alpha^2\sigma_a^2 + (1-\alpha)^2\sigma_b^2 + \left(\bar{X}_a^2 + \bar{X}_b^2\right)\sigma_a^2} \tag{15}$$

where

σ_m = standard deviation of the mixture
σ_a = standard deviation of component "a"
σ_b = standard deviation of component "b"
α = proportion of component "a" in the mixture
$\bar{X}_a$ = mean value for component "a"
$\bar{X}_b$ = mean value for component "b"
σ_α = standard deviation of the proportions

We can rewrite this for percent passing for a selected sieve for HMA mixtures consisting of a blend of new HMA materials with RAP:

$$\sigma_{PM} = \sqrt{w_R^2\sigma_{PR}^2 + w_N^2\sigma_{PN}^2 + \left(\bar{P}_R^2 + \bar{P}_N^2\right)\sigma_w^2} \tag{16}$$

where

σ_{PM} = standard deviation of percent passing for a selected sieve for the mixture with RAP
w_R = weight fraction of RAP in the mixture
σ_{PR} = standard deviation of percent passing for the selected sieve for the RAP
w_N = weight fraction of new materials (new HMA) in the mixture
= $(1 - w_R)$
σ_{PN} = standard deviation of percent passing for the selected sieve for the new HMA
$\bar{P}_R$ = mean value for RAP% passing for the selected sieve
$\bar{P}_N$ = mean value for new HMA% passing for the selected sieve
σ_w = standard deviation of the weight fractions, also called "batching variability,"

Equation 17 can be solved for the maximum amount of RAP that can be added to new material without increasing the standard deviation for percent passing on the selected sieve above a selected maximum value by application of the quadratic equation

$$Max.\ RAP\% = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \times 100\% \tag{17}$$

where

$Max.\ RAP\%$ = maximum amount of RAP that can be added to the mix, weight%
$a = \sigma_{PR}^2 + \sigma_{PN}^2$
$b = -2\,\sigma_{PN}^2$
$c = \sigma_{PN}^2 + (\bar{P}_{PN}^2 + \bar{P}_N^2)\,\sigma_w^2 - \sigma_{PM/Max}^2$
and $\sigma_{PM/Max}$ is the maximum allowable standard deviation for percent passing for the selected sieve.

Equations 1 and 2 can be rewritten for asphalt content rather than for aggregate percent passing, calculated using the following equation:

$$\sigma_{BM} = \sqrt{w_R^2\sigma_{BR}^2 + w_N^2\sigma_{BN}^2 + \left(\bar{B}_R^2 + \bar{B}_N^2\right)\sigma_w^2} \tag{18}$$

where

σ_{BM} = standard deviation of binder content (weight%) for the mixture with RAP
w_R = weight fraction of RAP in the mixture
σ_{BR} = standard deviation of binder content (weight%) for the RAP
w_N = weight fraction of new materials (new HMA) in the mixture
= $(1 - w_R)$
σ_{BN} = standard deviation of binder content (weight%) for the new HMA
$\bar{B}_R$ = mean value for the RAP binder content, weight%
$\bar{B}_N$ = mean value for new HMA binder content, weight%
σ_w = standard deviation of the weight fractions or "batching variability"

As for Equation 1, Equation 3 can also be solved for the maximum amount of RAP that can be added to new material without increasing the variability above a selected maximum by application of the quadratic equation:

$$Max.\ RAP\% = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \times 100\% \tag{19}$$

where

Max. RAP% = maximum amount of RAP that can be added to a mix without increasing the production variability, weight%

$a = \sigma_{BR}^2 + \sigma_{BN}^2$

$b = -2\sigma_{BN}^2$

$c = \sigma_{BN}^2 + (\bar{B}_R^2 + \bar{B}_N^2)\sigma_w^2 - \sigma_{BM/Max}^2$

and $\sigma_{BM/Max}$ is the maximum allowable standard deviation for binder content for the entire mix, including RAP.

The approach used in the Manual in applying Equations 17 and 19 to calculate the maximum allowable RAP in an HMA mix design is to assume that the maximum allowable standard deviation for the final HMA should be no larger than the standard deviation for the new materials in the mix. This is equivalent to saying that the amount of RAP should be limited so that the overall variability in HMA production is not increased above what it would normally be without the addition of any RAP. This approach simplifies some of the calculations, and as discussed below, if typical variability in aggregate% passing and binder contents are assumed, the mix designer can determine the maximum allowable RAP without knowing the existing HMA variability or the maximum allowable variability desired by the producer. The maximum amount of RAP that can be added to a mix simply becomes a function of the RAP variability.

A critical issue in this approach is what values to use for typical standard deviations for HMA production when applying Equations 17 and 19. The simplest and most conservative approach is to base the standard deviation values on information given in ASTM D 3515: *Standard Specification for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures.* Although this document does not specify standard deviations for aggregates and asphalt binder, it does give tolerances; these are shown in Table 22. The question becomes what is the ratio between these tolerances and typical standard deviations? The standard deviation values corresponding to those given in ASTM D 3515 must be much smaller than the tolerances, otherwise, plants would often violate these specified tolerances. For example, if typical standard deviation values were one-fourth the specified tolerance, HMA plants would exceed these limits one time in 20. This is probably too frequent; a somewhat smaller standard deviation is more likely—one-fifth of the specified tolerance range appears to provide reasonable estimates of typical standard deviation values in well-run HMA plants (typical variability of HMA production is discussed in more detail in Chapter 12 of the Manual and the Commentary). The resulting estimated typical standard deviations are also shown in Table 22.

A second complication arises from the uncertainty in estimating standard deviations. When several samples of RAP are taken and used to estimate the standard deviation, the resulting value is an estimate. In fact, unless the number of replicate samples is about 30 or higher, the uncertainty

Table 22. Production Tolerances for Hot Mix Plants as Given in ASTM D 3515.

Sieve Size	Tolerance	Typical Standard Deviation
> 12. 5 mm	± 8.0%	3.2%
4.75 and 9.5 mm	± 7.0%	2.8%
1.18 and 2.36 mm	± 6.0%	2.4%
0.300 and 0.600 mm	± 5.0%	2.0%
0.150 mm	± 4.0%	1.6%
0.075 mm	± 3.0%	1.2%
Asphalt binder	± 0.5%	0.2%

in the estimate can be quite large. To be conservative and to make certain that the variability in plant production will not be increased to an unacceptable level by the addition of RAP, an upper confidence limit for the standard deviation should be used, rather than the actual calculated value. This upper confidence limit is calculated using the following equation (*47*):

$$U = \sqrt{\frac{(n-1)s^2}{\chi^2(\alpha;n-1)}} \tag{20}$$

where

U = upper confidence limit for the standard deviation at 1–α confidence level; use in place of σ_{PR} and/or σ_{BR} in Equations 2 and 4.
n = number of measurements used in estimating the standard deviation s
χ^2 = chi-squared distribution with confidence level 1–α and n–1 degrees of freedom

A value for α of 0.20 (an 80% confidence level) appears to provide reasonable values for U and the resulting calculated maximum allowable RAP contents. One advantage to using an upper confidence limit U for the standard deviation, rather than the actual estimate, is the effect of sample size on the resulting value of U—the larger the number of RAP samples used to estimate the standard deviation, the lower will be the value of U and the greater the resulting maximum allowable RAP content. Producers who take large numbers of RAP samples to characterize their stockpile will be rewarded, while those using only a few samples will be severely limited in how much RAP they can use in their mixtures. For example, for the case where $\sigma_{PN} = \sigma_{PM/max} = 2.0$, and σ_{PR} is estimated to be 4.0, if the RAP standard deviation is based on $n = 5$ samples, the maximum allowable RAP content is 17%. However, if the RAP standard deviation is based on $n = 10$ samples, the maximum allowable RAP content increases to 27%.

A third important question in applying the equations for analyzing the effect of RAP on HMA production variability is what is a typical value for the standard deviation of the proportions, also called "batching variability." Information is given in *ASTM D 995: Standard Specification for Mixing Plants for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures* on the required accuracy of aggregate scales, control of automated plants, and related information that can be used to establish typical blending standard deviations for HMA plants:

- The accuracy of metering of asphalt binder must be within 1.0% when compared to another metering device or to within 0.5% when compared to test weights. Assuming a ±2 s precision applies to the comparison with test weights, this implies a blending standard deviation for asphalt binders of 0.25% or 0.0025.
- Aggregate scales for batch plants must also be accurate to within 0.5% when compared to test weights, again, implying a blending standard deviation of 0.25% or 0.0025.
- Automatic proportioning systems, for both batch plants and drum plants, must batch aggregate (other than mineral filler) to within ±1.5% of the total batch weight, or for continuous drum plants, to within ±1.5% of the mix production per drum rotation/unit time. Assuming that this tolerance refers to comparison with test weights or similarly accurate reference, and that a ±2 s precision is implied, this translates to a blending standard deviation of 0.75% or 0.0075. The required tolerance for mineral filler is ±0.5%, implying a blending standard deviation of 0.0025. For asphalt binder, the required tolerance is ±0.1%, implying a blending standard deviation of 0.0005.

Assuming that the tolerance for adding RAP to a mix is similar to that for aggregate, and conservatively applying the standard deviation calculated from the required tolerance for automated plants, the blending standard deviation for analyzing RAP variability should be 0.0075. It should be noted that this is based on the maximum permitted tolerance in automated plants and is thus

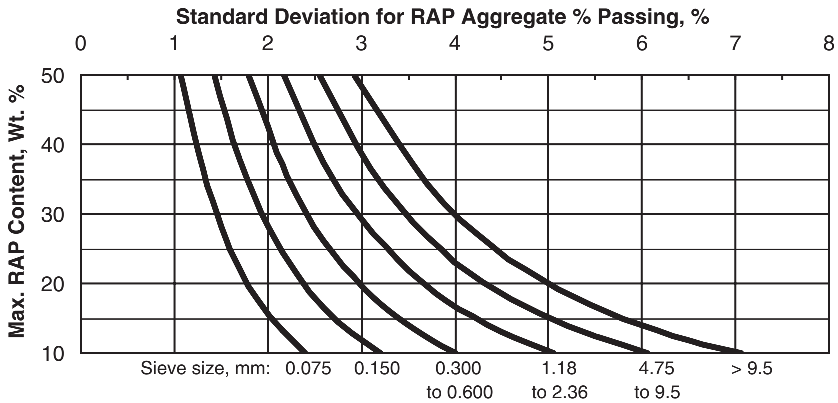

Figure 7. Maximum RAP content as a function of RAP aggregate sieve size and standard deviation (Figure 9-3 in the Mix Design Manual).

a conservative assumption. The actual blending variability in many plants will probably be lower than this.

HMA Tools calculates mean, standard deviation, the upper confidence limit for standard deviation, and the maximum allowable RAP content based on variability for up to four different RAP stockpiles. This analysis uses Equations 13 through 20 along with the assumptions described concerning typical variability in HMA production and batching variability.

Example 3. Determination of Maximum Allowable RAP Content Based on Variability Analysis Using the Graphical Approach

This example problem is essentially identical to the previous one, but in this case a simplified graphical approach is used in determining the maximum allowable RAP. This approach involves the use of four charts—Figures 9-3 through 9-6 in the Manual—to determine the maximum allowable RAP content. These charts are reproduced here for the convenience of the reader. The charts have been developed based on the analysis described above, with a sample size of N = 5; the sample size is small because it has been assumed that those producers wishing to use this simplified approach would probably not want to use large sample sizes of RAP. Figures 7 and 8 (9-3 and

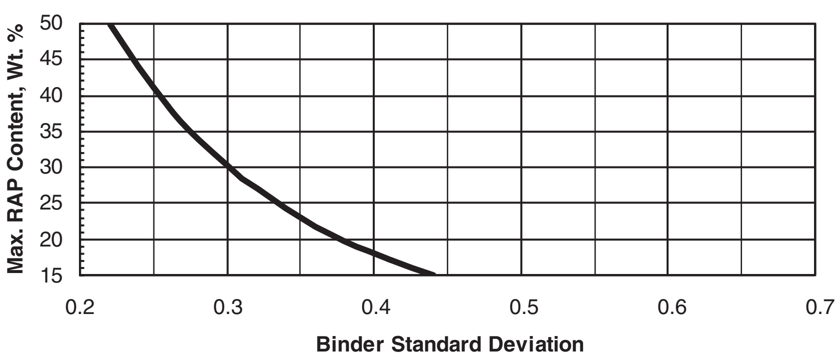

Figure 8. Maximum RAP content as a function of standard deviation for asphalt binder content (Figure 9-4 in the Mix Design Manual). *For n = 5 Samples from a single RAP stockpile.*

9-4 in the Manual) are for the case where only a single RAP stockpile is used in a mix design. The development of these charts was straightforward, involving a direct application of the equations given above. As an example of using these charts, if a given RAP stockpile has a standard deviation of 4.0% for aggregate passing the 0.60 mm sieve, the maximum allowable RAP content can be found from Figure 7 to be 30%. It must be emphasized that Figure 7 is based on standard deviations calculated using at least 5 samples—it should not be applied to standard deviations calculated using a lower number of samples. It can be used for standard deviations calculated using larger samples, but the results will be overly conservative. Also, in developing these figures it is assumed that the difference in percent passing between the RAP aggregate and the new aggregate will not exceed these limits: 30% for mineral filler; 40% for passing the 0.150 mm sieve, and 50% for all other aggregate sizes. If the differences exceed these limits, the difference term in Equations 15 through 18 starts to become significant and must be considered in the calculation. In using Figure 7 in an actual mix design, all aggregate sizes would be evaluated, and the overall maximum allowable RAP content would be the lowest value calculated for all sieve sizes. The maximum RAP content based on asphalt binder standard deviation should also be evaluated (Figure 8, or Figure 9-4 in the Manual)—again, the lowest calculated of these maximum RAP contents should be applied.

Figures 7 and 8 will in general not be accurate when more than one RAP stockpile is used in a mix design. This is because as RAP stockpiles are blended, the variability of the resulting stockpile will be reduced. The more stockpiles in a blend, the lower will be its variability when compared to the average variability for the materials in the blend. The standard deviation for % passing and binder content for a blend of three or more RAP stockpiles can be accurately estimated by sequential application of Equations 15 and 17, respectively. This calculation is done as follows. The standard deviation for % passing for a blend of Stockpiles 1 and 2 is first calculated. Then, the standard deviation is calculated for a second blend, made up of stockpile 3 and the blend of Stockpiles 1 and 2. If a fourth stockpile is used, the standard deviation for a blend of Stockpile 4 and the blend of Stockpiles 1, 2, and 3 is calculated. This is done for both aggregate % passing and binder content. This approach is very accurate and is used in HMA Tools in calculating the standard deviation values for RAP blends involving more than two stockpiles. Unfortunately, constructing graphs similar to Figures 7 and 8 for this situation is somewhat complicated. The approached used for the Manual was to develop an empirical relationship between the average standard deviation of a blend of stockpiles and the standard deviation calculated using Equations 15 and 17. The data was generated using a Monte Carlo approach. A total of 500 data points were generated, with simulated RAP stockpile blends composed of from two to four separate stockpiles, having a wide range of standard deviations along with a wide range of blend compositions. The relationships between average standard deviation and calculated standard deviation for % passing is shown in Figure 9, and for binder content in Figure 10. These plots include the regression function for predicted standard deviation, along with the 80% upper prediction limit. In order to provide a conservative estimate of standard deviation, the upper prediction limits were used in generating the charts for determining maximum RAP content in HMA designs using more than one stockpile. The equation for estimating the 80% upper prediction limit for standard deviation of a blend of RAP stockpiles for aggregate% passing is as follows:

$$SD(PP,\ 80\%\ UPL) = 0.70 \times \overline{SD}^{1.023} \tag{21}$$

where

$SD(PP,\ 80\%\ UPL)$ = Estimated standard deviation for % passing for the overall RAP stockpile blend, 80% upper prediction limit

$\overline{SD}$ = average standard deviation for % passing for the RAP stockpile blend

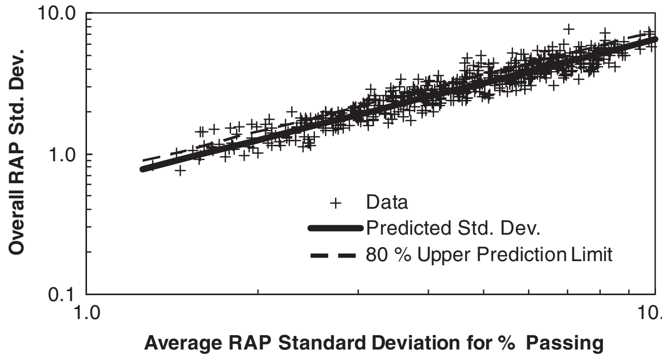

Figure 9. Relationship between average RAP standard deviation for % passing and calculated overall RAP standard deviation for % passing.

The 80% upper prediction limit for standard deviation of the binder content for a blend of RAP stockpiles can be estimated using the following equation:

$$SD(BC,\ 80\%\ UPL) = 0.69 \times \overline{SD}^{0.973} \tag{22}$$

where

$SD(BC, 80\% \ UPL)$ = Estimated standard deviation for binder content (weight%) for the overall RAP stockpile blend, 80% upper prediction limit

$\overline{SD}$ = average standard deviation for binder content (weight %) for the RAP stockpile blend

Using Equations 21 and 22, Figures 11 and 12 were developed, respectively, showing maximum allowable RAP content as a function of average RAP standard deviation values. It should be noted that these charts are conservative and like Figures 7 and 8 are based on standard deviation values calculated using N = 5 independent samples of RAP. HMA Tools uses the pertinent equations directly, without any assumptions or simplifications. HMA Tools will therefore provide more accurate estimates of maximum allowable RAP contents. Furthermore, in general, the estimated maximum RAP contents found using HMA Tools will be somewhat larger than those found with the charts. This is especially true when more than five samples of RAP are used in estimating standard deviation values, when more than one RAP is used in a mix design, or both.

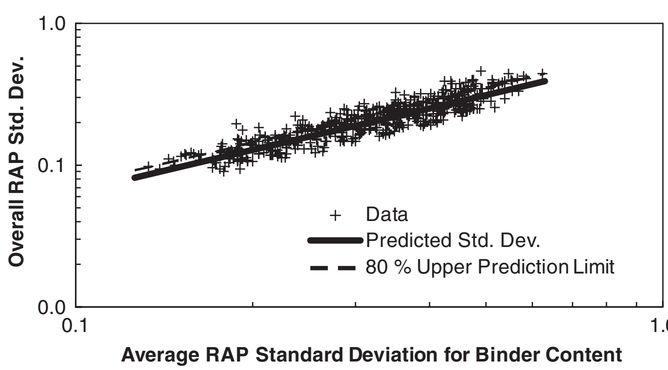

Figure 10. Relationship between average RAP standard deviation for binder content and calculated overall RAP standard deviation for binder content.

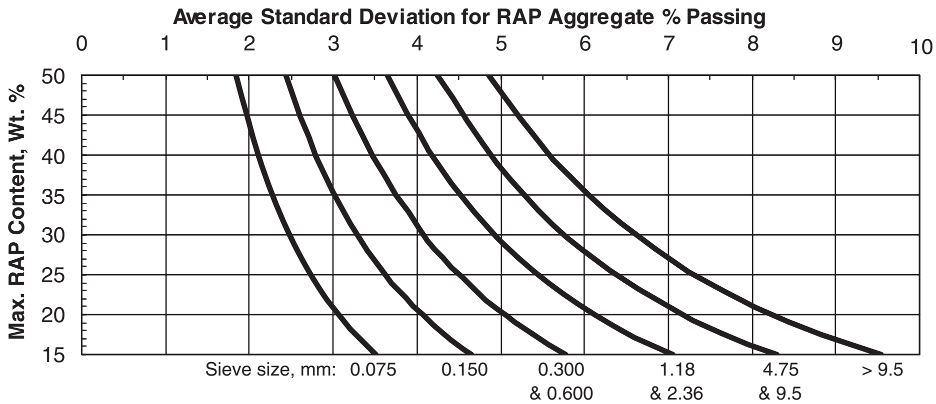

Figure 11. Maximum RAP content as a function of average standard deviation for aggregate % passing (Figure 9-5 in the Mix Design Manual). *For n = 5 Samples from a blend of RAP stockpiles, and no stockpile making up more than 70% of the RAP blend.*

Calculation of Aggregate Specific Gravity Values for RAP Stockpiles.

The bulk specific gravity of each aggregate stockpile used in an HMA mixture is needed for the computation of the voids in the mineral aggregate (VMA). Two methods can be used to determine the bulk specific gravity of the RAP aggregate (*46*). The first is to estimate the bulk specific gravity of the RAP aggregate from the RAP binder content, the maximum specific gravity of the RAP, and estimates of the binder absorption in the RAP and the specific gravity of the RAP binder. The second is to measure the bulk specific gravity of the coarse and fine fraction of the RAP aggregate after removing the binder with the ignition oven or solvent extraction. Details of these approaches are discussed below (*46*).

Estimating RAP Aggregate Bulk Specific Gravity

In this approach, the maximum specific gravity of the RAP is measured in accordance with AASHTO T 209. The maximum specific gravity is measured on a sample split from the representative sample formed for the RAP aggregate and binder analysis. The measured maximum specific gravity, the average RAP binder content from the variability analysis, and an estimate of the RAP binder specific gravity are then used to calculate the effective specific gravity of the RAP aggregate using Equation 23 (Equation 9-1 in the Manual) (*46*):

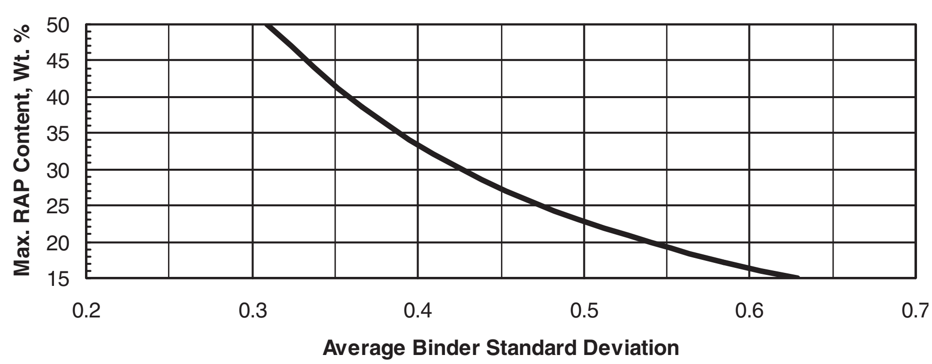

Figure 12. Maximum RAP content as a function of average standard deviation for asphalt binder content (Figure 9-6 in the Mix Design Manual). *For n = 5 Samples from a blend of RAP stockpiles, and no stockpile making up more than 70% of the RAP blend.*

$$G_{se} = \frac{(100 - P_b)}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)} \tag{23}$$

where

G_{se} = effective specific gravity of the RAP aggregate
G_{mm} = maximum specific gravity of the RAP measured by AASHTO T 209
P_b = RAP binder content, wt %
G_b = estimated specific gravity of the RAP binder

The bulk specific gravity of the RAP aggregate can then be estimated from Equation 24 (Equation 9-2 in the Manual), which is a rearranged version of the equation used in volumetric analysis to compute asphalt absorption.

$$G_{sb} = \frac{G_{se}}{\left(\frac{P_{ba} G_{se}}{100 \times G_b}\right) + 1} \tag{24}$$

where

G_{sb} = estimated bulk specific gravity of the RAP aggregate
G_{se} = effective specific gravity of the RAP aggregate from Equation 23
P_{ba} = estimated binder absorption for the RAP, wt% of aggregate
G_b = estimated specific gravity of the RAP binder

The overall error associated with this analysis is difficult to quantify. It depends on the precision of the maximum specific gravity measurement, the accuracy of the RAP binder content measurement, and the estimated RAP binder absorption and specific gravity. As shown in the analysis below, the accuracy of the RAP binder content in turn depends on the accuracy of the correction factor that was used to analyze the ignition oven data.

The single-operator precision of the maximum specific gravity test, AASHTO T 209, is 0.011 when the dry-back procedure is not required and 0.018 when it is. These are somewhat better than the single-operator precision of the aggregate bulk specific gravity tests which are 0.032 for fine aggregate (AASHTO T 84) and 0.025 for coarse aggregate (AASHTO T 85). The potential error associated with estimating the bulk specific gravity of the RAP binder is small. For a typical mixture it is only ±0.002 for a ±0.010 error in the bulk specific gravity of the binder. Potential errors associated with errors in the RAP binder content or the RAP binder absorption are significantly larger. These errors are shown in Figure 13 for RAP having a maximum specific gravity of 2.500, a total binder content of 4.0%, and binder absorption of 0.5%. In this case underestimating the absorbed binder by 0.3% results in an overestimation of the bulk specific gravity of the RAP aggregate of 0.020. Underestimating the total binder content of the RAP by 0.5% results in an underestimation of the bulk specific gravity of the RAP aggregate of 0.021.

Thus the accuracy of estimating the RAP aggregate specific gravity from the maximum specific gravity and binder content of the RAP depends mostly on the accuracy of the estimated correction factor used to determine binder content with the ignition oven and the accuracy of the assumed binder absorption. The correction factor for the ignition oven should not be in error by more than 0.3% and the assumed binder content should not be in error by more than 0.2% to obtain estimated RAP aggregate specific gravity values with similar accuracy as those measured in AASHTO T 84 and T 85. As discussed earlier, correction factors for the ignition oven can be established by performing both the ignition oven and solvent extraction analyses on split samples from at least three locations in the RAP stockpile.

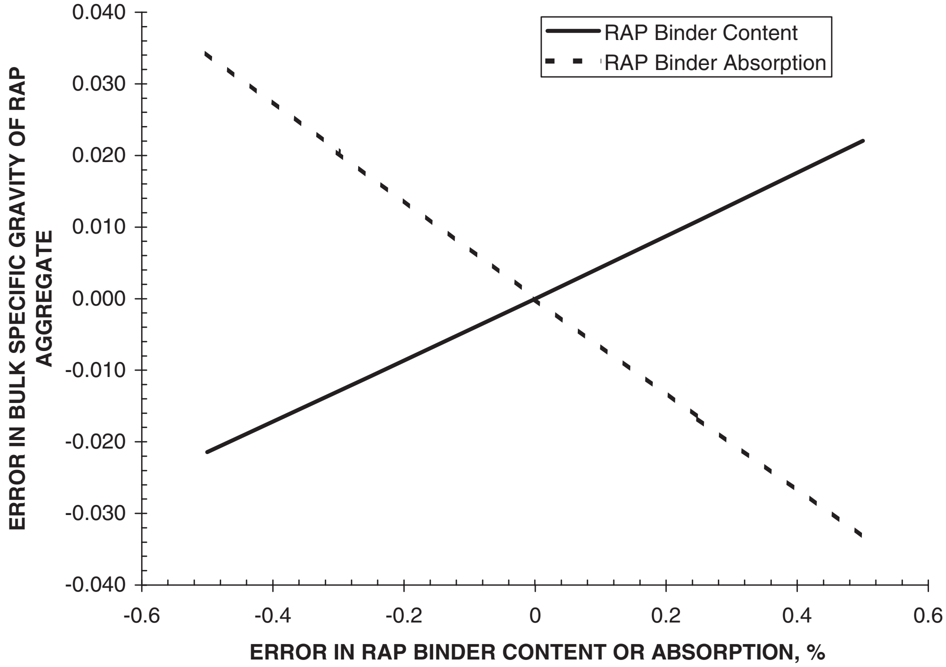

Figure 13. Potential errors in bulk specific gravity of the RAP aggregate for errors in RAP binder content and binder absorption (Figure 9-7 in the Mix Design Manual).

Measuring RAP Aggregate Specific Gravity

If a reasonable estimate of the binder absorption for the RAP is not available, the specific gravity of the RAP aggregate can be measured after removing the RAP binder using an ignition oven or solvent extraction. The specific gravities of the coarse and fine fractions of the RAP aggregate are measured in accordance with AASHTO T 85 and AASHTO T 84, respectively.

HMA Tools and RAP Aggregate Specific Gravity

HMA Tools has been designed so that either approach can be used to estimate RAP aggregate specific gravity values. If the specific gravity values are to be estimated from maximum theoretical specific gravity, binder content, and related information, the data is entered in cells C6:F9 in worksheet “RAP_Aggregates.” If actual measured values for RAP aggregate specific gravity are used, these are entered in cells C11:F14. The calculated values for bulk and apparent specific gravity for each of up to four RAP stockpiles then appear in cells C16:F17. If data for both methods are entered in the worksheet, HMA Tools will use the measured aggregate specific gravity values in estimating the RAP specific gravity values. The estimated water absorption for each RAP stockpile appears in cells C18:F18.

RAP Binder Properties

The section in the Manual on RAP binder properties is based on information and equations given in Appendix A of AASHTO M 323. The various equations and calculations described herein have been implemented in HMA Tools. Using HMA Tools to perform calculations related to RAP binder properties should give results identical to manual calculations carried out following the instructions given in Appendix A of AASHTO M 323.

CHAPTER 10

Design of Gap-Graded HMA Mixtures

Chapter 10 is based largely on the AASHTO Standards M 325-08: Standard Specification for Stone Matrix Asphalt and R 46-08: Standard Practice for Designing Stone Matrix Asphalt. However, there are several significant differences between the information provided in Chapter 10 and the requirements of these two AASHTO standards:

- In Table 10-1, giving requirements for coarse aggregate, the requirements for flat and elongated particles is given as a maximum of 10% at a 5 to 1 ratio, whereas M 325 has a maximum of 20% at a 3:1 ratio and 5% at a 5:1 ratio. The 10% maximum at a 5:1 ratio is used to provide consistency with the requirements for dense-graded HMA designed for traffic levels of 30 million ESALs or more (see Chapter 8 of the *Manual*).
- Also in Table 10-1, there is no absorption requirement, again, to maintain consistency with the requirements for dense-graded HMA given in Chapter 8.
- Also in Table 10-1, the requirements for fractured faces are 98/98 (one face/two faces), with the option of reducing this requirement to 95/95 if experience with local materials indicates that this will provide HMA with adequate rut resistance under very heavy traffic; this requirement is used, again, to provide consistency with the requirements for dense-graded HMA as given in Chapter 8 (the fractured faces requirement in M 325 is 100/95).
- In Table 10-2, giving requirements for fine aggregate, there are no requirements for liquid limit and plasticity as are listed in M 325. Instead, requirements for fine aggregate angularity and sand equivalency are given; these requirements are identical to those given in Chapter 8 for dense-graded HMA designed for traffic levels of 30 million ESALs or more.
- Table 10-11 is used explicitly to establish minimum asphalt binder contents; it is identical to Table X2.1 given in R 46, but according to R 46, the requirements in X2.1 are only to be used when the “standard” minimum binder content of 6.0% cannot be met.

CHAPTER 11

Design of Open-Graded Mixtures

The information given in this chapter comes directly from the following document (*48*):

Mallick, R. B., P. S. Kandhal, L. A. Cooley, Jr. and D. E. Watson. *Design Construction and Performance of New-Generation Open-Graded Friction Courses, NCAT Report 00-01.* National Center for Asphalt Technology. Auburn University. Auburn, Alabama. 2001.

The mix design procedure, tables, equations, and other critical information given in Chapter 11 are taken from this report.

CHAPTER 12

Field Adjustments and Quality Assurance of HMA Mixtures

Chapter 12 of the Manual is made up of two distinct parts. The first part of this chapter deals with how a typical HMA mix design, as developed in the laboratory must be adjusted during field production. The most important of these adjustments involves accounting for the increase in mineral dust that typically occurs during field production. The second part of the chapter deals with quality assurance (QA) of HMA. Although not strictly part of the HMA mix design process, it was thought that QA is such an important part of a typical HMA laboratory's work and is so closely related to the mix design process that the *Manual* would not be complete without addressing this topic. Information for this part of the chapter comes from various sources, but by far the most important are *Hot Mix Asphalt Construction, Instructor Manual, Part A: Lecture Notes* and the NHI course manual *Highway Materials Engineering, Module I: Materials Control and Acceptance—Quality Assurance (49, 50).*

Table 23 (Table 12-1 in the *Manual*) shows typical amounts of mineral dust generated during HMA plant production. This table is based in part on data from the NCAT test track; Figure 14 shows the increase in the passing 0.075-mm fraction of aggregate in QA data compared to the JMF gradation as a function of % passing the 2.36-mm sieve (*51*). For gradations having more than 35% passing the 2.36-mm sieve, the increase appears to mostly fall in the range of from 1.0 to 3.0%. However, as the % passing the 2.36-mm sieve decreases, the amount of dust generated during plant production appears to increase. A second factor affecting the amount of dust generated during plant production is aggregate hardness. Unfortunately the nature of the NCAT data does not allow development of such a relationship. The values shown in Table 23 are therefore based on engineering judgment. The L.A. abrasion values for different aggregate types are

Table 23. Typical amounts of mineral dust generated during HMA plant production for different aggregates and gradations (Table 12-1 in the Mix Design Manual).

Abrasion Resistance Level	L.A. Abrasion Loss *Wt.%*	Examples	% Retained on 2.36-mm sieve in theoretical aggregate blend:		
			< 25	25 to 35	> 35
Good	< 20%	Dense igneous rocks such as basalt, diabase and gabbro ("trap rock")	2.5	1.5	1.0
Moderate	20 to 35%	Good quality igneous rocks of moderate density such as granite, syenite, diorite; good quality dolomites, limestones and dolomitic limestones; most sandstones, graywackes, quartzites, slags, and crushed gravels	3.5	2.5	2.0
Poor	> 35%	Soft limestones, sandstones, graywackes and granite	4.5	3.5	3.0

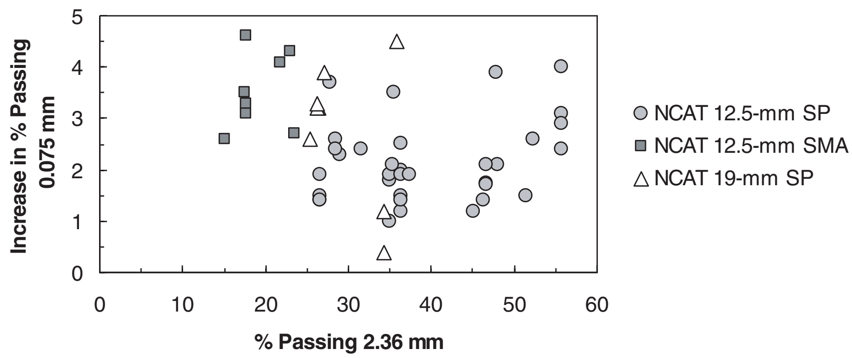

Figure 14. Increase in % passing the 0.075-mm Sieve during HMA Plant production as a function of % passing the 2.36-mm Sieve, for mixes placed in the first cycle of the NCAT test track.

based on a number of sources, including *NCHRP Report 405* and *NCHRP Report 557*. It should be emphasized that Table 23 is meant to provide approximate guidelines concerning the amount of dust generated during HMA production. Although such values cannot be predicted exactly, using estimated values in the mix design process will provide more reliable HMA mix designs than ignoring the amount of dust likely to be generated during plant production.

Table 24 (Table 12-2 in the *Manual*) gives precision values for commonly used tests on aggregates and HMA mixtures. These values are taken directly from AASHTO or ASTM standards for the various tests.

Equations 25 and 26 (Equations 12-2 and 12-3, respectively, in the *Manual*) for calculating the upper and lower control limits for an X-bar control chart are taken from *Hot Mix Asphalt Construction, Instructor Manual, Part A: Lecture Notes* (*49*). Similar equations appear in other documents dis-

Table 24. Single-operator and multi-laboratory precision for test results commonly used in HMA quality assurance and acceptance plans (Table 12-2 in the Mix Design Manual).

Test Procedure	Single Operator Std. Dev.	Single Operator D2S	Multi-laboratory Std. Dev.	Multi-laboratory D2S
Aggregate gradation, percent passing				
Coarse aggregate (CA)*	0.27 to 2.25	0.8 to 6.4	0.35 to 2.82	1.0 to 8.0
Fine aggregate (FA)*	0.14 to 0.83	0.4 o 2.4	0.23 to 1.41	0.6 to 4.0
Mineral Filler (in CA/ in FA)	0.10/0.15	0.28/0.43	0.22/0.29	0.62/0.82
Asphalt content, weight %				
Ignition oven	0.04	0.11	0.06	0.17
Quantitative extraction**	0.19 to 0.30	0.54 to 0.85	0.29 to 0.37	0.82 to 1.05
Maximum theoretical specific gravity	0.0040	0.011	0.0064	0.019
Bulk specific gravity, SSD	0.0124	0.035	0.0269	0.076
Bulk specific gravity, Paraffin-coated	0.028	0.079	0.034	0.095
Air void content, Vol. %***	0.5	1.5	1.1	3.0
Effective asphalt content, Vol. %***	0.3	0.9	0.6	1.6
Voids in mineral aggregate, Vol. %***	0.5	1.5	1.1	3.1
Voids filled with asphalt, Vol. %***	2.2	6.2	4.5	12.8
Dust/asphalt ratio, by weight***	0.05	0.13	0.09	0.25

* Lower values are for very high, very low, or both percent passing; higher values are for percent passing values close to 50%.
** Value depends on method used.
*** Typical values, estimated from data on aggregate gradation, aggregate and mixture specific gravity, and asphalt content using ignition oven. Values estimated using standard deviations for quantitative extraction vary slightly from these values.

cussing QA for HMA production and other industrial operations. These equations calculate control limits for the average (X-bar or $\bar{X}$) as a function of the overall range (R-bar or $\bar{R}$):

$$UCL = \bar{X} + (A_2 \times \bar{R}) \tag{25}$$

$$LCL = \bar{X} - (A_2 \times \bar{R}) \tag{26}$$

where

UCL = upper control limit
LCL = lower control limit
$\bar{X}$ = overall average or X-bar
$\bar{R}$ = overall range or R-bar
A_2 = a factor that depends on sample size n (given in Table 25)

Table 25 (Table 12-4 in the Manual) gives factors for calculating control limits for both X-bar charts and R charts. These numbers were also taken from *Hot Mix Asphalt Construction, Instructor Manual, Part A: Lecture Notes* (*49*), but can be found in other references on QA.

Tables 26 and 27 (12-5 and 12-6, respectively, in the Manual) show ranges for typical overall standard deviation for aggregate% passing, asphalt content, air voids, voids in the mineral aggregate (VMA) and voids filled with asphalt (VFA). These values are taken from a National Highway Institute (NHI) course manual on QA (*50*).

Equations 27 and 28 (Equations 12-4 and 12-5 in the *Manual*) are used to calculate lower and upper control limits for range charts (R charts) for quality assurance of HMA production:

$$LCL = D_3 \times \bar{R} = 0.0 \times 1.35 = 0.0 \tag{27}$$

$$UCL = D_4 \times \bar{R} = 2.12 \times 1.35 = 2.86 \tag{28}$$

where

LCL = lower control limit
UCL = upper control limit
D_3, D_4 = factors for computing control limits for standard deviation control charts; see Table 25
$\bar{R}$ = overall range

Table 25. Factors for computing control limits for control charts (Table 12-4 in the Mix Design Manual) (*49*).

Sample Size n	A_2	D_3	D_4
2	1.88	0.00	3.27
3	1.02	0.00	2.58
4	0.73	0.00	2.28
5	0.58	0.00	2.12
6	0.48	0.00	2.00
7	0.42	0.08	1.92

Table 26. Typical overall standard deviation values for aggregate gradation (Table 12-5 in the Mix Design Manual) (*50*).

Sieve Size	Typical Range for Overall Standard Deviation
19 mm	1.5 to 4.5%
12.5 mm	2.5 to 5.0%
9.5 mm	2.5 to 5.0%
4.75 mm	2.5 to 5.0%
2.36 mm	2.5 to 4.0%
1.18 mm	2.5 to 4.0%
0.60 mm	2.0 to 3.5%
0.30 mm	1.0 to 2.0%
0.15 mm	1.0 to 2.0%
0.075 mm	0.6 to 1.0%

Table 27. Typical overall standard deviation values for asphalt content, air voids, VMA, and VFA (Table 12-6 in the Mix Design Manual) (*50*).

Property	Typical Range of Value for Overall Standard Deviation
Asphalt content	0.15 to 0.30%
Air voids, from field cores	1.3 to 1.5%
Laboratory air voids	0.9%
VMA	0.9%
VFA	4.0%

These equations, like Equations 25 and 26 presented previously, are taken from *Hot Mix Asphalt Construction Instructor Manual, Part A: Lecture Notes* (*49*). As with much of the other information in Chapter 12, these equations can be found in numerous other references on QA.

The rules for interpreting statistical control charts (page 12-21) are based on those in reference *50.* The plan for investigating possible production problems as indicated by a control chart is based on the authors' engineering judgment and experience, as is much of the discussion for the last part of Chapter 12. The example of a stratified random sampling plan is based in part on the acceptance plan described in the Pennsylvania Department of Transportation's specification "Section 409—Superpave Mixture Design, Standard and RPS Construction of Plant-Mixed HMA Courses," as given in *Publication 408* (*52*). The example QA plan given toward the end of Chapter 12 is based on this same specification.

References

AASHTO Standards

M 29, Fine Aggregate for Bituminous Paving Mixtures
M 43, Standard Specification for Sizes of Aggregate for Road and Bridge Construction
M 320, Performance-Graded Asphalt Binder
M 323, Superpave Volumetric Mix Design
M 325, Standard Specification for Designing Stone Matrix Asphalt (SMA)
R 30, Mixture Conditioning of Hot-Mix Asphalt (HMA)
R 35, Superpave Volumetric Design for Hot-Mix Asphalt
R 46, Standard Practice for Designing Stone Matrix Asphalt (SMA)
T 2, Sampling of Aggregates
T 27, Sieve Analysis of Fine and Coarse Aggregate
T 84, Specific Gravity and Absorption of Fine Aggregate
T 85, Specific Gravity and Absorption of Coarse Aggregate
T 166, Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens
T 209, Theoretical Maximum Specification Gravity and Density of Bituminous Paving Mixtures
T 269, Percent Air Voids in Compacted Dense and Open Asphalt Mixtures
T 283, Resistance of Compacted Asphalt Mixture to Moisture-Induced Damage
T 320, Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester
T 321, Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending.
T 322, Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device
T 324, Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA)
TP 63, Determining the Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA)

Other Standards

NCHRP 9-29 PT 01, Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Simple Performance Test System
ASTM D 3515, Hot-Mixed, Hot-Laid Bituminous Paving Mixtures
ASTM D 3665, Random Sampling of Construction Materials
ASTM D 4460, Calculating Precision Limits Where Values are Calculated from Other Test Methods

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Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation