MS-2

7th Edition

Asphalt Mix Design Methods

asphalt institute

FOREWORD

Welcome to the 7th edition of MS-2 *Asphalt Mix Design Methods*. 2014 marks the 55th year of continuous publication of our mix design manual that has become the industry standard for the design of asphalt mixtures.

This manual remains a practical guide based on proven technologies, incorporating the most current information available at the time of this writing. Yet this edition offers significantly expanded explanation and guidance relative to our previous mix design manuals. AASHTO, ASTM and other published standards are referenced wherever applicable in addition to important research findings. It is intended for mix designers, pavement engineers, lab personnel and others involved in asphalt mixtures. It is formatted for easy use as an engineering textbook or as a laboratory workbook.

This edition represents a major shift from our previous mix design manuals by covering all methods in one comprehensive manual. It replaces the 6th edition of MS-2 (focused on the Marshall and Hveem methods) and our SP-2 manual (dedicated to Superpave mix design procedures and analysis). While Superpave has become the predominant method in the United States, Marshall and Hveem are still utilized in parts of the U.S. and certainly around the world. These three methods plus Stone Matrix Asphalt (SMA), Open Graded Friction Course (OGFC), with or without Reclaimed Asphalt Pavement (RAP), Recycled Asphalt Shingles (RAS), Warm Mix Asphalt (WMA) and others all share commonalty in their governing principles, fundamental properties, testing procedures and analysis. For this reason it made sense to merge MS-2 and SP-2 into one manual.

Topics are presented in the logical progression of mix design steps. Chapters 1 through 5 apply across all mix design methods; covering mix behavior, material selection, batching, specimen preparation, binder absorption, specific gravity testing, volumetric calculations, data interpretation and more. Chapter 6 covers the unique aspects of the Superpave system, while chapter 7 does the same for Marshall and chapter 8 does the same for Hveem. Chapters 9 through 11 are again generic for all dense graded mixes and design methods; covering moisture sensitivity, performance testing and the use of recycled materials. Chapter 12 discusses the many specialty type mixes utilized for unique applications, and chapter 13 wraps up the manual with guidance on verifying the JMF once field production starts.

MS-26, *The Asphalt Binder Handbook*, is the primary reference on asphalt binders, while our MS-4, *The Asphalt Handbook*, is a comprehensive manual on the use of asphalt. We believe this MS-2, 7th edition will serve the industry well for many years as the most thorough and current go-to-reference on asphalt mix design.

ACKNOWLEDGEMENTS

While it's impossible to mention everyone that contributed to the rewrite, review and redesign of this manual, we want to mention some of the key players.

Authoring individual chapters was a team effort between our lab and field engineering staff: Bob Humer, Mark Blow, Dave Johnson, Danny Gierhart, Phil Blankenship, Bob Horan, Wayne Jones, Greg Harder and Carlos Rosenberger. We augmented this in-house writing team with the expertise of Tim Murphy from Murphy Pavement Technology, Inc.

After many editing iterations led by Bob Humer and Mark Blow, the entire manuscript underwent a comprehensive review that included our Technical Advisory Committee and other outside experts. Extensive comments were submitted from Jim Scherocman – consulting engineer, Kevin Carlson of Jebro Inc, MeadWestvaco Corporation Asphalt Innovations, Road Science, Division of ArrMaz, PRI Asphalt Technologies Inc, Phillips 66 Company, as well as from Tim Murphy, Danny Gierhart and Bob Humer. All review comments were compiled by Kendal Butler, AI's Media and Publications Manager, and then thoroughly vetted for consideration by our in-house review team.

No surprise that there was lively debate among our engineers over a number of topics covered in this manual, but it's fair to say that the content presented herein represents the Asphalt Institute's collective position on best practice at the time of this writing.

Kendal Butler managed the overall design and proofing effort while graphics were created and illustrations managed by Mike Sonnenberg.

Successful teams are composed of individuals whose different backgrounds and expertise come together to create a synergy that is far more powerful than the individual talents. We are grateful to this winning team that produced a champion in our MS-2, 7th edition.

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CHAPTER 1

Introduction

1.0

General

Asphalt pavements are the predominant pavement type in the world. Asphalt is used for all types of applications—from residential streets to expressways, from parking lots to harbor facilities, from mastic roofing decks to water reservoir barriers, and from bike paths to airport runways.

Depending on traffic, climate, available materials and the location within the pavement structure, the type of mix selected and mix design criteria will be different. Moisture sensitivity tests and, more recently, performance tests are being added to better determine the final design mixture.

Successful mix design requires understanding the basic theory behind the steps and following the intent of the written instructions. It also includes having the proper training in laboratory techniques and effectively interpreting the results of laboratory tests. This manual was prepared with these goals in mind. It contains the latest information for the design of asphalt paving mixtures to meet the demands of modern traffic conditions and to ensure optimal performance of asphalt pavements.

1.1

Asphalt and hot mix defined

In different parts of the world, the term "asphalt" has different meanings. For instance, in Europe, asphalt is synonymous with what is called Hot Mix Asphalt (HMA) or Asphalt Concrete (AC) in the U.S. And the term "bitumen" in Europe is synonymous with asphalt, asphalt cement or asphalt binder in the U.S.

In this manual we will use "asphalt binder," "asphalt cement" and "bitumen" interchangeably to identify the liquid binder. Likewise, we will often use the terms "asphalt," "asphalt mixture," "hot mix asphalt" and "warm mix asphalt" to identify the mixture of aggregate and binder.

1.2

Classifications of mixes

Asphalt mixtures may be produced from a wide range of aggregate combinations, each having its own particular characteristics suited to specific design and construction uses. Mixtures consist of a combination of aggregate uniformly mixed and coated with asphalt binder. To dry the aggregates and to obtain sufficient fluidity of asphalt binder for proper mixing and workability, both the aggregate and the asphalt binder must be heated before mixing—hence the term "hot mix."

Dense-graded asphalt mixes

A dense-graded asphalt mix has a well-distributed aggregate gradation throughout the entire range of sieves used. It is the most commonly specified type of mix and can be used in the base, intermediate layers and surface of a pavement structure. Superpave, Marshall and Hveem are methods of designing dense-graded mixes.

Open-graded asphalt mixes

An open-graded layer is an asphalt mixture designed to have a large volume of air voids (typically 18 to 22 percent) so that water will readily drain through the pavement layer. It is used as an Open-Graded Friction Course (OGFC) to provide a skid-resistant pavement surface and as a porous base layer (also called Asphalt Treated Permeable Base, or ATPB) to provide for positive drainage under either an asphalt or Portland cement concrete pavement surface.

Gap-graded or Stone Matrix Asphalt (SMA)

Gap-graded or SMA is an asphalt mixture with a high-coarse aggregate content (typically 70 to 80 percent), a high asphalt content (typically more than 6 percent) and a high-filler content (approximately 10 percent by weight). The result is a durable mixture that has excellent stone-on-stone contact and that is very resistant to rutting.

Warm Mix Asphalt (WMA) is not a mix type, but refers to any mix produced at lower temperatures using a variety of technologies while maintaining the workability required to be successfully placed.

Many other specialty mixtures exist, and are described in chapter 12.

It is essential that the mixing facility produce the mix as similar as possible to the mix design, which is the purpose of chapter 13.

1.3

Objective of a mix design

The objective of a mix design is to determine the combination of asphalt cement and aggregate that will give long-lasting performance as part of the pavement structure. Mix design involves laboratory procedures developed to establish the necessary proportions of materials for use in the asphalt mixture. These procedures include determining an appropriate blend of aggregate sources to produce proper gradation of mineral aggregate and selecting the type and amount of asphalt cement to be used as the binder for that gradation. Well-designed asphalt mixtures can be expected to serve successfully for many years under a variety of loading and environmental conditions.

The mix design is just the starting point to assure that an asphalt pavement layer will perform as required. Together with proper construction practice, mix design is an important step in achieving well-performing asphalt pavements. In many cases, the cause of poorly performing pavements has been attributed to a poor or inappropriate mix design, or is due to the production of a mixture different from what was designed in the laboratory. Correct mix design involves adhering to an established set of laboratory techniques and design criteria. These techniques and criteria serve as the design philosophy of the governing agency. They are based on scientific research as well as many years of experience in observing the performance of asphalt pavements. It is critical that these laboratory methods be followed exactly as written.

Asphalt pavements perform well when they are designed, produced and constructed to provide certain desired properties. Those properties will vary based on project expectations and the position of that mix in the pavement structure. High-volume interstate highways with a high percentage of heavy trucks will require different properties than a parking lot or driveway.

The design of asphalt paving mixes, as with other engineering materials designs, is largely a matter of selecting and proportioning materials to obtain the desired properties in the finished construction product. The overall objective for the design of asphalt paving mixes is gradation of aggregates and binder content that yields a mix having:

- sufficient asphalt to ensure a durable pavement;
- sufficient mix stability to satisfy the demands of traffic without distortion or displacement;
- sufficient air voids in the total compacted mix to allow for a slight amount of additional compaction under traffic loading and a slight amount of thermal binder expansion without flushing, bleeding and loss of stability;
- a maximum void content to limit the permeability of harmful air and moisture into the mix;
- sufficient workability to permit efficient placement of the mix without segregation and without sacrificing stability and performance; and
- aggregate texture and hardness to provide sufficient skid resistance in unfavorable weather conditions.

The final goal of mix design is to select a unique design binder content that will achieve a balance among all of the desired properties. Ultimate pavement performance is related to durability, impermeability, strength, stability, stiffness, flexibility, fatigue resistance and workability. Within this context, there is no single asphalt content that will maximize all of these properties. Instead, an asphalt content is selected on the basis of optimizing the properties necessary for the specific conditions.

1.4

Evolution of mix design

The first recorded use of asphalt in pavements was in the city of Babylon, when between the years 625 B.C. and 604 B.C., "Procession Street" near King Nabopolassar's palace was paved.

The first asphalt pavements in the United States were built in Newark, New Jersey (1870), and in Washington, D.C. (1876), on Pennsylvania Avenue.

In 1908 Clifford Richardson stated that "gradations were arranged by chance, and mixes proportioned by rule of thumb and without reason." Richardson produced the first asphalt pavement engineering textbook, *The Modern Asphalt Pavement*, in 1905 while working for the Barber Asphalt Paving Company. He was considered the most well-known asphalt technologist of his time. He recognized the importance of air voids (V_a) and voids in the mineral aggregate (VMA). The Barber Company paved a product called sheet asphalt, a mixture of asphalt binder (from Trinidad) and clean angular sand and filler.

In contrast to Barber, Frederick Warren of the Warren Brothers Company produced mixes (more like HMA) with aggregates up to 3 inches in size. Warren called this mix "bitulithic" and received its patents in 1901 and 1903. Bitulithic was improved and later marketed as Warrenite. It used large stone aggregate, tightly graded to produce a mix with minimum voids and high stability. Because of the large-size aggregates and dense gradations, bitulithic mixes required less asphalt and were thus more economical. It also allowed the use of softer binders. In 1912 the Warren brothers filed a patent infringement suit in federal court in Topeka, Kansas. The court ruled that only mixes containing aggregates smaller than one-half inch could be produced without violating the Warren patents. This ruling was significant for the future developments in asphalt technology using smaller-size mixes.

Prevost Hubbard and Frederick Field of the Asphalt Institute developed a mix design procedure mainly for fine mixes. They determined a minimum Hubbard-Field stability value based on experience with pavements capable of resisting shoving. This method was developed in the 1920s and remained in wide use until the 1950s. Other mix design methods used in the 1950s were the Smith Triaxial method and the Texas Gyratory method.

In the 1930s, Francis Hveem, materials and research engineer for the California Division of Highways, developed a mix design procedure that introduced the use of a kneading compactor. Hveem felt the use of the kneading compactor was more representative of the field-produced mixes that were being compacted with steel and pneumatic-tire rollers. Additionally, Hveem recognized the need to have a mechanical test to evaluate the performance of the mix. That need led to the development of the Hveem Stabilometer, which is used to evaluate the ability of the mix (mainly the aggregate structure) to resist the shear forces applied by the traffic.

The Marshall Mix Design procedure was developed in the 1930s by Bruce Marshall of the Mississippi Highway Department. His belief was that laboratory compaction must produce a density that represents the in-place density after traffic. His recommendation was to select the asphalt content that produced the highest density, while checking stability and flow to assure minimum acceptable levels were met. World War II resulted in the need for runway construction in remote areas. This required a mix design procedure that used simple portable laboratory equipment. The U.S. Army Corps of Engineers adapted the Marshall method to design pavements that would support the increase in aircraft wheel loads and tire pressures of the new aircraft.

In the early 1950s, the initiation of the National Highway System saw state highway departments expanding the construction of high-volume roadways. It modified the Corps of Engineers procedures to design mixtures for highway pavements. Prior to the introduction of Superpave in the early 1990s, approximately 75 percent of state highway agencies were using the Marshall Mix Design procedure.

From 1987 to 1993, the Federal Highway Administration (FHWA) sponsored, and the Transportation Research Board (TRB) administered, the $150 million Strategic Highway Research Program (SHRP). The purpose of SHRP was to develop performance-based technology and specifications that would result in significant improvements in the way highways were designed and built to address the effect of increasing traffic on the nation's highway infrastructure.

Approximately $50 million of the SHRP funding was used to support an asphalt research program. The result of this research was the Performance Grading (PG) system for binders and a new mix design system called Superpave (SUperior PERforming Asphalt PAVEments). The first projects designed with Superpave technology were built in the early 1990s, and the procedure quickly became the standard procedure for the design of HMA pavement mixtures in the U.S. and Canada.

The Superpave mix design method uses a gyratory compactor to make test specimens and volumetrics to determine the optimum binder content. Moisture resistance, and in some cases, performance testing are included to verify the suitability of the designed mixture.

1.5

The structure of this manual

This manual goes chapter by chapter through materials, sample preparation, moisture sensitivity and mixture testing, volumetrics and mix performance testing as shown in **Figure 1.1**. Other considerations and specification requirements are added in each section, as appropriate.

For heavy-duty pavements, performance testing for rutting resistance and fatigue characteristics are sometimes included after/during moisture-sensitivity testing (see chapter 10).

Depending on the specific purpose of the mix, other gradings have been used with great success, such as gap-graded and open-graded aggregate compositions. The design philosophy and construction procedures of these mixes are different because of the additional void space incorporated between the larger particles. For such specialty mixes, refer to chapter 12.

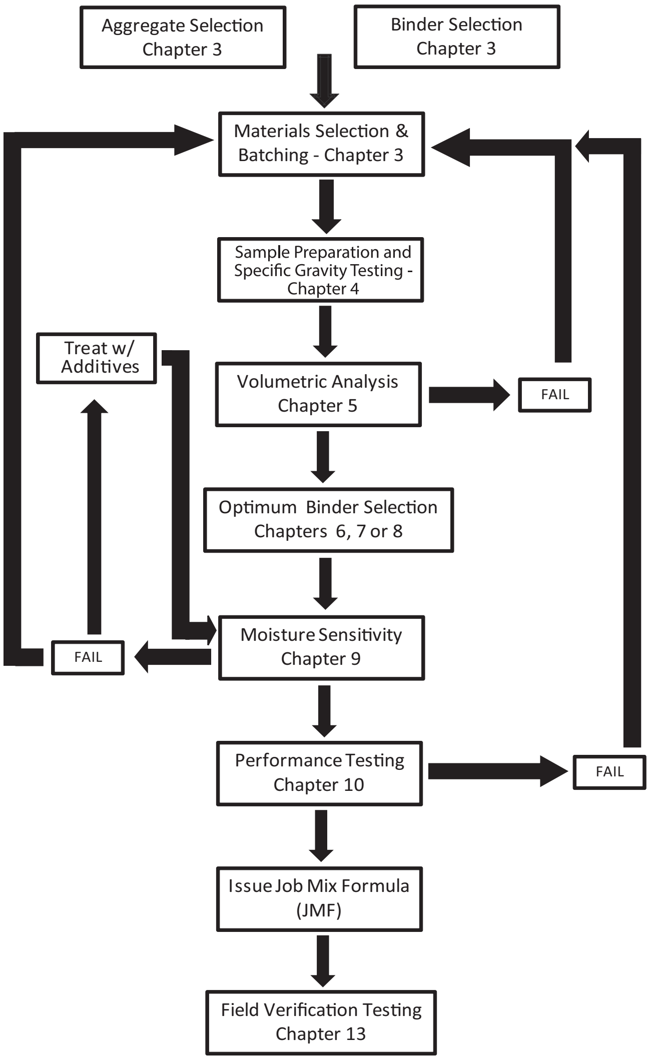


FIGURE 1.1 ***Basic Mix Design Flow Chart***

CHAPTER 2

Mixture Behavior

2.1

Objectives of asphalt paving mix design

A properly designed asphalt mixture provides a balance of engineering properties and economics that ensures a durable pavement that satisfies both its users and owners. Care should be exercised to consider the intended function of the pavement both overall and in terms of the specific location within the pavement structure where the mixture will be applied. Thus, for a surface layer it may be wise to use a mixture whose properties differ from those of a bottom or intermediate layers, each of which may also differ from the others.

The overall objective for the design of asphalt paving mixes is selecting the gradation of aggregates and proportioning of asphalt that yields a mix having the desirable properties listed in chapter 1, section 1.3.

2.1.1 Asphalt binder behavior

Three asphalt binder characteristics are important in asphalt mixture behavior: temperature susceptibility, viscoelasticity and aging characteristics. Asphalt's properties are temperature susceptible—asphalt is stiffer at colder temperatures. That is why a specified test temperature historically accompanies almost every asphalt binder and mixture test. Without specifying a test temperature, the test results are not effectively interpreted. For the same reason, asphalt binder behavior is also dependent on time of loading—asphalt is stiffer under a shorter loading time. The dependence of asphalt binder behavior on temperature and load duration means these two factors can be used interchangeably. That is, fast-loading rates can be simulated by low temperatures.

With the advent of the Performance Graded (PG) system, specifying a single temperature for a particular test became obsolete. Rather than specifying a test temperature for a physical property, a desired property parameter was set and the temperature that achieved the desired value was then determined via the prescribed test method. This was a fundamental change in binder testing philosophy. It allowed asphalt to be graded for the expected environmental conditions. Tests were incorporated in the PG system to reflect high temperature (rutting), low temperature (thermal cracking) and binder aging (fatigue) behaviors.

Asphalt binder is a viscoelastic material because it simultaneously displays both viscous and elastic characteristics. At high temperatures (e.g., greater than 200°F), asphalt cement acts almost entirely as a viscous fluid, displaying the consistency of a lubricant such as motor oil. At very low temperatures (e.g., below freezing), asphalt binder behaves mostly like an elastic solid, rebounding to its original shape when loaded and unloaded. At the intermediate temperatures found in most pavement systems, asphalt cement has characteristics of both a viscous fluid and an elastic solid.

Asphalt is chemically organic and reacts with oxygen from the environment. Oxidation changes the structure and composition of the asphalt molecules. Oxidation causes the asphalt to become more brittle, leading to the term "age hardening." Oxidation occurs more rapidly at higher temperatures and higher in-place air voids. A considerable amount of hardening occurs during HMA production, when the asphalt cement is heated to facilitate mixing and compaction. That is also why oxidation is more of a concern when the asphalt cement is used in a hot, desert climate.

The characteristics of asphalt cement under varying temperatures, rates of loading and stages of aging determine its ability to perform as a binder in the pavement system. More details on the PG and traditional testing methods are found in chapter 3. Even more explanation is available in the AI publication MS-26, The Asphalt Binder Handbook.

2.1.2 Mineral aggregate behavior

A wide variety of mineral aggregates is used to produce asphalt mixtures.

Natural aggregates including sand have been simply mined from river or glacial deposits and used without further processing to manufacture HMA. These are often called bank-run or pit-run materials.

Processed aggregate has been mined, quarried, crushed, separated in distinct size fractions, washed or otherwise processed to achieve certain performance characteristics of the finished HMA.

Synthetic aggregate is any man-made material that is not mined or quarried and is often an industrial byproduct, such as blast furnace slag. Occasionally, a synthetic aggregate will be included to enhance a particular performance characteristic of the HMA. For example, slag is sometimes used as a component to improve the skid resistance properties of HMA. Regardless of the source, processing method or mineralogy, aggregate must provide enough shear strength to resist permanent deformation. When a mix is overloaded, a shear plane develops, and aggregate particles slide past each other (see **Figure 2.1**), resulting in permanent deformation. Along this plane, the shear stress exceeds the shear strength of the mixture. Aggregate shear strength is critically important in HMA because it provides the mixture's primary rutting resistance.

Aggregate has relatively little cohesion. Thus, shear strength is primarily dependent on the resistance to movement, or inter-particle friction, provided by the aggregates. Angular, rough-textured aggregates provide more resistance than rounded, smooth-textured aggregates (see **Figure 2.2**). When a load is applied, the aggregate structure tends to be stronger because the load (confining pressure) holds the aggregate tighter together and increases shear strength. Even though an angular piece and rounded piece of aggregate may possess the same material strength, angular aggregate particles tend to lock together, resulting in a stronger mass of material. Rounded aggregate particles tend to slide past each other. The internal friction improves the ability of aggregate to interlock and create a mass that is almost as strong as the individual pieces.

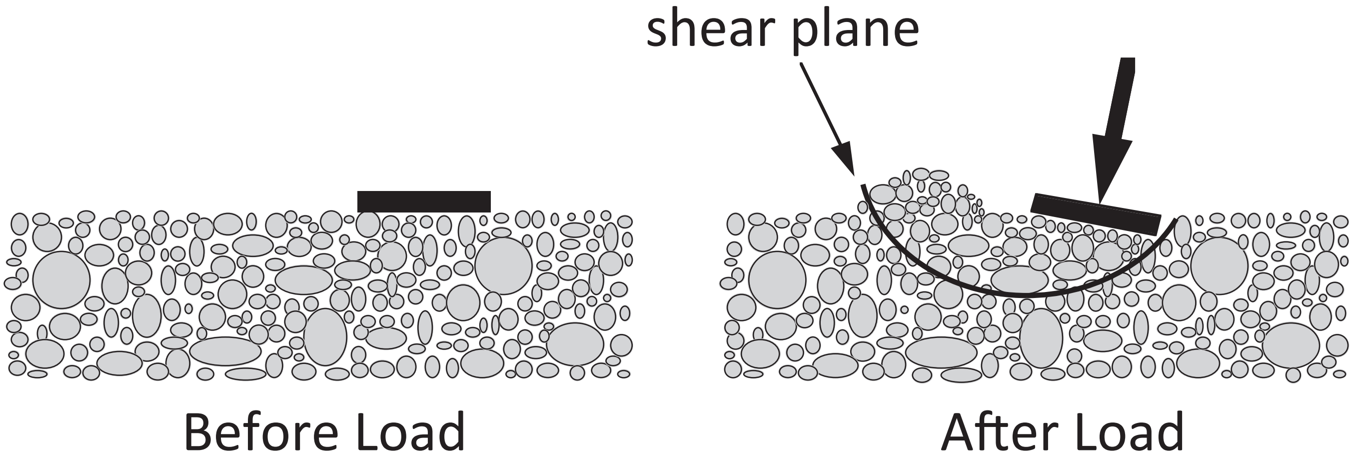


FIGURE 2.1 ***Shear Loading Behavior of Aggregate***

To ensure a strong aggregate blend for HMA, aggregate properties that enhance internal friction are typically specified. Normally, this is accomplished by specifying a certain percentage of crushed faces for the coarse portion of an aggregate blend and the fine aggregate angularity for the fine portion. In addition, the amount of natural sand in a blend is often limited because natural sands tend to be rounded, with poor internal friction.

2.1.3 Asphalt mixture behavior

When a wheel load is applied to a pavement, the primary stresses that are transmitted to the HMA are vertical compressive stress, shear stress within the asphalt layer and horizontal tensile stress at the bottom of the asphalt layer. The HMA must be internally strong and resistant to compressive and shear stress to prevent permanent deformation within the mixture. The material must also have enough tensile strength to withstand tensile stress at the base of the asphalt layer to resist crack initiation, which results in fatigue cracking after many load applications. Tensile strain at the edge of high pressure radial tires can cause top down cracking. The asphalt mixture must also resist contraction stresses from rapidly decreasing temperatures or extremely cold temperatures.

While the individual properties of HMA components are important, asphalt mixture behavior is best explained by considering asphalt cement and mineral aggregate acting together. One way to understand asphalt mixture behavior is to consider the primary asphalt pavement distress types that engineers try to avoid: permanent deformation, fatigue cracking and low temperature cracking. These are the distresses analyzed during mix design.

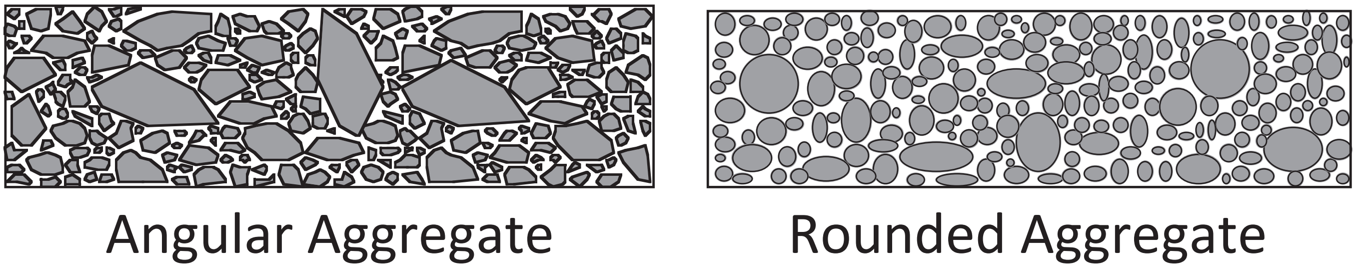
2.2

Desired properties considered for mix design

2.2.1 Resistance to permanent deformation—stability

Permanent deformation results from the accumulation of small amounts of unrecoverable strain (small deformations) from repeated loads applied to the pavement. Wheel path rutting is the most common form of permanent deformation. Resistance to permanent deformation is provided by designing and constructing a stable HMA pavement that will resist shoving and rutting under traffic. It will maintain its shape and smoothness under repeated loading. An unstable pavement develops ruts and shows other signs of mixture shifting.

Resistance to permanent deformation depends primarily on the internal friction provided by the aggregate particles and to a lesser extent the cohesion provided by the asphalt binder. Inter-particle friction among the aggregate particles is related to the shape and surface texture of both the fine and the coarse aggregate and the characteristics of the aggregate gradation. Cohesion results from the bonding ability and the stiffness characteristics of the asphalt binder. A proper degree of both inter-particle friction and cohesion in a mix prevents the aggregate particles from being moved past one another by the forces exerted by traffic. The use of more angular aggregate particles with rougher surface texture will increase the stability of the mix. Cohesion increases as the stiffness of the asphalt binder increases or when the pavement temperature decreases. The degree to which internal friction versus cohesion influences a mix's resistance to permanent deformation varies from mix to mix.


Angular Aggregate

Rounded Aggregate

FIGURE 2.2 ***Aggregate Stone Skeleton***

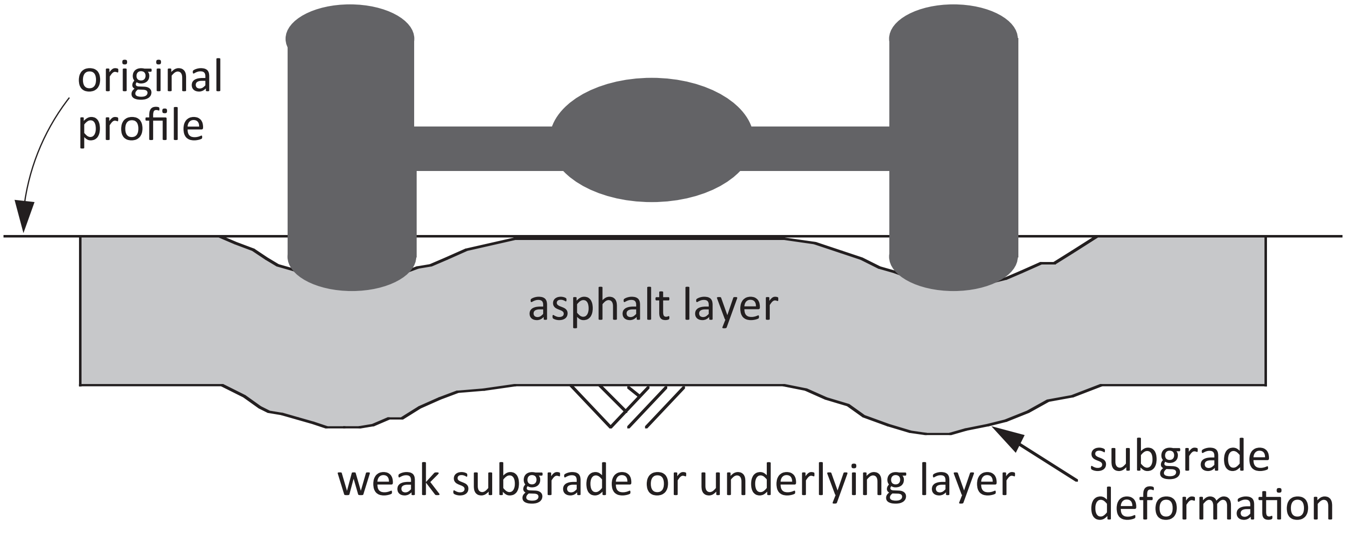


FIGURE 2.3 ***Rutting from Weak Subgrade***

Two primary causes of rutting are subgrade failure and inadequate mix stability.

First, deformation can occur in the subgrade or underlying layers such as base or sub-base rather than the asphalt layers (see **Figure 2.3**). It is typically caused by poor in situ subgrade quality or condition required for the pavement structure and loading conditions. Stiffer paving materials may reduce this type of rutting, but it is better to correct the subgrade or underlying layer instability.

In the case of an unstable mix, the deformation is limited to the asphalt layer and will result in an upheaval at the edge of the rut (see **Figure 2.4**). While this might suggest that rutting is an asphalt

FIGURE 2.4 ***Rutting from Weak Mixture***

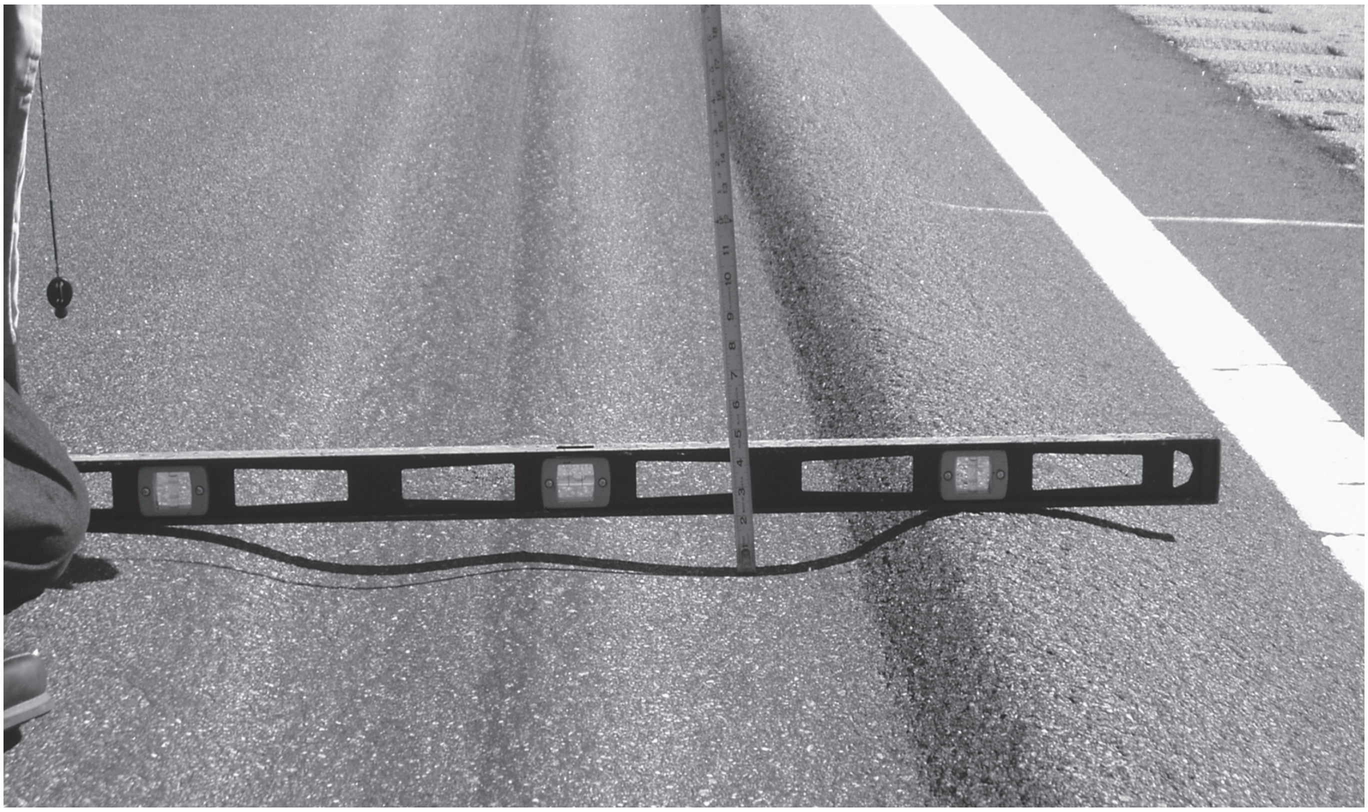


FIGURE 2.5 ***Fatigue (Alligator) Cracking***

binder problem, it is more correct to address rutting by considering the combined mineral aggregate and binder properties and through gradation and volumetric proportioning.

Asphalt mixture shear strength is mainly increased by selecting an aggregate that has a high degree of internal friction—one that is angular, has a rough surface texture and is graded to develop particle-to-particle contact. When a load is applied to the mixture, the aggregate particles lock tightly together and function more as a large, single, elastic mass. If improving the aggregates does not give sufficient improvement in mixture shear strength, a stiffer binder and/or a modified binder may be selected.

2.2.2 Fatigue resistance

Fatigue resistance is the pavement's resistance to repeated bending under wheel loads (traffic). The result of a fatigue failure is fatigue cracking, often called alligator cracking (see **Figure 2.5**). This type of cracking occurs when the pavement has been stressed to the limit of its life by repeated load applications. Although fatigue cracking is primarily related to an insufficient pavement thickness, air voids and asphalt binder properties have a significant effect on fatigue resistance. As the percentage of air voids in the pavement increases, either by design or by lack of compaction, pavement fatigue resistance is drastically reduced. Research and practice have shown that the use of a polymer-modified asphalt binder can significantly improve the fatigue resistance of an HMA mixture.

The thickness and strength characteristics of the pavement and the support of the subgrade also have a major impact upon pavement life and preventing fatigue or load-associated cracking. Thick, well-supported pavements deflect less under-traffic loading than thin or poorly supported pavements. Therefore, they have longer fatigue lives. For a perpetual pavement design, the endurance limit (the strain level below which the material will not fail in fatigue) may never be reached. Fatigue cracking typically initiates at the bottom of an asphalt layer and migrates toward the surface. This is the result of the high-tensile strain at the bottom of the HMA layer.

In recent years, fatigue cracking has been observed to begin at the top of the pavement and to migrate down. This cracking is due to high-tensile strains in the surface of the HMA. It is generally thought that thin HMA pavements experience fatigue cracking that starts at the bottom of the HMA and that thick HMA pavements experience fatigue cracking that starts at the HMA surface where it is more easily dealt with. Methods to overcome fatigue cracking are:

- adequately account for the number of heavy loads during design;
- use thicker pavements;
- provide adequate subgrade drainage;
- use pavement materials that are not easily weakened by moisture;
- use HMA that is resilient enough to withstand normal deflections; and
- use a modified binder.

Only selection of resilient materials can be strictly addressed during materials selection and mix design. The HMA must have enough tensile strength to withstand the applied tensile stress at the base of the asphalt layer, and be resilient enough to withstand repeated load applications without cracking. Thus, HMA must be designed to behave like an elastic material to overcome fatigue cracking. This is accomplished by placing an upper limit on the asphalt cement's stiffness properties, since the tensile behavior of HMA is strongly influenced by the asphalt cement. Simply put, soft asphalts have better fatigue properties than hard asphalts.

2.2.3 Low-temperature cracking

Low-temperature cracking (see **Figure 2.6**) normally occurs when the temperature at the surface of the pavement drops sufficiently to

FIGURE 2.6 ***Low-Temperature Cracking***

produce thermally induced stress in the HMA layer that exceeds the tensile strength of the asphalt mixture. The result is transverse cracks that are perpendicular to the roadway centerline and are often equally spaced. These cracks start at the surface and work their way downward. The magnitude, rate of cooling, frequency of the low-temperature occurrences and the stiffness of the asphalt binder are the major factors affecting the severity of the cracking. Low-temperature cracking can be the result of a single event or repetitive cycles of cold temperatures that result in a fatigue type of failure in the mixture. In general, the solution to this problem is the proper choice of binder. Using highly absorptive aggregates, or aggregates with high dust content, can aggravate low-temperature cracking.

2.2.4 Moisture resistance—impermeability

A major durability problem is associated with moisture damage, commonly referred to as "stripping." This typically is the result of water in combination with repeated traffic loadings, causing a scouring effect as the water is pushed into and pulled out of the voids in the pavement. Stripping involves water or water vapor getting between the asphalt film and the aggregates, thereby breaking the adhesive bond between the aggregate and the asphalt binder film. This will "strip" the asphalt from the aggregate.

The best line of defense against stripping is having sufficient binder in the mix and constructing an impermeable mat by achieving sufficient compaction.

Pavement failure due to stripping occurs in two stages: (1) stripping failure, and (2) the structural failure of the pavement under traffic. If stripping within the pavement becomes excessive, severe pavement deformation and failure will occur as the result of repeated loading. The failure of the pavement under traffic will generally occur in stages. The first stage will be a staining of the surface from fines. A local failure is followed by alligator cracking and potholes or by extensive rutting of the pavement in the wheel paths. Moisture-susceptible aggregates can be used provided they are treated with anti-stripping additives. A thorough discussion of moisture sensitivity of mixtures is provided in chapter 9.

2.2.5 Durability

The durability of an asphalt pavement is the ability to resist factors such as aging of the asphalt, disintegration of the aggregate and stripping of the asphalt film from the aggregate. These factors result from weather, traffic or a combination of the two.

Generally, the durability of a mixture can be enhanced by three methods:

- designing the mix using a dense gradation of moisture-resistant aggregate;
- maximizing the asphalt film thickness on the aggregate; and
- compacting the mixture to be impervious (which may be as low as 5 percent in-place air voids, depending on nominal maximum aggregate size and gradation).

Asphalt film thickness is related to the asphalt content, absorption characteristics of the aggregate and aggregate gradation. Thick asphalt films do not age and harden as rapidly as thin films. Also, increased film thickness effectively seals off a greater percentage of interconnected air voids in the pavement, making it difficult for water and air to penetrate. A certain percentage of air voids must remain in the pavement to allow for expansion of the asphalt in hot weather.

A dense gradation of sound, tough, moisture-resistant aggregate contributes to pavement durability. It provides closer contact among aggregate particles, enhancing the impermeability of the mixture. A sound, tough aggregate resists disintegration under traffic loading.

2.2.6 Skid resistance

Skid resistance is the ability of an asphalt surface to minimize skidding or slipping of vehicle tires, particularly when the roadway surface is wet. For good skid resistance, tire tread must be able to maintain contact with the aggregate particles and not ride on a film of water on the pavement surface (hydroplaning). Pavement skid resistance is typically measured at 40 miles per hour (65 kilometers per hour) with a standard tread tire under controlled wetting of the pavement surface.

A rough pavement surface with many little peaks and valleys will have greater skid resistance than a smooth textured surface. Best skid resistance is obtained with rough-textured aggregate in an open-graded mixture with an aggregate of about ⅜-inch (9.5 mm) to ½-inch (12.5 mm) maximum size. Besides having a rough surface, the aggregates must resist polishing (smoothing) under traffic. Calcareous aggregates (limestones) polish more easily than siliceous aggregates (quartz). Unstable mixtures that tend to rut or bleed present serious skid-resistance problems.

2.2.7 Workability

Workability describes the ease with which a paving mixture can be placed and compacted. Mixtures with good workability are relatively easy to place and compact; those with poor workability are difficult to place and compact. Changing mix design parameters, aggregate source and/or gradation can improve workability. However, mixes that are more workable are generally more prone to permanent deformation. Caution needs to be exercised to ensure a proper balance for a pavement's intended use.

Harsh mixtures (mixtures containing a high percentage of coarse aggregate and/or low asphalt content) have a tendency to segregate during handling and may be difficult to compact. To make a mixture more workable, the aggregate gradation can be adjusted by increasing the proportion of natural sand versus crushed fines or increasing the asphalt content of the mix. Care should be taken to ensure that the rutting resistance of the mix is not compromised in order to provide a workable mix. Many high-strength mixtures are harsh and difficult to compact. Too high a filler content can also affect workability, causing the mix to become gummy.

Workability is especially important when hand placement and raking (luting) around manhole covers, sharp curves and other obstacles are required. It is important that mixtures used in such areas be workable.

Mixtures that can be too easily worked or shoved are referred to as tender mixes. Tender mixes are too unstable to place and compact properly. They are often caused by:

- a shortage or excess of mineral filler;
- excessive medium-size sand;
- smooth, rounded aggregate particles; and/or
- moisture in the mix.

Although not normally a major contributor to workability problems, asphalt does have some effect on workability. Because the temperature of the mix affects the viscosity of the asphalt, a temperature that is too low will make a mix unworkable and a temperature that is too high may make it tender. Asphalt grade may also affect workability, as may the percentage of asphalt in the mix.

2.3

Volumetric characteristics of asphalt mixtures

When a mix design is conducted in the laboratory, the mix is analyzed to determine its probable performance in a pavement structure. Volumetric analysis is typically conducted on all mixtures regardless of the particular mix design methodology employed. The volumetric analysis focuses on the following five characteristics of the mixture and the influence those characteristics are likely to have on mix behavior:

- mix density;
- design air voids;
- voids in the mineral aggregate;
- voids filled with asphalt; and
- asphalt content.

2.3.1 Mix density

The density of an asphalt mixture is defined as the mass of mix per unit of volume. Two different mix densities are typically determined during the design and construction of an asphalt pavement. The bulk density and maximum density are measured during the mix design phase of the project using specified procedures discussed in further detail in chapter 4. Both values are needed to calculate the design air void content of laboratory-compacted samples when conducting a mix design. The bulk density can also be determined on in-place pavements. The bulk in-place density is typically determined by obtaining and measuring cores or through the use of other test methods. The density of mix compacted on the roadway is compared to the maximum density of the mix to determine the in-place (field compacted) air voids. Measurement of in-place air voids is very important because high density of the finished pavement (reduction of in-place air voids) is essential for lasting pavement performance. Mixture density and air void content are directly related.

2.3.2 Air voids

The durability of an asphalt pavement is a function of the air void content of the in-place HMA pavement. The lower the air voids, the less permeable the mixture becomes. An air void content that is too high provides passageways through the mix that allow damaging air and water to enter. An air void content that is too low can lead to rutting, shoving, flushing or bleeding.

The air voids in a paving mixture (see **Figure 2.7**) are small pockets of air between the asphalt-coated aggregate particles. A certain percentage of air voids is necessary in the finished HMA to allow for additional compaction under traffic and a slight amount of asphalt expansion because of temperature increases. In general, the design air void level in a laboratory-compacted sample of HMA is 4 percent. For special uses, a lower or higher design air void content may be specified.

Job specifications usually require that, for dense-graded mixes, pavement compaction achieve an air void content of less than 8 percent to minimize permeability.

2.3.3 Voids in the mineral aggregate

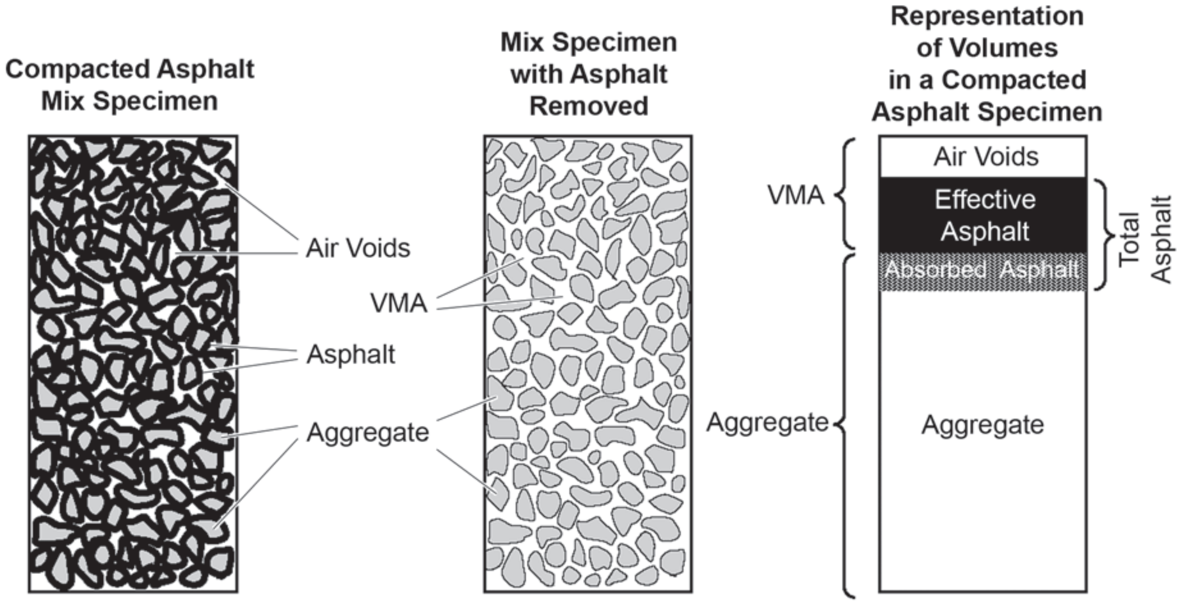
Given that mix designs typically aim for 4 percent air voids, voids in the mineral aggregate (VMA) must remain high enough to achieve an adequate asphalt film thickness, which results in a durable asphalt pavement. HMA mixtures with below-minimum VMA values will have thin films of asphalt and will provide an HMA pavement with low durability. Therefore, reducing asphalt content by lowering VMA is actually counterproductive and detrimental to pavement quality.

2.3.4 Voids filled with asphalt

Voids Filled with Asphalt (VFA) is the percentage of inter-granular void space between the aggregate particles (VMA) that contains or is filled with effective binder (see **Figure 2.7**). VFA is used to ensure proper asphalt film thickness in the mix. If it is too low, the mix will have poor durability, or if it is too high, the mix can be unstable. The acceptable range of VFA varies depending upon the traffic level. Higher traffic requires a lower VFA because mixture strength and stability are a greater concern. Lower traffic requires a higher range of VFA to increase HMA durability. A VFA that is too high, however, may yield a tender mix and cause bleeding.

2.3.5 Asphalt content

The optimum asphalt content of a mix is highly dependent on aggregate characteristics such as gradation and absorption. Aggregate gradation is directly related to optimum asphalt content. The finer the mix gradation, the greater the total



Note: For simplification, the volume of absorbed asphalt is not shown.

FIGURE 2.7 ***Voids in a Compacted HMA Mixture*** surface area of the aggregate will be, and the greater the amount of binder that will be required to uniformly coat the particles. Conversely, coarser mixes have less total aggregate surface area and demand less asphalt.

The relationship between aggregate surface area and optimum asphalt content is most pronounced when mineral filler containing high percentages of particles passing the No. 200 (0.075 mm) sieve is part of the mix. Variations in the amount and the size of mineral filler at a constant asphalt content will cause changes in mix properties, resulting in mixes with too little or too much asphalt. Small increases in filler content will use up additional asphalt and produce a dry, unstable mix. Small decreases have the opposite effect. Too little filler results in a mixture that is too rich. An abundance of very fine (less than 10–20 μm) particles in the filler is expected to act as an asphalt extender, resulting in a mix that seems to have too much asphalt.

Two terms in asphalt mix technology are used to express the methods of asphalt content: total asphalt content and effective asphalt content. Total asphalt content is the amount of asphalt that must be added to the mixture to produce the desired mix qualities. Effective asphalt content is based on the volume of asphalt not absorbed by the aggregate—it is asphalt that effectively forms a bonding film on the aggregate surfaces.

Aggregate absorption is critical in determining optimum asphalt content because enough asphalt must be added to the mix to allow for absorption while still coating the aggregate particles with an adequate film.

CHAPTER 3

Materials Selection and Aggregate Batching

The different types of asphalt mixtures are primarily made up of asphalt binder and mineral aggregates. Additionally, based on the local materials available, traffic loading and climate, other ingredients (or additives) may be required in the asphalt mixture, such as mineral fillers, fibers, liquid anti-strips, lime and/or recycled products. All quality pavements should be engineered to contain requirements for the following items:

- properly selected asphalt binder grades for the climate and traffic;
- aggregate characteristics including material quality and gradation;
- HMA volumetric requirements; and
- HMA performance criteria, if warranted.

This chapter will look at mixture types, asphalt binders, aggregates, aggregate gradations, aggregate blending and aggregate batching.

3.0

Mixture types

3.0.1 General

Dense-graded, flexible mixtures are generally divided into three major categories dependent upon their specific use: surface mixtures, binder or intermediate mixtures and base mixtures. Mixtures are typically designed with layer thickness, availability of aggregates and local history in mind. The maximum size aggregate is generally largest in the base, smaller in the intermediate (also referred to as "binder") course and the smallest in the surface course. However, this practice is not universal and it is not uncommon for a single mixture to be used for all of the layers. Nevertheless, any properly designed HMA mix can generally serve at any level in the pavement. Moreover, surface course mixtures may become "binder" mix if it is subsequently overlaid, so strength requirements should not be compromised regardless of the location of the mix within the pavement.

Generally, there is no single, uniform standard set of HMA classifications used by the various public agencies. There are similarities with respect to mixture types, but the geographic availability of materials and different climatic design requirements have led to various identifications. Each agency usually has its own designation for identifying various mixture types. While most HMA mixtures have a typical design use, these mixes offer a wide range of performance characteristics and there is substantial overlap of mixture application.

Recommendations on the minimum thickness for a single lift of dense-graded asphalt mixtures are four times the nominal maximum aggregate size (NMAS) for all mixtures with the exception of "fine" graded mixtures that may be placed at three times the NMAS. Historical pavement thickness guidelines of two times the "top size" are inappropriate for NMAS-defined gradations and are susceptible to poor pavement performance. Regardless of the mixture classification, the same degree of design, production and construction control procedures should be used to ensure proper performance of the pavement.

3.0.2 Surface course mixtures

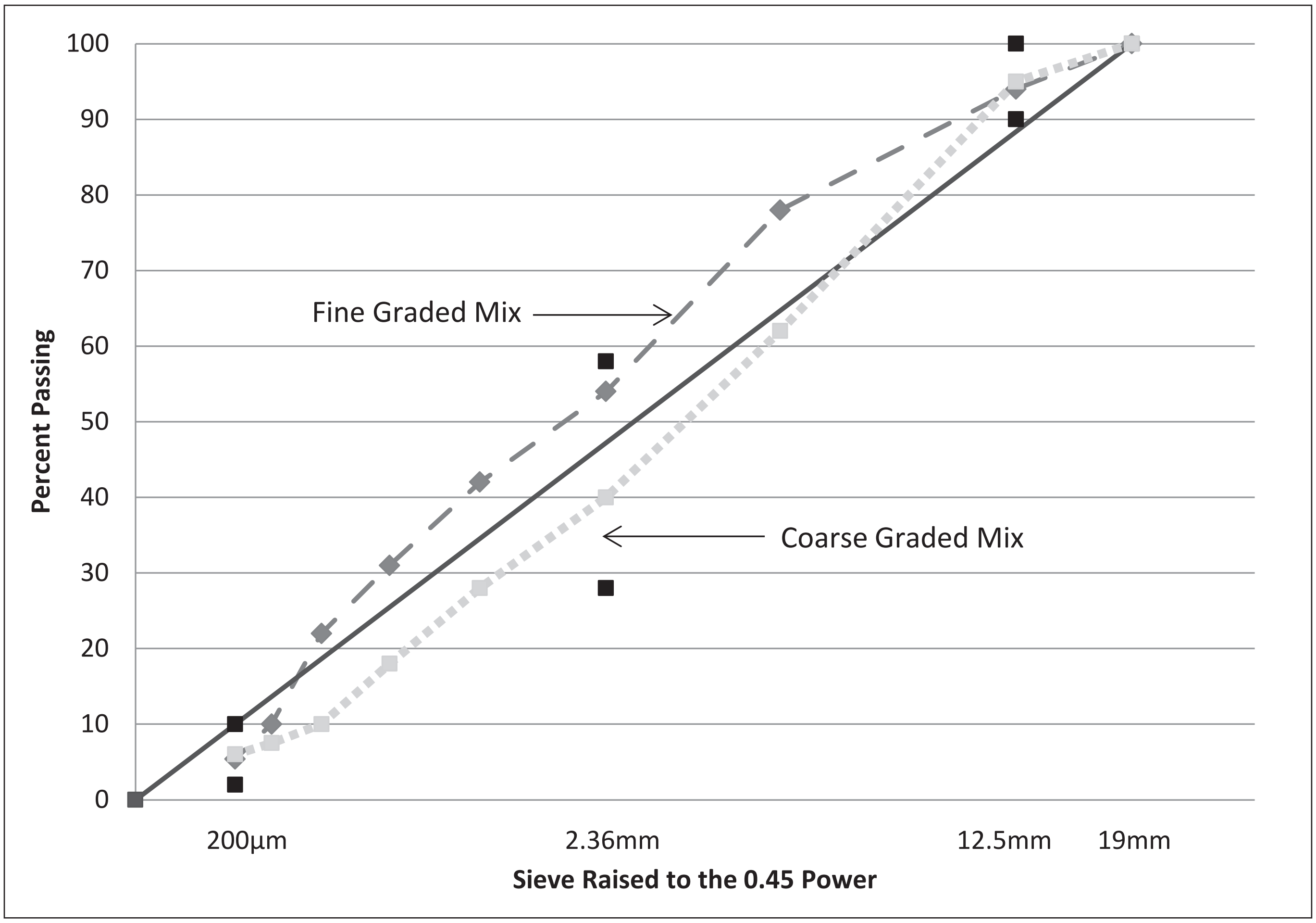
Surface course mixes must be designed to have sufficient stability and durability to both carry the anticipated traffic loads and to withstand the detrimental effects of air, water and temperature changes. In general, surface mixtures have greater asphalt content than binder or base mixtures due to their greater surface area and the higher VMA requirements of smaller maximum aggregate size mixtures.

The nominal maximum aggregate size is one size larger than the first sieve to retain more than 10 percent of the material. Maximum aggregate size (MAS) is one size larger than nominal maximum's size. Nominal maximum aggregate sizes for surface mixes generally vary from ⅜ to ¾ inch (9.5 to 19 mm). The choice of nominal maximum aggregate size is often predicated on the desired surface texture, with a smaller maximum aggregate size producing a smoother, tighter surface. **Figure 3.1** illustrates both a fine and coarse grading for a ½-inch (12.5 mm) NMAS Superpave mixture.

3.0.2.1 Open-Graded Friction Course considerations

A special type of surface mixture used for reducing hydroplaning and increasing skid resistance is an open-graded friction course (OGFC), also known as a porous friction course (PFC) or popcorn mix. The function of this mixture is to provide a free-draining layer that permits surface water to migrate laterally through the mixture to the edge of the pavement by maintaining in-place air voids at levels of 18 percent or higher. It is important that OGFC mixes are designed to not trap the moisture in the mix. The open-graded mixture also provides a skid-resistant surface as its coarse texture provides excellent friction between the pavement and the tire, while reducing noise. OGFCs contain a relatively high asphalt content,

FIGURE 3.1 ***Example of Fine and Coarse 12.5-mm Superpave Gradations***



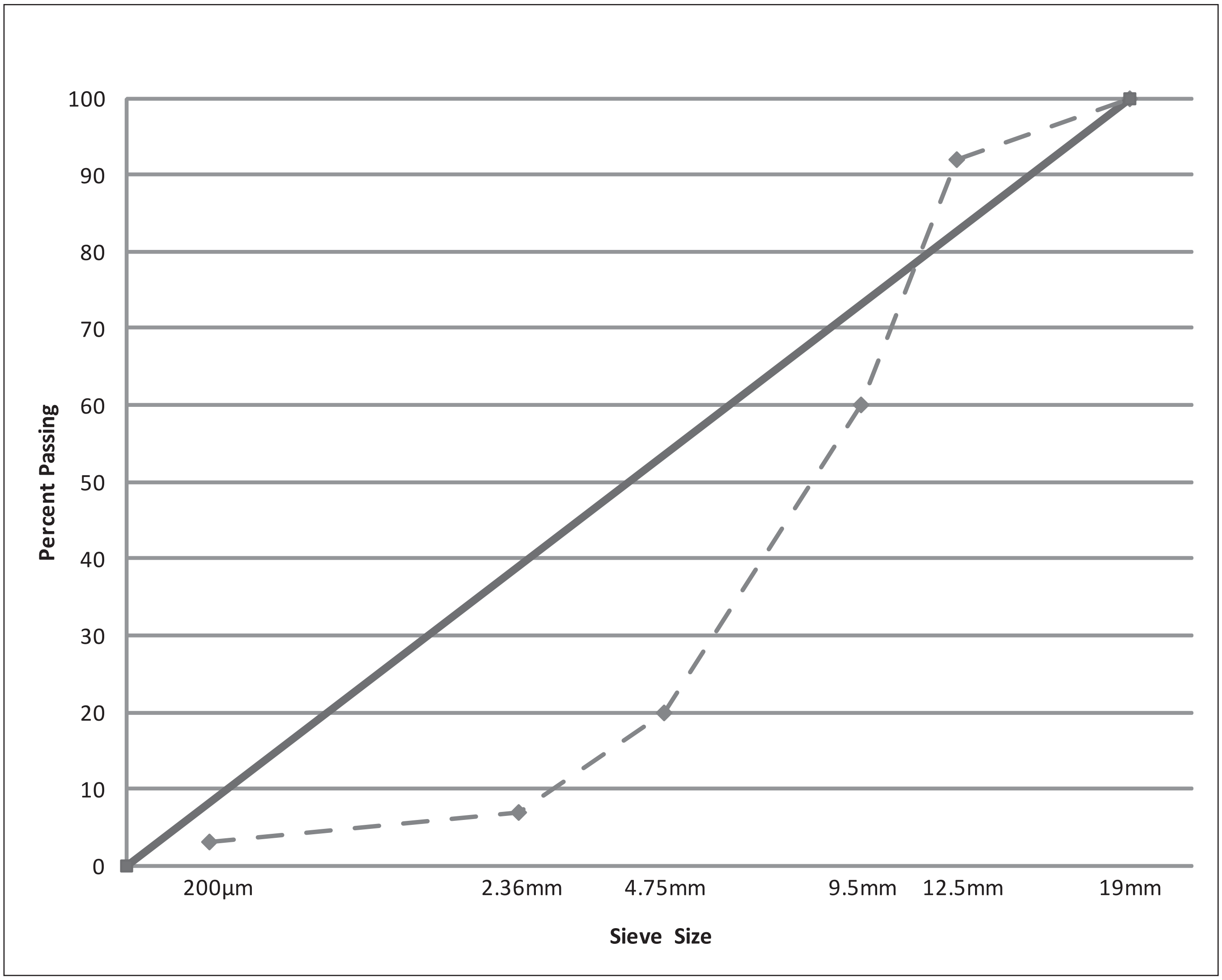


FIGURE 3.2 ***Typical Open-Graded Mixture Gradations***

using a ⅜- to ½-inch (9.5 to 12.5 mm) NMAS, with few aggregate fines to produce the open-graded mixture. Typically lift thicknesses are two to three times the NMAS. The mixes are placed only to facilitate rapid removal of surface water and with a minor improvement to the pavement's structure. OGFC surfaces should be removed prior to any subsequent overlays being placed. **Figure 3.2** shows the gradation of a typical ½-inch (12.5 mm) open-graded friction course. Additional guidance on the design of OGFCs can be found in Section 12.3.

3.0.2.2 Stone Matrix Asphalt considerations

Stone Matrix Asphalt (SMA) is a unique surface mixture that has been gaining favor with pavement engineers for use on high-volume roadways and intersections, as well as other areas where heavy loads may be expected. SMA is a durable, rut-resistant mixture that relies on stone-on-stone contact from a gap-graded aggregate skeleton—typically ⅜- to ½-inch (9.5 to 12.5 mm) NMAS (see **Figure 3.3**)—for strength, and a relatively high percentage of asphalt binder—often polymer-modified—for durability. To minimize drain down at elevated temperatures, a mastic is typically created using mineral filler and fibers in combination with the asphalt binder. The design air voids for an SMA are similar to dense-graded asphalt. In addition to their inherent strength, SMA surfaces are quieter and they have less spray than dense-graded pavements. Additional information on SMA is found in section 12.4.

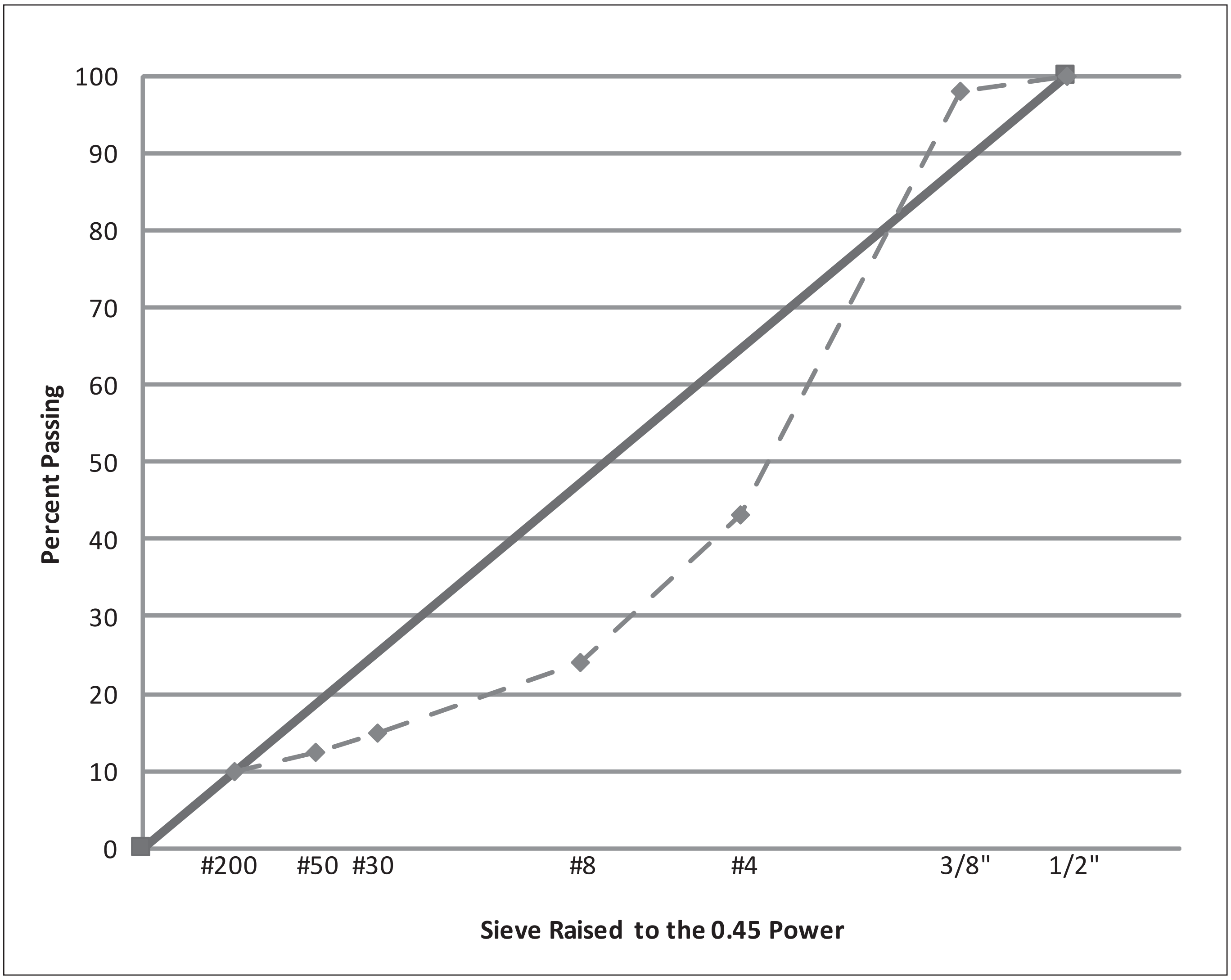


FIGURE 3.3 ***⅜-inch NMAS SMA Gradation***

3.0.2.3 Ultrathin bonded wearing course considerations

Ultrathin bonded wearing (UTBW) courses are yet another surface mixture that deserves special attention. (Please see chapter 12 for additional guidance on the design of ultrathin bonded wearing courses.) An ultrathin bonded wearing course is typically ⅜ to 1 inch of a gap-graded asphalt concrete intended to correct minor defects and improve ride while reducing noise. These are usually a part of a pavement preservation program for use on structurally sound materials. Ultrathin overlays are a single layer of HMA placed on a heavy asphalt emulsion layer or membrane. This emulsion is typically polymer modified, although unmodified materials have been used successfully. It is applied at a rate of about 0.20 ± 0.07 gallons per square yard (0.85 ± 0.3 liters per square meter). This heavy membrane seals small cracks and helps to adhere the overlay to the existing pavement. The grade of the asphalt binder, as with other pavement designs, is chosen based on the climate and traffic parameters for the project location. Nominal maximum aggregate size ranges from No. 4 to ½ inch (6.2 to 12.5 mm), with ⅜ inch being the most popular choice. Specialty equipment is required to place both the emulsion and the HMA in a single operation. The concept of the ultrathin bonded wearing course is that the heavy shot of asphalt membrane is wicked up, during construction, partially into the voids of the gap-graded mix.

3.0.3 Binder course mixtures

Binder mixes are often used as an intermediate layer between the surface mixture and the underlying asphalt mix or granular base. Binder mixes typically have a larger NMAS of ½ to 1.5 inches (19 to 38 mm), with a corresponding lower asphalt content. Binder and base mixes are often used interchangeably in pavement design and construction. Where heavy wheel loads are involved, a typical binder mix for highway construction can be used as a surface mix if a coarser surface texture will not be a concern. This approach has often been used in port facilities using heavy cargo-handling vehicles, in logging yards that use large log-handling vehicles, and for truck-unloading and industrial areas with high percentages of heavy trucks. Larger aggregate mixes (with less asphalt and sand contents) are often more resistant to the scuffing action of tight-radius, power steering turns.

Asphalt base mixes can be placed directly on the compacted subgrade or over a granular base. Base mixes are characterized by larger aggregate sizes that range up to 3 inches (75 mm). The relative asphalt content will be lower due to the larger maximum aggregate size and its smaller surface area. It is common, however, for a base mix to be designed to slightly lower air voids than the upper layers to ensure a greater amount of binder in the mix to assist in resisting the tensile strain it will encounter while in service. Maximum aggregate sizes for base mixtures are often established by the locally available material.

Base mixes can also be designed with an open gradation to facilitate drainage of water that may eventually enter the pavement structure. Such open-graded base mixtures, or Asphalt Treated Permeable Base (ATPB) mixes, are designed to provide an interconnecting void structure, using 100 percent crushed materials with maximum aggregate sizes of about 1.5 to 3 inches (38 to 75 mm). Positive, free drainage must be incorporated in the overall pavement design with these layers. See section 12.3 for more information.

3.1

Asphalt binder

A typical asphalt paving mixture consists of 4–7 percent asphalt binder. In the late 19th century, the penetration of asphalt with a weighted #2 sewing needle was introduced as an option for characterizing asphalt cements. ASTM adopted its first standard for this test in 1903 (ASTM D-5). The penetration grading system, refined over time, continues to be used in many locations worldwide. Additional grading systems were also developed. The next noteworthy system was the viscosity grading method (AC) developed by the Asphalt Institute and the Federal Highway Administration in the late 1950s. The AC system was further modified into the AR system by western U.S. states.

These were the dominant grading systems in the United States until the development of the Performance Graded (PG) System as part of the Superpave research of the late 1980s into the '90s. The PG System continues to undergo refinement as producers and agencies seek to properly classify today's highly engineered asphalt cements. This has most recently led to the introduction of the Multiple Stress Creep Recovery (MSCR) test. For most mix designers, determining the specific binder to be utilized on the project will have already been determined before the mix design phase. For more binder specification, selection and testing information, see Asphalt Institute's MS-26.

3.2

Mineral aggregate

Pavement experts from around the world have worked with industry to better shape the future of aggregate production based on aggregate properties that are important for long-term pavement performance. Since aggregate properties play an essential role in overcoming permanent deformation, much effort has been directed toward developing two categories of aggregate properties: consensus properties and source properties. Fatigue cracking and low-temperature cracking are less affected by aggregate characteristics. It is important to thoroughly understand the impacts of aggregate gradation for these distresses, and they shall be discussed throughout this manual. Chapter 5, section 5.12, contains a process called "Selecting a Design Aggregate Structure." This procedure can be very helpful in determining aggregate combinations that will meet or optimize mix design parameters.

A mix design may consist of simply verifying a previously performed design to a full investigative process dealing with many diverse aggregate materials and sources in search of an optimal combination of aggregates and binders. Regardless of the type of mix design being performed, actual data from field-produced stockpiles should accompany samples submitted in order for the designer to accurately determine the characteristics of the final mix. Samples used in the mix design must be representative of the materials produced and scheduled to be utilized on the project. Mix design samples that do not reasonably represent the average stockpile values for gradation or other properties listed in this section will result in a production mix substantially different than the mix design, in essence invalidating the mix design. The following aggregate properties should be evaluated by comparing the mix design data with available production data.

3.2.1 Consensus aggregate properties

Certain aggregate characteristics are critical to well-performing HMA and have been widely acknowledged by a wide range of industry experts. These characteristics are called the "consensus" properties and are as follows: coarse aggregate angularity (CAA), fine aggregate angularity (FAA, flat and elongated particles (F&E), and clay content (SE value).

The criteria for these consensus aggregate properties are based on traffic level and position within the pavement structure. Materials near the pavement surface subjected to high traffic levels require more stringent consensus properties. The criteria are intended to be applied to a proposed aggregate blend rather than individual components. However, many agencies currently apply such requirements to individual aggregates so undesirable components can be identified. The consensus properties are detailed in **Table 3.1**.

3.2.1.1 Coarse aggregate angularity

Coarse aggregate angularity (CAA) ensures a high degree of aggregate internal friction for rutting resistance by specifying a minimum percentage of angular particles in the asphalt mixture. The test method is ASTM D 5821, "Determining the Percentage of Fractured Particles in Coarse Aggregate." The test method determines the

TABLE 3.1 ***Aggregate Consensus Property Requirements***

Design ESALs[1] (In Millions)	Coarse Aggregate Angularity (CAA) (Percent), minimum		Uncompacted Void Content of Fine Aggregate Angularity (FAA) (Percent), minimum		Sand Equivalent (SE) (Percent), minimum	Flat and Elongated[3] (F&E) (Percent), maximum
	≤ 100 mm	> 100 mm	≤ 100 mm	> 100 mm		
< 0.3	55/–	–/–	–	–	40	–
0.3 to < 3	75/–	50/–	40	40	40	10
3 to < 10	85/80[2]	60/–	45	40	45	10
10 to < 30	95/90	80/75	45	40	45	10
≥ 30	100/100	100/100	45	45	50	10

NOTES:

1. Design ESALs are the anticipated traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years to choose the appropriate aggregate criteria.
2. 85/80 denotes that 85 percent of the coarse aggregate has one or more fractured faces and 80 percent has two or more fractured faces.
3. Criterion based upon a 5:1 maximum-to-minimum ratio.
4. Flat and elongated criteria do not apply to 4.75-mm NMAS mixes.
5. For 4.75-mm NMAS mixtures designed for traffic levels < 0.3 M ESALs, the minimum Uncompacted Void Content (FAA) is 40.
6. For 4.75-mm NMAS mixtures designed for traffic levels ≥ 0.3 M ESALs, the minimum Uncompacted Void Content (FAA) is 45.

If less than 25 percent of a layer is within 100 mm of the surface, the layer may be considered to be below 100 mm for mixture design purposes.

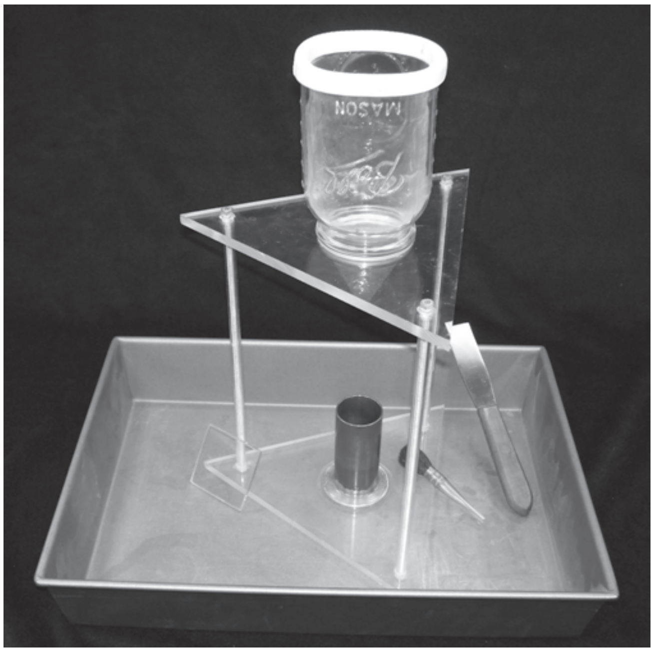
percentage of aggregate pieces larger than the #4 sieve (4.75 mm) meeting specified angularity criteria, either by mass or particle count. The reporting format gives both the percentage of aggregate with one or more fractured faces and with two or more fractured faces. For example, a reported value of "85/80" indicates that 85 percent of the sample has one or more fractured faces and 80 percent has two or more fractured faces. **Table 3.1** gives the required minimum values for coarse aggregate angularity as a function of traffic level and position within the pavement.

3.2.1.2 Fine aggregate angularity

Fine aggregate angularity (FAA) ensures a high degree of fine aggregate internal friction and rutting resistance. It is defined as the percent of air voids present in loosely compacted aggregates smaller than the #8 sieve (2.36 mm). The test method specified is AASHTO T 304, "Uncompacted Void Content of Fine Aggregate." This property is influenced by particle shape, surface texture and grading. Higher void contents typically mean more fractured faces.

In the test procedure, a sample of fine, washed and dried aggregate is poured into a small calibrated cylinder through a standard funnel (see **Figure 3.4**). By measuring the mass of fine aggregate (F) in the filled cylinder of known volume (V), the void content can be calculated as the difference between the cylinder volume and fine aggregate volume collected in the cylinder. The fine aggregate bulk (dry) specific gravity (G_{sb}) is used to compute the fine aggregate volume. **Table 3.1** gives the required minimum values for fine aggregate angularity (Uncompacted Void Content of Fine Aggregate) as a function of traffic level and position within the pavement.

FIGURE 3.4 ***Fine Aggregate Angularity Apparatus***



3.2.1.3 Flat and elongated particles

Flat and elongated particles (F&E) is the percentage by mass or by particle count of coarse aggregates that have a maximum-to-minimum dimension ratio greater than 5:1 (or other ratio, depending on the agency specification). Flat and elongated particles are undesirable because they have a tendency to break during construction and under traffic and they tend to reduce VMA. The test procedure used is ASTM D4791, which deals with flat and elongated particles, and is performed on coarse aggregate larger than the #4 sieve (4.75 mm).

The procedure uses a proportional caliper device (see **Figure 3.5**) to measure the dimensional ratio of a representative sample of aggregate particles. In **Figure 3.5**, the aggregate particle is first placed with its largest dimension between the swinging arm and fixed post at position (A). The swinging arm then remains stationary while the aggregate is placed between the swinging arm and the fixed post at position (B). If the aggregate passes through this gap, then it is counted as a flat and elongated particle. Maximum values for flat and elongated particles specified in AASHTO M 323 are given in **Table 3.1**.

3.2.1.4 Clay content (sand equivalent)

Clay content, more commonly described as sand equivalent (SE), is a percentage of clay material measured on the aggregate fraction that is finer than a #4 sieve (4.75 mm). It is measured by AASHTO T 176, "Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test (ASTM D2419)."

A sample of fine aggregate is mixed with a flocculating solution in a graduated cylinder and agitated to loosen clayey fines present in and coating the aggregate (see **Figure 3.6**). The flocculating solution forces the clay material into suspension above the granular aggregate. After a settling period,

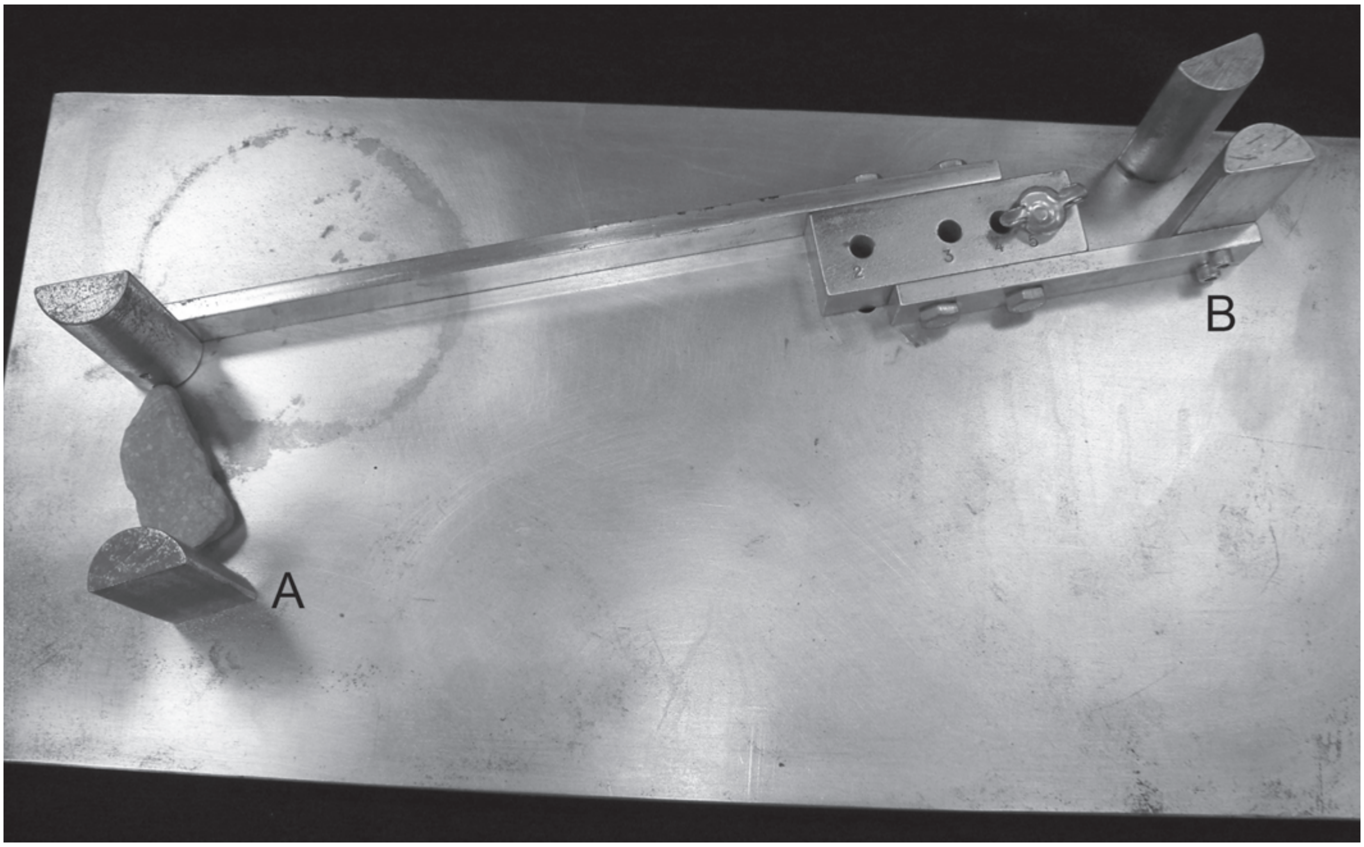


FIGURE 3.5 ***Measuring Flat and Elongated Particles***

the cylinder height of suspended clay and settled sand is measured. The sand equivalent value is computed as the ratio of the sand to clay height readings, expressed as percentage. In essence, this determines how sandy the fine aggregate fraction is.

3.2.2 Source aggregate properties

In addition to the consensus aggregate properties, certain other aggregate characteristics are critical. However, critical values of these properties could not be reached by consensus because needed values are source specific. Consequently, a set of source properties is recommended. Specified values are established by local agencies. While these properties are relevant during the mix design process, they may also be used for source acceptance control. Those properties are toughness, soundness and deleterious materials.

3.2.2.1 Toughness

Toughness tests estimate the resistance of coarse aggregate to abrasion and mechanical degradation during handling, construction and in-service. The most common toughness test is the Los Angeles Abrasion test (AASHTO T 96) or ASTM (C131 or C535) which measures the percent loss of material from the coarse aggregate fraction of a standardized test sample. It is performed by subjecting the coarse aggregate, usually larger than the #8 sieve (2.36 mm), to tumbling and the impact and grinding by steel spheres. The test result is the mass percentage of coarse material lost during the test due to the mechanical degradation. The maximum allowable loss value is typically a range from 30 to 45 percent. The higher the value, the more friable the coarse aggregate, and the greater the breakdown (degradation) of the aggregate from quarrying through stockpiling, HMA manufacturing and under the rollers. The lower the value, the better the skid resistance and tire chain wear resistance of the pavement.

Another test for toughness is Micro-Deval (AASHTO T 327 or ASTM D6928). Micro-Deval is similar to LA Abrasion except that the drum is much smaller and the sample size is 30 percent of that for the L.A. Abrasion test. Also, the aggregate is soaked in water before the test and water is used in the drum during the test. The test has been adopted by some agencies as a replacement for the Los Angeles Abrasion test.

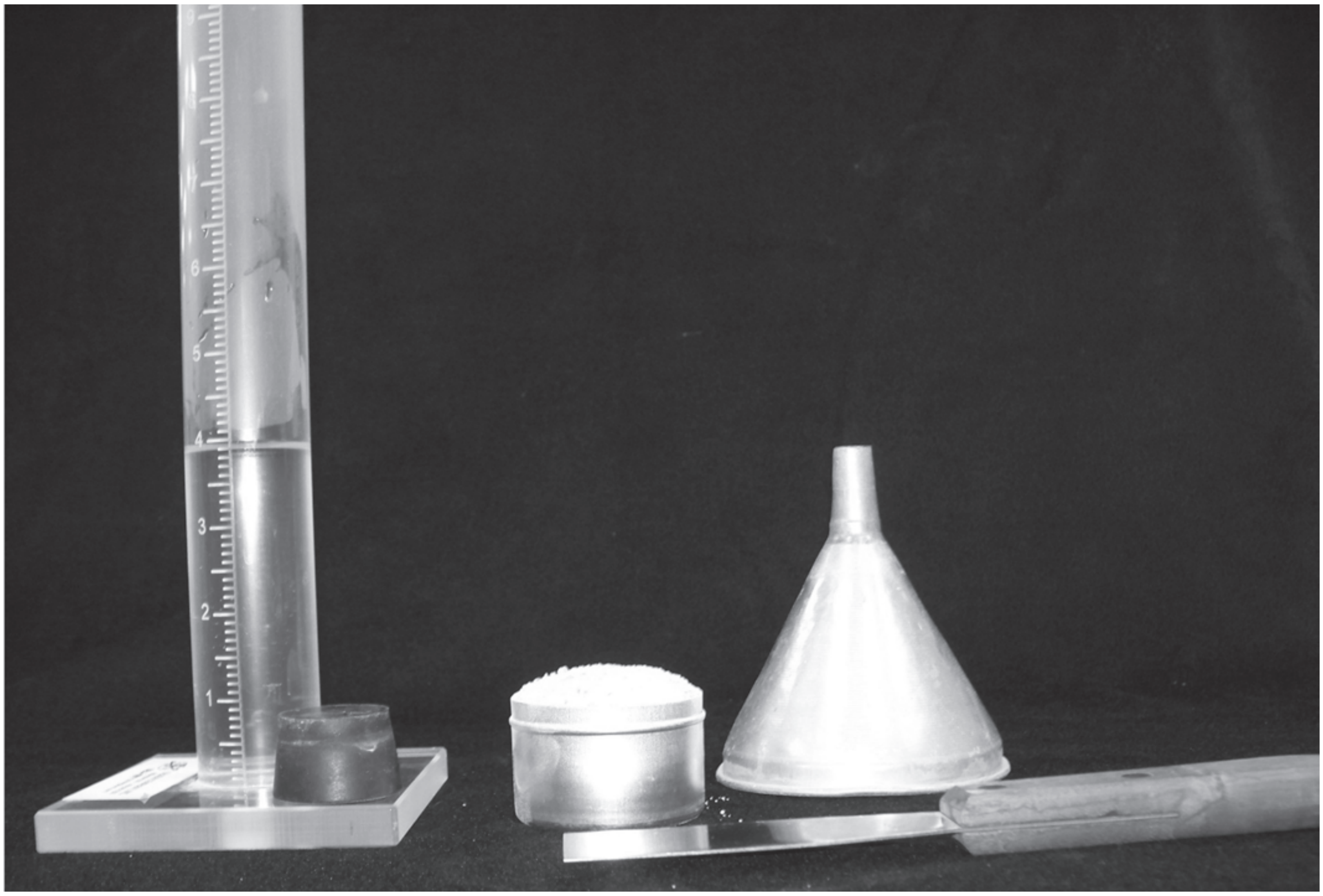


FIGURE 3.6 ***Sand Equivalent Test***

3.2.2.2 Soundness

Soundness tests estimate the resistance of aggregates to in-service weathering. The most common test is Soundness of Aggregate By Use of Sodium Sulfate or Magnesium Sulfate (AASHTO T 104 or ASTM C88) which measures the percent loss of material from an aggregate blend. It can be performed on both coarse and fine aggregate. The test is performed by exposing an aggregate sample to repeated immersions in saturated solutions of sodium or magnesium sulfate followed by oven drying. One immersion and drying is considered one soundness cycle. During the drying phase, salts precipitate in the permeable void space of the aggregate. Upon re-immersion, the salt rehydrates and exerts internal expansive forces that simulate the expansive forces of freezing water. The test result is total percent loss over various sieve intervals for a required number of cycles. Maximum loss values typically range from 10 to 20 percent for five cycles. Magnesium sulfate testing is typically more aggressive than sodium sulfate testing. It is typical for magnesium sulfate loss to be greater than sodium sulfate loss on the same aggregate.

3.2.2.3 Deleterious materials

Deleterious materials are defined as the mass percentage of contaminants such as clay lumps, shale, wood, mica and coal in the blended aggregate. The most common deleterious materials test is Clay Lumps and Friable Particles in Aggregate (AASHTO T 112, ASTM C142). The analysis can be performed on both coarse and fine aggregate. The test is performed by wet sieving aggregate size fractions over specified sieves. The mass percentage of material lost as a result of wet sieving is reported as the percent of clay lumps and friable particles. A wide range of criteria for maximum allowable percentage of deleterious particles exists. Values range from as little as 0.2 percent to as high as 10 percent, depending on the exact composition of the contaminant.

3.2.3 Gradation

It has long been established that the gradation of the aggregate is one of the factors that must be carefully considered in the design of asphalt paving mixtures. The purpose for establishing and controlling aggregate gradation is to provide a sufficient volume of voids in the asphalt-aggregate mixture to accommodate the proper asphalt film thickness on each particle and provide the design air void system to allow for thermal expansion of the asphalt within the mix. Minimum voids in the mineral aggregate (VMA) requirements have been established that vary with the nominal maximum aggregate size to help assure the correct volume of effective binder exists for each mix type.

The gradation of each aggregate material utilized in a mixture should be conducted using the washed sieve analysis procedures designated in AASHTO T 11 and T 27 or ASTM C117 and C136 to properly account for the -200 material. The results should be reported as an accumulative percent passing each respective specified sieve size and reported to the nearest whole percent passing. The exception is the percent passing the #200 sieve (0.075 mm), which should always be calculated and reported to the nearest 0.1 percent passing as shown in **Table 3.2**.

Typically, multiple stockpiles of aggregate are blended to meet the final specified requirements. A washed sieve analysis must be performed on every aggregate ingredient to be utilized in the mixture in order to calculate the final aggregate blend in the mixture to be designed. Most aggregate specifications are based on the final blend of the mixture.

Calculating a blended gradation, assuming all aggregate fractions have a similar Bulk Specific Gravity (G_{sb}):

$$P = (A \times a) + (B \times b) + (C \times c) + \ldots$$

where,

P = the blended percent passing for a given sieve

A,B,C, = the percent passing a sieve for an individual stockpile

a,b,c, = proportion of stockpile to be added in the blend, where total = 1.00.

Washed Sieve Analysis				
U.S. Standard Sieve Number	**Weight Retained**	**% Retained**	**% Passing**	**Report**
1 ½”	0.0	0.0	100.0	100
1”	0.0	0.0	100.0	100
¾”	0.0	0.0	100.0	100
½”	64.0	3.7	96.3	96
⅜”	111.1	6.5	89.8	90
#4	156.0	9.1	80.7	81
#8	201.1	11.7	68.9	69
#16	329.9	19.3	49.6	50
#30	266.1	15.5	34.1	34
#50	212.1	12.4	21.7	22
#100	198.8	11.6	10.1	10
#200	98.0	5.7	4.4	4.4
<200 (T)	75.0	4.4		
Total	1712.1			

A) Weight original sample	1712.1
B) Weight after wash	1646.9
C) Wash loss (A-B)	65.2
S) –200 from sieving	9.8
T) Total –200 (C + S)	75.0

TABLE 3.2 ***Washed Sieve Analysis***

The above-mentioned gradation and blending operations result in an aggregate size distribution based on percentage of mass. Volumetric properties such as air voids and VMA are directly impacted by the amount and size of aggregate particles and the resulting packing characteristics in the final mixture. A gradation can give insight to the final volumetric properties in a particular mixture. However, when the specific gravities of the individual aggregates differ or vary significantly (by 0.20 or more), the blended gradation, based on the mass of the aggregates, may have different volumetric characteristics when compared to an equivalent gradation of materials having similar specific gravities.

Consider the following example where you are given two different mixtures to compare. They both have similar gradations by mass and equivalent aggregate shape, strength and texture. One mix contains aggregates of similar specific gravity and the other contains aggregates of widely differing specific gravity. Based on the gradation, it would be reasonable to assume that the resulting volumetric properties would be similar, but the actual number and sizes of particles in the mixture are not similar and the resulting volumetrics in the compacted mixture will be different.

Field-produced mixtures with significantly different aggregate specific gravities than those used in the mix design will also yield different hot mix volumetric properties. This is one of the reasons why most specifications require a new mix design when the source (and characteristics) of any of the mix ingredients are changed.

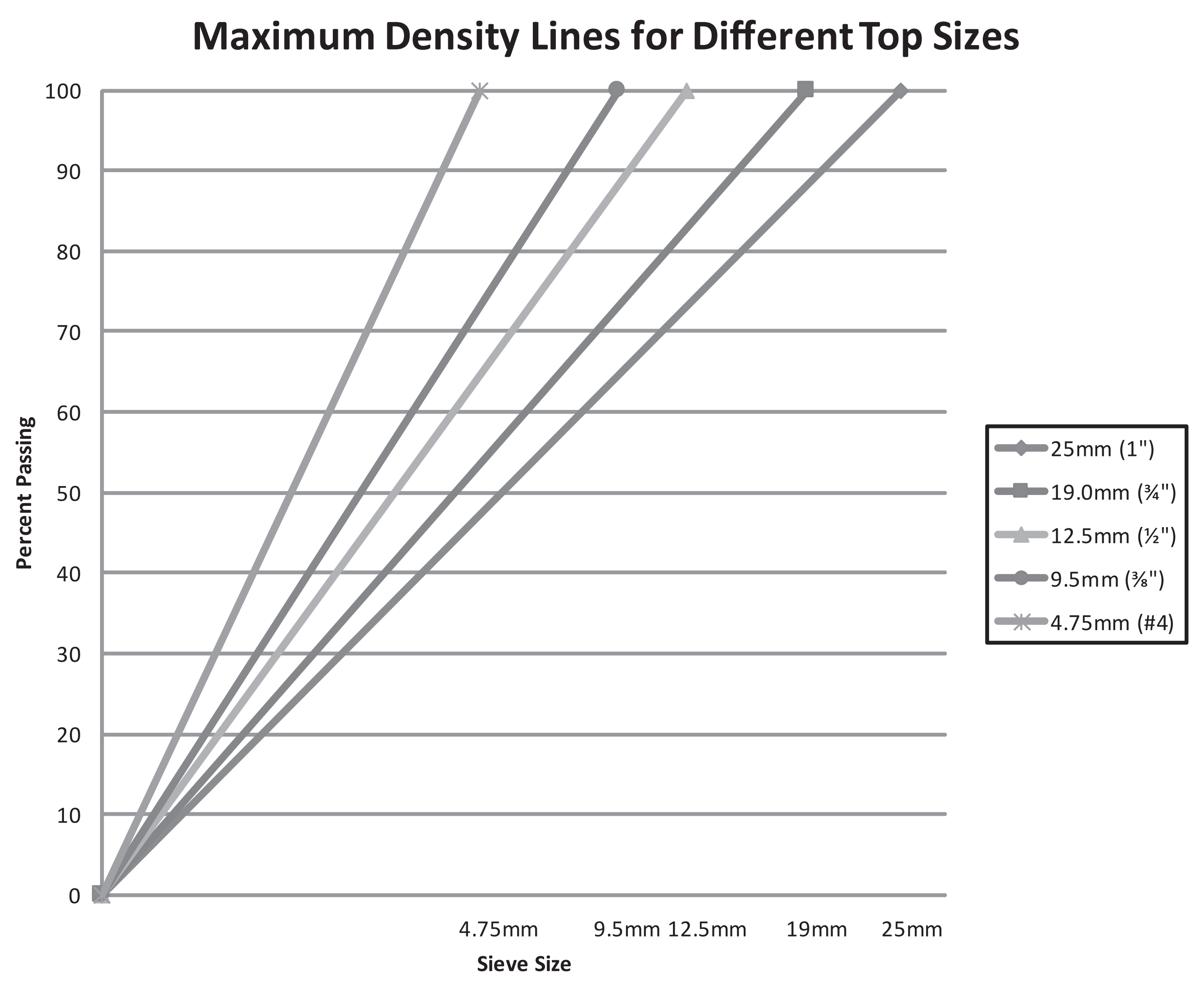
3.2.3.1 Plotting gradations

In the early 1960s, the Bureau of Public Roads (today's FHWA) introduced a gradation chart (see **Figure 3.7**) which is an especially useful tool in evaluating aggregate gradations. The chart uses a horizontal scale, which represents sieve size openings in inches (millimeters) raised to the 0.45 power, and a vertical scale that represents percent passing. An important feature of the 0.45 power chart is the maximum density line. The maximum density line plots as a straight line from the maximum aggregate size to the origin of the chart.

The maximum aggregate size is typically the largest sieve size through which 100 percent of the aggregate will pass. The maximum aggregate size is defined as one sieve size larger than the nominal maximum aggregate size (NMAS). The NMAS is defined as one sieve size larger than the first sieve size to retain more than 10 percent of the aggregate gradation.

The maximum density line was developed for rounded, uncrushed materials. The maximum density condition for crushed particles typically

FIGURE 3.7 ***0.45 Power Chart Showing Several Maximum Density Lines (MDLs) for Various Top Sizes***



Sieve Size Within Restricted Zone	Minimum and Maximum Boundaries of Sieve Size for Nominal Maximum Aggregate Size (Minimum and Maximum Percent Passing)									
	37.5 mm		25.0 mm		19.0 mm		12.5 mm		9.5 mm	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
0.300 mm	10.0	10.0	11.4	11.4	13.7	13.7	15.5	15.5	18.7	18.7
0.600 mm	11.7	15.7	13.6	17.6	16.7	20.7	19.1	23.1	23.5	27.5
1.18 mm	15.5	21.5	18.1	24.1	22.3	28.3	25.6	31.6	31.6	37.6
2.36 mm	23.3	27.3	26.8	30.8	34.6	34.6	39.1	39.1	47.2	47.2
4.75 mm	34.7	34.7	39.5	39.5	–	–	–	–	–	–

TABLE 3.3 ***Restricted Zone Boundaries***

occurs when the gradation is coarser than the plotted maximum density line and varies depending on the shape and texture of the aggregate material. As part of the mix design process, the aggregate gradation should be plotted on the 0.45 power gradation chart.

Two Superpave-specific features utilize the 0.45 power chart—control points and a restricted zone.

3.2.3.2 Control points

The control points define the type of mix and act as master ranges between which gradations must pass. Control points are placed at the nominal maximum size, an intermediate size (2.36 mm), and the smallest size (0.075 mm). Superpave control point limits vary depending on the nominal maximum aggregate size of the design mixture as shown in **Table 6.3** of chapter 6.

3.2.3.3 Restricted zone

From past experience it was known that if the gradation would pass through the restricted zone between the #8 and #50 sieves, it was an indication that there was possibly too much natural sand in the mix, resulting in possible mix tenderness. The restricted zone was introduced by the Superpave system and it has since been removed from AASHTO M 323. It resided along the maximum density line between the intermediate size (either 4.75 mm or 2.36 mm) and the 0.300-mm size. **Figure 3.8** shows the control points and restricted zone for a 12.5-mm Superpave mixture (12.5-mm nominal maximum and 19.0-mm maximum size). The restricted zone formed a zone through which it was generally recommended that the gradation not pass. The restricted zone was enforced to varying degrees.

National Cooperative Highway Research Project 09-14 showed that if the other Superpave aggregate properties were met, then violation of the restricted zone did not produce poor performing pavements. It was concluded that aggregate angularity and not size was key to pavement performance. Moreover, numerous field examples of pavements that were in violation of the restricted zone, but that were performing well, have been documented. However, a designer should understand the need to be cautious with gradations that pass through the restricted zone and utilize a significant amount of natural sand. Such gradations have a greater tendency to be tender and are to be approached with due care.

Aggregate gradation analysis and the combining of aggregates to obtain the desired gradation are important steps in hot mix asphalt mix design. The aggregate gradation must meet the gradation requirements of the project specifications and yield a mix design that meets the criteria of the mix design method. The gradation should also be made up of the most economical aggregates available that are of suitable quality.

3.2.4 Specific gravity

The specific gravity of an aggregate is the ratio of the weight of a unit volume of material to the weight of the same volume of water at 73.4°F (23.0°C). This property is used in mix volumetric calculations for voids determination. Also, bulk specific gravities are used in the computations for adjusting quantities of the aggregate components that are to be used in an HMA mix

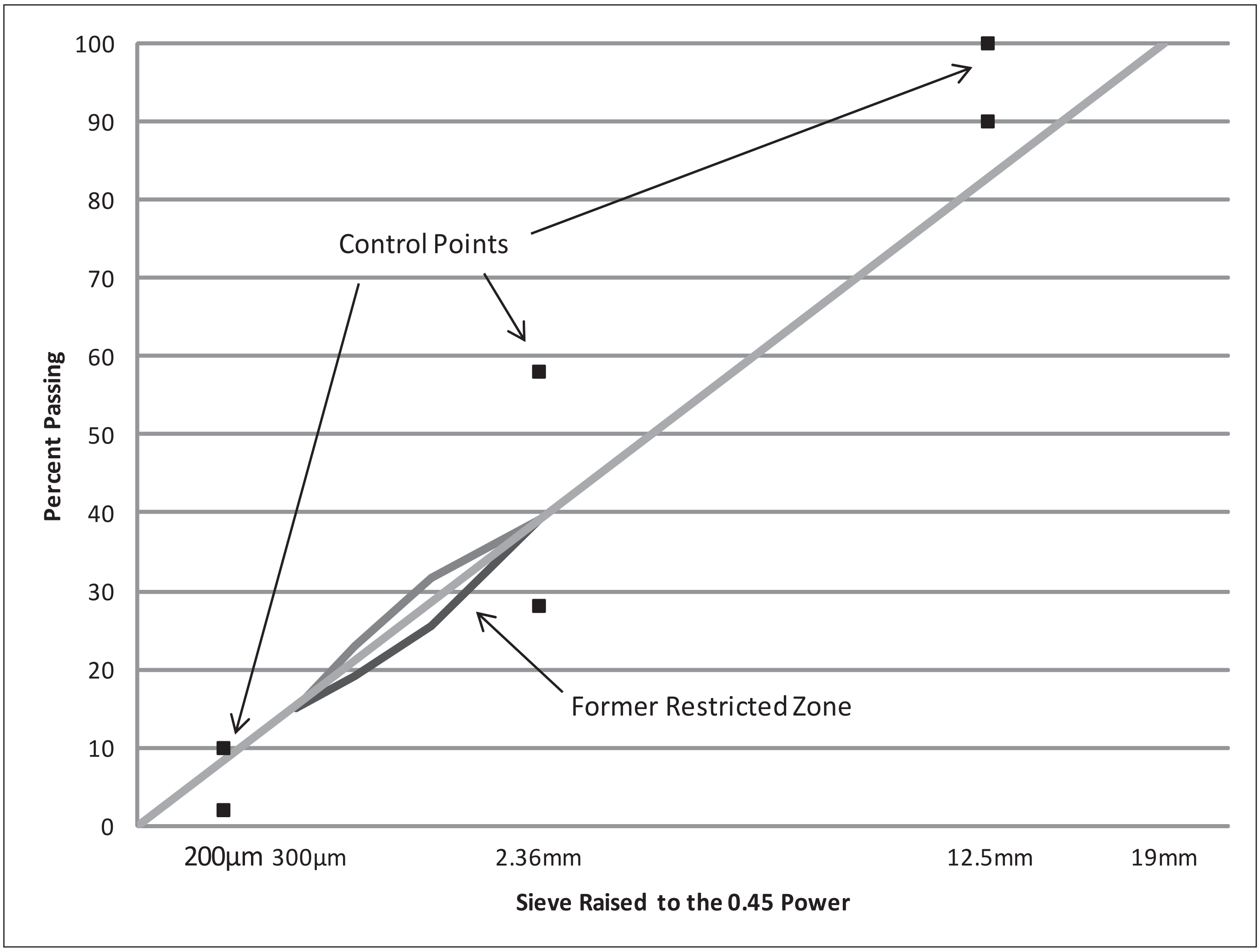


FIGURE 3.8 ***Gradation Control Points for 12.5 mm NMAS (19 mm MAS) with Former Restricted Zone***

because of the differing specific gravities of various aggregates.

The three generally accepted types of specific gravities for aggregate use in hot mix asphalt are the following:

- apparent specific gravity (G_{sa});
- bulk (dry) specific gravity (G_{sb}); and
- effective specific gravity (G_{se}).

Apparent specific gravity considers the volume as being the volume of the aggregate itself. It does not include the volume of any pores or capillaries that become filled with water after a 15- to 19-hour soaking. Bulk (dry) specific gravity considers the overall volume of the aggregate particle, including the pores that become filled with water after a 15- to 19-hour soaking. The effective specific gravity considers the overall volume of the aggregate excluding the volume of pores that absorb asphalt. Whereas bulk and apparent specific gravities can relate to individual aggregates or combined aggregates, effective specific gravity relates exclusively to the total combined aggregate structure in a mix of HMA. Additional information regarding the determination and use of aggregate specific gravities is discussed in chapter 5.

NOTE: *The accuracy of specific gravity measurements for mix designs is important. Unless specific gravities are determined to four significant figures (three decimal places), an error in air voids value of as much as 0.8 percent can occur. The Asphalt Institute recommends the use of weigh scales whose sensitivity will allow a mix batch weighing 1,000 to 5,000 grams to be measured to an accuracy of 0.1 gram.*

	5/8" Chips	Washed Screenings	Screenings	Natural Sand	Combined Aggregate
Stockpile %	35	20	30	15	
% Passing					
19.0 mm	100	100	100	100	100
12.5 mm	89	100	100	100	96.2
9.5 mm	52	100	100	100	83.2
4.75 mm	3	94	79	99	58.4
2.36 mm	1	71	46	98	43.1
1.18 mm	1	48	28	96	32.8
0.600 mm	1	28	19	89	25.0
0.300 mm	1	13	15	54	15.6
0.150 mm	1	5	13	10	6.8
0.075 mm	0.8	3.1	10.7	2.3	4.5

TABLE 3.4 ***Typical aggregate gradation data***

3.3

Aggregate batching and mix sample preparation

There are no AASHTO, ASTM or other widely accepted standards for the batching of aggregates for asphalt mix design. Many variations exist and are specified by some agencies. This section will detail several different batching methods.

An important part of any batching procedure is to completely dry the aggregates before beginning. Aggregate samples from the plant or quarry may come to the lab saturated with moisture. Fine aggregate stockpiles tend to absorb a higher percentage of moisture than coarser aggregate. Without completely drying the aggregate first, the absorbed moisture will increase the aggregate weight, resulting in inaccurate material proportions in the batch.

Prior to batching samples for the mix design, gradations should be performed on each material submitted. Aggregate material submitted for mix design should be accompanied with production test results. Submitted materials should then be compared with production test results to assure the materials submitted for mix design are representative of the materials that will be used in the project. Softer aggregates tend to degrade more during the production process than harder aggregates, often resulting in a finer gradation than the aggregate sample used for mix design. Many designers take this into account by determining the extent of the additional fines created during production and making a corresponding adjustment to the aggregate batch for mix design. Performing this adjustment may help avoid the drop in air voids and VMA often seen in plant mixes versus laboratory mixes. Therefore, a careful comparison of gradations and all other specified characteristics should be made prior to conducting a mix design. Conducting a mixture design using aggregate that does not meet a project specification or is not representative of field-produced material is of little use.

There are many different methods that can be utilized to prepare aggregate samples for mixture testing. They can range from very accurate and time-consuming to relatively quick but with less accuracy. Accurately prepared specimens that are representative of the final aggregate blend produced by the mixing plant will give the most reliable mixture design data. The mix designer should use the most accurate method practical that obtains representative and reproducible results and specimen with minimal variability. Many agencies specify certain methodologies for specimen preparation. Agency-specified procedures may deviate from the following methods described and should be thoroughly understood by the designer.

This manual will discuss three methods of batching sample specimens. Many variations exist. The following methods are known to obtain accurate specimen samples and minimize variability. The Asphalt Institute recommends that in order to achieve the highest level of accuracy and repeatability of laboratory-prepared specimens, individual samples should be batched rather than multiple specimens being batched, mixed and then divided into individual samples.

3.3.1 Method 1—Partial fractionation of individual stockpiles

The smallest practical sieve for a large capacity tray-type shaker is usually 2.36 or 1.18 mm. Fractionate each stockpile sample on each specified sieve through the #8 or #16, as determined by the designer, leaving the entire amount passing the smallest sieve as one fraction to be added during batching. The pan material using this method becomes all of the material that passes the 1.18-mm sieve (or whatever sieve is on the bottom of the stack). Large capacity, tray-type mechanical shakers have the capacity to fractionate samples up to 50 or more pounds. Care should be taken to prevent overloading of sieve screens. The amount allowed on each sieve is dependent on the nominal maximum aggregate size (NMAS), as outlined in ASTM C136.

A typical batching sheet for one stockpile would appear as shown in **Table 3.5** using the following equations, and must be repeated for each stockpile of aggregate to be used:

Determine the grams needed from each aggregate stockpile:

$$\begin{matrix}\text{Stockpile}\\ \text{grams needed}\end{matrix} = \frac{\text{Total specimen size} \times \text{bin split\%}}{100}$$

Determine the percent retained on each sieve:

$$\begin{matrix}\text{\% Retained on}\\ \text{a sieve}\end{matrix} = \begin{matrix}\text{\% Passing the next larger sieve} -\\ \text{\% Passing the sieve}\end{matrix}$$

Determine the amount to be contributed from each individual sieve fraction:

$$\begin{matrix}\text{Grams of each}\\ \text{fraction required}\end{matrix} = \frac{\text{\% Retained} \times \text{Grams Needed}}{100}$$

Care must be taken to assure that the large amounts of pan material shown are uniformly blended and added to the batch sample.

Total specimen size = 5000.0 grams				
⅝" Chips				
Bin Split %	35			
Grams needed	$5000.0\text{ g} \times .35 = \underline{1750.0}\text{ g}$			
	% Pass	**% Retained**	**Ind.**	**Cum.**
19.0 mm	100	0	0	0
12.5 mm	89	11	192.5	192.5
9.5 mm	52	37	647.5	840
4.75 mm	3	49	857.5	1697.5
2.36 mm	1	2	35	1732.5
1.18 mm	1	0	0	1732.5
Pan	0	1	17.5	1750

TABLE 3.5 ***Sample batching sheet 1***

3.3.2 Method Two—Pre-blending of samples prior to fractionation

This method utilizes a process where a sample of aggregate is prepared by combining a predetermined amount of every stockpile sample into one combined blend and then fractionates the combined aggregate sample into a chosen number of size fractions. This method mimics a batch plant, is relatively quick and easy, but has less flexibility and accuracy than Method 1. Method 2 is dependent on the mix design aggregates submitted being representative of field production and the accuracy of splitting out batching portions that are representative.

Because the aggregate is combined, then fractionated, the specific gravities of each aggregate must be reasonably similar. This method should not be used if lightweight or heavyweight aggregate is blended with regular aggregate. This method also loses some batching versatility because the bin splits are locked in once the combined aggregate is fractionated. Changing the percentages of aggregates used will require the process to be started over.

Stir each bucket or pan as well as possible and scoop the required amount from each bucket into a batching pan. Fractionate the resulting pan using a sieve shaker, making sure to limit the quantity of material on a given sieve so that all particles have an opportunity to reach the sieve openings a number of times during the sieving operation.

FIGURE 3.9 ***Fractionating aggregate on a large tray-type shaker***

After shaking, remove the material from each tray and place in a labeled pan designated for that particular size fraction. Make sure that each pan is as homogeneously mixed as possible.

This method can be adjusted to use whichever sieves are desired. For example, if the chosen sieves were 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm and the pan, it would produce six pans that contain fractionated, combined aggregates. If the chosen sieves were 12.5 mm, 4.75 mm, 0.600 mm and the pan, it would produce four pans that contain fractionated, combined aggregates.

Example:

For six 5,000-gram batches, a total of 33,000 grams will need to be combined in proportion to the cold feed bin percentages and then sieved. Note that this large of a batch may require the aggregate to be fractionated in multiple batches to avoid blinding the sieves:

Scoop ⅝" Chips:	33,000 × 0.35 = 11,550 grams
Scoop Washed Screenings:	33,000 × 0.20 = 6,600 grams
Scoop Screenings:	33,000 × 0.30 = 9,900 grams
Scoop Natural Sand:	33,000 × 0.15 = 4,950 grams
Add to Check:	33,000 grams

For this example, the following trays were used: 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm and the pan. When fractionating with fewer than the five tray slots typically present, fill the extra slots at the top with sieve trays larger than

Size Fraction	JMF Combined Aggregate (% passing)	% Retained on Each Fraction	Total Batch Size	Grams of Each Fraction Required	Cumulative Weight for Batching
+ 12.5 mm	96.2	3.8	5,000	190.0	190.0
+ 9.5 mm	83.2	13.0		650.0	840.0
+ 4.75 mm	58.4	24.8		1240.0	2080.0
+ 2.36 mm	43.1	15.3		765.0	2845.0
+ 1.18 mm	32.8	10.3		515.0	3360.0
Pan	0	32.8		1640.0	5000.0
			Total Weight	5000.0	

TABLE 3.6 ***Sample batching sheet 2***

the largest particle size of the blend or relief sieves if necessary. Fractionate each pan, placing each resulting size fraction into a separate pan.

Calculate the number of grams required of each size fraction to make each 5,000-gram batch as previously described in Method 1 and summarized in **Table 3.6**.

In this example, there will be sufficient material to batch six 5,000-gram batches.

Methods 1 and 2 both involve batching a combination of fines (also referred to as "pan material"). Even though the particles are small, there may be some level of segregation. If the designer is concerned about obtaining a representative sample from the pan material, the fines can be thoroughly mixed with 3 percent water to make the fines' combination more homogeneous. Then batch 3 percent more of the moist fines than would be required of dry fines to account for the extra moisture. The moisture will be driven away when the aggregates are placed in the oven to bring them up to mixing temperature.

In the above example, suppose there were 10,740 total grams of unfractionated fines (–1.18-mm material) from the 33,000 grams that were fractionated. Add 322.2 grams of water (10,740 × 3 percent) to the total fines and stir until the resulting moist fines appear homogeneously mixed. Then, instead of adding 1,640 grams of dry fines into the 5,000-gram batch, add 1,640 × 1.03 = 1,689.2 grams of moist fines for a total of 5,049.2 grams in the batch. In the oven, 49.2 grams of water will evaporate, leaving a 5,000-gram unsegregated dry batch.

3.3.3 Method 3—Total fractionation of all aggregate materials

Fractionate each aggregate source with a sieve shaker on every specified sieve, with each aggregate fraction individually batched for each specimen. After shaking, remove the material from each tray and place in a separate, labeled pan. Using the gradation data shown in **Table 3.4** for four aggregate sources and the standard nest of 11 sieves will result in 44 pans from which to batch.

EXAMPLE:

Fractionate each aggregate individually and keep each fraction in a separate, labeled pan or bucket.

Batching will be done using the entire range of sieves in a standard nest, plus the material passing the last sieve in the stack (#200) referred to as "pan material."

Calculate the number of grams required of each size fraction to make a 5,000-gram batch.

First, calculate the number of grams needed for each aggregate source according to the percentage used. Next, calculate the percentage retained on each sieve and multiply it by the number of total number of grams of the aggregate source. Finally, add the number of grams of each fraction cumulatively for batching.

It is recommended to calculate the cumulative weights as shown in **Table 3.7** so that each fraction can be added without re-taring the scale each time an aggregate fraction is added to a batch specimen. Taring the scales after adding each new fraction can result in significant error, especially when using methods requiring batching from dozens of pans.

	⅝" Chips			Washed Screenings			Screenings			Natural Sand		
% Used	35			20			30			15		
% Passing												
# of Grams	1750			1000			1500			750		
	% Pass	Ind.	Cum.	% Pass	Ind.	Cum.	% Pass	Ind.	Cum.	% Pass	Ind.	Cum.
19.0 mm	100	0.0	0.0	100	0.0	1750.0	100	0	2750.0	100	0.0	4250.0
12.5 mm	89	192.5	192.5	100	0.0	1750.0	100	0	2750.0	100	0.0	4250.0
9.5 mm	52	647.5	840.0	100	0.0	1750.0	100	0	2750.0	100	0.0	4250.0
4.75 mm	3	857.5	1697.5	94	60.0	1810.0	79	315	3065.0	99	7.5	4257.5
2.36 mm	1	35.0	1732.5	71	230.0	2040.0	46	495	3560.0	98	7.5	4265.0
1.18 mm	1	0.0	1732.5	48	230.0	2270.0	28	270	3830.0	96	15.0	4280.0
0.600 mm	1	0.0	1732.5	28	200.0	2470.0	19	135	3965.0	89	52.5	4332.5
0.300 mm	1	0.0	1732.5	13	150.0	2620.0	15	60	4025.0	54	262.5	4595.0
0.150 mm	1	0.0	1732.5	5	80.0	2700.0	13	30	4055.0	10	330.0	4925.0
0.075 mm	0.8	3.5	1736.0	3.1	19.0	2719.0	10.7	34.5	4089.5	2.3	57.8	4982.8
Pan	0	14.0	1750.0	0	31.0	2750.0	0	160.5	4250.0	0	17.2	5000.0

TABLE 3.7 ***Sample batching sheet 3***

It should be noted that Method 3 is susceptible to significant error when batching materials with high levels of dust. It is not uncommon for elevated amounts of static dust (–#200) to “cling” onto and remain in the larger fractionated sizes of material. In order to compensate for this potential error, a prepared trial batch should be analyzed using a washed sieve analysis. The amount of dust (–#200) in excess of that desired will need to be removed from the –#200 pan material added during batching. This reduction in pan material will require an adjustment (increase) on the sieve fractions that are determined to retain this fugitive dust (typically the +#50 through +#200) in order to obtain an exact batch weight. A thorough understanding of the material being utilized by the designer is necessary when using Method 3. Mix designers are encouraged to use Methods 1 or 2 for ordinary production mix design.

3.3.4 Batching other ingredients

Many different types of additives may be added to a batched specimen. The additives may include reclaimed asphalt, recycled shingles, lime, ground tire rubber or a number of other specialty products. The manner in which additives are incorporated into the final mixture must consider the temperature to which the aggregate will be heated. Standard hot mix asphalt aggregates are typically heated to levels well over 150°C (300°F).

RAP and RAS are typically batched separately from the virgin aggregates. Prolonged exposure to elevated temperature can cause the materials to conglomerate and may alter the binder properties in the recycled materials. RAP and RAS materials are typically prepared and preheated to 110°C (230°F) for no more than two hours and then added at the time of mixing. Caution must be exercised, as any excess moisture remaining in the recycled products will be released as steam in an uncontrolled manner that can cause injury. Preheating of recycled products should eliminate the majority of the moisture and raise the temperature to a high enough level to assure thorough mixing with the virgin aggregate.

Hydrated lime is often used as a treatment for the prevention of moisture damage and is discussed in chapter 9. Hydrated lime is typically added to the prepared samples prior to heating and mixing. Hydrated lime may be added as either a dry component or as slurry. Each method has been effective in improving AASHTO T 283 testing. The method of adding hydrated lime to the mixture design sample should duplicate the method utilized at the hot plant. See chapter 9 for more details on methods to incorporate hydrated lime in field-produced mix.

FIGURE 3.10 ***Batching into distinct piles***

There is a multitude of other less-common additives and modifiers that are added to the aggregate or binder during the laboratory mix design process, many of which are proprietary in nature. The supplier guidelines for use and incorporation must be followed very carefully unless otherwise specified by the owner or agency.

3.3.5 Batching hints

Dried aggregate is often scooped from either pans or buckets. If scooping from a pan, flat-bottomed scoops are used to obtain the entire cross-section of material, including the fines that typically settle to the bottom of the pan. If scooping from buckets, round-bottomed scoops can be used.

Placing each aggregate fraction in the batching pan in distinct piles will aid in batching (as shown in **Figure 3.10**). If the batch weight is accidentally exceeded for any particular aggregate, it is easier to remove some from a distinct pile rather than accidentally removing a portion of another aggregate fraction when they are all piled together.

CHAPTER 4

Laboratory Mixture Testing

4.1

Introduction

Laboratory asphalt mixture testing is primarily conducted to determine two fundamental asphalt mix properties the bulk specific gravity of the mixture (G_{mb}) and the theoretical maximum specific gravity of the mix (G_{mm}). These values are utilized in calculating volumetric properties discussed in chapter 5 which are specified requirements for most asphalt mix design procedures. Laboratory asphalt mix specimens are also prepared for further mixture testing such as moisture sensitivity or other recommended or specified performance tests. The specific gravity of aggregates, binders and asphalt mixtures are determined using standardized procedures.

4.2

Selection of trial binder contents, compaction temperatures and mixing times

Most asphalt mixture design procedures are conducted by preparing multiple trial mixtures at different binder content percentages. Decisions to be made regarding what binder contents to use in the mix design process include the number of trial binder contents, the increments between binder contents and the range of binder contents to be covered by the trial blends.

Using too few binder increments for each aggregate blend can lead to uncertainty about the design results. For example, basing a mix design on a single trial binder percentage would allow no check on reliability of the test results, plus no sense of how the mix volumetrics change with different binder contents. Using too many binder increments would make the design process unnecessarily long. If the material source is unknown to the designer, use five trial binder contents. If the designer is familiar with the material source, the number of trials may be reduced to no less than three.

Likewise, the increments between trial binder percentage points should be far enough apart to provide a distinction between points, but close enough together to avoid lengthy interpolation. The most commonly selected increment between binder percentage points is 0.5 percent. However, if the designer is unfamiliar with aggregates being used, or if other materials or additives are used such as RAP, RAS, lime, WMA, etc., more points at smaller increments over a wider range may be needed.

The ultimate goal is to prepare batches that bracket the anticipated design binder content. The designer should evaluate the aggregate type, the aggregate structure and any additional materials or additives before selecting the beginning and ending binder points. There are several guidelines to keep in mind:

- binder demand increases as the nominal maximum aggregate size of the mix decreases;
- absorptive aggregates have a greater binder demand;

- for a given nominal maximum aggregate size, a fine aggregate gradation will require more binder than a coarse aggregate gradation;
- if higher VMA is anticipated due to hard, angular aggregates, more binder will be required;
- mixes with a higher P_{200} tend to require more binder than those with a lower P_{200}; and
- ask the manufacturer for guidance when proprietary additives are used.

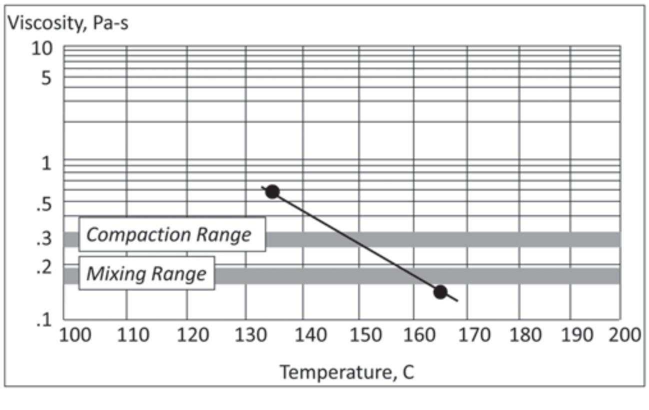
In the end, the designer should never extrapolate a higher or lower design binder content from outside the range of trial points.

4.2.1 Mixing and compaction temperatures

For years, asphalt mix design procedures have used equiviscous temperature ranges for selecting laboratory mixing and compaction temperatures. The purpose of using equiviscous mixing and compaction temperatures in laboratory mix design procedures is to normalize the effect of asphalt binder stiffness on mixture volumetric properties. By using this procedure, a particular asphalt mixture of the same aggregate structure will exhibit very similar volumetric properties regardless of whether a hard or soft asphalt binder is used.

In the equiviscous method, the viscosity of the asphalt binder is determined at two test temperatures, establishing a relationship between temperature and viscosity as shown in **Figure 4.1**. Compaction temperatures are determined where the viscosity-temperature line crosses the compaction viscosity range of 0.28 ± 0.03 Pa-s. Mixing temperatures are determined where the viscosity-temperature line crosses the mixing viscosity range of 0.17 ± 0.02 Pa-s. This method still works well for unmodified asphalt binders.

FIGURE 4.1 ***Mixing and compaction temperature chart***



However, the increased usage of highly modified asphalt binders frequently resulted in unusually high equiviscous mixing and compaction temperatures, often over 350°F (177°C). At these excessively high mixing temperatures, there are potential problems with binder degradation and increased binder stiffening (aging) during the mixing process. Additionally, high compaction temperatures lead to potential problems in obtaining accurate density data due to low mix stiffness, excess absorption of asphalt binder into some types of aggregates and drain-down of asphalt binder in some coarse mixes.

Recent research outlined in NCHRP Report 648, "Mixing and Compaction Temperatures of Asphalt Binders in Hot-Mix Asphalt," suggests that modified asphalt binders should be tested following one of two procedures: the DSR Phase Angle Procedure or the DSR Steady Shear Flow Procedure.

The DSR Phase Angle Procedure is performed by conducting a shear frequency sweep from 0.1 to 100 radians per second (rad/s) at a minimum of three temperatures, developing a master curve at 176°F (80°C), and determining the frequency where the phase angle equals 86 degrees. This frequency is then input into an equation to calculate mixing and compaction temperatures.

The DSR Steady Shear Flow Procedure uses a shear stress sweep from 50 to 1000 Pa at a minimum of three test temperatures to determine the steady state viscosity. This viscosity is then plotted as a function of temperature, from which the mixing and compaction temperatures are derived.

Note that the recommended procedures for determining laboratory mixing and compaction temperatures do not apply to asphalt binders that have been modified with Ground Tire Rubber (GTR). Users should refer to other existing practices or supplier recommendations to determine appropriate mixing and compaction temperatures for GTR-modified asphalt binders.

Laboratory mixing and compaction temperatures are intended for determining design volumetric properties of the asphalt mixture and are not intended to represent field mixing and compaction temperatures at the project level. The laboratory mixing and compaction temperatures shall apply equally to the mix design, the Quality Control (QC) and Quality Acceptance (QA) test specimen in order to have a valid comparison of their volumetric properties. In an asphalt mix facility, the mixing temperature can best be defined as the temperature at which the aggregate can be sufficiently dried and uniformly coated, not to exceed 350°F (177°C). The compaction temperature for an asphalt mix is usually in the range of 275–310°F (135–155°C) and is based solely on the ability of the compaction equipment to achieve adequate in-place density.

In summary, for unmodified asphalt binders, the laboratory mixing and compaction temperature can be determined using the equiviscous method. For modified asphalt binders, the laboratory mixing and compaction temperature can be determined using either the DSR Phase Angle Procedure or the DSR Steady Shear Flow Procedure. Additionally, the supplier's recommendation may also be used. The mixing and compaction temperatures they have determined for their individual products have proven to be appropriate. Regardless of the selected procedure, the Asphalt Institute recommends that laboratory mixing temperatures do not exceed 350°F (177°C).

4.2.2 Laboratory mixing operations

The goal of laboratory mixing operations is to produce uniform batches of properly coated HMA mixtures. Mixing is typically done with either a planetary mixer (see **Figure 4.2**) with wire whips or a five-gallon bucket mixer (see **Figure 4.3**).

The following points are important to remember in the mixing operation:

- Place the aggregate and binder in the oven at the mixing temperature for at least two hours before mixing. To avoid excessive aging of the binder, do not allow it to stay at the mixing temperature for much over the time needed to bring it to temperature and complete the mixing operation.
- Place all mixing bowls, whips and molds in the oven 30 minutes to an hour before mixing. Keep enough molds in the oven to rotate their use, always keeping a hot mold available.
- Keep the binder in smaller containers (no more than a gallon) to avoid aging it from constant reheating. Transfer the hot binder to quart cans or other small containers as needed to make it safer and easier to pour. Small stainless steel pitchers from commercial restaurant supply stores work very well.
- Place the spatula blade on a hot plate, making sure that the wooden handle does not touch the hot plate (usually a heavier nonflammable object is placed on the blade to keep the spatula from slipping off).
- Keep a stack of paper squares next to the scale to intercept the poured binder stream when it is about to reach the proper weight and to dip out excess binder.
- It is good practice to place something on the scale to protect the electronics from overheating when the hot bowl/bucket is placed on it.
- "Butter" the mixing bowl and whip by mixing a dummy batch before subsequent design batches to coat the equipment with binder as an aid in maintaining consistent binder contents.
- Make sure to have an oven set at the compaction temperature ready to receive the freshly mixed batches.

FIGURE 4.2 ***Planetary benchtop mixer***

FIGURE 4.3 ***Five-gallon bucket mixer***

When you are ready to mix, place the mixing bowl/bucket on the scale and tare it. Pour the heated aggregate batch into the bowl and verify the required weight (see **Figure 4.4**). Form a crater in the center of the aggregate to receive the binder and keep it from flowing to the edges of the bowl. Add the correct number of grams of binder, dipping out any excess binder with a folded paper dipper. Mix until all of the aggregate is thoroughly coated. ASTM D6926 suggests mixing for approximately 60 seconds for single specimen batches and approximately 120 seconds for multiple specimen batches. When transferring the mixture from the mixing bowl to the conditioning pans, make sure to thoroughly scrape the mixed fines and asphalt from the bowl and whips into the conditioning pans with a hot spatula.

4.2.3 Mixture conditioning

The absorptive characteristics of aggregates used in asphalt mixes can greatly impact laboratory tests and resulting volumetric calculations. Current laboratory procedures for molding Marshall specimens (AASHTO T 245) do not stipulate mixture conditioning prior to compaction. Superpave laboratory compaction procedures specified in AASHTO T 312 call for conditioning of loose mix prior to compaction according to AASHTO R 30. R 30 specifies a 2-hour conditioning period prior to laboratory compaction. Aggregate absorption characteristics can impact both G_{mb} and G_{mm} values. The maximum theoretical specific gravity test procedure, AASHTO T 209, requires a mixture conditioning period of at least 2 hours. In an effort to provide the most accurate mixture design results, the Asphalt Institute recommends the consideration that all G_{mb} and G_{mm} mixture samples be conditioned a minimum of 2 hours prior to the compaction of G_{mb} samples and the cooling and testing of G_{mm} samples, regardless of the mix design procedure utilized. Aggregate sources that have high water absorption values (above 2.0 percent) should be conditioned for an extended period of time (up to 4 hours). **Figure 4.5** demonstrates how different levels of absorption in the aggregate can greatly affect the increase of G_{mm} over time.

FIGURE 4.4 ***Weighing binder into mixing bowl***

AASHTO R 30 gives the standard practice for mixture conditioning. It differentiates between mixture conditioning for volumetric mix design,

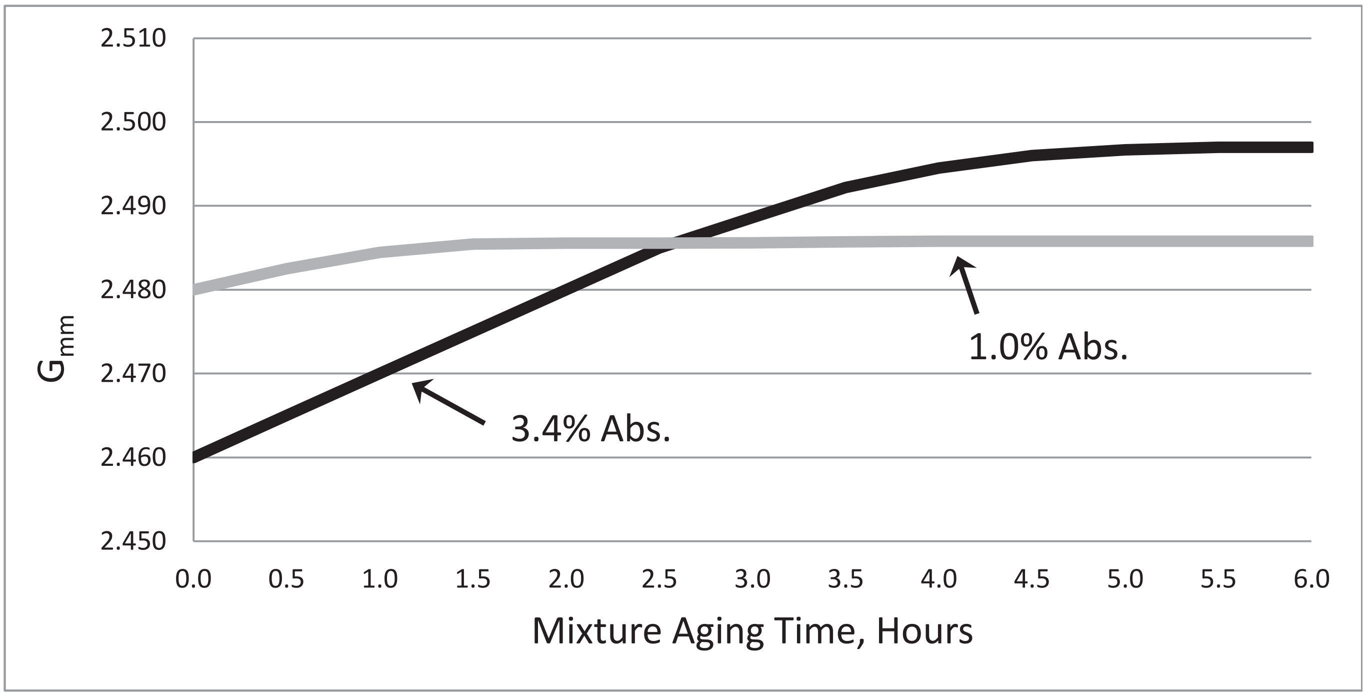


FIGURE 4.5 ***Effects of absorption on G_{mm}***

short-term conditioning for mixture mechanical property testing and long-term conditioning for mixture mechanical property testing. The purpose of the conditioning for volumetric mix design is to allow for binder absorption during the mix design process. The short-term conditioning for mixture mechanical property testing is designed to simulate the plant-mixing and construction effects on the mixture. The long-term conditioning for mixture mechanical property testing is designed to simulate the aging that the mixture would experience over 7–10 years of service life.

4.2.3.1 Equipment

The equipment required for conditioning the mixture includes:

(a) oven—a forced-draft oven capable of maintaining the desired temperature setting within ± 5.4°F (3°C);
(b) thermometers—having a range of 122°F (50°C) to 500°F (260°C), readable to 1.8°F (1°C); and
(c) miscellaneous—a shallow metal pan for heating uncompacted asphalt mix, a metal spoon or spatula, timer and gloves for handling hot materials.

4.2.3.2 Procedure

(a) Place the mixture in a shallow metal pan and spread it to an even thickness between 25 and 50 millimeters in depth. Place the mixture and pan, as shown in **Figure 4.6**, in the oven for 2 hours ± 5 minutes at a temperature equal to the mixture's compaction temperature ± 5.4°F (3°C). *Note that the conditioning time may need to be increased to be more representative of field conditions when higher-absorptive aggregates (more than 2 percent) are used, subject to agency approval.*

FIGURE 4.6 ***Mixture conditioning for volumetric mix design***

(b) Stir the loose mixture every 60 ± 5 minutes to maintain uniform conditioning.
(c) Remove the mixture and pan from the oven after 2 hours ± 5 minutes. The mixture is now conditioned for further testing.

The procedure for short-term conditioning for mixture mechanical property testing is similar to that for volumetric mix design, but the conditioning time is 4 hours ± 5 minutes, and the oven temperature, 275°F (135°C) ± 5.4°F (3°C).

4.3

Laboratory compaction

As part of all detailed mix design procedures, the asphalt mixture is compacted by some method into specimens for further testing. The laboratory compaction effort is intended to replicate the ultimate or final compacted condition of the pavement after being exposed to several years of traffic loading. Experience has shown that pavements that maintain an air void level of around 4 percent provide the best long-term performance in the field. As a result, choosing the appropriate level of compaction in the laboratory is crucial in designing a well-performing mixture.

Several different compaction methods, specimen shapes and sizes have been utilized over the last several decades. The designer should understand that different compaction methods affect the aggregate orientation and density profiles within the specimen. Therefore, different types (not meaning different models within the same type) of compaction methods should not be used for mix comparison. The goal of laboratory compaction is to both simulate the compaction expected in the field and ensure sufficient repeatability for acceptable consistency between different labs and technicians.

The steps preliminary to specimen preparation are:

- ensure all materials used in the design process have been sampled according to an accepted procedure and are representative of the materials to be used on the project;
- ensure all materials proposed for use meet the physical requirements of the project specifications;
- determine the appropriate mixing and compaction temperatures, either through the use of a viscosity versus temperature chart or by local agency specification; and
- determine the bulk specific gravity of all aggregates used in the blend and the specific gravity of the asphalt binder for performing density and voids analyses.

These requirements are matters of routine testing, specifications and laboratory technique that must be considered for any mix design method. Refer to chapter 3 for the preparation and analysis of aggregates. The following sections outline the three most common compaction methods for volumetric mix design and quality control testing:

- Gyratory—Superpave;
- Impact—Marshall; and
- Kneading—Hveem.

Detailed discussion of these compactors and their use can be found in their respective chapters (chapters 6, 7, 8) for each mix design method. In addition, vibratory and rolling wheel compactors that are typically used for mixture performance testing are covered in chapter 10.

4.3.1 Gyratory—Superpave

While gyratory compactors have been in use in the construction of laboratory asphalt specimens since the 1930s, the Superpave gyratory compactor (SGC) was developed under the SHRP program in the late 1980s and early 1990s as part of the Superpave mix design system. Predecessors to the SGC included the Texas Gyratory Testing Machine, the French Laboratoire Central des Ponts et Chaussées (LCPC) Gyratory Compactor and the Gyratory Testing Machine as developed by the Corp of Engineers.

The SGC is a computer-controlled device, offering direct feedback to a designer as it produces its specimens via compaction curves. Engineers and designers can utilize this information to design better asphalt mixtures that reduce the potential for problematic mixtures in the field, such as tender mixtures. The SGC was designed with 150-mm (6 inch) molds to be capable of accommodating large aggregates. Extensive research during the SHRP studies determined that the SGC was more representative of the compaction that occurs on asphalt pavements, in the field, during construction and eventual traffic loading.

Use of the Superpave system, and thus the SGC, has proven so successful that it is now used in all 50 states and elsewhere worldwide. Superpave compaction test methods are designated in ASTM and AASHTO as:

- **ASTM D6925** Standard Test Method for Preparation and Determination of the Relative Density of HMA Specimens by means of the Superpave Gyratory Compactor; and
- **AASHTO T 312** Preparing and Determining the Density of HMA Specimens by means of the Superpave Gyratory Compactor.

Further information and details on the SGC can be found in chapter 6 of this manual.

4.3.2 Impact hammers—Marshall

Marshall impact hammers compact asphalt mix specimens by the impact generated from dropping a 10-pound weight 18 inches along a shaft to an attached circular foot, which in turn imparts the load to asphalt mix contained in a 4-inch diameter mold assembly. The compaction equipment is portable, simple to use and has been used with reasonable success for many years, although many designers do not believe this method accurately represents field compaction.

The concepts of the Marshall method for designing paving mixtures were formulated by Bruce Marshall, a former Bituminous Engineer with the Mississippi State Highway Department. The U.S. Army Corps of Engineers, through extensive research and correlation studies, improved and added certain features to Marshall's test procedure and ultimately developed mix design criteria. Marshall test procedures were standardized by ASTM D1559, "Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus," but that procedure was withdrawn in 1998. The current ASTM Standards governing Marshall testing are:

- **D6926** Standard Practice for Preparation of Bituminous Specimens Using Marshall Apparatus;
- **D6927** Standard Test Method for Marshall Stability and Flow of Bituminous Mixtures; and
- **D5581** Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (6 inch-Diameter Specimen).

AASHTO still maintains Standard Method of Test **AASHTO T 245**, "Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus," which deals with 4-inch (10.16 cm) molds only.

4.3.3 Kneading compactors—Hveem

The California Kneading Compactor, often referred to as the Hveem compactor, employs a kneading action which allows densification by particle reorientation, which many designers believe effectively simulates the field compaction of rollers.

The concepts of the Hveem method of designing paving mixtures have been developed and advanced under the direction of Francis N. Hveem, a former Materials and Research Engineer for the California Department of Transportation. Over the years, certain features have been improved and others added. The test procedures and their application have been developed through extensive research and correlation studies on asphalt highway pavements. They are applicable to dense-graded paving mixtures using either asphalt cement or cutback asphalt and containing aggregates up to 25 mm (1 inch) maximum size.

Hveem method test procedures have been designated in two ASTM Standards and two AASHTO Standards:

- **ASTM D1560** Standard Test Methods for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus;
- **ASTM D1561** Standard Practice for Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor;
- **AASHTO T 246** Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus; and
- **AASHTO T 247** Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor.

4.4

Determining bulk specific gravity, G_{mb}

Determining the bulk specific gravity (G_{mb}) of lab-molded specimens and roadway cores is a fundamental component of asphalt mix design and testing. All specific gravity computations involve a mass divided by a volume multiplied by the unit mass of water. In this case, the mass includes both the mass of the aggregate and the mass of the binder. The volume includes the effective volume of the aggregate, the volume of the binder and the volume of the air voids within a compacted specimen.

There are three main standardized methods for determining the bulk specific gravity in the laboratory. The most common is intended for relatively nonabsorbent (impermeable), dense-graded specimens and uses the saturated surface-dry (SSD) mass in the calculation. This method is outlined in the following standards:

- **ASTM D2726** Bulk Specific Gravity and Density of Non-Absorptive Compacted Bituminous Mixtures; and
- **AASHTO T 166** Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens.

There are two methods intended for relatively absorbent (permeable) specimens. The first is the CoreLok® method, which is outlined in the following procedures:

- **ASTM D6752** Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method; and
- **AASHTO T 331** Bulk Specific Gravity and Density of Compacted Hot Mix Asphalt (HMA) Using Automatic Vacuum Sealing Method.

Like the CoreLok method, the paraffin-coated method is intended to be used for compacted mixture specimens with water absorption (infiltration) greater than 2.0 percent by volume. It seals the asphalt surface in a similar manner as the vacuum-sealed bags in the previous method. The test procedure is standardized in the following:

- **ASTM D1188** Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Coated Samples; and
- **AASHTO T 275** Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Using Paraffin-Coated Specimens.

4.4.1 Determining G_{mb} using the SSD method

The SSD method is intended to be used for compacted mixture specimens with water absorption less than or equal to 2.0 percent of the specimen volume as designated in AASHTO T 166 or ASTM D2726.

The water absorption can be determined as follows:

$$\text{\% water absorbed by volume} = 100 \times \frac{(B - A)}{(B - C)} \qquad \text{(Eq. 4.1)}$$

where:
A = dry mass of the specimen in air
B = saturated surface-dry mass of the specimen in air
C = mass of the specimen in water

This calculation must be done on samples suspected of excessive absorption in order to determine which method to use for calculation of G_{mb}. Consideration should be given to preparing an extra specimen for absorption determination. If the specimen is absorptive, and the internal voids become wetted, the sample will not be usable in CoreLok testing.

After mixing, aging and compacting the mixture, the mass of the sample is determined in air (dry), while submerged in water, and then in air again after drying the surface (saturated surface dry). The mass of the oven-dry specimen is being determined in **Figure 4.7**. The next step is to place the specimen

Figure 4.7 ***Determination of dry mass of specimen***

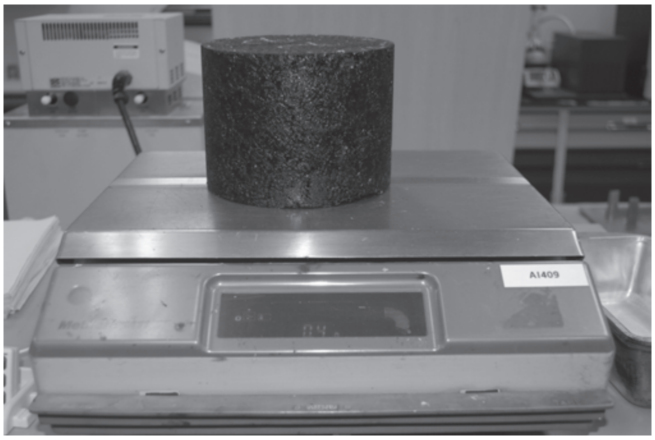


Figure 4.8 ***Determination of SSD mass of specimen***

in the water bath directly below the scale (not shown) and determine its mass under water. The last step is to determine the mass of the saturated surface dry specimen in air. The saturated surface-dry (SSD) mass is obtained by quickly blotting the sample so that the surface is not shiny (see **Figure 4.8**). The bulk specific gravity is the mass of the sample divided by the mass (volume) of water it displaces.

$$G_{mb} = \frac{A}{(B - C)} \qquad \text{(Eq. 4.2)}$$

where:

A = dry mass of the specimen in air
B = saturated surface-dry (SSD) mass of the specimen in air
C = mass of the specimen in water at 77°F (25°C)

The "mass" part of the standard specific gravity formula is the dry mass of the specimen in air. The "volume" part of the formula is determined in the denominator of the above formula, "(B – C)." The surface of the specimen has thousands of small irregularities, so the volume cannot be accurately computed by the standard formula for the volume of a cylinder. Archimedes' Principle is used in this method to determine the volume. It says that the buoyant force on an object is equal to the mass of the water it displaces. The buoyant force plus the immersed mass equals the SSD mass in air. When tested at 77°F, the mass of displaced water in grams is equal to the volume of water in cubic centimeters, therefore the formula "(B–C)" accurately represents the volume of the specimen.

4.4.2 Determining G_{mb} using the automatic vacuum sealing method (CoreLok©)

The CoreLok method is intended to be used for specimens with water absorption more than 2.0 percent by volume as designated in AASHTO T 331 or ASTM D6752.

The CoreLok test is conducted by the water-displacement method using compacted specimens vacuum-sealed in special puncture-resistant bags.

The term "saturated" in SSD means that the volume of water-permeable internal voids is included in the total specimen volume. However, a problem may occur with coarse dense-graded and gap-graded mixtures. The problem is greatly exacerbated in open-graded mixtures. With most conventional mixes, the surface tension of the water keeps it from flowing out of the internal voids and the voids are therefore included in the volume measurement. With larger void spaces or internal void spaces interconnected with the surface of the specimen, the mass of the water overcomes the surface tension and flows out of the specimen while drying the surface, resulting in a non-saturated sample which causes errors with the water-displacement method.

To overcome this problem, the CoreLok vacuum-sealing device (see **Figure 4.9**) is often used. The CoreLok device uses an automatic vacuum chamber with a puncture-resistant bag (see **Figure 4.10**), which tightly conforms to the sides of the sample and preserves the internal voids, resulting in a more accurate G_{mb} determination.

Again, the bulk specific gravity is the mass of the sample divided by the volume of water it displaces. With the CoreLok, the extra factor of the bag must be accounted for.

$$G_{mb} = \frac{A}{(B - C) - \frac{(B - E)}{F}} \qquad \text{(Eq. 4.3)}$$

where:

A = dry mass of the specimen in air
B = sealed specimen mass in air
C = mass of the sealed specimen in water
E = initial mass of the specimen in air
F = specific gravity of the bag

4.4.3 Determining G_{mb} using paraffin coating

AASHTO procedures specify that paraffin coating can be accomplished either through dipping a

Figure 4.9 ***CoreLok© Device***

compacted asphalt specimen into hot paraffin or by pressing and sealing pre-manufactured paraffin sheets the specimen with, called Parafilm. The Asphalt Institute recommends that samples be dipped in hot paraffin.

The calculations are very similar to the vacuum-sealed method:

$$G_{mb} = \frac{A}{(D - E) - \frac{(D - A)}{F}} \quad \text{(Eq. 4.4)}$$

where:

A = dry mass of the specimen in air
D = mass of the dry specimen plus paraffin coating in air
E = mass of the dry specimen plus paraffin coating in water
F = specific gravity of the paraffin at 25°C (77°F)

4.4.4 Determining theoretical maximum specific gravity, G_{mm}

Determining the theoretical maximum specific gravity (G_{mm}) of loose asphalt mixtures is another

Figure 4.10 ***Asphalt Mix Sample in Sealed Bag***

fundamental component of asphalt mix design and testing that involves a mass divided by a volume multiplied by the unit mass of water. In this case, the mass includes both the mass of the aggregate and the mass of the binder. The volume includes only the effective volume of the aggregate and the volume of the binder. If G_{mb} and G_{mm} samples had the same dry weight in air, the numerators of the specific gravity equation would be the same for G_{mb} and G_{mm}, but the denominator of the G_{mm} calculation is smaller because it does not include the volume of air. Therefore, G_{mm} must always be a larger number than G_{mb}. Theoretically, if a G_{mb} sample could be compacted until 0 percent air voids remain, the G_{mb} and G_{mm} would be equal.

The most commonly used practice for determining the theoretical maximum specific gravity is standardized in the following ASTM and AASHTO test methods:

- **ASTM D2041** Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures; and
- **AASHTO T 209** Theoretical Maximum Specific Gravity and Density of Hot Mix Asphalt (HMA).

There are three basic steps in determining the theoretical maximum specific gravity (see **Figure 4.11**). The loose mix is warmed and separated into loose, individually coated aggregates. A minimum mass, specified in T 209, of the dry loose mix is split out and placed in a metal bowl or calibrated pycnometer and covered with water. A vacuum lid is fitted and secured to the bowl or

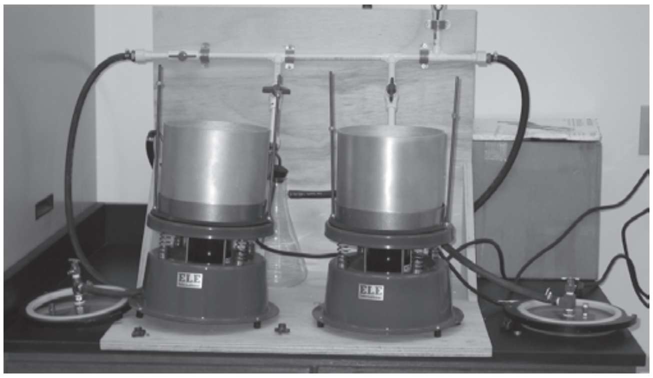


Figure 4.11 ***Equipment for maximum specific gravity***

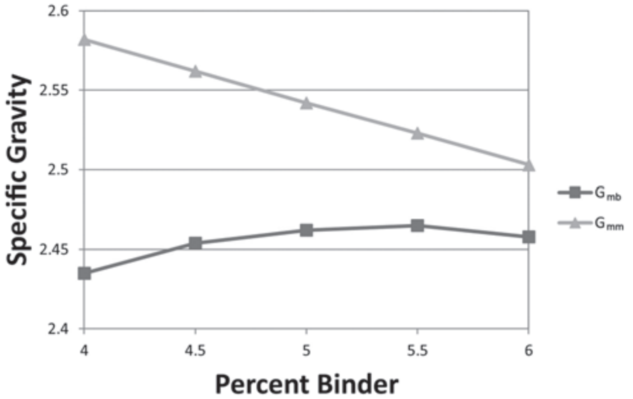


Figure 4.12 ***G_{mm} and G_{mb} relationship to binder content***

pycnometer and placed on a vibratory shaker table. A vacuum pump is started and the manometer or absolute pressure gauge reading is used to determine the proper vacuum adjustment. Once the proper (almost absolute, 27.5 mm Hg) vacuum is obtained, the shaker table is started. This provides gentle agitation to help in the removal of any air between particles. The agitation ensures that the air in the mixture is as close as possible to zero. The theoretical maximum specific gravity is calculated using the equation for the specific procedure utilized. G_{mm} is the mass of the coated aggregate divided by the volume of coated aggregate. Air voids are calculated from the bulk and maximum specific gravities (G_{mb} and G_{mm}). The ratio of these two specific gravities is actually the percent by volume of solids (in decimal form).

A common source of error with this test is that technicians do not calibrate (verify) the mass of the vacuum container filled with water often enough. This is not usually a problem in labs where only distilled water is used for the test, but field labs often have water tanks that serve the entire lab and are refilled periodically, sometimes from different sources. Because it only takes a few minutes to calibrate, more consistent results will be generated if the vacuum containers are calibrated daily or even before each test.

4.5

Effect of binder content on G_{mb} and G_{mm}

The effects of asphalt binder content (P_b) on G_{mb} and G_{mm} are illustrated in **Figure 4.12**. It is important to remember that G_{mb} is measured on a compacted mixture sample. As P_b increases, more lubricity is added to the mixture which allows the specimen to compact and slightly reduce the volume, while at the same time the mass of the specimen is also increasing as the binder fills the voids within the compacted aggregate structure. The slight reduction in volume in combination with the increasing mass causes the specific gravity (density) of the compacted sample to increase. As the voids become filled with binder, the volume of the sample begins to increase. This increasing volume is due entirely to the additional binder being added which begins to reduce the overall specific gravity of the compacted specimen.

The effects of increasing P_b on G_{mm} are quite different. As the P_b increases the percent stone (P_s) decreases. Since there is no compaction or air voids involved with the measurement of G_{mm}, the volume of a G_{mm} sample always increases because the volume of binder being added is roughly 2.5 times the volume of stone that is being removed from the mixture. This makes the G_{mm} property very sensitive to binder content. This also shows the importance of obtaining representative samples of mix when conducting G_{mm} testing. If a sample is segregated and is too coarse, the P_b will be artificially low, resulting in a G_{mm} value that is too high. If the segregated sample is too fine compared to the mixture being produced, the binder content of the material tested will be high and the resulting G_{mm} test result too low.

CHAPTER 5

Volumetric Properties of Compacted Paving Mixtures

5.1

General

Since the fundamental performance properties are not directly measured in a normal mix design, asphalt content is selected on the basis of a measured parameter that best controls the pavement performance. Considerable research has determined that air void content is this parameter. An acceptable air voids range of 3 to 5 percent is most often used. Within this range, 4 percent air voids is typically considered the best initial estimate for a design that balances the desired performance properties. Slight refinements are then considered in the analysis of the mix test results.

The volumetric properties of a compacted paving mixture are important criteria by which the quality of an asphalt mixture is evaluated. The volumetric properties are determined using the mass and/or volume measurements of a mixture and its constituent components (binder, aggregate, air). Volumetrics have historically provided a good indication of the mixture's probable performance during its service life.

The term "volumetric," as applied in the asphalt industry, actually uses measurements of an asphalt mixture by both mass (M) and volume (V) to determine various percentages (P). Volumetric properties are often specified design elements of the total mix, the aggregate only or the binder only. The relationship between mass and volume is determined by the material's specific gravity (G). Specific gravity is a dimensionless number defined as the ratio of the density of a material to the density of water (assumed to be 1.000 g/cm^3 at temperatures used in asphalt testing).

$$G = \frac{\frac{m}{v}}{\rho} \; or \; \frac{m}{v \times \rho} \qquad \text{(Eq. 5.1)}$$

where:
G = specific gravity
m = mass of the material
v = volume of the material
ρ = density of water

Since ρ is assumed to be a constant 1 g/cm^3, any specific gravity can be calculated from the appropriate mass and volume. If any two of the three are quantities, G, m, or v, are known through testing, the third can be easily calculated. The principal of using specific gravity is essential in that actual laboratory measurements are by mass, yet mixture criteria are a function of volume. Usage of specific gravity principles enables us to readily calculate mass or volumetric data based on the available data.

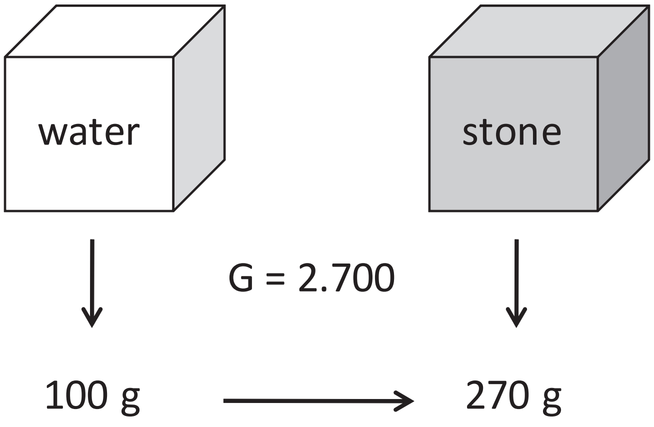
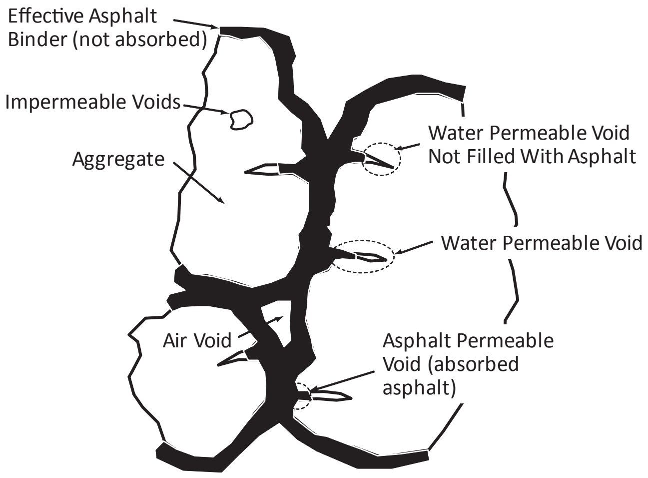


FIGURE 5.1 ***Specific Gravity Example***

As shown in **Figure 5.1**, a specific gravity of 2.700 means that the stone weighs 2.7 times the same volume of water.

Several different specific gravity types are determined in the volumetric analysis of an asphalt mixture. Each specific gravity uses a particular mass (binder and/or aggregate) and a particular volume (air, binder or aggregate or some combination). Some specific gravity values use an aggregate volume that also includes the water-permeable voids, while others include only the portion of the water-permeable voids not filled with absorbed asphalt. (Because liquid water is always at a lower viscosity than asphalt binder, the asphalt can never penetrate the aggregate voids as much as water.) **Figure 5.2** represents a microscopic view of the aggregate, asphalt and air mixture.

FIGURE 5.2 ***Representation of Microscopic View of Aggregate, Asphalt, and Air Mixture***



It can be helpful to represent a compacted mixture in terms of the different masses and volumes used in volumetric calculations. This representation, shown in **Figure 5.3**, is called a phase diagram. It breaks down the components into air, effective asphalt (nonabsorbed), absorbed asphalt and aggregate.

The various specific gravity types discussed in sections 5.2.1 through 5.2.3 can be either directly determined or calculated from laboratory tests on the binder, aggregates or asphalt mixture. Several equations have been derived to allow calculation of each volumetric property without the use of a phase diagram. The equations shown in this chapter are simplified, as the value of ρ is assumed to be 1.000.

5.2

Nomenclature and definitions

During the design process, various laboratory tests are used to determine the specific gravity of the asphalt mixture and its components. The standard nomenclature for most volumetric properties uses:

- a beginning capital letter to identify the property type;
- followed by a subscripted lowercase letter identifying the material; and
- sometimes followed by a second subscripted lowercase letter giving more detail about the nature of the property.

Beginning capital letter:
G - specific gravity
M - mass
V - volume
P - percent

First lowercase subscripts:
a - air
b - binder
s - stone (aggregate)
m - mix

Second lowercase subscripts:
a - absorbed (binder only)
a - apparent (aggregate only)
b - bulk
e - effective
m - maximum

The following definitions are the most commonly used in volumetric analysis:

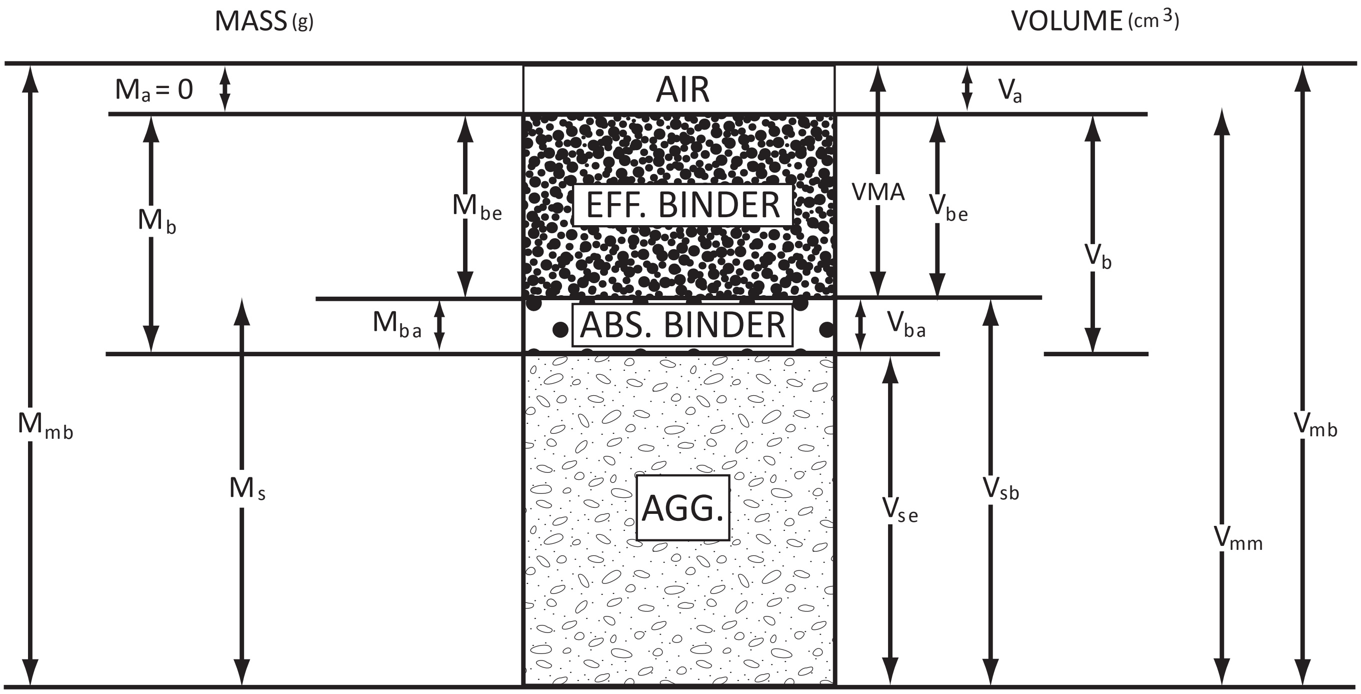


FIGURE 5.3 ***Phase Diagram***

5.2.1 Binder specific gravity

Binder Specific Gravity G_b As determined for asphalt binder by ASTM D70 or AASHTO T 228, the ratio of the mass of a unit volume of binder to the mass of the same volume of water. Binder specific gravity typically ranges from 1.00 to 1.05.

5.2.2 Aggregate specific gravities

Bulk (dry) Specific Gravity G_{sb} As determined for aggregate by ASTM C127 and C128 or AASHTO T 84 and T 85, the ratio of the oven-dry mass of a unit volume of aggregate (including both the impermeable and water-permeable void volumes) to the mass of the same volume of water.

Apparent Specific Gravity G_{sa} As determined for aggregate by ASTM C127 and C128 or AASHTO T 84 and T 85, the ratio of the oven-dry mass of a unit volume of aggregate (including only the impermeable void volumes) to the mass of the same volume of water.

Effective Specific Gravity G_{se} As calculated for aggregate from the results of ASTM D2041 or AASHTO T 209, the ratio of the oven-dry mass of a unit volume of aggregate (including both the impermeable void volumes and the water-permeable voids not filled with absorbed asphalt) to the mass of the same volume of water.

NOTE: *G_{sb}, G_{sa} and G_{se} each use the same mass (oven-dry aggregate), but they use different volumes. Because volume is in the denominator of the specific gravity equation, the smallest volume necessarily results in the largest specific gravity. Since the volumes can only be the same if there is zero aggregate absorption, the following inequality always exists:*

$$G_{sa} \geq G_{se} \geq G_{sb}$$

5.2.3 Mixture specific gravities

Theoretical Maximum Specific Gravity G_{mm} As determined for loose asphalt mixtures by ASTM D2041 or AASHTO T 209, the ratio of the oven-dry mass of a unit volume of asphalt mixture (including the volumes of the aggregate and binder only) to the mass of the same volume of water.

Bulk Specific Gravity G_{mb} As determined for compacted asphalt mixtures by ASTM D2726 or AASHTO T 166, the ratio of the oven-dry mass of a unit volume of asphalt mixture (including the volumes of aggregate, binder and air) to the mass of the same volume of water. G_{mb} is applicable to any laboratory- or field-compacted specimen including cores, beams, slabs, etc.

5.2.4 Mixture volumetric parameters

Percent Air Voids P_a The volume of air voids in a compacted mixture, expressed as a percentage of the total mix volume. Many agencies refer to this percentage by the term V_a because it is a percentage by volume instead of a percentage by mass. However, the identical term V_a is also used to represent the volume of air voids in an asphalt mixture, expressed in cubic centimeters. Other agencies use the term VTM (Voids in Total Mix) to avoid the conflict.

Voids in the Mineral Aggregate (VMA) The voids created by the aggregate structure of a compacted asphalt mixture, expressed as a percentage of the total mix volume. VMA represents the volume of air voids and effective (nonabsorbed) asphalt binder.

Voids Filled with Asphalt (VFA) The percentage of the VMA filled with effective (nonabsorbed) asphalt binder.

Percent Aggregate P_s The total percentage of aggregate in the asphalt mixture, expressed as a percentage of the total mix mass.

Percent Binder P_b The total percentage of asphalt binder in the asphalt mixture, expressed as a percentage of the total mix mass. Note that $P_s + P_b = 100\%$.

Percent Binder Effective P_{be} The functional portion of the asphalt binder that coats the aggregate in the asphalt mixture but is not absorbed into the aggregate, expressed as a percentage of the total mix mass.

Percent Binder Absorbed P_{ba} The portion of the asphalt binder that is absorbed into the aggregate, expressed as a percentage of the total aggregate mass.

5.3

Bulk (dry) specific gravity of aggregate

It is recommended that the bulk (dry) specific gravity (G_{sb}) of each aggregate be determined on samples submitted for mix design. Some stockpiles will be essentially coarse (retained on the No. 4 [4.75 mm] sieve), some will be fine (passing the No. 4 [4.75 mm] sieve) and some will have both coarse and fine portions.

5.3.1 Determining coarse aggregate G_{sb}

The coarse G_{sb} is determined using AASHTO T 85 or ASTM C127. The size of the test sample is specified and determined by the nominal maximum aggregate size. This procedure requires that the dry aggregate be saturated to determine the volume of the aggregate plus the water-permeable voids.

$$G_{sb} = \frac{m}{v\rho} = \frac{\text{mass of oven-dry aggregate}}{(\text{volume of aggregate} + \text{water-permeable voids}) \times \rho} \qquad \text{(Eq. 5.2)}$$

Notice that this equation mirrors the equation in the test procedure:

$$G_{sb} = \frac{A}{B - C} \qquad \text{(Eq. 5.3)}$$

where:

G_{sb} = bulk (dry) specific gravity of the aggregate

A = mass of the oven-dry test sample

B = mass of the saturated surface-dry (SSD) test sample in air

C = mass of the saturated sample in water

(ρ is not shown because its numerical value is 1)

Therefore, B–C = volume of the aggregate plus the water-permeable voids.

5.3.2 Determining fine aggregate G_{sb}

The fine G_{sb} is determined using AASHTO T 84 or ASTM C128. The dry aggregate is again saturated to account for the volume of the aggregate plus the water-permeable voids. Note that the procedure allows saturation by the addition of 6 percent moisture as an alternative to total submersion. This option allows the aggregate to be dried to an SSD condition much quicker than using the submerged option. If the designer is using aggregates with a high water absorption (3-4 percent), the Asphalt Institute recommends total submersion.

After the fine aggregate has been dried to a saturated surface-dry (SSD) condition (as specified in AASHTO T 84), the volume of the SSD fine aggregate is determined by submerging the sample in a volumetric flask (pycnometer) for de-airing. It is suggested in AASHTO T 84 to remove the fine

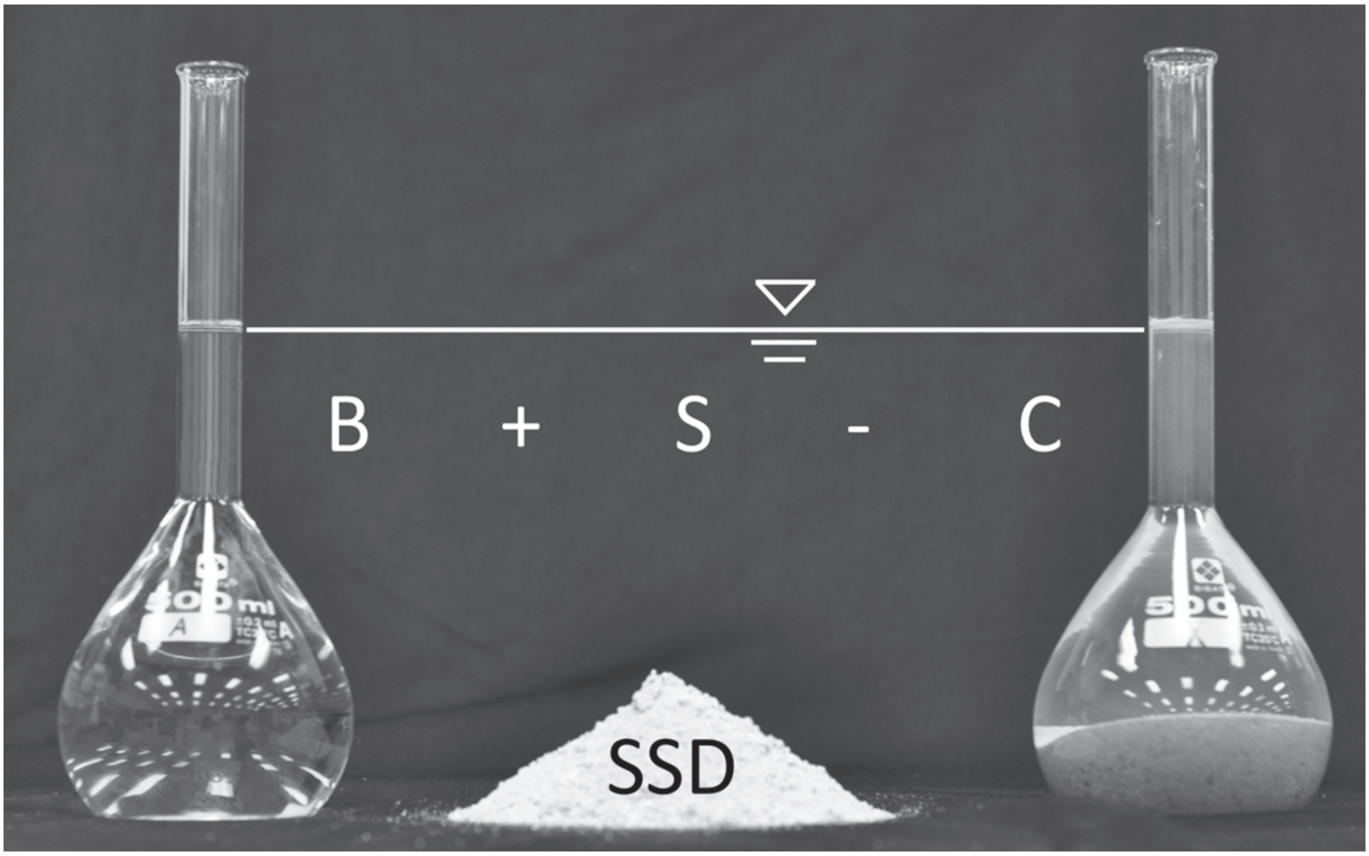


FIGURE 5.4 ***Illustration of Displaced Water Volume***

aggregate and water from the pycnometer and dry to a constant mass. The determination of the dry mass of the aggregate in this manner can be messy, has the potential for loss of material and results in a sample that is covered with water and will take a long time to dry. The procedure allows a sample of the same mass (± 0.2 grams) to be obtained at the time the SSD material is placed in the pycnometer. This second sample can then be used to determine the oven-dry mass quicker and more easily.

$$G_{sb} = \frac{m}{v\rho} = \frac{\text{mass of oven-dry aggregate}}{(\text{volume of aggregate + water-permeable voids}) \times \rho} = \frac{A}{B+S-C} \qquad \text{(Eq. 5.4)}$$

where:

G_{sb} = bulk (dry) specific gravity of the aggregate
A = mass of the oven-dry test sample
B = mass of the pycnometer filled with water
S = mass of the saturated surface-dry (SSD) specimen
C = mass of pycnometer with specimen and water to calibration mark

This time, B + S – C = volume of the aggregate plus the water-permeable voids as shown in **Figure 5.4**.

5.3.2.1 Determining mineral filler G_{sb}

The bulk specific gravity of mineral filler is difficult to determine accurately. However, the apparent specific gravity (G_{sa}) of mineral filler is more easily determined. This can be done for filler only, as the amount of mineral filler added is typically small and the difference between G_{sb} and G_{sa} is relatively small. Agency approval would be necessary for this substitution.

Mixture Components					
Material	Specific Gravity	Test Methods		Mix Composition	
	Bulk	AASHTO	ASTM	Percent by Mass of Total Mix	Percent by Mass of Total Aggr.
Asphalt Binder	1.030 (G_b)	T 228	D70	5.3 (P_b)	5.6 (P_b)
Coarse Aggregate	2.716 (G_1)	T 85	C127	37.9 (P_1)	40.0 (P_1)
Fine Aggregate	2.689 (G_2)	T 84	C128	56.8 (P_2)	60.0 (P_2)
Mineral Filler	--	T 100	D854	–	–
Paving Mixture Bulk specific gravity of compacted paving mixture specimen, G_{mb} = 2.442 Maximum specific gravity of paving mixture, G_{mm} = 2.535					

TABLE 5.1 ***Design Data for the Following Example Calculations***

5.3.3 Determining the composite G_{sb} for one stockpile

For stockpiles that include both a coarse and fine fraction, one value must be determined for the stockpile. The average G_{sb} can be calculated as follows:

$$G_{sb} = \frac{P_{coarse} + P_{fine}}{\frac{P_{coarse}}{G_{coarse}} + \frac{P_{fine}}{G_{fine}}} \qquad \text{(Eq. 5.5)}$$

where:

G_{sb} = bulk (dry) specific gravity of the aggregate

P_{coarse} = percentage by weight retained on the No. 4 (4.75 mm) sieve

P_{fine} = percentage by weight passing the No. 4 (4.75 mm) sieve

G_{coarse} = bulk (dry) specific gravity of the coarse fraction

G_{fine} = bulk (dry) specific gravity of the fine fraction

5.3.4 Calculate the G_{sb} for the aggregate blend

Once the bulk (dry) specific gravity for each stockpile has been determined, the combined bulk (dry) specific gravity for the total aggregate blend is calculated as follows:

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{Gn}} \qquad \text{(Eq. 5.6)}$$

where:

G_{sb} = bulk (dry) specific gravity of the aggregate

P_1, P_2, P_n = percentages by weight of aggregates 1, 2, through n

G_1, G_2, G_n = bulk (dry) specific gravity of aggregates 1, 2, through n

This equation is useful for estimating G_{sb} during trials in the design process. The calculated coarse and fine G_{sb} can be verified by batching the combined aggregates, splitting them on the 4.75-mm sieve and determining the coarse and fine G_{sb} for the design. This process of splitting the aggregate blend on the 4.75-mm sieve and only running G_{sb} values on the coarse and fine fractions of the blend is often utilized for mix design verification and quality control testing on plant-produced mix in the field.

Using the data for two stockpiles in **Table 5.1**:

$$G_{sb} = \frac{40 + 60}{\frac{40}{2.716} + \frac{60}{2.689}} = \frac{100}{14.73 + 22.31} = 2.700 \qquad \text{(Eq. 5.7)}$$

The equation format for calculating the combined bulk (dry) specific gravity uses the weighted harmonic mean. This method is necessary because the criteria being averaged involve a ratio. In this case, the percentages are all by weight, but the specific gravity is a ratio of the density of the material to the density of water.

The equation for calculating the combined average absorption uses the weighted arithmetic mean, because each absorption is a percentage by weight, with no supplemental ratio involved.

Assuming a blend of three very dissimilar materials, **Figure 5.5** demonstrates the need to use the weighted harmonic mean:

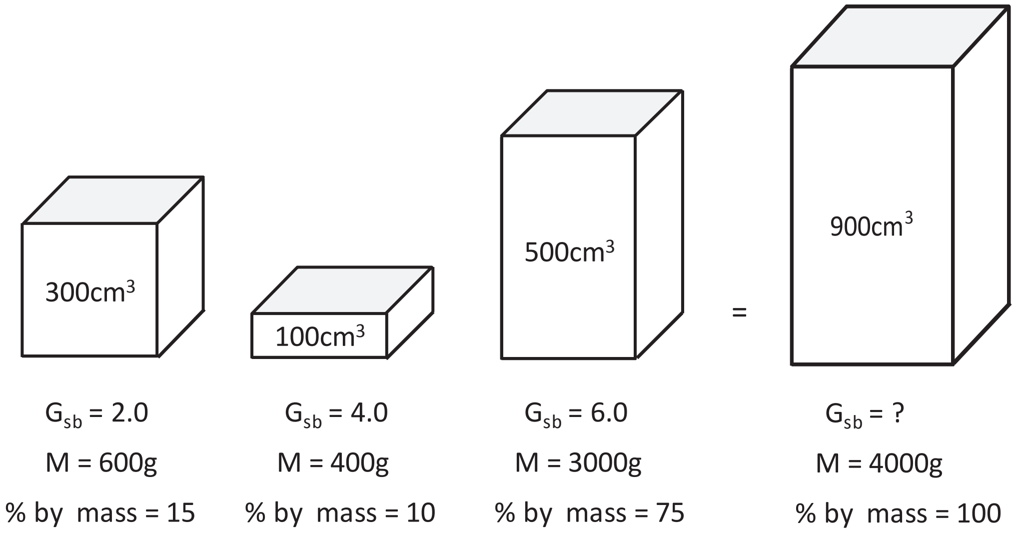
5.3.5 Calculate the G_{sa} and absorption for the aggregate blend

Laboratory testing to determine the bulk specific gravity (G_{sb}) also provides data to easily determine two additional aggregate properties, the apparent specific gravity (G_{sa}) and the water absorption of the aggregate. These calculations are not required to determine mixture volumetric properties; however, they are valuable tools for the mix designer to monitor. The absorption of the aggregate indicates several characteristics of the final mixture. Highly absorptive aggregates will require additional binder to fill the permeable voids in the aggregate, which increases cost. It is not uncommon for aggregates to absorb a binder amount equal to 40–80 percent of the water-permeable voids.

In order to determine these values for the total blend of aggregate, the methodology used will depend on the manner in which the G_{sb} was determined. If G_{sb} testing was conducted on individual stockpiles, then the G_{sa} and absorption will need to be determined for each stockpile and then combined to determine the final values for the blend. If individual G_{sb} samples were determined for the coarse and fine fractions of any individual stockpile, then the equation shown in section 5.3.3 can be used to determine the G_{sa} and absorption values for the stockpile.

Once G_{sa} values of individual stockpiles are known, they will need to be combined into one value for the blend. This can be accomplished using

FIGURE 5.5 ***Weighted Arithmetic Mean vs. Weighted Harmonic Mean***



$$\text{Average } G_{sb} = \frac{\text{Total mass}}{\text{Total volume} * \rho} = \frac{4000\text{g}}{900\text{cm}^3 * 1\text{ g/cm}^3} = 4.44$$

$$\text{Weighted Arithmetic Mean} = \frac{(15)(2.0)+(10)(4.0)+(75)(6.0)}{100} = 5.20$$

$$\text{Weighted Harmonic Mean} = \frac{100}{\frac{15}{2.0} + \frac{10}{4.0} + \frac{75}{6.0}} = 4.44$$

the same equation shown in section 5.3.4, but for G_{sa} instead of G_{sb}. The equation for calculating the absorption of the blend is shown in section 5.3.5.2.

If G_{sb} data are determined directly from the blend (during a mixture verification or a field sample obtained from the belt carrying aggregate into the plant), the designer can simply use the equation from section 5.3.3 to directly determine the G_{sb}, G_{sa} and absorption data for the blend.

5.3.5.1 Apparent specific gravity (G_{sa})

G_{sa} is the ratio of the mass of the oven-dry aggregate to the volume of the aggregate excluding the volume of the voids occupied by absorbed water. The G_{sa} volume is less than the volume used to calculate the G_{sb}; therefore, the G_{sa} value will always be larger than the G_{sb} value.

$$G_{sa} = \frac{m}{v \times \rho} = \frac{\text{mass of oven-dry aggregate}}{(\text{bulk volume of aggregate} - \text{volume of water-permeable voids}) \times \rho} = 2.700 \qquad \text{(Eq. 5.8)}$$

Notice that this equation mirrors the equation in the test procedure:

$$G_{sa} = \frac{A}{A - C}$$

where:

G_{sa} = apparent specific gravity of the aggregate
A = mass of the oven-dry test sample
C = mass of the saturated sample in water
(ρ is not shown because its numerical value is 1)

Therefore, A – C = apparent volume of the aggregate minus the water-permeable voids.

5.3.5.2 Water absorption (A)

The amount of water absorption is also easily determined from the Gsb test data. The absorptiveness of aggregate is of significant interest to the mixture designer and specifier. Absorption can be an indicator regarding aggregate quality along with increased binder demand. The binder absorption is typically 40–80 percent of the water absorption rate. The water absorption rate is calculated by the following equation as outlined in AASHTO T 85

$$\text{Absorption, \%} = \frac{(B - A)}{A} \times 100 \qquad \text{(Eq. 5.9)}$$

where:

B = mass of the saturated surface-dry sample
A = mass of the oven-dry test sample

The average water absorption for the total aggregate blend as shown in AASHTO T 85 is calculated as follows:

$$A = \frac{(P_1)(A_1) + (P_2)(A_2) + \ldots + (P_n)(A_n)}{100} \qquad \text{(Eq. 5.10)}$$

where:

P_1, P_2, P_n = percentages by weight of aggregates 1, 2, through n
A_1, A_2, A_n = absorption of aggregates 1, 2, through n

5.4

Calculating G_{mm} at trial binder contents

The theoretical maximum specific gravity of an asphalt mixture (G_{mm}), as described in chapter 4, is the specific gravity of the binder coated aggregate only, with no air voids.

In order to calculate the volumetric properties of a mixture, a G_{mb} and G_{mm} must be determined at each trial binder percentage utilized in the mix design. Compaction procedures provide the G_{mb} values for each sample, which are then averaged for each trial binder content. The appropriate G_{mm} value must also be determined at each trial binder percentage. Some designers elect to prepare samples and conduct G_{mm} testing at each trial binder percentage. As previously discussed, the G_{mm} directly accounts for the volume of asphalt binder absorbed by the aggregate. The Asphalt Institute considers the asphalt absorption constant and not dependent on the amount of binder added to the mix, as long as the binder content added to the mixture exceeds the absorption value of the aggregate. This position allows the designer to prepare and determine the G_{mm} at one laboratory trial binder content. The G_{mm} values for the remaining trial binder contents, or at any binder content, can then be calculated by computing an effective specific gravity of the aggregate. G_{se} is a constant that can be used to back-calculate G_{mm} at any asphalt binder content.

5.4.1 Determining the G_{se} — effective specific gravity of the aggregate

The effective specific gravity is the ratio of the oven-dry mass of a unit volume of aggregate (including

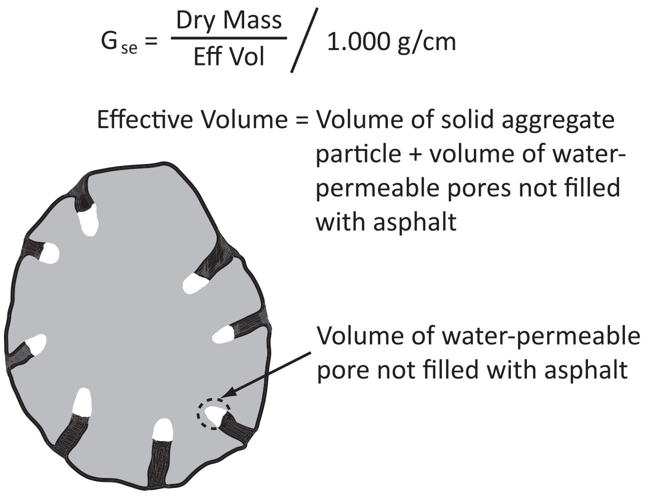


FIGURE 5.6 ***Illustration of Effective Aggregate Volume***

both the solid volume of the aggregate and the water-permeable voids not filled with absorbed asphalt as shown in **Figure 5.6**) to the mass of the same volume of water. When only one G_{mm} is conducted in the laboratory, the designer should select a binder content that is equal to or greater than the anticipated design binder content in order to assure thorough coating and minimize water intrusion into the aggregate during vacuum testing. After finding the average of two G_{mm} samples at a single binder content (or if desired, at every trial binder percentage), G_{se} can be calculated using the following equation:

$$G_{se} = \frac{P_s}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \qquad \text{(Eq. 5.11)}$$

where:

G_{se} = effective specific gravity of aggregate
P_s = percentage of aggregate by total mix weight
P_b = percentage of binder by total mix weight, at which the G_{mm} test was performed
G_{mm} = maximum specific gravity of paving mixture
G_b = specific gravity of binder

Using the data in **Table 5.1**:

$$G_{se} = \frac{94.7}{\frac{100}{2.535} - \frac{5.3}{1.030}} = 2.761 \qquad \text{(Eq. 5.12)}$$

5.4.2 Determining G_{mm} at other binder contents

The G_{se} is then used to calculate G_{mm} at each of the other binder contents. This step is not necessary if the designer has performed G_{mm} testing at each trial binder percentage. G_{mm} can be visually determined from the phase diagram in **Figure 5.3** and is defined by the following relationship:

$$G_{mm} = \frac{M_{mb}}{V_{mm}\rho} \qquad \text{(Eq. 5.13)}$$

The calculation of G_{mm} at other binder contents, utilizing $G_{se,}$ is accomplished with the following equation:

$$G_{mm} = \frac{100}{\frac{P_s}{G_{se}} + \frac{P_b}{G_b}} \qquad \text{(Eq. 5.14)}$$

where:

G_{mm} = maximum specific gravity of asphalt mixture
P_s = percentage of aggregate by total mix weight
P_b = percentage of binder by total mix weight
$P_s + P_b = 100$
G_{se} = effective specific gravity of aggregate
G_b = specific gravity of binder
M_{mb} = bulk mass of paving mixture (which would be the same as M_{mm}, since the air has no mass), typically in g
V_{mm} = volume of aggregate and binder, typically in cm^3
ρ = density of water, 1.000 g/cm^3

For example, the following illustrates how to calculate G_{mm} at a binder content of 5.3%, and then 0.5% higher, using the data in **Table 5.1** and the G_{se} calculated in section 5.4.1.

$$G_{mm} = \frac{100}{\frac{94.7}{2.761} + \frac{5.3}{1.030}} = 2.535 \qquad \text{(Eq. 5.15a)}$$

$$G_{mm} = \frac{100}{\frac{94.2}{2.761} + \frac{5.8}{1.030}} = 2.516 \qquad \text{(Eq. 5.15b)}$$

Note that as the binder content increases, G_{mm} *always* decreases. This is because the percentage of aggregate, which has a higher specific gravity, necessarily decreases for a unit volume with an increase in the percentage of binder, which has a lower specific gravity.

5.5

Percent air voids in compacted mixture

Keep in mind that this manual defines P_a as the percentage of air voids by volume and V_a as the measured volume of air voids. They consist of the small air spaces between coated aggregate particles. The property P_a can be visually determined from the phase diagram in **Figure 5.3** and is defined by the following relationship:

$$P_a = 100 \times \frac{V_a}{V_{mb}} \qquad \text{(Eq. 5.16)}$$

Although the P_a can be calculated several different ways, the following equation is most commonly used:

$$P_a = 100 - \frac{100 \times G_{mb}}{G_{mm}} \qquad \text{(Eq. 5.17)}$$

where:

P_a = air voids in compacted mixture, percentage of total volume
G_{mm} = maximum specific gravity of paving mixture
G_{mb} = bulk specific gravity of paving mixture

P_a in a laboratory-compacted mixture is an important part of selecting the proper binder content of the asphalt mixture. While AASHTO M 323, "Superpave Volumetric Mix Design," sets the design air voids at 4.0 percent, some agencies have lowered the target to 3.5 percent or even 3.0 percent in an effort to force more binder into the mix. A reasonable rule of thumb says that for each 1.0 percent decrease in the air void content for a given aggregate structure, the design binder content increases 0.3 to 0.4 percent.

Using the data in **Table 5.1**:

$$P_a = 100 - \frac{100 \times 2.442}{2.535} = 3.7 \qquad \text{(Eq. 5.18)}$$

5.6

Percent VMA in compacted mixture

The voids in the mineral aggregate, VMA, are defined as the intergranular void space between the aggregate particles in a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume. The VMA is calculated on the basis of the bulk specific gravity of the aggregate and is expressed as a percentage of the bulk volume of the compacted paving mixture. Therefore, the VMA can be calculated by subtracting the volume of the aggregate determined by its bulk specific gravity from the bulk volume of the compacted paving mixture.

VMA can be visually determined from the phase diagram in **Figure 5.3** and is defined by the following relationship

$$VMA = 100 \times \frac{V_a + V_{be}}{V_{mb}} \qquad \text{(Eq. 5.19)}$$

VMA is most readily calculated utilizing the following equation:

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}} \qquad \text{(Eq. 5.20)}$$

where:

VMA = voids in the mineral aggregate
G_{mb} = bulk specific gravity of paving mixture
P_s = percentage of aggregate by total mix weight
G_{sb} = bulk (dry) specific gravity of the aggregate
V_a = volume of voids in compacted mixture, typically in cm^3
V_{be} = volume of the effective (nonabsorbed) binder, typically in cm^3
V_{mb} = total volume of compacted mixture, typically in cm^3

Using the data in **Table 5.1** and the combined bulk (dry) aggregate specific gravity from section 5.3.4:

$$VMA = 100 - \frac{2.442 \times 94.7}{2.700} = 14.3\% \qquad \text{(Eq. 5.21)}$$

The equations shown above are for analyzing mixture compositions that are determined as percent by weight of the total mixture.

If the mixture composition is determined as percent by weight of aggregate, the following equation must be utilized to calculate VMA:

$$VMA = 100 - \frac{G_{mb}}{G_{sb}} \times \frac{100}{100 + P_b} \times 100 \qquad \text{(Eq. 5.22)}$$

Because the VMA does not include the water-permeable voids in the aggregate, the bulk dry G_{sb} must be utilized in calculating VMA. **Table 5.2** illustrates the effects of using other aggregate

Example Asphalt Mixture Data:

Bulk Specific Gravity of Compacted Mixture (G_{mb})	2.406
Max. Theoretical Specific Gravity of Compacted Mixture (G_{mm})	2.494
Asphalt Content, percentage by weight of total mix (P_b)	5.1
Specific Gravity of Asphalt Cement (G_b)	1.011

Aggregate Specific Gravity Test Parameters:
A = 3357.8 g (Mass of oven-dry aggregate in air)
B = 3439.8 g (Mass of saturated surface-dry aggregate in air)
C = 2173.1 g (Mass of saturated aggregate in water)

Aggregate Specific Gravity	Specific Gravity Calculation	Example Value	Resulting VMA (%)	Calculated VMA Filled With:	Correct? (Yes/No)
Bulk Dry - G_{sb} *(Determined from Aggregate Test)*	$\frac{A}{B-C}$	2.651	13.9	Air + Effective Binder	Yes
Bulk SSD - G_{sb}, SSD *(Determined from Aggregate Test)*	$\frac{B}{B-C}$	2.716	15.9	Not Applicable *(SSD aggregate weight in numerator results in inaccurate volume calculation)*	No
Effective - G_{se} *(Calculated from Gmm)*	$\frac{P_s}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}}$	2.708	15.7	Air + Effective Binder + Absorbed Binder	No
Apparent - G_{sa} *(Determined from Aggregate Test)*	$\frac{A}{A-C}$	2.834	19.4	Air + Effective Binder + Water-Permeable Voids in Aggregate	No

TABLE 5.2 ***Proper Aggregate Specific Gravity for Use in VMA Calculation***

specific gravities in calculating VMA. VMA criteria recommended in chapters 6 and 7 are based on bulk dry G_{sb}.

As the nominal maximum aggregate size of the mix decreases, the surface area of the total aggregate structure increases as shown in **Figure 5.7**. Therefore, the percentage of binder necessary to adequately coat the particles increases. Since the target air voids (Pa) typically remains the same, the VMA must increase to allow sufficient room for the additional asphalt binder.

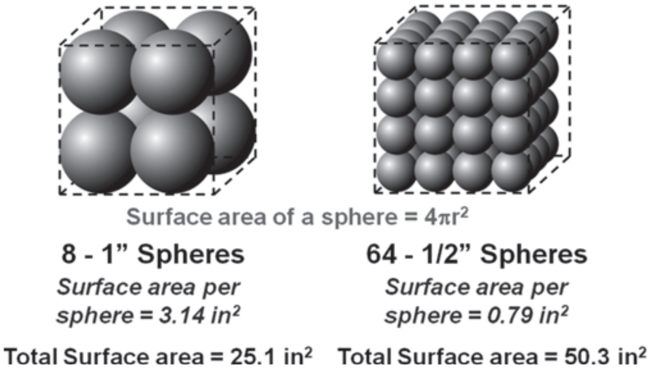
5.6.1 Factors affecting VMA

Many factors affect the VMA in a compacted paving mixture. In fact, anything that impacts the ability of the compactor to consolidate the mixture in the mold will affect the resulting VMA. Some of the more notable factors are discussed below.

Minor Factors

- Binder type—Stiffer binders, whether neat or modified, can increase the resistance to the compaction in the laboratory or in the field. The resistance to compaction can be minimal at temperatures greater than 300°F, but will increase as the binder temperature decreases due to the resulting increase in viscosity.
- Binder quantity—Asphalt binder will add lubrication to the mix and increase the ability of the aggregate structure to consolidate. Small changes in binder content, at or near the design binder content, typically will have minimal effect on the compacted VMA

FIGURE 5.7 ***NMAS affects binder content***



content. Large variations from the design binder content will have greater effects as discussed in section 5.11.

- Sample temperature—As the mixture temperature cools, the overall mixture viscosity will increase. This increasing mixture viscosity will increase the resistance to compaction in the mold and in the field, thus resulting in an increased VMA condition. Many warm mix additives are available that will reduce the viscosity of a mixture at lower temperatures. This technology is discussed in chapter 12.
- Aggregate shape, strength and texture—These values are very subjective and difficult to measure. More cubical or angular materials will increase the resistance to compaction. Rougher surface textures will also provide the same results. Aggregate strength is critical since a weak aggregate can degrade or break down during compaction, thus changing the gradation and greatly impacting VMA.

Major Factors

- Type and amount of laboratory compactive effort—It is important to remind the designer that VMA is basically the total void space between the aggregate particles in a compacted asphalt mixture. The type of compactor and number of compactive repetitions utilized in the mix design process will have a significant impact on the resulting VMA. Gyratory compactors utilized in the Superpave system impart significantly more energy into a specimen than traditional impact hammers used in the Marshall method and will result in lower VMA for any given blend of aggregate. Intuitively, a higher number of gyrations or number of blows will also decrease the VMA in a compacted specimen.
- Aggregate gradation—The gradation of an aggregate blend is perhaps one of the most influential factors governing VMA. A thorough understanding of the gradation plotting tools described in chapter 3 will assist the designer in formulating a blend to meet VMA criteria. It is very difficult to predict the VMA of a mixture based solely on gradation. All of the factors previously described in this section will impact the final VMA result. The final VMA of different blends cannot be determined until the mixtures are compacted with the specified compactor at the specified number of repetitions. The Bailey method is an excellent tool that will predict the change in VMA in response to gradation changes, with all of the other factors remaining constant. Detailed information on the Bailey method is available at www.asphaltinstitute.org.

5.7

Percent VFA in compacted mixture

The voids filled with asphalt (VFA) is the percentage by volume of the VMA that is filled with the effective binder. VFA, like VMA, also tends to increase as the mix becomes finer and gains more total aggregate surface area. The VFA can be calculated with either of the following equations.

VFA can be visually determined from the phase diagram in **Figure 5.3** and is defined by the following relationship:

$$VFA = 100 \times \frac{V_{be}}{V_{be} + V_a} \qquad \text{(Eq. 5.23)}$$

VFA is most readily calculated with the following equation:

$$VFA = 100 \times \frac{VMA - P_a}{VMA} \qquad \text{(Eq. 5.24)}$$

where:

VFA = voids filled with asphalt
VMA = voids in the mineral aggregate
P_a = air voids in compacted mixture, percentage of total volume
V_{be} = volume of the effective (non-absorbed) binder, typically in cm^3
V_a = volume of voids in compacted mixture, typically in cm^3

Using the data in **Table 5.1**, the resulting P_a in section 5.5, and the resulting VMA in section 5.6 :

$$VFA = 100 \times \frac{14.3 - 3.7}{14.3} = 74.1 \qquad \text{(Eq. 5.25)}$$

5.8

Binder absorption

The percent binder absorption (P_{ba}) is the percentage by mass of binder that is absorbed into the aggregate. It is assumed that the amount of binder absorbed into the aggregate is a constant value; therefore, it is calculated based on the mass of the aggregate. Note that if the absorption was calculated based on the total mass of the mix, the percent absorption would change based on the amount of binder added to the mix.

P_{ba} can be visually determined from the phase diagram in **Figure 5.3** and is defined by the following relationship:

$$P_{ba} = 100 \times \frac{M_{ba}}{M_s} \qquad \text{(Eq. 5.26)}$$

P_{ba} is most readily calculated with the following equation:

$$P_{ba} = 100 \times \frac{(G_{se} - G_{sb})}{(G_{se} \times G_{sb})} \times G_b \qquad \text{(Eq. 5.27)}$$

where:

P_{ba} = absorbed binder, percentage by mass of aggregate
G_{se} = effective specific gravity of aggregate
G_{sb} = bulk (dry) specific gravity of the aggregate
G_b = specific gravity of binder
M_{ba} = mass of the absorbed asphalt, typically in grams
M_s = mass of the aggregate, typically in grams

Using the data in **Table 5.1** along with the resulting combined G_{sb} in section 5.3.4, and the resulting G_{se} in section 5.4.1:

$$P_{ba} = 100 \times \left[\frac{(2.761 - 2.700)}{(2.761 \times 2.700)}\right] \times 1.030 = 0.8 \qquad \text{(Eq. 5.28)}$$

5.9

Effective binder content of a paving mixture

The effective binder content (P_{be}) of a paving mixture is the percentage by mass of binder that stays on the outside of aggregate particles and is not absorbed. It is effective or usable, as the "glue" that binds the mix together and governs the performance of an asphalt paving mixture.

Note that P_{be} is expressed as a percentage of the total mix mass. That means that mathematically, $P_{ba} + P_{be} \neq P_b$, the total binder content, because P_{ba} is a percentage of the total aggregate and P_{be} is a percentage of the total mix. However, the mass of the total aggregate and the mass of the total mix are so close in magnitude that in a practical sense, when calculated to the nearest 0.1 percent, the absorbed and effective binder contents added together usually equals the total binder content. It can be calculated as follows:

From the phase diagram, P_{be} can be defined as:

$$P_{be} = 100 \times \frac{M_{be}}{M_{mb}} \qquad \text{(Eq. 5.29)}$$

P_{be} is most readily calculated with the following equation:

$$P_{be} = P_b - \frac{P_{ba}}{100}\, P_s \qquad \text{(Eq. 5.30)}$$

where:

P_{be} = effective binder, percentage by mass of mix
P_b = total binder, percentage by mass of mix
P_{ba} = absorbed binder, percentage by mass of aggregate
P_s = total aggregate, percentage by mass of mix
M_{be} = mass of the effective binder, typically in grams
M_{mb} = mass of the total mix, typically in grams

Using the data in **Table 5.1** and the resulting P_{ba} in section 5.8:

$$P_{be} = 5.3 - \frac{0.8}{100}\, 94.7 = 4.5 \qquad \text{(Eq. 5.31)}$$

5.10

Dust to binder ratio

The dust to binder ratio ($P_{0.075}/P_{be}$) of a paving mixture, sometimes referred to as the "dust proportion," is the ratio of the percentage of aggregate passing the 0.075-mm (No. 200) sieve ($P_{0.075}$) to the effective binder (P_{be}). The typical allowable range for this property is 0.6–1.2, with the following exceptions:

- for 4.75-mm mixes, the allowable range is 0.9–2.0; and
- for coarse-graded mixes whose gradation plots below the Primary Control Sieve (PCS) on a 0.45 power chart, the allowable range may be increased to 0.8–1.6.

In general, this property addresses the workability of asphalt mixtures. A low $P_{0.075}/P_{be}$ often results in a tender mix, which lacks cohesion and is difficult to compact in the field because it tends to move laterally under the roller. Mixes tend to stiffen as the $P_{0.075}$ increases, but too much will also result in a tender mix. A mix with a high $P_{0.075}/P_{be}$ will often exhibit a multitude of small stress cracks during the compaction process, called check-cracking. This property is usually calculated for dense-graded mixes only.

Using the P_{be} in section 5.9 and an example $P_{0.075}$ of 4.9 percent:

$$\frac{P_{0.075}}{P_{be}} = \frac{4.9}{4.5} = 1.1 \qquad \text{(Eq. 5.32)}$$

5.11

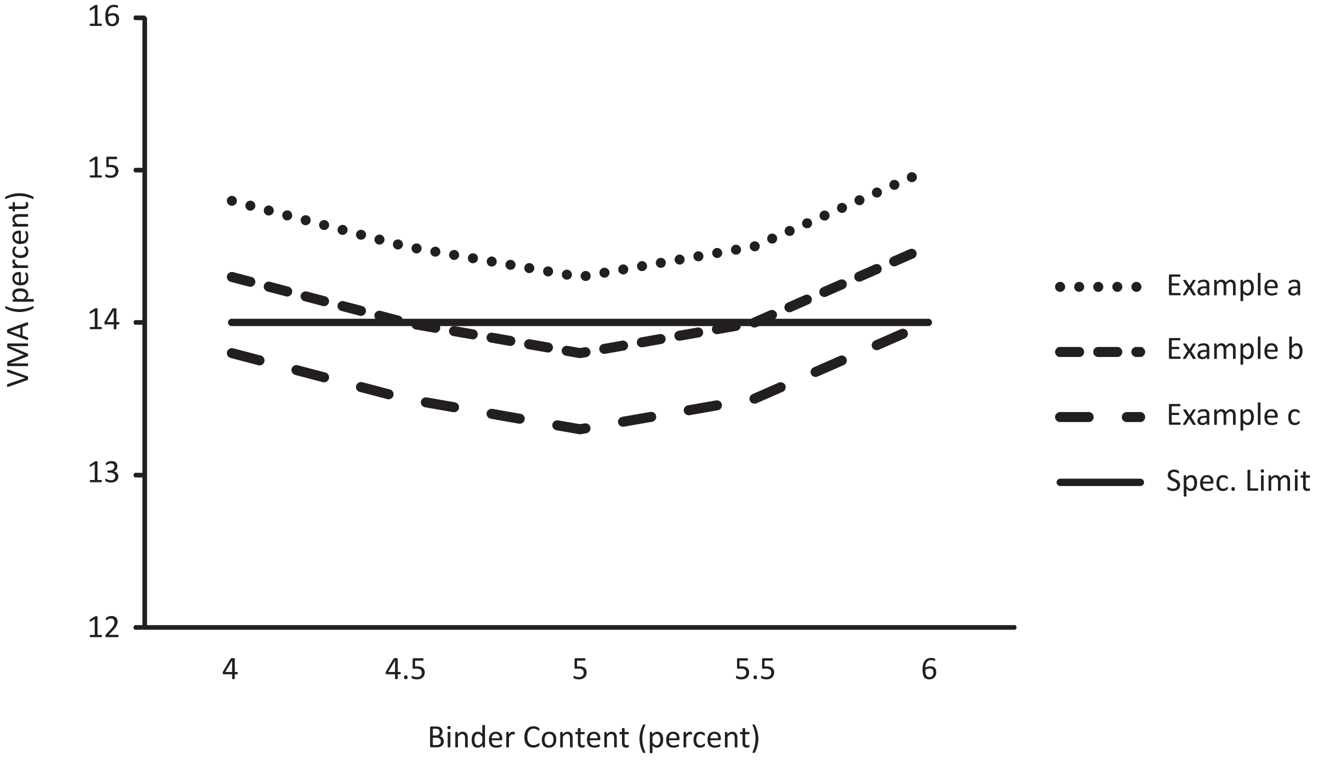
Discussion on volumetric properties

5.11.1 Evaluation of VMA curve

In many cases, the most difficult mix design property to achieve is a minimum amount of voids in the mineral aggregate. The goal is to furnish enough space for the asphalt cement so it can provide adequate adhesion to bind the aggregate particles, but without bleeding when temperatures rise and the asphalt expands. Normally, the curve exhibits a flattened U-shape, decreasing to a minimum value and then increasing with increasing asphalt content, shown in **Figure 5.8, example a**.

This dependency of VMA on asphalt content appears to be a contradiction to the definition. One might expect the VMA to remain constant with varying asphalt content, thinking that the air voids would simply be displaced by asphalt cement. In reality, the total volume changes across the range of asphalt contents; the assumption of a constant unit volume is not accurate. With the increase in asphalt, the mix actually becomes more workable and compacts more easily, meaning more weight

FIGURE 5.8 ***Relationship Between VMA and Specification Limit***



can be compressed into the unit volume. Therefore, up to a point, the bulk density of the mix increases and the VMA decreases.

At some point as the asphalt content increases (the bottom of the U-shaped curve), the VMA begins to increase because relatively higher specific gravity material (aggregate) is displaced and pushed apart by the lower specific gravity material (asphalt cement). It is recommended to avoid asphalt contents on the "wet," or right-hand, increasing side of this VMA curve, even if the minimum air void and VMA criteria are met. Design asphalt contents in this range have a tendency to bleed and/or exhibit plastic flow when placed in the field. Any amount of additional compaction from traffic leads to inadequate room for asphalt expansion, loss of aggregate-to-aggregate contact and eventually, rutting and shoving in high-traffic areas. Ideally, the design asphalt content should be selected slightly to the left of the low point of the VMA curve.

In some mixes, the bottom of the U-shaped VMA curve is very flat, meaning that the compacted mixture is not as sensitive to asphalt content in this range as some other factors. In the normal range of asphalt contents, compactability is influenced more by aggregate properties. However, at some point, the quantity of asphalt will become critical to the behavior of the mix, and the effect of asphalt will dominate as the VMA increases drastically.

When the bottom of the U-shaped VMA curve falls below the minimum criteria level required for the nominal maximum aggregate size of the mix (see **Figure 5.8**, **example b**), this is an indication that changes to the job-mix formula are necessary. Specifically, the aggregate grading should be modified to provide additional VMA; suggestions are provided in section 5.12. The design asphalt content should not be selected at the extremes of the acceptable range, even if the minimum criteria are met. On the left-hand side, the mix would be too dry, prone to segregation and would probably be too high in air voids. On the right-hand side, the mix would be expected to rut.

If the minimum VMA criteria are completely violated over the entire asphalt content range (curve is completely below minimum, **Figure 5.8**, **example c**), a significant redesign and/or change in material sources is warranted. Section 5.12, regarding the Design Aggregate Structure, provides an additional process for determining aggregate structures that will meet VMA specifications.

5.11.2 Effect of compaction level

At the same asphalt content, both air voids (P_a) and voids in the mineral aggregate (VMA) decrease with higher compactive effort. The three levels of compaction of the Marshall mix procedure (35, 50 and 75 blows) can be used to illustrate the consequences of this fact. As shown in **Figure 5.9**, **example a**, not only do the magnitudes of the VMA values change with different levels of compaction, but also the asphalt content value at which the minimum VMA occurs shifts. If a mix is designed slightly to the left of minimum VMA at a compaction level of 50 blows and the pavement actually endures heavier traffic than expected (closer to a 75-blow design level), then the same asphalt content now plots on the right-hand or "wet" side of the minimum VMA point for the mix if it had been designed using 75-blow compaction. At the higher traffic level, this mix will be susceptible to rutting.

This scenario can also work in the opposite direction. If a mix, designed at a compaction level of 75 blows as shown in **Figure 5.9**, **example b**, is placed in a pavement with much lower volumes of traffic, then the final percentage of air voids (P_a) will be considerably higher than planned. This condition could lead to a more open, permeable mix allowing air and water to pass through easily. The consequence of this situation is a mix that hardens prematurely, becomes brittle and cracks at an early age or the aggregate ravels out of the mix because of the loss of asphalt adhesion. This condition may also lead to stripping as discussed in chapter 9.

For this reason, it is important that the compactive effort used to simulate the design traffic expected in the pavement be selected accordingly in the laboratory. Similarly, the mixture must be constructed with appropriate compaction effort in the field to produce adequate initial density regardless of climatic conditions.

It is also important to note that the VMA criteria do not change based on the level of compaction. The reasoning for having sufficient VMA (providing space for the asphalt and air voids) is consistent regardless of the traffic level for which the mixture is being designed.

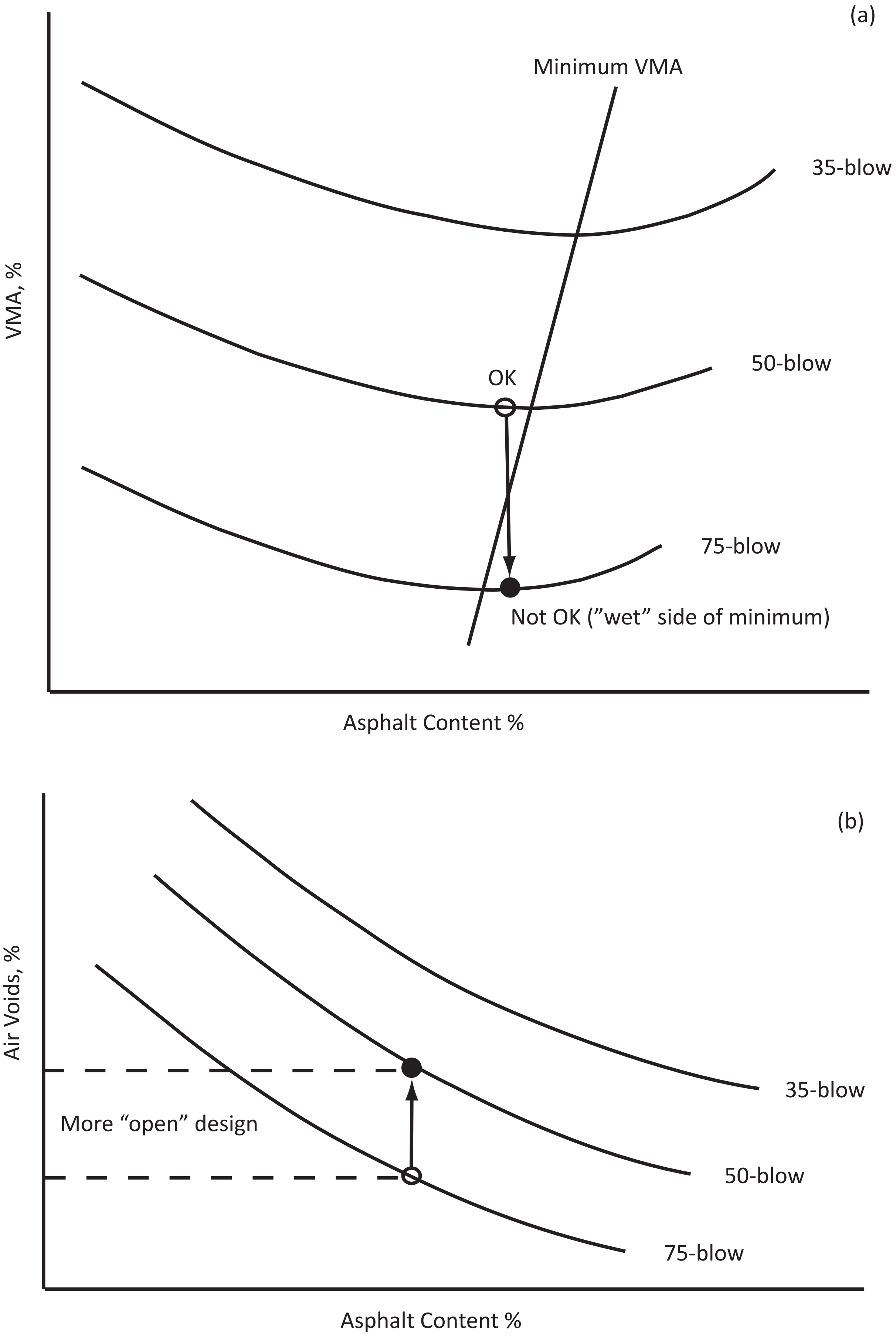


FIGURE 5.9 ***Effect of Marshall Compactive Effort on VMA and Air Voids***

5.11.3 Effect of air voids

It should be emphasized that the design level of air voids (4 percent) is the level desired after several years of traffic. This design level of air voids does not vary based on traffic; the laboratory compactive effort varies and is selected for the expected traffic. This design air void range will normally be achieved if the mix is designed at the correct compactive effort and the percent air voids after construction is no more than 8 percent. Some consolidation with traffic is expected.

The consequence of a change in any factor or in the mix design procedure will be a loss of performance or service life. It has been shown that mixtures that ultimately consolidate to less than 2 percent air voids can be expected to rut and shove if placed in heavy traffic locations. Several factors may contribute to this occurrence, such as: an arbitrary or accidental increase in asphalt content at the mixing facility, or an increased amount of ultra-fine particles passing the 75-µm (No. 200) sieve, which can act as an asphalt extender, just to name a few.

Similarly, problems can occur if (after years of traffic) the final air void content of the pavement is above 5 percent, or if initially constructed with over 8 percent air voids. Brittleness, premature cracking, raveling and stripping are all possible under these conditions.

The overall objective is to limit adjustments of the design asphalt content to less than 0.5 percent air voids from the median of the design criteria (4 percent air voids). If “dryer” or “richer” mixtures are desired, the laboratory compaction should be changed to fit the pavement type being considered in the design.

5.11.4 Effect of voids filled with asphalt

Although VFA, VMA and P_a are all interrelated and only two of the values are necessary to solve for the other, including the VFA criteria helps prevent the design of mixes with marginally acceptable VMA. The main effect of the VFA criteria is to limit maximum levels of VMA, and subsequently, maximum levels of asphalt content.

VFA also restricts the allowable air void content for mixes that are near the minimum VMA criteria. Mixes designed for lower traffic volumes will not pass the VFA criteria with relatively high percent air voids (5 percent) even though the air void criteria range are met. The purpose is to avoid less durable mixes in light traffic situations.

Mixes designed for heavy traffic will not pass the VFA criteria with relatively low percent air voids (less than 3.5 percent) even though that amount of air voids is within the acceptable range. Because low air void contents can be very critical in terms of permanent deformation (as discussed previously), the VFA criteria help to avoid those mixes that would be susceptible to rutting in heavy traffic situations.

The VFA criteria provide an additional factor of safety in the design and construction process in terms of performance. Since changes can occur between the design stage and actual construction, an increased margin for safety is desirable.

5.12

Selecting a design aggregate structure

Meeting the minimum VMA requirement at the design binder content is often difficult to achieve. It is not uncommon for a mixture designer to complete a laboratory mixture design only to learn that the VMA does not meet the volumetric criteria specified. The Superpave mix design process originally included a procedure called Selecting a Design Aggregate Structure (DAS) to assess the ability of multiple gradations to meet specified VMA criteria at the design binder content. This process can be used to evaluate different aggregate combinations before or after the initial mixture design is completed. This process can also be very useful in optimizing material combinations for Marshall or Superpave mix designs. The process can be summarized as an analysis of multiple aggregate combinations at one estimated design binder content during a single laboratory session.

The process of conducting a design aggregate structure evaluation requires the individual aggregate characteristics, specific gravity and gradation properties to be determined on each aggregate material. These procedures are discussed in chapter 3. Trial blends are then established by mathematically combining the gradations and other aggregate properties of the individual aggregates

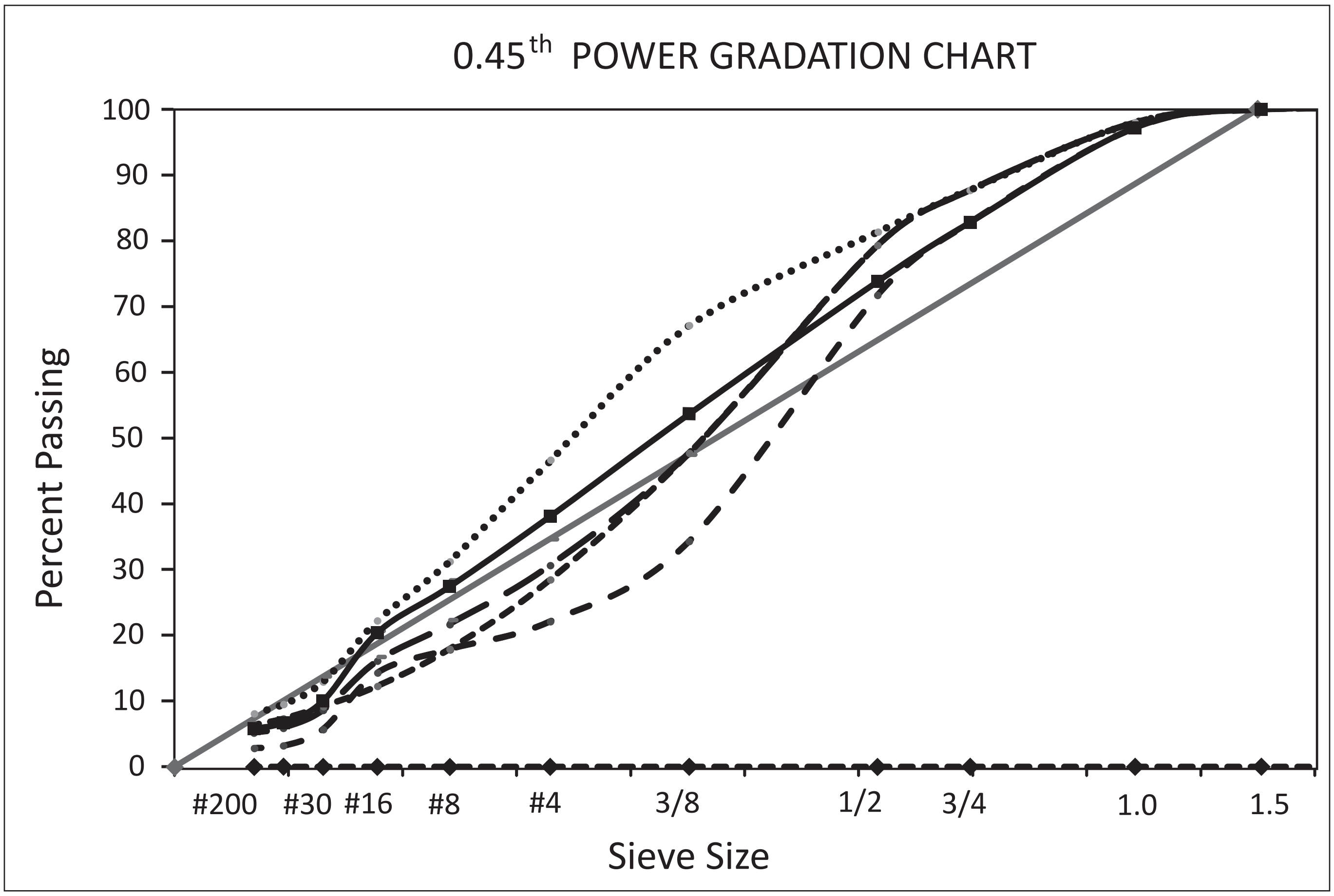


FIGURE 5.10 ***Multiple Gradations Used in D.A.S.***

for each proposed blend. Typically the blended properties are then compared to the specification criteria required for the mixture to be designed. The number of material combinations can be limitless, depending on the characteristics of aggregate materials and the purpose for which a design aggregate structure is being performed. Often when conducting a DAS for a typical project mixture design, a designer may evaluate multiple gradations that range from the high to the low side of gradation range allowed in the contract, while maintaining the aggregate properties to meet the specified criteria. An example plot of multiple trial gradations is shown in **Figure 5.10**.

Specified aggregate properties can be estimated by mathematical calculation. However, consideration should be given to actually batching and testing for those parameters that may be borderline or are at risk of failure during actual production, such as FAA.

After the aggregate properties have been evaluated, the next step is to compact specimens to determine the volumetric properties of each trial blend. The asphalt binder content for each trial blend must be determined. The goal is to estimate the percent binder required for the compacted specimen to achieve the design air void content, which is 4 percent if you are conducting a standard Superpave mixture design. Selection of a binder content that results in a compacted specimen that is within +/−2 percent of the design binder content is usually sufficient for the DAS. Some designers utilize existing methods such as those discussed in sections 8.3 through 8.8. Others will rely on past experience, intuition or simply use a fixed value such as 5 percent binder. A highly accurate initial estimate is not critical for this procedure, as an estimated design binder content will be calculated based on the results obtained from the laboratory specimen compacted at this initial binder content selected.

The binder content of mixtures generally varies based on the maximum aggregate size, gradation and aggregate properties. The absorption rate

Nominal Maximum Aggregate Size (mm)	Trial Asphalt Binder Content (%)
37.5	3.5
25.0	4.0
19.0	4.5
12.5	5.0
9.5	5.5

TABLE 5.3 ***Default Binder Contents for DAS***

for the aggregate plays a large role in the amount of asphalt needed to obtain the design air void content in the mixture. The values in **Table 5.3** may be useful for aggregate blends having combined aggregate bulk specific gravity of approximately 2.650. Aggregate combinations having significantly higher G_{sb} may need less asphalt and those with a lower G_{sb} may need more asphalt.

It is recommended that two accurately batched specimens be prepared for initial G_{mb} and G_{mm} testing on each proposed blend. Other tests necessary to verify aggregate properties or characteristics of concern should also be prepared at this time. The following step-by-step process is conducted for each trial blend prepared. The basic DAS process is suitable for Marshall and Superpave designed mixtures. Additional steps are required that are specific to gyratory compacted mixtures.

$$P_{a_{initial}} = \frac{G_{mm_{initial}} - G_{mb_{initial}}}{G_{mm_{initial}}} \times 100 \qquad \text{(Eq. 5.33)}$$

where,

$P_{a\ initial}$ = % air voids in the mix at $P_{b\ initial}$

$G_{mb\ initial}$ = bulk specific gravity of the mix at $P_{b\ initial}$

$G_{mm\ initial}$ = theoretical maximum specific gravity of the mix at $P_{b\ initial}$

Step 3—Calculate $\text{VMA}_{\text{initial}}$

$$VMA_{initial} = 100 - \frac{G_{mb_{initial}} \times P_s}{G_{sb}} \qquad \text{(Eq. 5.34)}$$

where,

$\text{VMA}_{\text{initial}}$ = voids in the mineral aggregate at $P_{b\ initial}$

$G_{mb\ initial}$ = bulk specific gravity of the mix at $P_{b\ initial}$

P_s = percent aggregate in the specimen

G_{sb} = bulk specific gravity of the aggregate blend

The following three steps are for gyratory compacted mixes. The steps shown are complete for mixtures being compacted to N_{des}. If the mixtures are compacted to N_{max}, steps 5 and 6 must be followed to calculate the corrected N_{des} and N_{ini}.

Step 4—Calculate the $\%G_{mm}$ @ N_{des}– for gyratory compacted mixes only

$$\%G_{mm}@N_{des} = 100 - P_{a_{initial}} \qquad \text{(Eq. 5.35)}$$

where,

$\%G_{mm}$ @ N_{des} = percent of theoretical maximum specific gravity at N_{des} using $P_{b\ initial}$

$P_{a\ initial}$ = % air voids in the mix at N_{des} using $P_{b\ initial}$

Step 5—Calculate the Correction Factor for G_{mb} @ N_{ini}–for gyratory compacted mixes only

$$C = \frac{Ht.@N_{des}}{Ht.@N_{ini}} \qquad \text{(Eq. 5.36)}$$

where,

C = the correction factor

$\text{Ht.@}N_{des}$ = height of the specimen at N_{des}

$\text{Ht.@}N_{ini}$ = height of the specimen at N_{ini}

Step 6—Calculate the G_{mb} @ N_{ini}– for gyratory compacted mixes only

$$G_{mb}@N_{ini} = C \times \left(G_{mb}@N_{des}\right) \qquad \text{(Eq. 5.37)}$$

Step 7—Calculate $P_{b\ estimated}$

$$P_{b_{estimated}} = P_{b_{initial}} - 0.4 \times \left(4.0 - Va_{initial}\right) \qquad \text{(Eq. 5.38)}$$

where,

$P_{b\ estimated}$ = the asphalt content needed to obtain a mixture at the design P_a

$P_{b\ initial}$ = the initial binder content of the trial blend

$P_{a\ initial}$ = percent air voids in the mix at $P_{b\ initial}$

0.4 = a value derived from the slope of the P_a curve which equates to a 0.4% change in binder = 1% change in air voids

4.0 = the design air void level

$P_{b\ estimated}$ is an estimated value of how much binder should have been added to the trial blend to achieve the design air voids in the compacted mixture. The following steps calculate the typical volumetric properties in the mixture assuming that $P_{b\ estimated}$ had been added to the trial blend.

Step 8—Calculate VMA$_{estimated}$

$$VMA_{estimated} = VMA_{initial} + C \times \left(4.0 - P_{a_{initial}}\right)$$

(Eq. 5.39)

where,

$VMA_{estimated}$ = the estimated VMA had the trial blend used $P_{b\ estimated}$

$VMA_{initial}$ = Voids in the mineral aggregate at $P_{b\ initial}$

C (Constant) = 0.1 if V_a is less than 4.0%

= 0.2 if V_a is greater than 4.0%

$P_{a\ initial}$ = % air voids in the mix at $P_{b\ initial}$

4.0 = the design air void level

The VMA in a compacted mixture is a result of the aggregate properties, binder properties, temperature and perhaps the most significant, the type and amount of compactive effort exerted on the specimen. Minute changes in binder content in close proximity to the design binder content have little effect on VMA, but are accounted for in the above-referenced C value.

Step 9—Calculate VFA$_{estimated}$

$$VFA_{estimated} = 100 \times \frac{\left(VMA_{estimated} - 4.0\right)}{VMA_{estimated}} \quad \text{(Eq. 5.40)}$$

where,

$VFA_{estimated}$ = the estimated VFA had the trial blend used $P_{b\ estimated}$

$VMA_{estimated}$ = the estimated VMA had the trial blend used $P_{b\ estimated}$

4.0 = the design air void content

Step 10—Calculate P$_{be\ estimated}$

The effective asphalt binder content is calculated using:

$$P_{be_{estimated}} = P_{b_{estimated}} - \left(P_s \times G_b\right) \times \frac{G_{se} - G_{sb}}{G_{se} \times G_{sb}}$$

(Eq. 5.41)

where,

$P_{be\ estimated}$ = the estimated effective binder content had the trial blend used $P_{b\ estimated}$

P_s = aggregate content, percent by total mass of the mixture

G_b = specific gravity of the asphalt

G_{se} = effective specific gravity of the aggregate

G_{sb} = bulk specific gravity of the aggregate

$P_{b\ estimated}$ = the asphalt content needed to obtain a mixture at 4.0% air voids

Step 11—Calculate Dust Proportion

The requirement for the dust proportion is calculated as the percent by mass of the material passing the 0.075-mm sieve (by wet sieve analysis) divided by the effective asphalt binder content (expressed as percent by mass of mix).

$$DP = \frac{P_{0.075}}{P_{be_{estimated}}} \quad \text{(Eq. 5.42)}$$

where,

$P_{be\ estimated}$ = the estimated effective binder content had the trial blend used $P_{b\ estimated}$

$P_{0.075}$ = aggregate content passing the 0.075-mm sieve

Finally, for Superpave gyratory compacted mixtures:

Step 12—Calculate %G$_{mm,estimated@Nini}$

$$\%Gmm@Nini_{estimated} = \%Gmm@Nini_{initial} - \left(4.0 - P_{a_{initial}}\right)$$

(Eq. 5.43)

where,

4.0 = the design air void content

Careful analysis of the DAS results will provide the designer with valuable information upon which to make design changes or decisions on whether or not to proceed with a full mixture design and performance testing.

CHAPTER 6

Superpave HMA Mix Design System

6.0

Introduction

In 1987 a five-year FHWA study to improve the performance of HMA pavements was started. The result was the Superpave (SUperior PERforming Asphalt PAVEments) system.

This system was developed to give pavement engineers and contractors the tools they needed to improve the performance of HMA pavements. The Superpave system as implemented consisted of two interrelated elements: an asphalt binder specification thoroughly discussed in Asphalt Institute publication MS-26 and a mix design system that specifies aggregate criteria and volumetric properties.

A mixture performance and analysis system was developed utilizing the Superpave Shear Tester (SST), but due to complexities of the equipment and procedure was not implemented as part of the final Superpave mix design procedure. The following summarizes the current Superpave mix design procedure:

1. selecting asphalt and aggregate materials that meet the design criteria;
2. developing an aggregate blend that will meet the Superpave requirements;
3. mixing and short-term aging the selected asphalt binder and aggregate blend;
4. compacting specimens utilizing the Superpave gyratory compactor according to expected traffic levels;
5. analyzing the volumetric properties of the mixture (using the procedures described in chapter 5);
6. selecting the best aggregate and asphalt blend that meets the specified criteria for the mixture; and
7. mixture performance testing for moisture sensitivity.

Additional mixture performance testing described in chapter 10 is recommended to assure confidence in the final design but is not currently specified in the Superpave system.

The procedures and calculations for conducting a mix design containing Recycled Asphalt Pavement (RAP) and Recycled Asphalt Shingles (RAS) are outlined in chapter 11.

6.1

Superpave materials selection and mix design criteria

Superpave is a system. A pavement mixture will not perform successfully on the roadway unless the appropriate binder and aggregate materials are incorporated into the work.

6.1.1 Asphalt binder

The asphalt binder to be used in the mix is chosen based on environmental conditions and traffic loadings using the procedures described in Asphalt Institute publication MS-26. The grade of asphalt binder to be incorporated into the HMA is usually specified during the project design and is not a variable considered during the mix design phase. If the mix designer is responsible for selecting the proper binder grade, please refer to and become familiar with MS-26.

6.1.2 Mineral aggregate specifications

Mix design procedures prior to Superpave did not contain detailed aggregate requirements. Aggregate specifications were typically left up to local, specifying agencies or pavement owners. During the development of Superpave, pavement experts agreed that certain aggregate characteristics were essential to design a well-performing HMA. These characteristics were broken into two categories: consensus properties and source properties. A detailed discussion of these properties and test procedures are included in chapter 3.

6.1.2.1 Consensus aggregate properties

The consensus aggregate properties specified in the Superpave system are the following: coarse aggregate angularity; fine aggregate angularity; flat and elongated particles; and clay content (Sand Equivalent). The criteria for these properties are based on the traffic level and position within the pavement structure. The materials near the pavement surface (≤ 100 mm) are subjected to higher stress levels and require materials of higher quality than mixtures that are located lower in pavement section (> 100 mm). The following criteria, in **Table 6.1**, are applied to the final aggregate blend rather than individual components.

6.1.2.2 Source aggregate properties

The Superpave mix design system also allows for aggregate characterization of what are called source properties. These "source properties" are requirements specific to local areas because of the variety of aggregate types available in different geographic areas, and are not broad, all-inclusive requirements like Superpave consensus properties. Source property tests are typically specified to

TABLE 6.1 ***Superpave Aggregate Consensus Requirements***

20-Year Design ESALs^a (in millions)	Coarse Aggregate Angularity (Percent), minimum^c AASHTO T 335 (CAA)		Uncompacted Void Content of Fine Aggregate Angularity (Percent), minimum AASHTO T 304 (FAA)		Sand Equivalent (Percent), minimum AASHTO T 176 (SE)	Flat and Elongated^c (Percent), maximum ASTM D4791 (F&E)
	≤ 100 mm^f	> 100 mm^f	≤ 100 mm	> 100 mm^f		
< 0.3	55/-	-/-	-^{d}	-	40	-
0.3 to < 3	75/-	50/-	40^e	40	40	10
3 to < 10	85/80^b	60/-	45	40	45	10
10 to < 30	95/90	80/75	45	40	45	10
≥ 30	100/100	100/100	45	45	50	10

NOTES:

a Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years and choose the appropriate N_{design} levels.

b 85/80 denotes that 85 percent of the coarse aggregate has one or more fractured faces and 80 percent has two or more fractured faces.

c This criterion does not apply to 4.75-mm nominal maximum aggregate size mixtures.

d For 4.75-mm nominal maximum aggregate size mixture designed for traffic levels below 0.3 million ESALs, the minimum Uncompacted Void Content is 40.

e For 4.75-mm nominal maximum aggregate size mixture designed for traffic levels equal to or above 0.3 million ESALs, the minimum Uncompacted Void Content is 45.

f If less than 25 percent of a construction lift is within 100 mm of the surface, the lift may be considered to be below 100 mm for mix design purposes.

20-Year Design Equivalent Single Axle Loads (ESALs in millions)	Los Angeles Abrasion (Max. %) AASHTO T 96	Sodium or Magnesium Sulfate Soundness (Max. %) AASHTO T 104	Deleterious materials*	
			Clay Lumps/ Friable Particles AASHTO T 112	Lightweight Particles AASHTO T 113
< 0.3	45	25	<5	<5
0.3 to < 3	40	20	<4	<4
3 to < 10	30	15	<3	<3
10 to < 30	30	15	<2	<2
≥ 30	25	<10	<1	<1
*Specific tests and property requirements to be determined locally				

TABLE 6.2 ***Recommended Superpave Source Property Tests and Typical Requirements***

set limits on aggregate properties like toughness, soundness and deleterious materials. **Table 6.2** lists some recommended source property tests and typical requirements.

6.1.2.3 Gradation

Superpave gradation requirements are shown in **Table 6.3**. Control points have been established to define the type of mix. It is important to note that the recommended gradation control points are limited in number when compared to traditional HMA gradation specifications. The following gradation control points should be regarded as a requirement for mixture design only. Actual field production testing may and should consist of a larger nest of sieves with tighter tolerances than the limits shown in the following table. The Superpave gradation requirements shown are only useful if the aggregates meet the consensus and source properties and the completed asphalt mixture meets the volumetric and/or performance requirements. Implementing a specification for compliance or quality control based solely on the following gradation parameters without mixture and aggregate property requirements may result in catastrophic failure.

Mixtures containing Reclaimed Asphalt Pavement (RAP), Recycled Asphalt Shingles (RAS) or any other recycled product can be designed to meet the aggregate and mixture property requirements specified in the Superpave system. However, significant alterations of typical virgin aggregate materials may be required. The final aggregate materials must be chosen based on the blended materials meeting the gradation and consensus and source requirements, as specified.

If the designer is unfamiliar with the materials being considered for design, they are strongly encouraged to investigate the detailed laboratory procedure outlining the design of an aggregate structure (DAS) as discussed in chapter 5. Significant laboratory time can be saved by utilizing this procedure to evaluate which material combinations have the best chance of meeting Superpave mixture requirements, especially the VMA requirement.

6.1.3 Superpave mixture requirements

6.1.3.1 N_{ini}, N_{des}, N_{max}

Prior to the development of the Superpave mix design procedure, laboratory compaction data were only available on samples compacted to the final design density. The modern instrumentation of SGCs, and the ability to measure sample height after every gyration, enables the designer to estimate a G_{mb} value at any specimen height or number of gyrations. AASHTO R 35 designates specified levels of laboratory compaction (density) at three different levels of gyration based on expected traffic loading as shown in **Table 6.4** and referenced as $N_{initial}$, N_{design} and $N_{maximum}$. These terms, more commonly designated as N_{ini}, N_{des} and N_{max}, refer to the number of gyrations estimated to result in different levels of field densification.

N_{ini} is a measure of the compactability of the mix and is based on the estimated field density obtained behind the screed before compaction. Mixtures that compact too quickly may be inherently tender to compact and would therefore be undesirable. The N_{ini} density specifications range from 89 to 91.5 percent of G_{mm}, depending on the design traffic level. Mixtures with a high percentage

NMAS	37.5 mm		25 mm		19 mm		12.5 mm		9.5 mm		4.75 mm	
Sieve, mm	**Min. % P**	**Max. % P**	**Min. % P**	**Max. % P**	**Min. % P**	**Max. % P**	**Min. % P**	**Max. % P**	**Min. % P**	**Max. % P**	**Min. % P**	**Max. % P**
50.0	100	-	-	-	-	-	-	-	-	-	-	-
37.5	90	100	100	-	-	-	-	-	-	-	-	-
25.0	-	90	90	100	100	-	-	-	-	-	-	-
19.0	-	-	-	90	90	100	100	-	-	-	-	-
12.5	-	-	-	-	-	90	90	100	100	-	100	-
9.5	-	-	-	-	-	-	-	90	90	100	95	100
4.75	-	-	-	-	-	-	-	-	-	90	90	100
2.36	15	41	19	45	23	49	28	58	32	67	-	-
1.18	-	-	-	-	-	-	-	-	-	-	30	55
0.600	-	-	-	-	-	-	-	-	-	-	-	-
0.300	-	-	-	-	-	-	-	-	-	-	-	-
0.150	-	-	-	-	-	-	-	-	-	-	-	-
0.075	0.0	6.0	1.0	7.0	2.0	8.0	2.0	10.0	2.0	10.0	6.0	13.0

TABLE 6.3 ***Superpave Gradation Requirements***

of natural sand or low FAA and CAA properties may fail this requirement. Although there is no minimum density at N_{ini} gyrations specified, the opposite is also true. Very coarse, harsh mixes may have densities at N_{ini} around 82 to 83 percent of G_{mm}, be inherently difficult to compact and have a higher risk of permeability. **Table 6.4** shows the Superpave compaction parameters based on expected traffic levels from AASHTO R 35.

N_{des} is the number of gyrations specified to reach the target density of the mix and is based on the estimated field density in the middle of its service

TABLE 6.4 ***Superpave Gyratory Compaction Effort***

20-Year Design ESALs* (in millions)	**Compaction Parameters**			**Typical Roadway Applications**
	$N_{initial}$	N_{design}	$N_{maximum}$	
< 0.3	6	50	75	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 considered local in nature, not regional, intrastate or interstate. Special-purpose roadways serving recreational sites or areas may also be applicable to this level.
0.3 to < 3	7	75	115	Applications include collector roads or access streets. Medium-trafficked city streets and the majority of county roadways may be applicable to this level.
3 to < 30	8	100	160	Applications include many two-lane, multilane, divided and partially or completely controlled access highways. Among these are medium to heavily trafficked city streets, many state routes, U.S. highways and some rural interstates.
≥ 30	9	125	205	Applications include the vast majority of the U.S. Interstate System, both rural and urban in nature. Special applications such as truck-weighing stations or truck-climbing lanes on two-lane roadways may also be applicable to this level.
*Laboratory compaction parameters should be based on 20-year design ESALs, regardless of the pavement design life of the roadway.				

life. AASHTO M 323 specifies the target density to be 96.0 percent of theoretical maximum specific gravity, or 4.0 percent air voids. Some agencies have increased the target density at N_{des} gyrations or lowered the number of design gyrations in an attempt to get a higher percentage of binder into the mix. Both methods will work if the aggregate structure is unaltered. However, the best way to ensure a higher percentage of binder gets into the mix without compromising the aggregate structure is to raise the minimum VMA requirement.

N_{max} is an estimate of the final density expected in the field at the end of its service life after years of further densification by traffic. It is capped at 98 percent for all traffic levels. Low air voids in the field result in a plastic mix that ruts easily and should be avoided. Originally, specimens were compacted to N_{max} gyrations and the density levels after N_{ini} and N_{des} gyrations were back-calculated. The current practice is to compact specimens to N_{des} gyrations, measure the density at that level directly, then back-calculate the density at N_{ini} gyrations. Separate specimens at the design binder content are then compacted to N_{max} gyrations and the density measured directly to check compliance with specifications.

The number of design gyrations was established by comparing lab-molded densities at different gyration levels to resulting field densities. Once the design gyration levels were established, the initial and maximum gyration levels were established by the following equations:

$$N_{ini} = (N_{des})^{0.45}$$

$$N_{max} = (N_{des})^{1.10}$$

Table 6.5 lists the specified levels of densification along with all of the required Superpave volumetric parameters.

6.2

Test equipment

6.2.1 Superpave gyratory compactor

The SGC consists of (see **Figure 6.1**):

- reaction frame, rotating base and motor;
- loading system, loading ram and pressure gauge;
- height measuring and recordation system;
- mold and base plate; and
- specimen extruding device.

The main parameters governing the compaction effort are:

- the vertical pressure, 600 ± 18 kPa;
- the gyration angle of the mold, 1.16 ± 0.02° (internal);
- gyration rate, 30.0 ± 0.5 gyrations per minute; and
- the number of gyrations, variable—based on expected traffic level.

6.2.1.1 Gyratory calibration and angle verification

Although all parameters need to be regularly and accurately calibrated, the gyration angle has the greatest effect on specimen compaction. As the angle increases, the compaction effort increases. Likewise, if the gyration angle decreases, specimen compaction will decrease. Relatively small changes in the gyratory angle can make significant changes in the compaction level. It is critical to maintain a well-calibrated compactor to assure no deviation in the internal gyration angle. **Figure 6.2** demonstrates the compaction parameters. In the fabrication of the SGC specimens, a constant consolidation pressure is applied to the sample while the mold gyrates. To provide specimens of consistent density, it is very important that the SGC maintain a constant pressure and a known constant angle of gyration during the compaction process. Each model of SGC uses a different method of setting, inducing and maintaining the angle of gyration. Each model also uses a unique calibration system to measure the angle of gyration externally in relation to the mold. These calibration systems cannot be used universally on all models of SGCs. Inconsistencies in the density of asphalt specimens produced by different SGCs can often be attributed to variations in the angle of gyration. This can be very troublesome when comparing mix designs prepared by different compactors or during quality control when the quality control specimens are manufactured using one brand of compactor and the acceptance specimens are manufactured using another brand of compactor. AASHTO has developed a procedure for evaluating the internal angle of an SGC, AASHTO TP 344 "Evaluation of the Superpave Gyratory Compactor (SGC) Internal Angle of Gyration Using Simulated Loading."

20-Year Design ESALs[a] (in millions)	Required Relative Density During Mix Design, Percent of Theoretical Maximum Specify Gravity (% G_{mm})			Voids in the Mineral Aggregate (VMA), Percent Minimum					Voids Filled with Asphalt (VFA)[b] Range, Percent	Dust-to-Binder Ratio (DP) Range[c]
	$N_{initial}$	N_{design}	$N_{maximum}$	Nominal Maximum Aggregate Size (NMAS), mm						
				37.5	25.0	19.0	12.5	9.5		
< 0.3	≤ 91.5	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	70–80 [d,e]	0.6–1.2
0.3 to < 3	≤ 90.5								65–78 [f]	
3 to < 10	≤ 89.0								65–75 [e,f,g]	
10 to < 30										
≥ 30									65–75 [g]	

NOTES:

a Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years.

b For 37.5-mm nominal maximum aggregate size mixtures, the specified lower limit of the VFA shall be 64 percent for all design traffic levels.

c For 4.75-mm nominal maximum aggregate size mixtures, the dust-to-binder ratio shall be 1.0 to 2.0, for design traffic levels < 3 million ESALs, and 1.5 to 2.0 for design traffic levels ≥ 3 million ESALs.

d For 4.75-mm nominal maximum aggregate size mixtures, the relative density (as a percent of the theoretical maximum specific gravity) shall be within the range of 94.0 to 96.0 percent.

e For design traffic levels < 0.3 million ESALs and for 25.0-mm nominal maximum size mixtures, the specified lower limit of the VFA range shall be 67 percent, and for 4.75-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 67 to 69 percent.

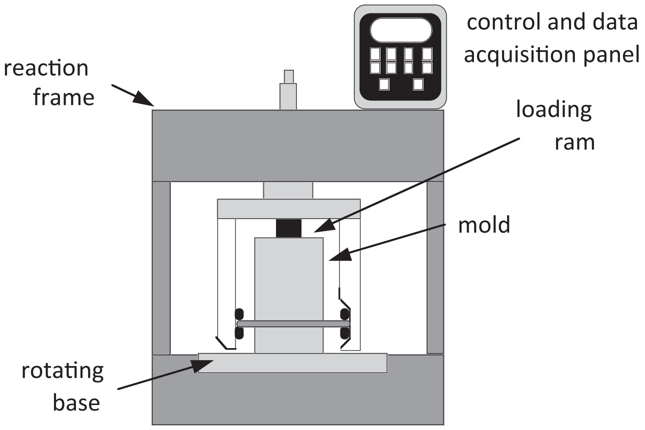
f For design traffic levels > 0.3 million ESALs and for 4.75-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 66 to 67 percent.

g For design traffic levels ≥ 3 million ESALs and for 9.5-mm nominal maximum aggregate size mixtures, the specified VFA range shall be 73 to 76 percent.

If the aggregate gradation passes beneath the specified PCS Control Point, the dust-to-binder ratio range may be increased from 0.6–1.2 to 0.8–1.6 at the agency's discretion.
Mixtures with VMA exceeding the minimum value by more than 2 percent may be prone to flushing and rutting. Unless satisfactory experience with high VMA mixtures is available, mixtures with VMA greater than 2 percent above the minimum should be avoided.

TABLE 6.5 ***Superpave Mixture Requirements***

FIGURE 6.1 ***Schematic of a typical SGC***



6.2.1.2 Gyratory mold verification

The SGC molds were designed to be 150 mm (6 inches) in diameter with minimum 7.5-mm thick steel walls and base plates of minimum C48 Rockwell Hardness to resist deformation under loading. The mold provides confinement for the mix during compaction. An accepted rule of thumb is that the mold diameter should be at least four times the maximum aggregate size of the mixture. The edge effects of compacting large-stone mixes in the mold versus how they compact in the field should be considered for NMAS mixes of 1-½ inches and greater. Although the 150-mm

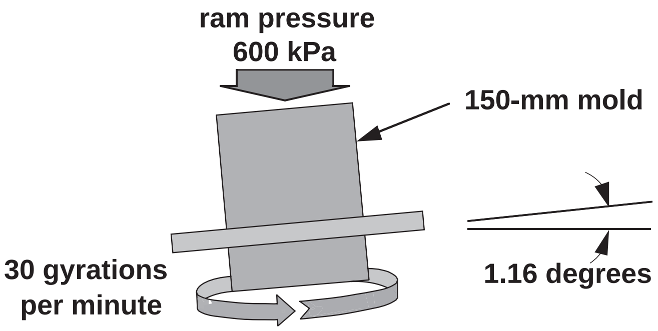


FIGURE 6.2 ***SGC Mold Configuration***

mold is the standard, 100-mm molds (4 inches) have been produced and used for various reasons.

The designer should be aware that the SGC and molds wear over time. The molds and equipment should be cleaned after each use, maintained according to manufacturer's recommendations and inspected regularly for signs of wear. This issue can be serious enough to create significant error. AASHTO T 312, "Standard Method of Test for Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor," which is the standard procedure for compacting samples with the SGC, has been recently updated to include "Annex A," a section that addresses mold diameter measurement. The issue concerns properly measuring the inside mold diameter in the area where the majority of the wear occurs, which is approximately in the middle to lower section of the mold. Designers are urged to become familiar with the procedure as it includes the use of a "Bore Gauge" to determine the acceptability of SGC molds.

6.2.1.3 Gyratory height instrumentation

The SGC has the ability to accurately measure the specimen height to the nearest 0.1 mm once per gyration during compaction. This function is used to estimate the mixture density at low levels of compaction and to document the compaction rate of the mixture. **Figure 6.3** shows a specimen's compaction characteristics based on its rate of compaction.

6.2.2 Additional test equipment

Ancillary test equipment (calibrated as needed) required in the preparation of Superpave asphalt mixtures includes:

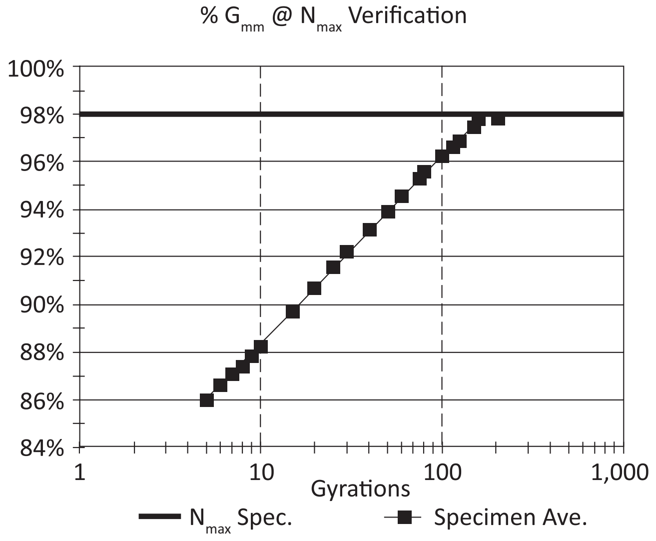


FIGURE 6.3 ***SGC N_{max} Mixture Compaction Characteristics***

1. ovens—thermostatically controlled, for heating aggregates, asphalt and equipment;
2. mechanical mixer—commercial bread dough mixer 10-liter (10-qt.) capacity or larger, equipped with metal mixing bowls and wire whips, or a 5-gallon laboratory bucket mixer;
3. flat-bottom metal pans for heating aggregates and conditioning mixtures;
4. round metal pans—approximately 10-liter capacity for mixing asphalt and aggregate;
5. scoops for batching aggregates;
6. containers—gill-type tins, beakers or pouring pots, for heating asphalt;
7. thermometers or other thermometric devices—armored, glass or dial-type with metal stem, 10°C to 235°C, for determining temperature of aggregates, asphalt and asphalt mixtures;
8. balances—10-kg capacity, sensitive to 1 g, for weighing aggregates and asphalt; 10-kg capacity, sensitive to 0.1 g, for weighing compacted specimens;
9. large mixing spoon or small trowel;
10. large spatula;
11. welders gloves (or similar) for handling hot equipment;
12. paint, markers or crayons, for identifying test specimens;
13. paper disks, 150 mm, for compaction;
14. fans, for cooling compacted specimens; and
15. computer/printer for data collection and recording.

6.3

Specimen preparation and compaction

6.3.1 Preparation of aggregate

The following samples must be prepared to conduct a Superpave mix design:

- a minimum of 8 specimens to be compacted to N_{des};
- a minimum of 2 specimens for maximum theoretical specific gravity (G_{mm}) testing;
- a minimum of 6 specimens for moisture sensitivity testing AASHTO T 283;
- a minimum of 2 specimens for N_{max} verification; and
- any other samples required for specified performance testing.

Three aggregate sample sizes are used in conducting a Superpave mix design, depending on their final use.

For compacted G_{mb} samples, the specimen size is 150 mm (diameter) by 115 mm (height) and requires two samples of approximately 4,700 grams of aggregate to be prepared for each trial asphalt content tested plus 2 additional specimens for N_{max} verification testing.

A minimum of two samples need to be prepared for determination of maximum theoretical specific gravity (G_{mm}). These samples remain uncompacted, and their size varies by the nominal maximum aggregate size (NMAS) of the mixture and ranges from 1,000 to 2,500 grams as specified in AASHTO T 209/ASTM D204. These samples will be mixed at an asphalt binder content at or above the final estimated design binder content. The G_{mm} values for the remaining trial binder contents may be back-calculated according to the equations shown in chapter 5.

Moisture damage testing using AASHTO T 283 requires a minimum of 6 specimens with a height of 95 mm. These samples require approximately 3,700 grams of aggregate.

6.3.2 Preparation of binder

Samples of asphalt binder to be used in the mix design shall be representative of the materials to be used in the project and should be of a size that is manageable in the laboratory. Care should be taken not to overheat the binder nor maintain elevated temperatures for an extended period of time. The mixture design will require the molding of mixture specimens at a minimum of four different binder contents. It is recommended that specimens be molded at the anticipated design binder content and at –0.5, +0.5 and +1.0 percent binder bracketing the anticipated design binder content.

6.3.3 Laboratory mixing and compaction temperatures

The mixing and compaction temperatures shall be determined according to chapter 4.2.1. Preheat the aggregate samples in an oven set approximately 15°C higher than the mixing temperature. Two to four hours are required for the aggregate to reach the mixing temperature. Preheat the asphalt binder to the predetermined mixing temperature. The time required for this step varies depending on the container size and method of heating. Care should be taken to adjust the preheating time in a manner that does not expose the binder to elevated temperatures for an extended period of time.

6.3.4 Preparation of mixtures

- Place the hot mixing bowl on a balance and zero the balance.
- Charge the mixing bowl with the heated aggregates and mix thoroughly.
- Form a crater in the blended aggregate and weigh the required asphalt in the mixture to achieve the desired batch weight.
- Remove the mixing bowl from the scale and mix the asphalt and aggregate using a mechanical mixer.
- Mix the specimen until the aggregate is thoroughly coated (30–90 seconds).
- Place the mix in a flat shallow pan at an even thickness ranging between 25 and 50 mm.
- Place the mix and pan in the conditioning oven at a temperature equal to mixture's specified compaction temperature ± 3°C.
- Condition the prepared mixture in accordance with chapter 4, section 4.2.3.
- Repeat this procedure until the desired number of specimens is produced.
- Proper timing of the gyratory compaction steps can be achieved by spacing

approximately 20 minutes between mixing each specimen.
- At the end of the short-term aging period, proceed to AASHTO T 209/ASTM D2041 if the mixture is to be used to determine maximum theoretical specific gravity. Otherwise, proceed with compaction.

6.3.5 Compaction of volumetric specimens

6.3.5.1 Molding procedure for specimens at a specified number of gyrations

Prepare at least two replicate specimens for each trial binder content at the N_{des} gyration level. G_{mb} specimen heights after compaction should be 115 ± 5 mm. Generally, placing approximately 4,700 grams of mixture into the mold for blends with aggregate specific gravities between 2.55 and 2.70 will result in a compacted specimen meeting height requirements.

Prepare the SGC while the mixture specimens are conditioning in the oven. Turn on the main power to the SGC for the manufacturer's recommended warm-up period. Further preparations include verifying the compaction pressure of 600 kPa, verifying the gyration internal angle of 1.16°, and verifying the gyration speed of 30 revolutions per minute. Set the SGC to the proper number of gyrations according to Table 6.4. Lubricate any bearing surfaces according to manufacturer instructions.

Approximately 45 to 60 minutes before compaction of the first specimen, place the molds and base/top plates in an oven set at the compaction temperature. Remove the mold and base plate from the oven, place the base plate inside the mold and place a paper disk (to prevent the mix from sticking to the base plate) on top of the base plate.

Place the conditioned mixture at the compaction temperature in the mold in one lift using a material handling chute, taking care to avoid segregation of the mix. Level the mix, place another paper disk on top of the mix and place the top plate (if required) in the mold. Load the charged mold into the SGC, centering it under the loading ram.

Depending on the SGC make and model, the following steps will either be performed automatically after pressing a "start" button or be initiated by the user:

- Lower the loading ram until it reaches a pressure of 600 kPa;
- Apply the 1.16° angle; and
- Proceed with gyrations to the preset number.

During compaction, the ram loading system will maintain a constant pressure of 600 kPa. The specimen height will be continually monitored during compaction, and a height measurement is recorded after each gyration.

The compactor will cease compacting after reaching the user-specified number of gyrations, the angle of gyration will be released and the loading ram will be raised. Remove the mold containing the compacted specimen from the compactor and slowly extrude the specimen from the mold. Most mixes can be removed immediately, but some mixes may require a 5- to 10-minute cooling period before they can be removed without distortion.

Remove the paper disks from the top and bottom of the specimen and then allow the specimen to cool on a flat surface, undisturbed. Place the mold and base/top plates back in the oven to reach compaction temperature for the next specimen. Additional molds will avoid the delay caused by this step. Repeat the compaction procedure for each specimen. Identify each specimen with a suitable marker.

6.3.5.2 Molding procedure for specimens at a specified height

The actual molding procedure is the same, except that the SGC is set to "height" mode instead of "# of gyrations" mode. This will cause the molding process to stop after reaching the specified height, rather than stopping after a specified number of gyrations. Some procedures, such as AASHTO T 283, call for specimens to be molded to a specific height with a specific percentage of air voids. To accomplish this, the designer must determine the appropriate mass of asphalt mixture to place in the mold that will result in the proper air void content at the proper height.

There are multiple ways to make this determination. One method requires the designer to mix and mold two extra specimens. A slightly higher-than-the-final-anticipated mass is placed in one mold and a slightly lower-than-the-final anticipated mass is placed in the other. After each specimen is molded to the specified height, the first specimen will have slightly lower-than-target air voids and the second specimen will have slightly higher-than-target air voids. The designer

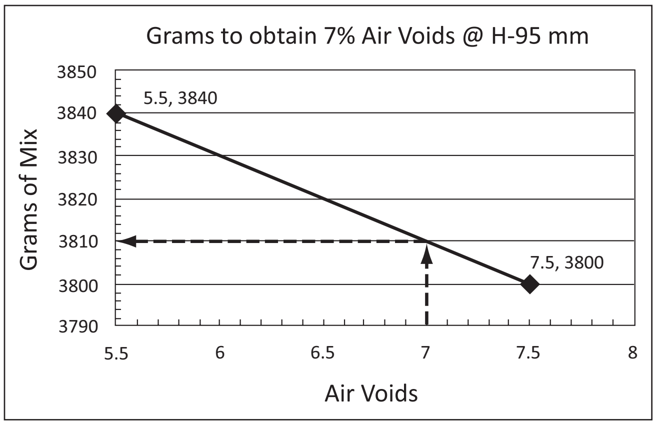


FIGURE 6.4 ***Graphical Determination of Mixture Mass at Specified Height***

can then interpolate the masses to achieve the proper air voids. For example:

Target height = 95.0 mm	Extra Specimen No. 1 Mass	Extra Specimen No. 2 Mass
Target air voids = 7.0%	3840 g	3800 g

After molding and weighing, Specimen 1 has an air void content of 5.5 percent and Specimen 2 has an air void content of 7.5 percent. The solution can be determined using a graph similar to **Figure 6.4**.

To calculate the amount of mix needed to result in 7.0 percent air voids after compaction: The target air void content is 0.5 percent lower than the air void content at 3,800 g. The air void content at 3,840 g is 2.0 percent lower than at 3,800 g. Therefore, the number of grams, *n*, more than 3,800 to reach "x" at 7.0 percent air voids is:

$n = (3840 - 3800) * [(7.5 - 7.0)/(7.5 - 5.5)] = 10\text{ g}$
$x = 3800 + 10 = 3810\text{ g}$

6.4

Superpave data analysis

6.4.1 Determine %G_{mm} at N_{ini} for each binder percentage

An additional step unique to Superpave is the calculation of %G_{mm} at N_{ini}, N_{des} and N_{max}. Using the data provided by the SGC, an estimated mixture bulk specific gravity, $G_{mb,estimated}$, can be calculated based on the diameter of the mold and the height of the specimen. This $G_{mb,estimated}$ value can be calculated for every gyration. The final product of the compaction process is an actual specimen with known height at N_{des}.

Step 1—Obtain the height of the Specimen at N_{des} (h_{Ndes}).

Step 2—Obtain the height of the Specimen at N_{ini} (h_{Nini}).

Estimating the $G_{mb,estimated}$ at N_{ini} using the smooth mold dimensions would not account for surface imperfections in the sample. These surface imperfections are accounted for when the sample compacted to N_{des} is actually weighed in the lab. By using the following correction factor, the estimated G_{mb} at N_{ini} can be more accurately calculated.

Step 3—Calculate the correction factor C, as follows:

$C = h_{Ndes} / h_{Nini}$

Step 4—Calculate $G_{mb,\ estimated}$ @ N_{ini}

$G_{mb,\ est}\ @\ N_{ini} = C \times G_{mb}\ @\ N_{des}$

Step 5—Determine the average G_{mb} @ N_{ini} value for each trial binder content.

Step 6—Calculate %G_{mm} values as follows:

$$\%G_{mm} = \frac{G_{mb}\ @\ N_{ini}}{G_{mm}\ (at\ the\ trial\ P_h)} \times 100$$

or

$$\%G_{mm} = \frac{G_{mb}\ @\ N_{des}}{G_{mm}\ (at\ the\ trial\ P_h)} \times 100$$

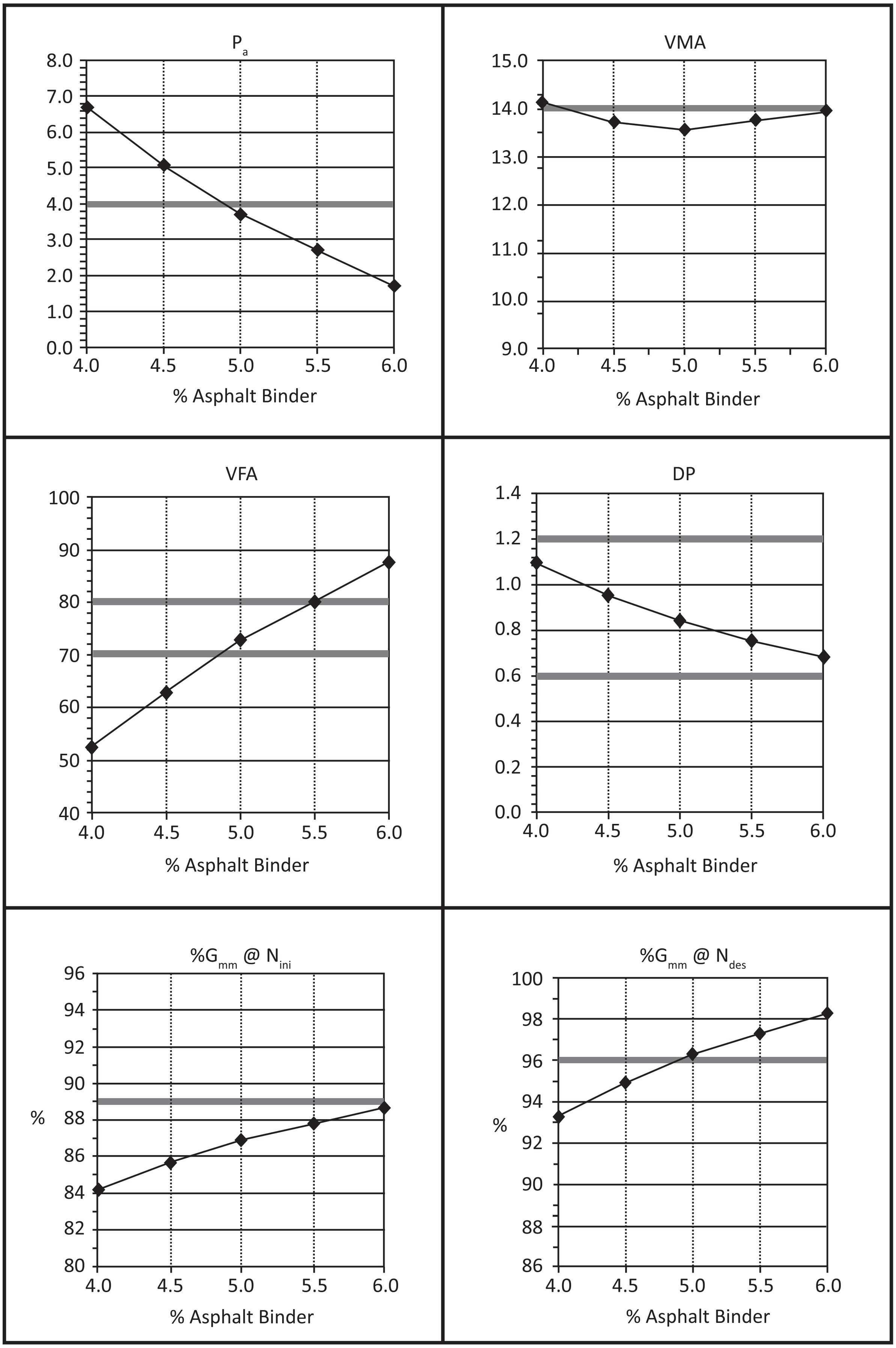
or

$$\%G_{mm} = \frac{G_{mb}\ @\ N_{max}}{G_{mm}\ (at\ the\ trial\ P_b)} \times 100$$

6.4.2 Superpave volumetric analysis

The remaining volumetric properties for P_a, VMA, VFA, P_{be} and DP are calculated as shown in chapter 5, sections 5.4 through 5.10. The results are plotted to provide curves of asphalt content versus %G_{mm} @ N_{ini}, % G_{mm} @ N_{des}, air voids (P_a), voids in the mineral aggregate (VMA), voids filled with asphalt (VFA) and dust proportion (DP). Typical curves contained in the Asphalt Institute Mix Design Program SW-2 are shown in **Figure 6.5**. This example is for a 12.5-mm mix and its appropriate criteria from **Table 6.5**. Note the minimum VMA requirement of 14 percent is not met.

FIGURE 6.5 ***Typical Superpave Mix Design Curves Form (Example for a 12.5-mm mix)***



6.5

Design asphalt binder content

After the data have been plotted, the designer will pick an asphalt binder content that will provide 4.0 percent air voids at N_{des}, which is equal to 96.0 percent $G_{mm}@N_{des}$. The designer must then verify the mixture conforms to the specifications by determining the values for percent $G_{mm}@N_{ini}$, voids in the mineral aggregate (VMA), voids filled with asphalt (VFA) and dust proportion (DP) from the other curves. The recommended Superpave volumetric mixture design requirements are shown in **Table 6.5**

6.6

N_{max} determination

The N_{max} parameter must be determined and verified that the $G_{mm}@N_{max}$ does not exceed 98 percent. Early versions of the Superpave mix design system required designers to compact all of the mixture specimens discussed above to N_{max}. The values for N_{ini} and N_{des} are then both back-calculated as was discussed above for N_{ini}. Empirical evidence from experienced mix designers has determined that the %G_{mm} at N_{max} rarely fails at the design binder content when the Superpave aggregate and mixture criteria are met. Designers prefer to actually measure the G_{mb} at N_{des} and verify the N_{max} parameter only at the design binder content, at the time of moisture sensitivity testing, which cannot be conducted until the design binder content is known.

Prepare four aggregate samples that will be mixed at the design binder content determined during the mix design. Two samples will be compacted to N_{max} and two loose samples will be measured for G_{mm}. The average of these two sets of data will be used to directly calculate the %$G_{mm}@N_{max}$. Verify that the result is less than 98 percent of the G_{mm} at the design binder content.

If the designer has a concern that the N_{max} may fail, N_{max} samples may be compacted during the mix design in lieu of N_{des} samples. The percent $G_{mm,est}@N_{des}$ would then need to be calculated just as the percent $G_{mm,est}@N_{ini,}$ values were calculated in steps 3 and 4 shown above.

6.7

Moisture sensitivity testing

The next step in the mixture design process is to conduct moisture sensitivity testing. The mixture must meet all of the aggregate and volumetric criteria prior to proceeding. All samples prepared for performance testing will be mixed at the design binder content determined in this chapter. The Superpave system requires that the designed mixture meet AASHTO T 283 moisture sensitivity requirements as covered in chapter 9.

6.8

Performance testing

Although not a formal requirement of the Superpave design system, the Asphalt Institute recommends that strong consideration be given to conducting some type of additional mixture performance testing, especially on critical, high-volume projects. Additional performance testing such as Hamburg, Asphalt Pavement Analyzer and other procedures are discussed in chapter 10.

CHAPTER 7

Marshall Method of Mix Design

7.1

General

The Marshall method of mix design is for dense-graded HMA mixes. It is used almost everywhere in the world. It is the predominant mix design method for airport pavements.

For a single selected aggregate gradation, five different asphalt contents are tested for various volumetric and strength criteria to select the optimum binder content. The test results should always be reported as the average for three compacted, "identical" specimens. The selection of the optimum binder content requires engineering judgment, depending on traffic, climate and experience with the local materials used. In most cases, the optimum binder content should be selected for which the compacted specimens have 4 percent air voids.

7.1.1 Applicable standards

The Marshall test procedures have been standardized by the American Society for Testing and Materials (ASTM) and by the American Association of State Highway and Transportation Officials (AASHTO). Procedures are given by:

- ASTM D6926, "Preparation of Bituminous Mixtures Using Marshall Apparatus";
- ASTM D6927, "Standard Test Method for Marshall Stability and Flow of Bituminous Mixtures"; and
- AASHTO T 245, "Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus."

AASHTO T 245 agrees with ASTM D6926 except for provisions for a mechanically operated hammer. AASHTO T 245, paragraph 2.3, note 2, says "Instead of a hand-operated hammer, and associated equipment described in sections 2.3, 2.4, and 2.5, a mechanically operated hammer may be used provided it has been calibrated to give results comparable with the hand-operated hammer."

The original Marshall method is applicable only to hot mix asphalt paving mixtures containing aggregates with maximum sizes of 25 mm (1 in.) or less. A modified Marshall method has been developed for aggregates with maximum sizes up to 38 mm (1.5 in.). Procedures for 6-inch-diameter specimen are given by ASTM D5581. The differences between the modified method for 6-inch-diameter specimen and the original are discussed in section 7.5. The Marshall method is intended for laboratory design and field control (chapter 13) of asphalt hot mix dense-graded paving mixtures. Because the Marshall stability test is empirical in nature, the meaning of the results in terms of estimating relative field behavior is lost when any modification is made to the standard procedures. An example of such modification is preparing specimens from reheated or remolded materials. If reheating cannot be avoided, a correlation should be made to adjust the compactive effort on the reheated mix to match the volumetric properties (such as VMA and % air voids) of the compacted mix which was not reheated.

7.1.2 Outline of method

The procedure for the Marshall method starts with the preparation of test specimens. Steps preliminary to specimen preparation are:

- all materials proposed for use meet the physical requirements of the project specifications;
- aggregate blend combinations meet the gradation requirements of the project specifications; and
- for the purpose of performing density and voids analyses, the bulk specific gravity of all aggregates used in the blend and the specific gravity of the asphalt cement are determined.

These requirements are matters of routine testing, specifications and laboratory technique that must be considered for any mix design method. Refer to chapters 3, 4 and 5 for the preparation and analysis of aggregates and compacted specimens.

The Marshall method uses standard test specimens of 63.5-mm (2 ½-in.) height by a 101.6-mm (4-in.) diameter. These are prepared using a specified procedure for heating, mixing, and compacting the asphalt-aggregate mixture. The two principle features of the Marshall method of mix design are a density-voids analysis and a stability-flow test of the compacted test specimens.

The stability of the test specimen is the maximum load resistance in Newtons (lb.) that the standard test specimen will develop at 60°C (140°F) when tested as outlined. The flow value is the total deformation, in units of 0.25 mm (1⁄100 in.), occurring in the specimen between no load and the point of maximum load during the stability test.

7.2

Preparation of test specimens

7.2.1 General

In determining the design asphalt content for a particular blend or gradation of aggregates by the Marshall method, a series of test specimens is prepared for a range of different asphalt contents so that the test data curves show well-defined relationships. Tests should be planned on the basis of 0.5 percent increments of asphalt content, with at least two asphalt contents above the expected design value and at least two below this value.

The "expected design" asphalt content can be based on any or all of these sources: experience, computational formula, or performing the centrifuge kerosene equivalency and oil soak tests in the Hveem procedure (see section 8.3). The expected design asphalt content, in percent by total weight of mix, could then be estimated to be approximately equivalent to the percentage of aggregate in the final gradation passing the 75-µm (No. 200) sieve.

One example of a computational formula is this equation:

$$P = 0.035a + 0.045b + Kc + F$$

Where:

P = approximate asphalt content of mix, percent by weight of mix
a = percent of mineral aggregate retained on 2.36-mm (No. 8) sieve
b = percent of mineral aggregate passing the 2.36-mm (No. 8) sieve and retained on the 75-µm (No. 200) sieve
c = percent of mineral aggregate passing 75-µm (No. 200) sieve
K = 0.15 for 11–15 percent passing 75-µm (No. 200) sieve
0.18 for 6–10 percent passing 75-µm (No. 200) sieve
0.20 for 5 percent or less passing 75-µm (No. 200) sieve
F = 0 – 2.0 percent. Based on the absorption of light or heavy aggregate, in the absence of other data, a value of 0.7 is suggested.

To provide adequate data, at least three test specimens are prepared for each asphalt content selected. Therefore, a Marshall mix design using five different asphalt contents will normally require at least 15 test specimens. Each test specimen will usually require approximately 1.2 kg (2.7 lb.) of aggregate. Assuming some minor waste, the minimum aggregate requirements for one series of test specimens of a given blend and gradation will be approximately 23 kg (50 lb.). About 4 liters (1 gal.) of asphalt cement will be adequate.

7.2.2 Equipment

The equipment (calibrated as needed) required for the preparation of test specimens is:

- flat-bottom metal pans for heating aggregates;
- round metal pans or a mixing bowl, approximately 4-liter (4-qt.) capacity, for mixing asphalt and aggregate;

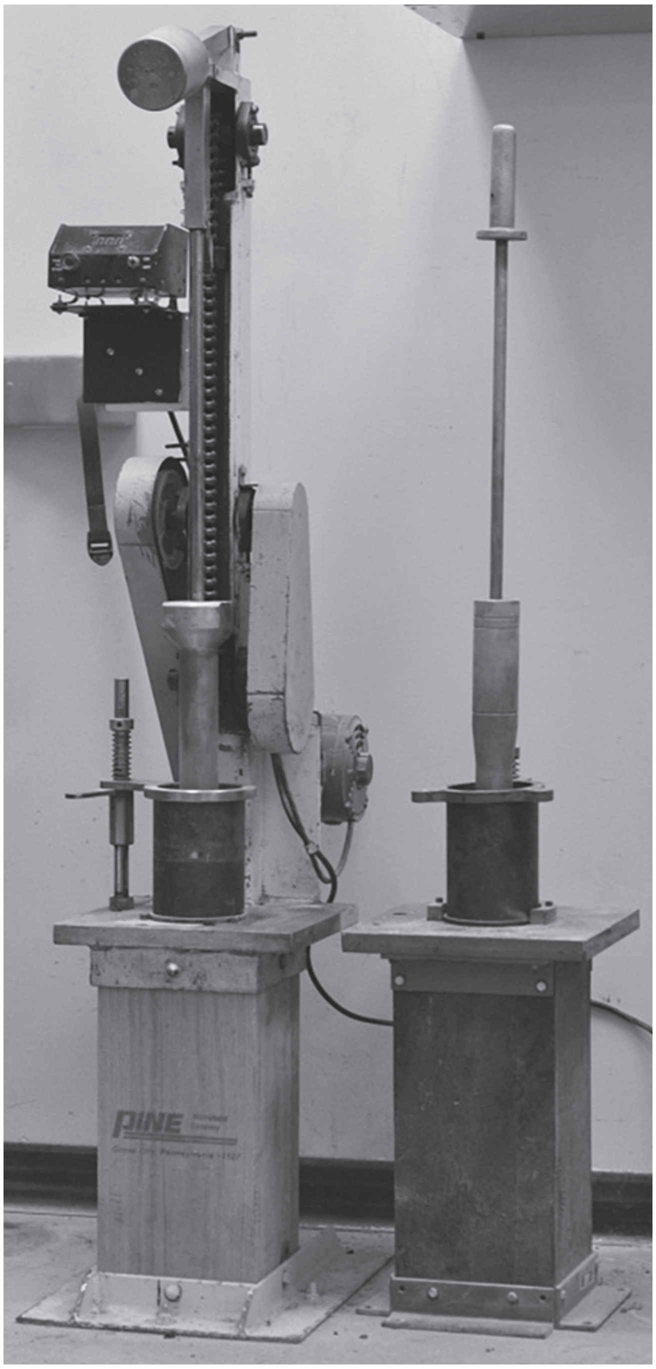


FIGURE 7.1 ***Manual and Mechanical Hammer Configurations***

- oven and hot plate, preferably thermostatically controlled, for heating aggregates, asphalt and equipment;
- scoop for batching aggregates;
- containers: gill-type tins, beakers, pouring pots and sauce pans for heating asphalt;
- thermometers or other thermometric devices: armored, glass or dial-type with metal stem, 10°C (50°F) to 235°C (450°F), for determining temperature of aggregates, asphalt and asphalt mixtures;
- balances: 5-kg capacity, sensitive to 1 g, for weighing aggregates and asphalt, and 2-kg capacity, sensitive to 0.1 g, for weighing compacted specimens;
- large mixing spoon or small trowel;
- large spatula;
- mechanical mixer (optional): commercial bread dough mixer 4-liter (4-qt.) capacity or larger, equipped with two metal mixing bowls and two wire stirrers, or an equivalent type mixer (see section 4.2.2);
- compaction pedestal (see **Figure 7.1**), consists of a 200 × 200 × 460 mm (8 × 8 × 18 in.) wooden post capped with a 305 × 305 × 25 mm (12 × 12 × 1 in.) steel plate. The wooden post should be oak, pine or other wood having a dry weight of 670 to 770 kg/m^3 (42 to 48 pcf). The wooden post should be secured by four angle brackets to a solid concrete slab. The steel cap should be firmly fastened to the post. The pedestal should be installed so that the post is plumb, the cap level, and the entire assembly free from movement during compaction. Compaction hammers can be either manually or mechanically operated, as shown in **Figure 7.1.** Mechanically operated hammers drop the hammer at a rate of 64 ± 4 blows per minute. Mechanical hammers can also have single or multiple hammer and mold sets for compacting single or multiple specimens at a time. Some mechanically operated hammers are designed with a rotating base mechanism which rotates at 18 to 30 revolutions per minute;
- compaction mold, consisting of a base plate, forming mold and collar extension. The forming mold has an inside diameter of 101.6 mm (4 in.) and a height of approximately 75 mm (3 in.); the base plate and collar extension are designed to be interchangeable with either end of the forming mold. ASTM D5581 references a 154.2-mm (6-in.) mold with a 114.3-mm (4.5-in.) height for use with aggregate up to 1.5-inch NMAS and should conform to requirements of ASTM D6926;
- compaction hammer, consisting of a flat, circular tamping face, 98.4 mm (3 ⅞ in.) in diameter and equipped with a 4.5-kg (10-lb.)

weight constructed to obtain a specified 457-mm (18-in.) height of drop, should conform to requirements of ASTM D6926;
- mold holder, consisting of spring-tension device designed to hold compaction mold centered in place on the compaction pedestal, should conform to requirements of ASTM D6926;
- paper disks, 100 mm (4 in.);
- steel specimen extractor, in the form of a jack and a disk with a diameter not less than 100 mm (3.95 in.) and 13 mm (0.5 in.) thick for extruding compacted specimen form mold. The steel specimen extractor for the 6-inch mold is 151.5 to 152.5 mm (5.950 to 5.990 in.) in diameter and 13 mm (0.5 in.) thick;
- welders' gloves, for handling hot equipment. Rubber gloves for removing specimens from water bath; and
- marking crayons, for identifying test specimens.

NOTE: *See additional equipment requirements in section 7.3.2.*

7.2.3 Preparation of test specimens

These steps are recommended for preparing Marshall test specimens.

(a) *Number of specimens*—prepare at least three specimens for each combination of aggregates and binder content.

(b) *Preparation of aggregates*—dry aggregates to constant weight at 105°C to 110°C (220°F to 230°F) and separate the aggregates by dry sieving into the desired size fractions. These size fractions are recommended:

- 25.0 to 19.0 mm (1 to ¾ in.)
- 19.0 to 9.5 mm (¾ to ⅜ in.)
- 9.5 to 4.75 mm (⅜ in. to No. 4)
- 4.75 to 2.36 mm (No. 4 to No. 8)
- Passing 2.36 mm (No. 8)

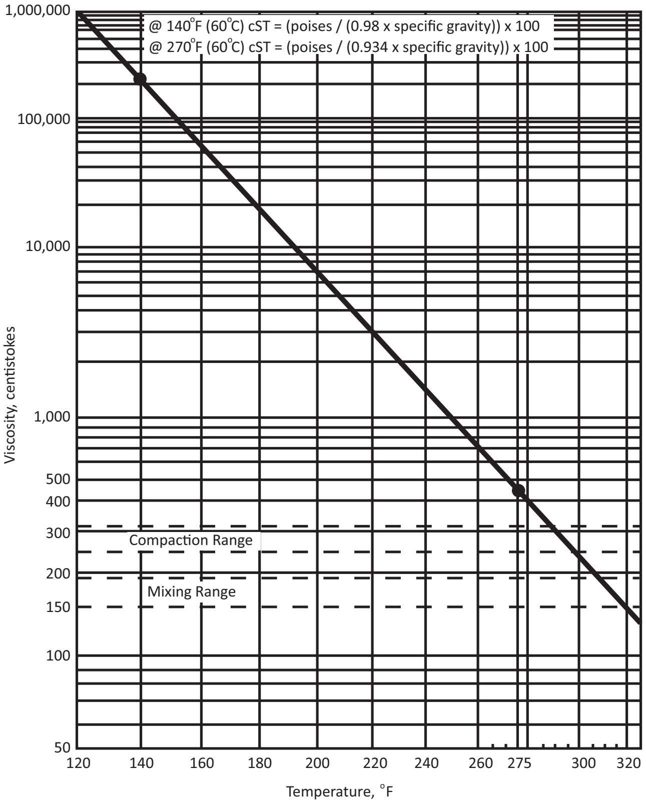
Refer to chapter 3, section 3.3, for batching procedure details.

(c) *Determination of mixing and compaction temperature*—the temperature to which the asphalt must be heated to produce viscosities of 170 ± 20 centistokes kinematic and 280 ± 30 centistokes kinematic shall be established as the mixing temperature and compaction temperatures, respectively. These temperatures can be estimated from a plot of the viscosity (log-log centistokes scale) versus temperature (log degrees Rankine scale, °R = °F + 459.7) relationship for the asphalt cement to be used. An example plot is shown in **Figure 7.2** (see section 4.2.1. as well as MS-26 pg. 205). For mixing and compaction temperatures for modified binders, refer to section 4.2.1.

(d) *Preparation of mold and hammer*—thoroughly clean the specimen mold assembly and the face of the compaction hammer and heat them in a water bath or on the hot plate to a temperature between 95 and 150°C (200 and 300°F).

(e) *Preparation of mixtures*—weigh into separate pans for each test specimen the amount of each aggregate size fraction required to produce the required gradation and a batch that will result in a compacted specimen 63.5 ± 1.27 mm (2.5 ± 0.05 in.) in height. This will normally be about 1.2 kg (2.7 lb.). See chapter 3 for

FIGURE 7.2 ***Determination of Mixing and Compaction Temperatures***



suggested methods of calculating batch weights. It is generally desirable to prepare a trial specimen prior to preparing the aggregate batches. If the trial specimen height falls outside the height limits, the amount of aggregate used for the specimen should be adjusted using:

for International System of Units (SI),

$$\text{Adjusted mass of aggregate} = \frac{63.5 \times (\text{mass of aggregate used})}{\text{Specimen height (mm) obtained}}$$

for U.S. Customary Units,

$$\text{Adjusted mass of aggregate} = \frac{2.5 \times (\text{mass of aggregate used})}{\text{Specimen height (in.) obtained}}$$

Place the pans in the oven or on the hot plate and heat to a temperature not exceeding 28°C (50°F) above the mixing temperature specified in (c). (If a hot plate is used, provision should be made for dead space, baffle plate or a sand bath beneath the pans and the hot plate to prevent local overheating.) Charge the mixing bowl with the heated aggregates and dry mix thoroughly. Form a crater in the dry blended aggregate and weigh the required amount of asphalt cement into the mixture in accordance with the calculated batch weights. At this point the temperature of the aggregate and the asphalt must be within the limits of the mixing temperatures established in paragraph (c). Asphalt cement should not be held at mixing temperatures for more than one hour before using. Mix the aggregate and asphalt cement, preferably with a mechanical mixer or by hand with a trowel, as quickly and thoroughly as possible to yield a mixture having a uniform distribution of asphalt. (Details on mixing and buttering the mixing bowl are discussed in section 4.2.2.)

NOTE: *Current ASTM and AASHTO procedures do not require any aging or curing of the mixture prior to Marshall compaction. The Asphalt Institute recommends that Marshall mixes be conditioned according to AASHTO R 30. Reference is made to this discussion in section 4.2.3.2.*

(f) *Packing the mold*—place a filter or non-absorbent paper disk cut to size in the bottom of the mold. Place the entire batch in the mold with collar, and then spade the mixture vigorously with a heated spatula or trowel 15 times around the perimeter and 10 times over the interior. Smooth the surface to a slightly rounded shape. The temperature of the mixture immediately prior to compaction shall be within the limits of the compaction temperature established in paragraph (c); otherwise, it shall be discarded. In no case shall the mixture be reheated.

(g) *Compaction of specimens*—place a paper disk on top of the mix and place the mold assembly on the compaction pedestal in the mold holder. As specified according to the design traffic category (see **Table 7.2**), apply 35, 50 or 75 blows with the compaction hammer using a free fall of 457 mm (18 in.). Hold the axis of the compaction hammer as nearly perpendicular to the base of the mold assembly as possible during compaction. Remove the base plate and collar, and reverse and reassemble the mold. Apply the same number of compaction blows to the face of the reversed specimen. After compaction, remove the base plate and the paper disks and allow the specimen to cool at room temperature until no deformation will result when removing it from the mold. When more rapid cooling is desired, electric fans may be used, but not water unless the specimen is in a plastic bag. Remove the specimen from the mold by means of an extrusion jack or other compression device, and then place on a smooth, level surface until ready for testing. Normally, specimens are allowed to cool overnight.

7.2.4 Correlation of the manually and mechanically operated hammers

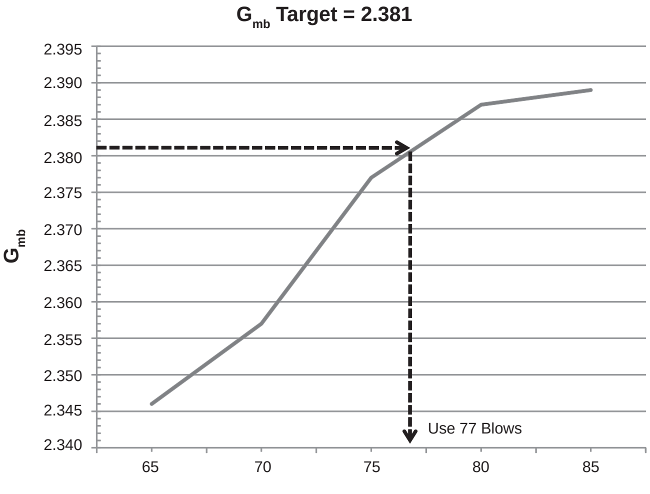
There is a tendency for manually operated hammers to yield a higher specimen density than mechanically operated hammers. It has been theorized that a slight kneading effect takes place when the manually operated hammer is dropped at a slight angle from the true-vertical action of the automated hammers. If variations of the standard manual Marshall hammer (e.g., mechanical lift, slanted face and rotating base) are used, correlations with the standard Marshall compaction procedure must be made. This is equally the case if mix samples are reheated before compaction. See section 7.1.1.

The Asphalt Institute recommends using only mechanical hammers for mix design and field verification.

The Asphalt Institute recommends that the asphalt pavement owner establish a governing compactor which establishes the job mix formula parameters and resulting construction tolerances. The Asphalt Institute further recommends that all laboratory compactors utilized in the quality control, acceptance or assurance of field constructed pavements be correlated to the governing compactor utilized in the mixture design approved by the pavement owner.

An example of such a correlation is adjusting the number of blows to result in the same volumetric properties. Using the same mix and compaction temperature as in the mix design, compact triplicate specimens at five different blow counts. As a minimum, specimens shall be compacted at the mix design number of blows, ± 5 blows and ± 10 blows. Draw a "number of blows vs. G_{mb} curve" and determine the number of blows required to get the same G_{mb} obtained by the governing compactor used in the mix design. This new correlated blow count shall be used for all further testing when utilizing this correlated compactor. The range of blow counts must be large enough to include the results of the governing compactor without extrapolation of the correlation curve. An example of a correlation curve is shown below in **Figure 7.3**.

FIGURE 7.3 ***Compactor correlation curve***



7.3

Test procedure

7.3.1 General

In the Marshall method, each compacted test specimen is subjected to these tests and analysis in the order listed:

- specimen height determination;
- bulk specific gravity determination;
- density and voids analysis; and
- stability and flow test.

7.3.2 Bulk specific gravity determination

The bulk specific gravity test may be performed as soon as the freshly compacted specimens have cooled to room temperature. This test is performed according to ASTM D1188, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens" or ASTM D2726, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens." (See chapter 4.)

7.3.3 Marshall testing

7.3.3.1 Equipment for stability and flow tests

The equipment required for the testing of the 101.6 -mm (4-in.) diameter by 63.5-mm (2-½ in.) height specimens is as follows:

- The Marshall testing machine is a compression testing device conforming to ASTM D6927. It is designed to apply loads to test specimens through cylindrical segment testing heads [inside radius of curvature of 51 mm (2 in.)] at a constant loading rate of 51 mm (2 in.) per minute. Two perpendicular guide posts are included to allow the two segments to maintain horizontal positioning and free vertical movement during the test. It is equipped with a calibrated proving ring for determining the applied testing load, a Marshall stability testing head for use in testing the specimen, and a Marshall flow meter (or automatic recording device) for determining the amount of deformation at the maximum load in the test. A universal testing machine equipped with suitable load and deformation indicating devices may be used instead of the Marshall testing frame.

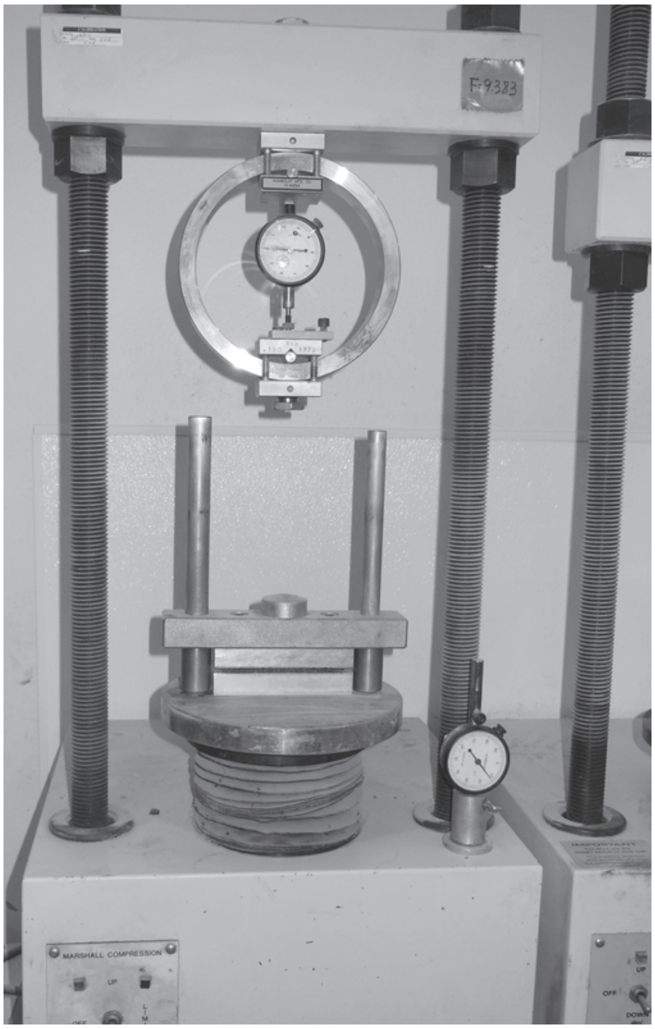
- The water bath must be at least 150 mm (6 in.) deep and thermostatically controlled to 60°C ± 1°C (140°F ± 1.8°F). The tank should have a perforated false bottom or be equipped with a shelf for suspending specimens at least 50 mm (2 in.) above the bottom of the bath.

7.3.3.2 *Stability and flow test procedures*

After the bulk specific gravity of the test specimens has been determined, the following stability and flow tests are performed:

(a) determination of specimen height;
(b) immerse specimen in water bath at 60°C ± 1°C (140°F ± 1.8°F) for 30 to 40 minutes before testing, or in an oven at the same temperature for 120 to 130 minutes; and
(c) use an automatic recording device, or use a proving ring and flow meter (as shown in **Figure 7.4**). Place the flow meter over the marked guide rod and "zero" the flow meter while holding it firmly against the upper segment of the testing head while the load is being applied.

FIGURE 7.4 ***Marshall Proving Ring and Flow Meter***



NOTE: *The same assembly of the testing head and flow meter must be used in testing all specimens. Specimens should be 101.6 ± 0.25 mm (4.00 ± 0.01 in.). Otherwise, an initial and final reading of the flow meter is required for the determination of the flow value.*

(d) Thoroughly clean the inside surfaces of the testing heads. Temperature of heads shall be maintained between 21.1 and 37.8°C (70 and 100°F) using a water bath, when required. Lubricate guide rods with a thin film of oil so that the upper test head will slide freely without binding. If a proving ring is used to measure applied load, check to see that the dial indicator is firmly fixed and "zeroed" for the "no-load" position.
(e) With the testing apparatus ready, remove the test specimen from water bath and carefully dry surface with a towel. Place specimen in lower testing head and center; then fit upper testing head into position and center complete assembly in loading device. Place flow meter over marked guide rod as noted in (c) above.
(f) Apply testing load to specimen at a constant rate of deformation, 51 mm (2 in.) per minute, until failure occurs. The point of failure is defined when the maximum load reading is obtained. The total force in Newtons (N) required to produce failure of the specimen shall be recorded as its Marshall stability value.
(g) While the stability test is in progress (if not using an automatic recording device), hold the flow meter firmly in position over the guide rod and remove immediately when the load begins to decrease, take reading and record. This reading is the flow value for the specimen, expressed in units of 0.25 mm (1⁄100 in.). For example, if the specimen deformed 3.8 mm (0.15 in.), the flow value is 15.
(h) The entire procedure for both the stability and flow measurements, starting with the removal of the specimen from the water bath, shall be completed within a period of 30 seconds.
(i) The Marshall stability is corrected for specimens with a height different than 63.5 mm (see **Table 7.1**).

7.3.3.3 Marshall stability and flow considerations

Marshall stability is the peak resistance load obtained during a constant rate of deformation.

Marshall flow is a measure of the deformation (elastic plus plastic) of the specimen determined during the stability test. The determination of the Marshall flow with an automatic recording device will typically produce a plot comparable to the stylized **Figure 7.5**.

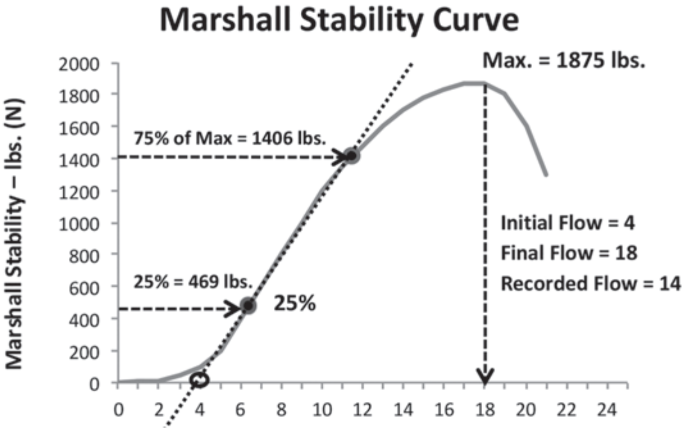
The bottom portion of the Marshall stability versus Marshall flow curve shows the effects of irregularities on the specimen surface until full contact (seating) of the testing heads and the specimen surface is achieved. Therefore, when using an automatic recording device, the recorded Marshall flow must be corrected by subtracting the flow portion during "seating" of the specimen (as shown in **Figure 7.5**). To determine the correct start of the flow reading, a tangent line shall be drawn connecting two points on the stability–flow curve, representing 25 percent and 75 percent of Marshall stability. Where this tangent line intersects the *x*-axis is the start of Marshall flow.

No correction is necessary when using a proving ring and flow meter, since the flow meter has been "zeroed out" on a calibrated 4.00-inch metal disk or specimen.

If the flow at the selected ~~optimum~~ binder content is above the upper specified limit, the mix is considered too plastic or unstable. If the flow is below the lower specified limit, the mix is considered too brittle.

The stability and flow results are highly dependent on binder grade, binder quantity and aggregate structure. The stability and flow criteria given in **Table 7.2** were determined during an era that pre-dates binder modifiers (such as with polymers and/or tire rubber) and requires careful evaluation when determining the acceptability of mixture properties.

FIGURE 7.5 ***Determining Marshall Flow***



Mixes with modified binders may exhibit significant deformation before the resistance to loading peaks (Marshall stability), artificially elevating flow values. Therefore, the upper limit of the flow criteria should be waived when polymer or rubber-modified binders are used. When concerns arise over elevated flow values, consideration should be given to conducting alternative mixture performance testing as discussed in chapter 10.

7.3.4 Density and voids analysis

After the completion of the stability and flow test, a density and voids analysis is made for each series of test specimens. (The calculations for the voids analysis are fully described in chapter 5.)

(a) Average the bulk specific gravity values for all test specimens of a given asphalt content; values obviously in error shall not be included in the average. The average value of bulk specific gravity for each binder content shall be used in further computations of voids data.

(b) Determine the average unit weight for each asphalt content by multiplying the average bulk specific gravity value by the density of water (1,000 kg/m^3 [62.4 pcf]). (See **Figure 7.7**.)

(c) Determine the theoretical maximum specific gravity (G_{mm} by ASTM D2041) for at least two asphalt contents, preferably on mixes at or near the design asphalt content. An average value for the effective specific gravity of the total aggregate is then calculated from these values. This value may then be used for calculation of the maximum specific gravity of mixtures with different asphalt contents, as discussed in section 5.4.2.

(d) Using the effective (G_{se}) and bulk specific gravity (G_{sb}) of the total aggregate, the average bulk specific gravities of the compacted mix (G_{mb}), the specific gravity of the asphalt (G_b), and the maximum specific gravity of the mix (G_{mm}) determined above in (c), calculate the percent absorbed asphalt (P_{ba}) by weight of dry aggregate, percent air voids (P_a), percent voids filled with asphalt (VFA) and percent voids in the mineral aggregate (VMA). These values and calculations are more fully described in chapter 5.

7.4

Interpretation of test data

7.4.1 Preparation of test data

Prepare the stability and flow values and void data.

(a) Measured stability values for specimens that depart from the standard 63.5 mm (2 ½ in.) thickness shall be converted to an equivalent 63.5 mm (2 ½ in.) value by means of a conversion factor. Applicable correlation ratios to convert the measured stability values are set forth in **Table 7.1**. Note that the conversion may be made on the basis of either measured thickness or measured volume.

(b) Average the flow values and the final converted stability values for all specimens of given asphalt content. Values that are obviously in error shall not be included in the average.

(c) Prepare separate graphical plots for the following values and connect plotted points with a smooth curve that obtains the "best fit" for all values, as illustrated in **Figure 7.6**:

- percent air voids (P_a) versus asphalt content;
- percent voids in mineral aggregate (VMA) versus asphalt content;
- percent voids filled with asphalt (VFA) versus asphalt content;
- unit weight of total mix versus asphalt content;
- stability versus asphalt content; and
- flow versus asphalt content.

(d) Determine the optimum binder content and properties of the mix by using these graphs, as further discussed in section 7.4.5.

7.4.2 Trends and relations of test data

By examining the test property curves plotted for section 7.4.1 (c) on **Figure 7.6**, information can be learned about the sensitivity of the mixture to asphalt content. The test property curves have been found to follow a reasonably consistent pattern for dense-graded asphalt paving mixes, **but variations do and will occur**. Trends generally noted are:

- The stability value increases with increasing asphalt content up to a maximum, after which the stability decreases.
- The flow value consistently increases with increasing asphalt content.
- The curve for unit weight of total mix follows the trend similar to the stability curve, except that the maximum unit weight normally (but not always) occurs at slightly higher asphalt content than the maximum stability.
- The percent of air voids, P_a, steadily decreases with increasing asphalt content, ultimately approaching a minimum void content.
- The percent voids in the mineral aggregate, VMA, generally decreases (because of better compaction) to a minimum value, and then increases with increasing asphalt content, because the aggregate is starting to be pushed apart by excessive binder in the mix.
- The percent voids filled with asphalt, VFA, steadily increases with increasing asphalt content because the VMA is being filled with asphalt.

7.4.3 Criteria for a satisfactory paving mix

Deciding whether the asphalt paving mix will be satisfactory at the selected design asphalt content is guided by applying certain limiting criteria to the mixture test data. The Marshall method mix design criteria in **Table 7.2** are recommended by the Asphalt Institute.

7.4.4 Determination of preliminary design asphalt content

The optimum binder content of the asphalt paving mix is selected by considering **all of the data** discussed previously. In general, the Asphalt Institute recommends choosing the asphalt content at the median of the percent air voids limits, which is 4 percent. If all of the calculated and measured mix properties at this asphalt content meet the mix design criteria in **Table 7.2**, then this is the optimum binder content for the mix design.

However, if not all of the design criteria are met, then some adjustment or compromise is necessary or the mix may need to be redesigned (see Footnote 7 of **Table 7.2**). A number of considerations are discussed in section 7.4.5 that

Volume of Specimen, cm^3	Approximate Thickness of Specimen		Correlation Ratio
	mm ______	in. ______	
200 to 213	25.4	1	5.56
214 to 225	27.0	1 1⁄16	5.00
226 to 237	28.6	1⅛	4.55
238 to 250	30.2	1 3⁄16	4.17
251 to 264	31.8	1¼	3.85
265 to 276	33.3	1 5⁄16	3.57
277 to 289	34.9	1⅜	3.33
290 to 301	36.5	1 7⁄16	3.03
302 to 316	38.1	1½	2.78
317 to 328	39.7	1 9⁄16	2.50
329 to 340	41.3	1⅝	2.27
341 to 353	42.9	1 11⁄16	2.08
354 to 367	44.4	1¾	1.92
368 to 379	46.0	1 13⁄16	1.79
380 to 392	47.6	1⅞	1.67
393 to 405	49.2	1 15⁄16	1.56
406 to 420	50.8	2	1.47
421 to 431	52.4	2 1⁄16	1.39
432 to 443	54.0	2⅛	1.32
444 to 456	55.6	2 3⁄16	1.25
457 to 470	57.2	2¼	1.19
471 to 482	58.7	2 5⁄16	1.14
483 to 495	60.3	2⅜	1.09
496 to 508	61.9	2⅞	1.04
509 to 522	63.5	2½	1.00
523 to 535	65.1	2 9⁄16	0.96
536 to 546	66.7	2⅝	0.93
547 to 559	68.3	2 11⁄16	0.89
560 to 573	69.8	2¾	0.86
574 to 585	71.4	2 13⁄16	0.83
586 to 598	73.0	2⅞	0.81
599 to 610	74.6	2 15⁄16	0.78
611 to 625	76.2	3	0.76

NOTES:

1. The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 63.5-mm (2.5-in.) specimen.
2. Volume-thickness relationship is based on a specimen diameter of 101.6 mm (4 in.).

TABLE 7.1 ***Stability Correlation Ratios*** should be weighed even if all the design criteria are met.

EXAMPLE

Assume the data shown in **Figures 7.6** and **7.7** represent Marshall mix design laboratory tests on dense-graded HMA to be used in a heavy traffic area. The mixture contains a ¾-inch nominal maximum particle size aggregate. At 4 percent air voids, the mixture properties are:

• asphalt content, %	4.7
• stability, lb.	2,300
• flow, 0.01 in.	9
• VMA, %	13.9
• VFA, %	70.0

Comparing these values to the criteria in **Table 7.2**, it is evident that this mixture is acceptable for use in heavy traffic areas.

7.4.5 Selection of design binder content

The Asphalt Institute recommends that the final selected mix design should be one whose aggregate structure and binder content, compacted to the design number of blows, results in 4 percent air voids and satisfactorily meets all of the other established criteria in **Table 7.2**. Deviations from the recommended design criteria should be clearly specified in the project documents and must be appropriate for the intended use of the asphalt mixture. The mixture should contain as much asphalt binder as possible to maximize

TABLE 7.2 ***Marshall Mix Design Criteria***

Marshall Method Criteria[1]	Light Traffic[3] Surface & Base		Medium Traffic[3] Surface & Base		Heavy Traffic[3] Surface & Base	
	Min	Max	Min	Max	Min	Max
Compaction, number of blows each end of specimen	35		50		75	
Stability[2], N (lb.)	3336 (750)	–	5338 (1200)	–	8006 (1800)	–
Flow[2,4,5], 0.25 mm (0.01 in.)	8	18	8	16	8	14
Percent Air Voids[7]	3	5	3	5	3	5
Percent Voids in Mineral Aggregate (VMA)[6]	See Table 7.3					
Percent Voids Filled With Asphalt (VFA)	70	80	65	78	65	75

NOTES:

1. All criteria, not just stability value alone, must be considered in designing an asphalt paving mix.
2. Hot mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed 100 mm (4 inches) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.
3. Traffic classifications
    Light Traffic conditions resulting in a 20-year Design ESAL < 10^4
    Medium Traffic conditions resulting in a 20-year Design ESAL between 10^4 and 10^6
    Heavy Traffic conditions resulting in a 20-year Design ESAL > 10^6
4. The flow value refers to the point where the load begins to decrease. When an automatic recording device is used, the flow should be corrected as shown in section 7.3.3.3.
5. The flow criteria were established for neat asphalts. The flow criteria are often exceeded when polymer-modified or rubber-modified binders are used. Therefore, the upper limit of the flow criteria should be waived when polymer-modified or rubber-modified binders are used.
6. Percent voids in the mineral aggregate are to be calculated on the basis of the ASTM bulk specific gravity for the aggregate, as discussed in chapter 5.
7. Percent air voids should be targeted at 4 percent. This may be slightly adjusted if needed to meet the other Marshall criteria.

Nominal Maximum Particle Size[1,2]		Minimum VMA, percent		
		Design Air Voids, Percent[3]		
mm	in.	3.0	4.0	5.0
1.18	No. 16	21.5	22.5	23.5
2.36	No. 8	19.0	20.0	21.0
4.75	No. 4	16.0	17.0	18.0
9.5	⅜	14.0	15.0	16.0
12.5	½	13.0	14.0	15.0
19.0	¾	12.0	13.0	14.0
25.0	1.0	11.0	12.0	13.0
37.5	1.5	10.0	11.0	12.0
50	2.0	9.5	10.5	11.5
63	2.5	9.0	10.0	11.0

NOTES:

1. Standard Specification for Wire Cloth Sieves for Testing Purposes, ASTM E11 (AASHTO M 92)
2. The nominal maximum particle size is one size larger than the first sieve to retain more than 10 percent.
3. Interpolate minimum voids in the mineral aggregate (VMA) for design air void values between those listed.

TABLE 7.3 ***Minimum Percent Voids in Mineral Aggregate (VMA)***

durability, while also maintaining the stability required to support the intended loads for the life of the pavement.

If the Asphalt Institute recommended design criteria are modified, experienced designers are encouraged to carefully evaluate project variables such as traffic type and loading, location, materials and climatic region. Two common methods of modifying the mixture design parameters are changing the target air void content and/or changing the laboratory compaction level. For example, the FAA P-401 specifies a 3.5 percent design air void content to maximize durability, and it also specifies a compaction level of 75 blows for heavy aircraft along with stringent volumetric parameters to ensure the necessary stability.
A usage with much lighter loads, such as a bike path, may specify 3 percent air voids and reduce the compaction to only 35 blows for increased durability.

Approximate Height		Specimen Volume	Correlation Ratio
(mm)	(in.)	(cc)	
88.9	3½	1608 to 1626	1.12
90.5	3 9/16	1637 to 1665	1.09
92.1	3⅝	1666 to 1694	1.06
93.7	3 11/16	1695 to 1723	1.03
95.2	3¾	1724 to 1752	1.00
96.8	3 13/16	1753 to 1781	0.97
98.4	3⅞	1782 to 1810	0.95
100.0	3 15/16	1811 to 1839	0.92
101.6	4	1840 to 1868	0.90

TABLE 7.4 ***Stability Corrections for Large Stone Marshall Mixes in 6" Molds***

For a discussion on how to evaluate such parameters as VMA, compaction level, air voids and VFA, reference is made to section 5.11.

7.5

Modified Marshall method for large aggregate

A modified Marshall method for 6-inch diameter specimens has been standardized by ASTM D5581 for mixes composed of aggregates with maximum size up to 38 mm (1.5 in.). This procedure is documented in the 1990 “Proceedings of the Association of Asphalt Paving Technologists (AAPT).” The procedure is basically the same as the original Marshall mix design method except for these differences that are due to the larger specimen size:

- The hammer weighs 10.2 kg (22.5 lb.) and has a 149.4-mm (5.88-in.) flat tamping face. Only a mechanically operated device is used for the same 457-mm (18-in.) drop height.
- The specimen has a 152.4-mm (6-in.) diameter by 95.2-mm (3.75-in.) height.
- The batch weights are typically 4,050 g (8.9 lb.).
- The equipment for compacting and testing (molds and breaking heads) are proportionately larger to accommodate the larger specimens.

- The mix is placed in the mold in two approximately equal increments, with spading performed after each increment.
- The number of blows needed for the larger specimen is 1.5 times (75 or 112 blows) of that required for the smaller specimen (50 or 75 blows) to obtain equivalent compaction.
- The design criteria should be modified as well. The minimum stability should be 2.25 times, and the range of flow values should be 1.5 times the criteria listed in **Table 7.2**.
- Similar to the normal procedure as shown in **Table 7.1**, if the actual specimen thickness varies from 92.5 mm (3.75 in.), the correction values as listed in **Table 7.4** should be used to convert the measured stability values to an equivalent value for a specimen with a 92.5-mm (3.75-in.) thickness.

FIGURE 7.6 ***Test Property Curves for Hot Mix Design Data by the Marshall Method***

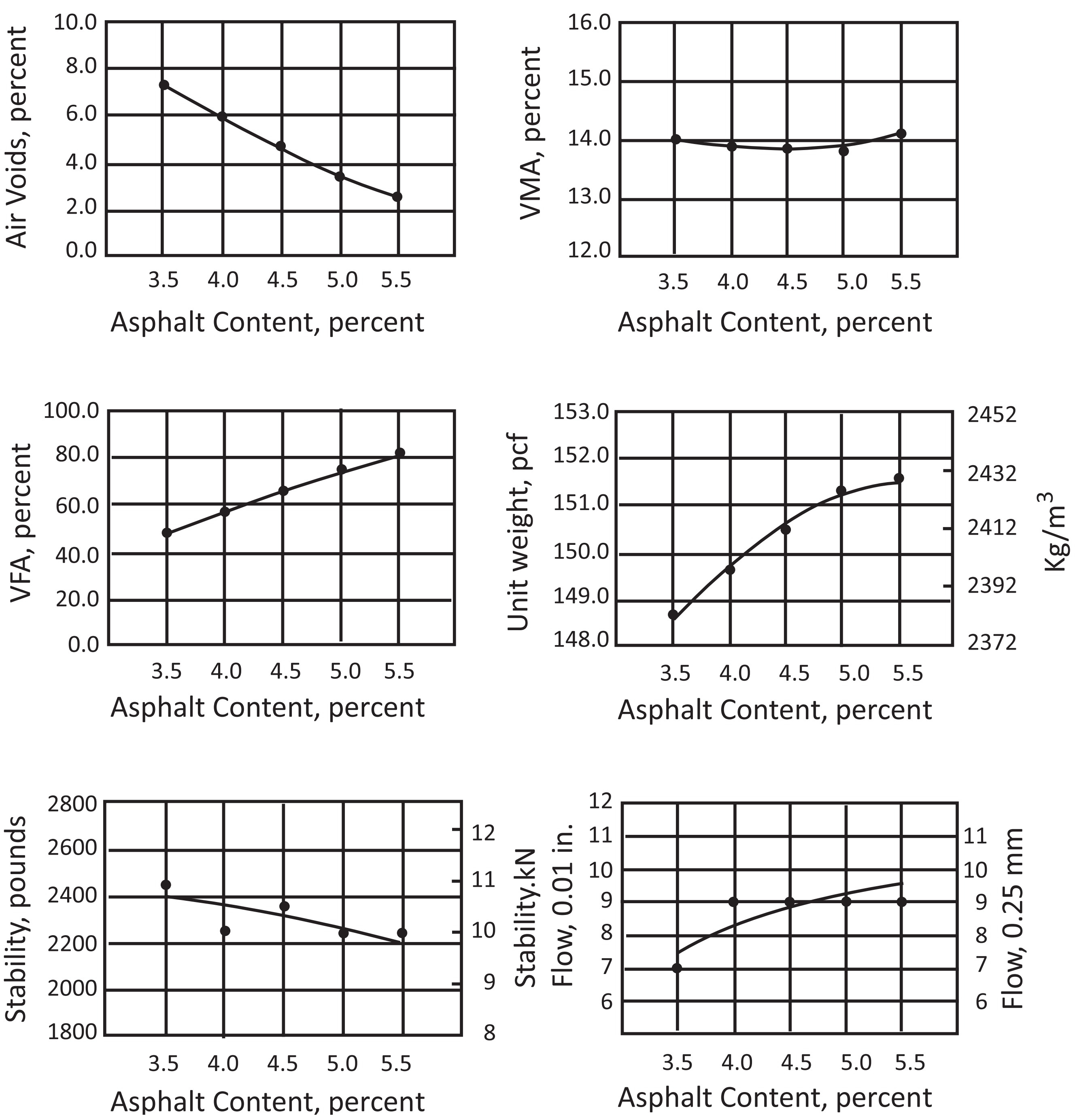


FIGURE 7.7 ***Test Report Showing Typical Lab Data for Design by the Marshall Method***

Compaction: 75 Blows
Specific Gravity of AC: 1.030
Bulk S.G. Aggregate: 2.674

Grade AC: AC-20
Absorbed AC of Aggregate: 0.6%
Effective S.G. Aggregate: 2.717

Project:
Location:
Date:

Trial
Mix:

		Mass, grams										**Stability, lbs. (N)**		
% AC by wt. of mix, Spec. No.	**Spec. Height in. (mm)**	**In Air**	**In Water**	**Sat. Surface Dry In Air**	**Bulk Volume, cc**	**Bulk S.G. Specimen**	**Max. S.G. (Loose Mix)**	**Unit Weight, pcf (Mg/m^3)**	**% Air Voids**	**%VMA**	**%VFA**	**Measured**	**Adjusted**	**Flow 0.01 in. (0.25 mm)**
3.5 – A		1240.6	726.4	1246.3	519.9	2.386		148.9				2440	2440	8
3.5 – B		1238.7	723.3	1242.6	519.3	2.385		148.8				2420	2420	7
3.5 – C		1240.1	724.1	1245.9	521.8	2.377		148.3				2510	2510	6
Average						2.383	2.570	148.7	7.3	14.0	48.0		2457	7
4.0 – A		1244.3	727.2	1246.6	519.4	2.396		149.5				2180	2180	7
4.0 – B		1244.6	727.0	1247.6	520.6	2.391		149.2				2260	2260	8
4.0 – C		1242.6	727.9	1244.0	516.1	2.408		150.2				2310	2310	8
Average						2.398	2.550	149.6	6.0	13.9	57.1		2250	7.7
4.5 – A		1249.3	735.8	1250.2	514.4	2.429		151.2				2420	2420	9
4.5 – B		1250.8	728.1	1251.6	523.5	2.389		149.1				2310	2314	9
4.5 – C		1251.6	735.3	1253.1	517.8	2.417		150.8				2340	2340	9
Average						2.412	2.531	150.5	4.7	13.9	66.1		2358	9
5.0 – A		1256.7	739.8	1257.6	517.8	2.427		151.4				2290	2290	10
5.0 – B		1258.7	742.7	1259.3	516.6	2.437		152.0				2190	2190	10
5.0 – C		1258.4	737.5	1259.1	521.6	2.418		150.5				2240	2240	9
Average						2.425	2.511	151.3	3.4	13.8	75.2		2240	9.7
5.5 – A		1263.8	742.6	1264.3	521.7	2.422		151.2				2210	2210	11
5.5 – B		1258.8	741.4	1259.4	518.0	2.430		151.6				2300	2300	10
5.5 – C			742.5	1259.5	517.0	2.435		152.0				2210	2240	10
Average						2.429	2.493	151.6	2.5	14.1	82.1		2240	10.3

CHAPTER 8

Hveem Method of Mix Design

8.1

General

The Hveem mix design method has been extensively used in the southern and western states of the U.S., where hot climates and desert areas are prevalent. The Hveem method of mix design is specifically useful for the design of rut-resistant mixes.

More so than the Marshall method, the Hveem method tests the stability of the aggregate structure in a compacted mix specimen. Typically, the optimum binder content determined with the Hveem method is approximately 0.3 percent lower than that determined with the Marshall method.

For a given aggregate gradation, one specimen is made for each of four binder contents and tested for various criteria. Using a decision pyramid, the optimum binder content is determined in three steps, selecting each time the highest binder content that meets the criteria.

8.1.1 Development and application

The concepts of the Hveem method of designing paving mixtures were developed and advanced in the 1930s under the direction of Francis N. Hveem, a former Materials and Research Engineer for the California Department of Transportation. The Hveem method has been refined over a period of years as certain features have been improved and other features added. The test procedures and their application have been developed through extensive research and correlation studies on asphalt highway pavements.

The Hveem method is applicable to paving mixtures using either asphalt cement or cutback asphalt and containing aggregates up to 25 mm (1 in.) maximum size. The method presented here is applicable to the design of dense-graded HMA paving mixtures.

The Hveem method test procedures have been standardized by the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM). Test procedures are found in ASTM D1560, “Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus,” and ASTM D1561, “Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor.” The testing procedures presented in this manual are basically the same as those of the ASTM test methods. The stabilometer can be used to establish R-values for treated and untreated soils, California Department of Transportation (CT) Test 301,” and stability values for bituminous mixtures (CT 304).

8.2

Outline of method

The Hveem method begins with the preparation of test specimens. Steps preliminary to specimen preparation are:

(a) determining that the proposed materials meet the physical requirements of the project specifications;
(b) assuring that the aggregate blend combinations meet the gradation requirements of the project specifications; and
(c) having an ample supply of aggregates, which have been dried and sized into fractions.

These requirements are matters of routine testing, specifications and laboratory technique, which must be considered but are not unique to any particular mix design method. The reader should refer to chapter 3 for the schedule of preparation and analysis of aggregates.

It should be noted, however, that the maximum size aggregates used in the test mixes should not exceed 25 mm (1 in.). In the event that the specifications for the paving mix being considered require aggregate sizes greater than 25 mm, up to 25 percent of oversized rock may be screened out and replaced by an equal weight of the largest aggregate passing the 1-inch sieve. However, this screening process can have a significant effect on the magnitude of the stabilometer values, depending on the size, amount and shape of the larger aggregate pieces.

The Hveem method uses standard test specimens of 63.5 mm (2 ½ in.) height by 101.6-mm (4 in.) diameter; these are prepared using a specified procedure for heating, mixing and compacting the asphalt-aggregate mixtures. The principal features of the Hveem method of mix design are the surface capacity and Centrifuge Kerosene Equivalent (CKE) test on the aggregates to estimate the initial asphalt requirements of the mix, followed by the stabilometer test, a cohesiometer test*, a swell test and a density voids analysis on test specimens of the compacted paving mixtures. The stabilometer test utilizes a special triaxial-type testing cell for measuring the resistance of the compacted mix to lateral displacement under the vertical loading, and the swell test measures the resistance of the mix to the action of water. The specimens are maintained at 60°C (140°F) for the stability test, whereas the swell test is performed at room temperature.

8.3

Approximate asphalt content by the centrifuge kerosene equivalent method

The first step in the Hveem method of mix design is to determine the "approximate" asphalt content. This can be based on past experience or determined by the Centrifuge Kerosene Equivalent method.**

With a calculated surface area and the factors obtained by the CKE method for a particular aggregate or blend of aggregates, the approximate asphalt content is determined by using a series of charts. These charts are presented in this chapter, accompanied by typical examples to demonstrate their application.

8.3.1 Equipment

The equipment and materials required for determining the approximate asphalt content (see **Figure 8.1**) are:

- small sample splitter for obtaining representative samples of fine aggregate;
- pans, 114 mm (4 ½ in.) diameter by 25 mm (1 in.) deep;
- kerosene, 4 liters (1 gal.);
- oil, SAE No. 10, lubricating, 4 liters;
- beakers, 1500 ml;
- metal funnels, 89 mm (3 ½ in.) top diameter, 114 mm (4 ½ in.) height, 13 mm (½ in.) orifice with piece of 2.00 mm (No. 10) sieve soldered to bottom of opening;

---

* The cohesiometer test is sometimes used for fine mixes such as sand mixes wherein cohesion, or tensile strength, is of major or primary importance. The cohesiometer test is also sometimes used for the design of cold mixes containing emulsified asphalt. The cohesiometer value is seldom used and, therefore, this test is not included in this manual. If the cohesiometer value is desired, the test is described in ASTM D1560.

**The development of the method for determining design asphalt content is outlined in "Establishing the Oil Content for Dense-Graded Bituminous Mixtures" by F. N. Hveem, California Highways and Public Works, July-August, 1942, and also presented in the "Proceedings of The Association of Asphalt Paving Technologists," Volume 13, 1942.

FIGURE 8.1 ***Apparatus for Hveem CKE Tests***

- timer;
- centrifuge, manual or power-driven, complete with cups, capable of producing 400 times gravity as seen in **Figure 8.1;** and
- filter papers, 55 mm diameter.

NOTE: *See sections 8.4.2 and 8.5.2 for additional Hveem method equipment requirements.*

8.3.2 Surface area

The gradation of the aggregate or blend of aggregates employed in the mix is used to calculate the surface area of the total aggregate. This calculation consists of multiplying the total percent passing each sieve size (in decimal form) by a "surface-area factor" as set forth in **Table 8.1**. Sum these products and the total will represent the equivalent surface area of the sample in terms of square meters per kilogram (square feet per pound). It is important to note that all the surface-area factors must be used in the calculation. Also, if a different series of sieves is used, different surface-area factors are necessary.

NOTE: *These surface-area factors have been used to calculate an average film thickness using the volume of asphalt binder in the mix. Although this determination of asphalt film thickness can provide a broad, relative indication of mix durability, the Asphalt Institute strongly recommends against comparing this calculated value with specific mix design criteria because of inherent inaccuracies. These surface-area factors do not take into account the specific aggregate shape, but are intended only as an index factor. In addition, in a compacted mixture, some of the asphalt and fine particle mastic is actually shared by adjacent particles rather than each being in an isolated state as assumed.*

8.3.3 CKE procedure for fine aggregate

The centrifuge kerosene equivalent method involves these steps:

(a) Place exactly 100 grams of dry aggregate (representative of the passing 4.75-mm [No. 4] material being used) in a tared centrifuge cup assembly fitted with a screen and a disk of filter paper.

(b) Place bottom of centrifuge cup in kerosene until the aggregate becomes saturated.
(c) Centrifuge the saturated sample for two minutes at a force of 400 times gravity. (For the manual centrifuge, this force can be developed by turning the handle approximately 45 revolutions per minute.)
(d) Weigh sample after centrifuging and determine the amount of kerosene retained as a percent of the dry aggregate weight; this value is called the Centrifuge Kerosene Equivalent (CKE). (Note: Duplicate samples are always prepared in order to balance the centrifuge and to check results. The average of the two CKE values is used unless there is a large discrepancy, in which case the test is rerun.)
(e) If the specific gravity of the aggregate samples is greater than 2.70 or less than 2.60, make a correction to the CKE value using the formula at the bottom of the chart in **Figure 8.2**.

8.3.4 Surface capacity test for coarse aggregate

The surface capacity (or "oil soak") test for the larger aggregate involves these steps:

(a) Place exactly 100 grams of dry aggregate that passed the 9.5-mm (⅜-in.) sieve and is retained on the 4.75-mm (No. 4) sieve into a metal funnel (this fraction is considered to be representative of the coarse aggregate in the mix).
(b) Immerse sample and funnel into a beaker containing SAE No. 10 lubricating oil at room temperature for 5 minutes.
(c) Allow to drain for 2 minutes.
(d) Remove funnel and sample from oil and drain for 15 minutes at a temperature of 60°C (140°F).
(e) Weigh the sample after draining and determine the amount of oil retained as a percent of the dry aggregate weight. (Note: Duplicate samples are prepared to check results. Average value is used unless there is a large discrepancy, in which case the test is rerun.)

TABLE 8.1 ***Surface Area (SA) Factors***

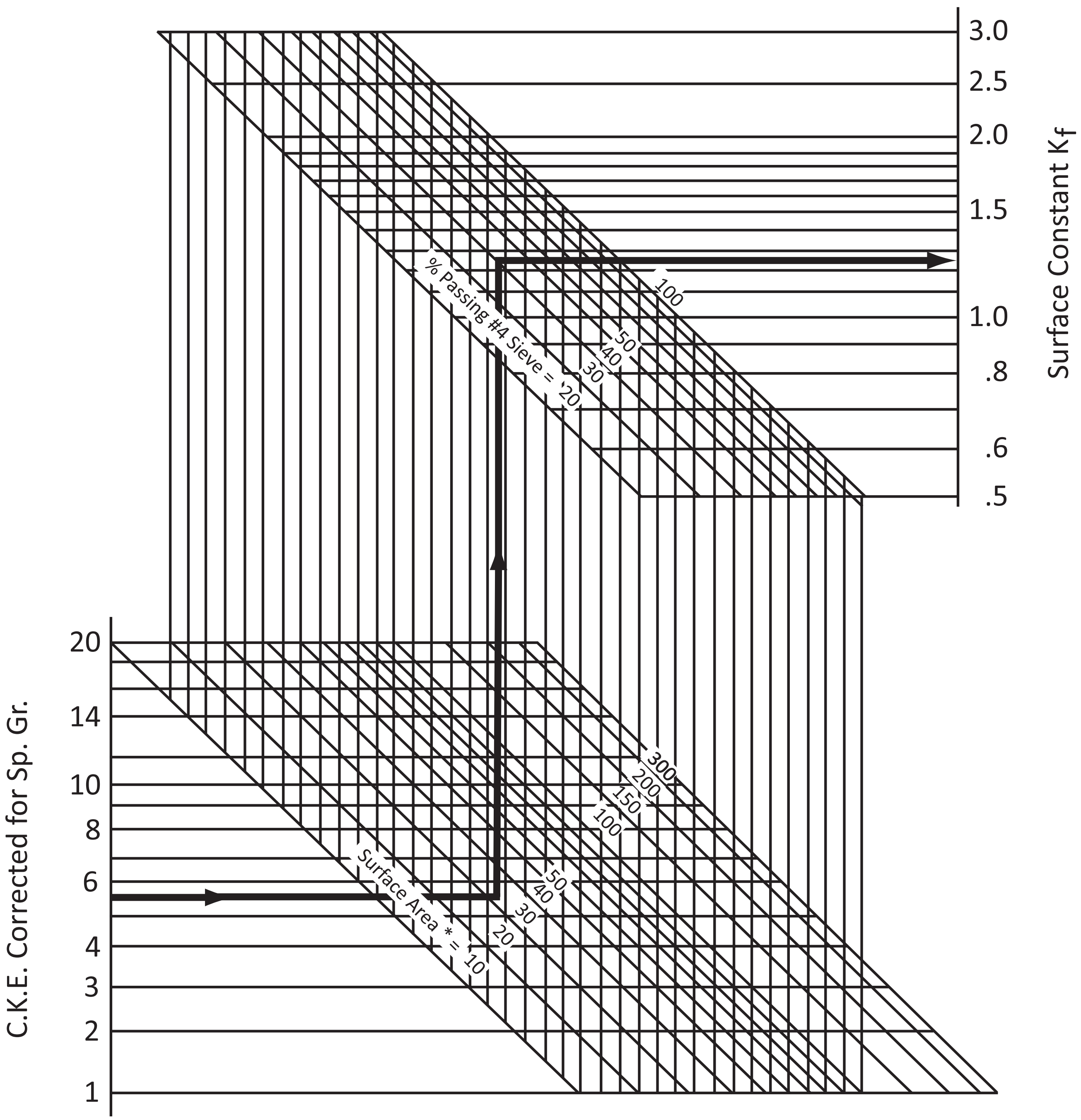
Total % Passing Sieve No.	Maximum Size	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	600 µm (No. 30)	300 µm (No. 50)	150 µm (No. 100)	75 µm (No. 200)
S.A. m^2/kg	.41	.41	.82	1.64	2.87	6.14	12.29	32.77
S.A. (ft^2/lb)	(2)	(2)	(4)	(8)	(14)	(30)	(60)	(160)
• Surface-area factors shown are applicable only when all the above-listed sieves are used in the sieve analysis.								

This example tabulation demonstrates the calculation of surface area by this method.

Sieve Size	Percent Passing	S.A. Factor m^2/kg	S.A. Factor (ft^2/lb)	S.A. m^2/kg	S.A. (ft^2/lb)
19.0 mm (¾ in.) 9.5 mm (⅜ in.)	100 }* 90	.41	(2)	.41	(2.0)
4.75 mm (No. 4)	75	.41	(2)	.31	(1.5)
2.36 mm (No. 8)	60	.82	(4)	.49	(2.4)
1.18 mm (No. 16)	45	1.64	(8)	.74	(3.6)
600 µm (No. 30)	35	2.87	(14)	1.00	(4.9)
300 µm (No. 50)	25	6.14	(30)	1.54	(7.5)
150 µm (No. 100)	18	12.29	(60)	2.21	(10.8)
75 µm (No. 200)	6	32.77	(160)	1.97	(9.6)
Total surface area				8.67	(42.3)

*Because of the relatively small surface area of larger aggregate sizes, a single surface-area factor of 0.41 m^2/kg (2 ft^2/lb) is used to account for the surface area of all of the material retained on the 9.5 mm sieve, regardless of the maximum aggregate size.

Chart for determining K_f from C.K.E.



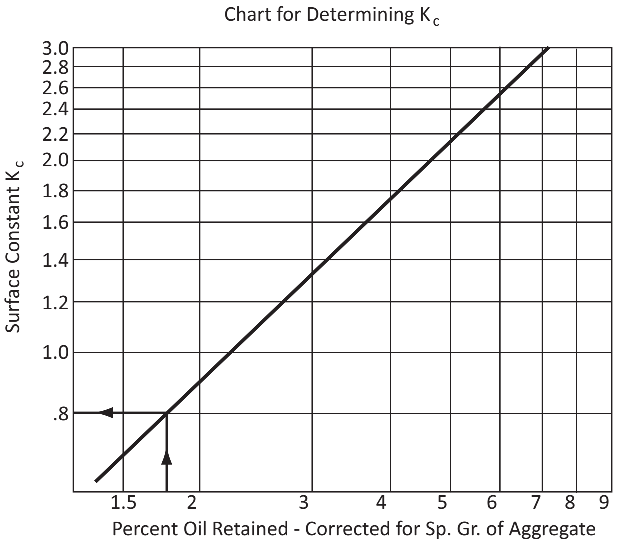
$$\text{C.K.E. Corrected} = \text{C.K.E.} = X \ \frac{\text{sp. gr. fine}}{2.65}$$

NOTE: Do not confuse this correction to C.K.E. with that used in Fig. 8.5

$$\text{*Surface area, } \frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$$

Courtesy of California Department of Transportation

FIGURE 8.2 ***Chart for Determining Surface Constant for Fine Material, K_f from C.K.E., Hveem Method of Design***



Material Used: Aggregate - Passing 9.5 mm (3/8"), Ret. 4.75 (#4) Sieve
Oil - SAE 10

$$\% \text{ Oil Ret. Corrected} = \% \text{ Oil Ret.} \times \frac{\text{sp. gr. of Course Aggregate}}{2.65}$$

FIGURE 8.3 ***Chart for Determining Surface Constant for Coarse Material K_c, from Coarse Aggregate Absorption, Hveem Method of Design***

(f) If the specific gravity of the aggregate is greater than 2.70 or less than 2.60, make a correction to the percent oil retained using the formula at the bottom of the chart in **Figure 8.3**.

8.3.5 Estimated design asphalt content

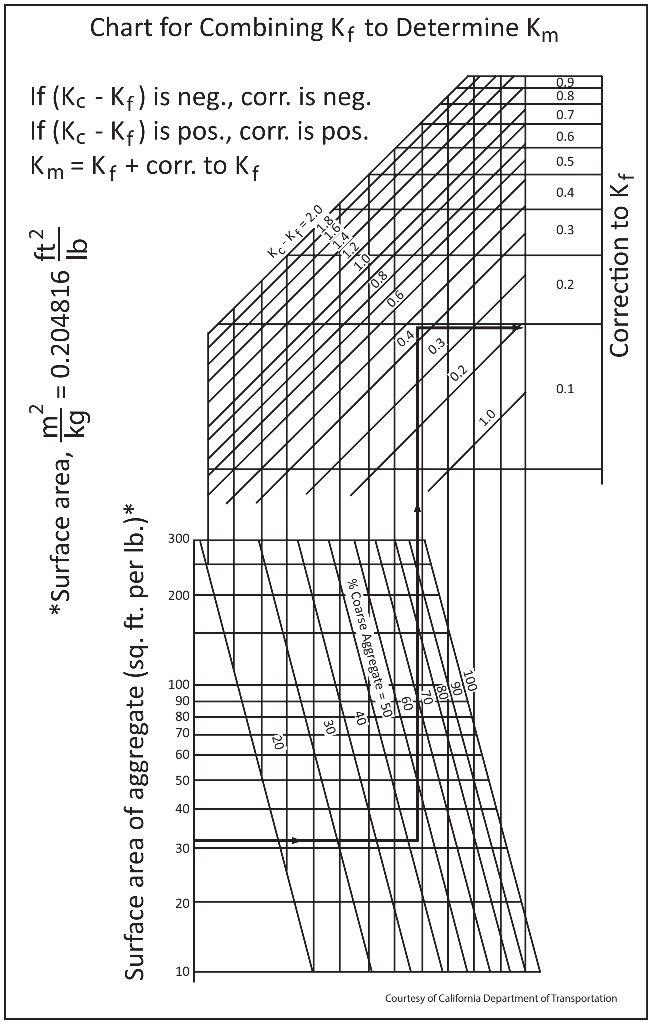
These steps are used to make a preliminary estimate of the design asphalt content:

(a) Using the CKE value obtained and the chart in **Figure 8.2**, determine the value K_f (surface constant for fine material).

(b) Using the percent oil retained and the chart in **Figure 8.3**, determine the value K_c (surface constant for coarse material).

(c) Using the values obtained for K_f and K_c and the chart in **Figure 8.4**, determine the value K_m (surface constant for fine and coarse aggregate combined). $K_m = K_f +$ correction to K_f. The correction to K_f obtained from **Figure 8.4** is positive if $(K_c - K_f)$ is positive and is negative if $(K_c - K_f)$ is negative.

(d) The next step is to determine the approximate bitumen ratio for the mix based on cutback asphalts of RC-250, MC-250 and SC-250 grades. With values obtained for K_m, surface area and average specific gravity, use Case 2 procedures of the chart in **Figure 8.5** to determine the oil ratio.



Courtesy of California Department of Transportation

FIGURE 8.4 ***Chart for Combining K_f and K_c to Determine Surface Constant for Combined Aggregate, K_m, Hveem Method of Design***

(e) Determine the asphalt content (bitumen ratio) for the mix (see **Figure 8.6**) corrected for the grade to be employed, using the surface area of the sample, the grade of asphalt and the oil ratio from **Figure 8.5**.

EXAMPLE:

To demonstrate the use of the charts in **Figures 8.2** through **8.6**, assume that these conditions apply to paving mix using AC-10 viscosity grade asphalt cement:

- Specific Gravity, coarse (bulk)[1] = 2.45
- Specific Gravity, fine (apparent)[2] = 2.64
- Percent Passing No. 4 = 45

$$\text{Avg. Sp. Gr.}^3 = \frac{100}{\frac{55}{2.45} + \frac{45}{2.64}} = 2.53$$

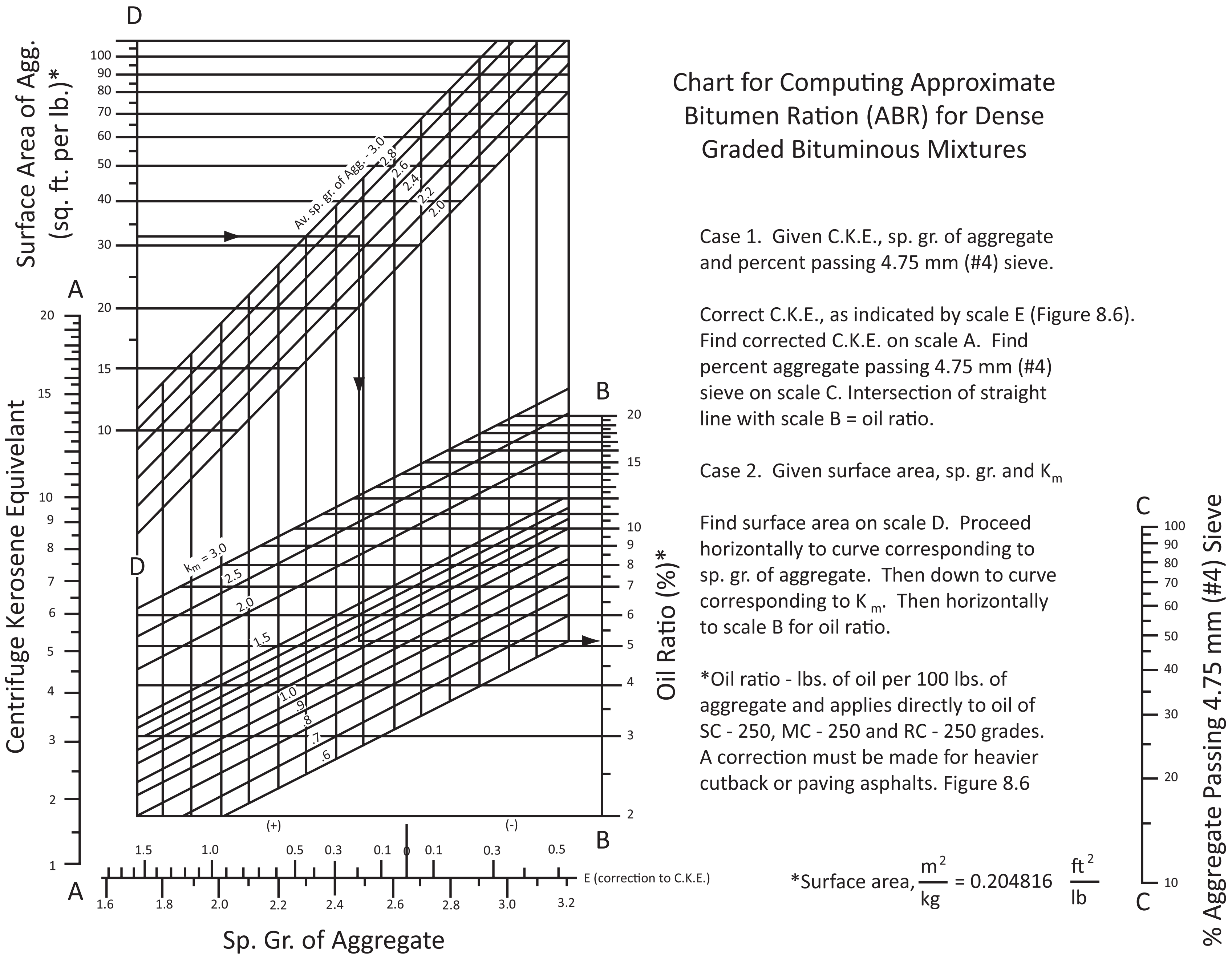


Chart for Computing Approximate Bitumen Ration (ABR) for Dense Graded Bituminous Mixtures

Case 1. Given C.K.E., sp. gr. of aggregate and percent passing 4.75 mm (#4) sieve.

Correct C.K.E., as indicated by scale E (Figure 8.6). Find corrected C.K.E. on scale A. Find percent aggregate passing 4.75 mm (#4) sieve on scale C. Intersection of straight line with scale B = oil ratio.

Case 2. Given surface area, sp. gr. and K_m

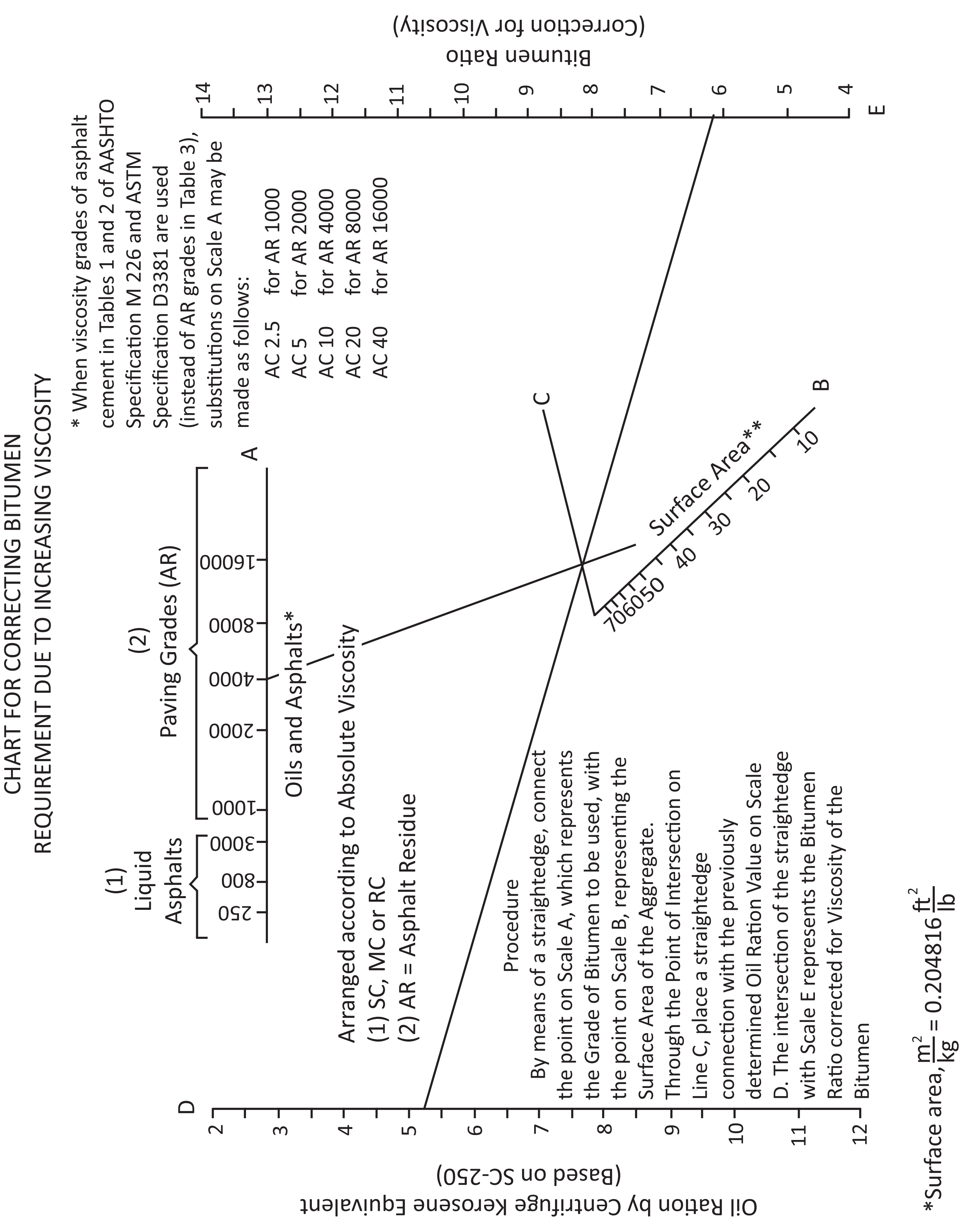
Find surface area on scale D. Proceed horizontally to curve corresponding to sp. gr. of aggregate. Then down to curve corresponding to K_m. Then horizontally to scale B for oil ratio.

*Oil ratio - lbs. of oil per 100 lbs. of aggregate and applies directly to oil of SC - 250, MC - 250 and RC - 250 grades. A correction must be made for heavier cutback or paving asphalts. Figure 8.6

*Surface area, $\frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$

Courtesy of California Department of Transportation

FIGURE 8.5 ***Chart for Computing Oil Ratio for Dense-Graded Bituminous Mixtures, Hveem Method of Design***



* When viscosity grades of asphalt cement in Tables 1 and 2 of AASHTO Specification M 226 and ASTM Specification D3381 are used (instead of AR grades in Table 3), substitutions on Scale A may be made as follows:

AC 2.5	for AR 1000
AC 5	for AR 2000
AC 10	for AR 4000
AC 20	for AR 8000
AC 40	for AR 16000

Procedure

By means of a straightedge, connect the point on Scale A, which represents the Grade of Bitumen to be used, with the point on Scale B, representing the Surface Area of the Aggregate. Through the Point of Intersection on Line C, place a straightedge connection with the previously determined Oil Ration Value on Scale D. The intersection of the straightedge with Scale E represents the Bitumen Ratio corrected for Viscosity of the Bitumen

$$*\text{Surface area}, \frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$$

Courtesy of California Department of Transportation

FIGURE 8.6 ***Chart for Correcting Bitumen Requirement Due to Increasing Viscosity of Asphalt, Hveem Method of Design***

- Surface Area of Aggregate Grading = 6.6 m^2/kg (32.4 ft^2/lb)
- CKE = 5.6
- Percent Oil Retained, coarse = 1.9
- (corrected for specific gravity, this value is 1.7 percent; see **Figure 8.3**)
- From **Figure 8.2**, determine K_f as 1.25.
- From **Figure 8.3**, determine K_c as 0.8.
- From **Figure 8.4**, determine K_m as 1.15.
- From **Figure 8.5**, determine the oil ratio for liquid asphalt as 5.2 percent.
- From **Figure 8.6**, determine design asphalt content (bitumen ratio) for AC-10 asphalt as 6.1 percent by weight of dry aggregate.

NOTES:

1. California DOT Manual of Tests, No. CT206, which is a modification of AASHTO T 85 (ASTM C127).
2. California DOT Manual of Tests, No. CT208, which is a modification of AASHTO T 133 (ASTM C188).
3. As defined by California DOT Manual of Tests, No. CT303.

8.4

Preparation of test specimens

8.4.1 General

In designing a paving mix by the Hveem method, a series of stabilometer test specimens is prepared for a range of asphalt contents both above and below the approximate design asphalt content indicated by the CKE procedure.

For hot mix designs using an average aggregate, tests should be scheduled by preparing one specimen with the amount of asphalt as determined by the CKE procedure: two above the CKE amount in 0.5 increments, and one 0.5 percent below the CKE (total of four specimens, each with a different asphalt content).

For mixes thought to be critical (i.e., sensitive to asphalt content), the steps in asphalt content are lowered to 0.3 percent and tests are scheduled for the approximate asphalt content indicated by the CKE procedure: three above the CKE amount in 0.3 percent increments and one 0.3 percent below the CKE amount.

For highly absorptive aggregates and noncritical mixes, increase the steps in asphalt content to 1.0 percent (versus 0.5 percent) and use more specimens as necessary.

Regardless of these general rules for preparing stabilometer specimens, the series of test specimens should have at least one specimen containing an excess of asphalt as indicated by moderate or heavy flushing after compaction.

In addition, two swell test specimens are later prepared at the same design asphalt content as determined from tests on the series of specimens prepared for stabilometer tests.

Therefore, for a normal mix design study, a total of six test specimens will usually be required. Although each test specimen will normally require only 1.2 kg (3 lb.) of aggregate, the minimum aggregate requirements for a series of test specimens should be at least 18 kg (40 lb.) to provide for additional tests that may be required.

8.4.2 Equipment for preparation of Hveem test specimen

The equipment required for the preparation of test specimens is:

- pans, with a minimum dimension of 250 mm (10 in.) diameter by 50 mm (2 in.) deep, for quartering and mixing fine aggregate, or an equivalent 8 × 12 inch rectangular pan;
- pans, 200 mm (8 in.) diameter by 45 mm (1 ¾ in.) deep, for batching and heating aggregates;
- pans, 305 mm (12 in.) diameter by 64 mm (2 ½ in.) deep, for mixing aggregate and asphalt;
- pans, 280 mm (11 in.) by 180 mm (7 in.) by 25 mm (1 in.) for curing mix;
- large sample splitter for mixing and quartering fine aggregate;
- electric hot plate, with a surface measuring at least 460 mm (18 in.) by 305 mm (12 in.), for heating aggregates, asphalt and equipment as required;
- large oven, thermostatically controlled, capable of 110°C (230°F) temperature, with a tolerance of +/- 5°F;

FIGURE 8.7 ***Mechanical Kneading Compactor for the Preparation of Hveem Test Specimens***

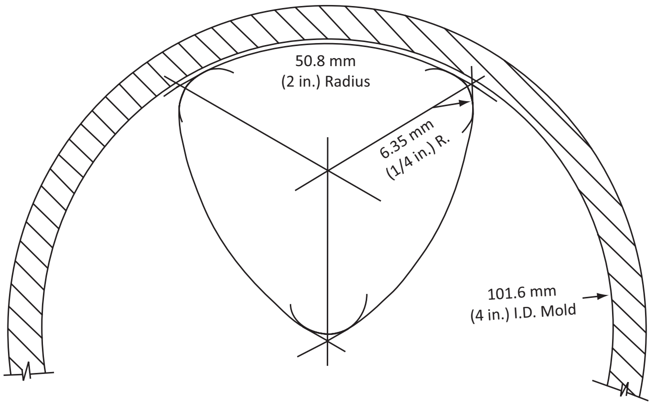


FIGURE 8.8 ***Diagram of Tamping Foot for Mechanical Kneading Compactor***

- large oven, thermostatically controlled, capable of 60°C (140°F) temperature with a tolerance of +/- 5°F;
- large oven for drying and preheating aggregates, capable of temperatures up to 165°C (325°F);
- large scoop for handling hot aggregates;
- beakers or metal cans, 800 ml, for adding asphalt;
- thermometer, armored, 35°C (100°F) to 205°C (400°F);
- balance, minimum 5 kg capacity, sensitive to 0.1 g for weighing aggregates and asphalt;
- small pointed mixing trowel;
- large mixing spoon;
- mechanical mixer (optional);
- mechanical compactor (see **Figure 8.7**) designed to consolidate the material by a series of individual "kneading action" impressions made by a roving ram having a face shaped as a sector of a 101.6-mm (4-in.) diameter circle (see **Figure 8.8**). The compactor must be capable of exerting a force of 3.45 MPa (500 psi) beneath the tamper foot. Accessories with the compactor should include two mold holders, an insulated mix feeder trough 460 mm (18 in.) long by 102 mm (4 in.) wide by 64 mm (2 ½ in.) deep, a paddle shaped to fit the trough, and a round-nosed steel rod 9.5 mm (⅜ in.) diameter by 406 mm (16 in.) long;
- steel compaction molds, 101.6 mm (4 in.) inside diameter × 127 mm (5 in.) high × 6.4 mm (¼ in.) wall thickness;
- paper disks, heavy paper, 100 mm (4 in.) in diameter, to place in bottom of mold during compaction;
- hydraulic compression machine, 222 kN (50,000 lb.) capacity;
- steel shim, 6.4 mm (¼ in.) thick by 19.0 mm (¾ in.) wide by 63.5 mm (2 ½ in.) long; and
- gloves, heavy and sturdy, for handling hot equipment.

8.4.3 Batch weights

These guidelines are suggested for estimating the aggregate requirements:

(a) Compute batch weights for the blend and gradation of aggregates desired. Suggested procedures for computing batch weights are presented in chapter 3.

(b) The necessary dry weight of the aggregate for the stabilometer specimens is that which will produce a compacted specimen 63.5 +/- 1.3

mm (2.5 +/- 0.05 in.) in height. This volume of aggregate will normally weigh about 1,200 grams. To determine the exact batch weight, it is generally desirable to prepare a trial specimen prior to preparing the actual aggregate batches. If the trial specimen height falls outside the limits, the amount of aggregate used for the specimen may be adjusted using:

For International System of Units (SI),

$$\text{Adjusted mass of aggregate} = \frac{63.5\ (\text{mass of aggregate used})}{\text{Specimen height (mm) obtained}}$$

For U.S. Customary Units,

$$\text{Adjusted weight of aggregate} = \frac{2.5\ (\text{weight of aggregate used})}{\text{Specimen height (in.) obtained}}$$

8.4.4 Preparation of batch mixes

These steps are provided as a guide in preparing the mixtures for testing:

(a) Weigh the various-sized fractions of dry aggregates into suitable pans in accordance with the calculated batch weights. (See chapter 4).

(b) Thoroughly mix each individual batch of aggregate and preheat in oven to desired mixing temperature. Asphalt should be preheated at the same time. The temperature of the aggregate and the asphalt at the time mixing begins is indicated below for the paving grade of asphalt cement being used.

PG grades and modified asphalt binders should be heated as directed in section 4.2.1.

Grade	Temperature Range Minimum	Maximum
AC-2.5, AR-1000 or 200-300 Pen.	99°C (210°F)	121°C (250°F)
AC-5, AR-2000 or 120-150 Pen.	110°C (230°F)	135°C (275°F)
AC-10, AR-4000 or 85-100 Pen.	121°C (250°F)	149°C (300°F)
AC-20, AR-8000 or 60-70 Pen.	132°C (270°F)	163°C (325°F)
AC-40, AR-1600 or 40-50 Pen.	132°C (270°F)	163°C (325°F)

(c) When the aggregates and asphalt have reached the desired mixing temperature, form a crater in the aggregates and weigh in asphalt in accordance with the calculated batch weights.

(d) Place pan containing aggregates and asphalt for batch mix on hot plate to maintain mixing temperature. Vigorously mix aggregates and asphalt by hand with a pointed trowel or by mechanical mixing until all particles are coated. Take special precaution not to overheat the materials.

(e) After mixing is complete, transfer the batch mix to a suitable flat pan and cure for two or three hours at a temperature of 146 ± 3°C (295°F ± 5°F) in an oven equipped with forced draft air circulation. Caltrans uses the CT 304 procedure to cure the mixture for 15 to 18 hours at 60 ± 2.8°C (140°F ± 5°F).

(f) After curing is complete, place batch mix in heating oven and reheat mixture to 110°C (230°F). The batch mix is then ready for compaction.

8.4.5 Compaction

The compaction of the test specimen is accomplished by means of the Hveem Kneading Compactor, a mechanical compactor that imparts a kneading action type of consolidation by a series of individual impressions made with a ram having a face shaped as a sector of a 101.6-mm (4-in.) diameter circle (see **Figure 8.8**). With each push of the ram, a pressure of 3.45 Mpa (500 psi) is applied, subjecting the specimen to a kneading compression over an area of approximately 2000 square millimeters (3.1 square inches). Each pressure application is maintained for approximately 0.4 seconds. The detailed compaction procedure follows.

8.4.5.1 Stabilometer specimens

(a) Preheat the compaction molds, feeder trough and round-nosed steel rod to approximately the mix compaction temperature.

(b) Heat the compactor foot to a temperature that will prevent the mix from adhering to it. The temperature of the compactor foot may be controlled by a variable transformer.

FIGURE 8.9 ***Transfer of Mix to Mold***

FIGURE 8.10 ***Rodding Mix in Mold***

(c) Place the compaction mold in the mold holder and insert a 100-mm (4-in.) diameter paper disk to cover the base plate. The steel shim is temporarily placed under the edge of the mold, so the base plate will act as a free-fitting plunger during the initial compaction operation.

(d) Spread the prepared mixture uniformly on the preheated feeder trough. Using a paddle that fits the shape of the trough, transfer approximately one-half of the mixture to the compaction mold (see **Figure 8.9**).

(e) Rod the portion of the mix in the mold 20 times in the center of the mass and 20 times around the edge with the round-nosed, steel rod (see **Figure 8.10**). Transfer the remainder of the sample to the mold and repeat the rodding procedure.

(f) Place mold assembly into position on the mechanical compactor and apply approximately 20 tamping blows at 1.7 Mpa (250 psi) pressure to achieve a semi-compacted condition of the mix so that it will not be unduly disturbed when the full load is applied. The exact number of tamping blows to accomplish the semi-compaction shall be determined by observation. The actual number of tamping blows may vary between 10 and 50, depending upon the type of material, and it may not be possible to accomplish the compaction in the mechanical compactor because of undue movement of the mixture under the compactor foot. In these instances, use a 178 kN (40,000 lb.) static load applied over the total specimen surface by the double-plunger method, in which a free-fitting plunger is placed below and on top of the sample. Apply the load at the rate of 1.3 mm (0.05 in.) per minute and hold for 30 ± 5 seconds.

(g) After the semi-compaction, remove the shim and release mold tightening screw sufficiently to allow free up-and-down movement of mold and about 3 mm (⅛ in.) side movement of mold.

(h) To complete compaction in the mechanical compactor, increase compactor foot pressure to 3.45 Mpa (500 psi) and apply 150 tamping blows.

(i) Place the mold and specimen in an oven at 60°C (140°F) for 1 hour, after which a “leveling off” load of 56 kN (12,600 lb.) is applied by the “double-plunger” method (head speed = 6 mm/min. [0.25 in./min.]) and released immediately. (Note: The specimen shall not be pushed to the opposite end of the mold.)

8.4.5.2 *Swell test specimens*

(a) Prepare the compaction mold by placing a paraffin-impregnated strip of ordinary wrapping paper 19 mm (¾ in.) wide around the inside of the mold 13 mm (½ in.) to 19 mm (¾ in.) from the bottom to prevent water from escaping from between the specimen and the mold during the water immersion period of the test. The paper strip is dipped in melted paraffin and applied while hot. Compaction molds are not preheated for swell test specimens.

(b) The remainder of the compaction procedure for swell test specimens is the same for the stabilometer test specimens except when compaction is completed in the mechanical compactor, remove mold and specimen from compactor, invert mold and push specimen to the opposite end of mold. Apply a 56 kN (12,600 lb.) static load (head speed 6 mm/min. [0.25 in./min.]) with the "original" top surface supported on the lower platen of the testing press. It is advisable to place a piece of heavy paper under the specimen to prevent damage to the lower platen.

8.5

Test procedures

8.5.1 General

In the Hveem method, the compacted test specimens are used in these tests and analyses and are normally performed in the order listed:

(a) stabilometer test;
(b) bulk density determination; and
(c) swell test.

The swell test is performed only on specimens prepared for this purpose; the stabilometer and bulk density tests are performed on each of all other test specimens. **Figure 8.15** shows a suggested test report form for recording test data and results.

8.5.2 Equipment for Hveem swell and stability testing

The equipment required for the testing of the 101.6-mm (4-in.) diameter specimens is:

(a) bronze disks, perforated, 98.4 mm (3 ⅞ in.) diameter by 3.2 mm (⅛ in.) thick, with adjustable stem, for swell measurement (see **Figure 8.11**);
(b) dial gauge, mounted on tripod, with reading accuracy to 0.025 mm (0.001 in.) (see **Figure 8.11**);
(c) scale (ml), graduated to read the water volume on top of the specimen, in the 101.6-mm (4-in.) diameter mold at 25 ml intervals, for the purpose of measuring the percolation of water during swell test;
(d) round aluminum pans, 190 mm (7 ½ in.) diameter by 64 mm (2 ½ in.) deep;
(e) The Hveem stabilometer (see **Figures 8.12** and **8.13**) is a triaxial testing device consisting

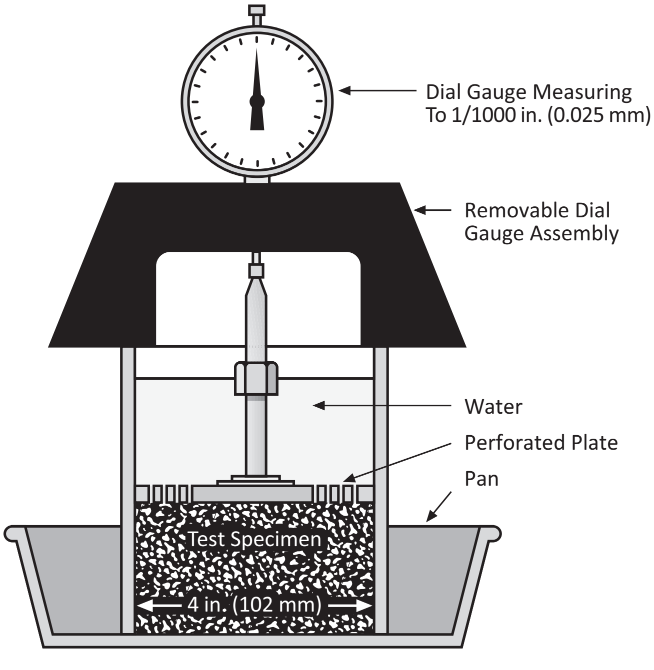
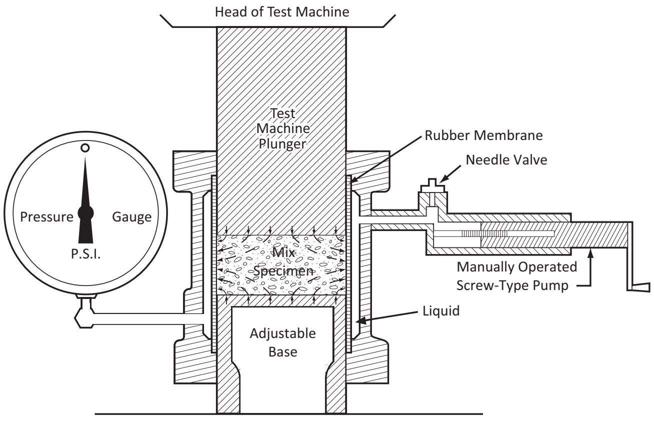


FIGURE 8.11 ***Swell Test Apparatus***



Note: The specimen is given lateral support by the flexible sidewall, which transmits horizontal pressure to the liquid. The magnitude of the pressure can be read on the gauge.

FIGURE 8.12 ***Diagrammatic Sketch Showing Principal Features of Hveem Stabilometer***

FIGURE 8.13 ***Hveem Stabilometer***

essentially of a cylindrical metal body shell (containing a liquid which registers the horizontal pressure developed by a compacted test specimen as a vertical load is applied), a flexible rubber diaphragm, a pressure gauge, a screw-type hand pump assembly and an air chamber fitted with a needle valve for adjusting the quantity of air in the system. The pump provides a means of obtaining a standard quantity of air in the liquid system of the stabilometer; and

(f) scale or other measuring device to accurately determine the height of the compacted test specimen.

8.5.3 Stabilometer test procedures

The stabilometer measures the lateral pressure transmitted through the specimen from the applied vertical load. The ratio of a given applied unit compressive stress to the transmitted lateral or horizontal pressure is used to determine an index, on a scale ranging from 0 to 100, of the ability of the material under test (at the test temperature) to resist deformation.

Frequent calibration of the stabilometer should be made during the day as temperature change has considerable effect upon the pressure exerted within the hydraulic system. See section 8.9, (Preparing the Hveem stabilometer for testing.) This is especially true when changing from R-value testing to bituminous testing.

These are the steps for measuring the Hveem stability (refer to **Figures 8.12** and **8.13**):

(a) Place specimens for stabilometer tests (compacted and contained in mold) in oven at 60 ± 3°C (140 ± 5°F) for 3 to 4 hours.
(b) Adjust compression machine for head speed of 1.3 mm/min. (0.05 in./min.) with no load applied.
(c) Check displacement of stabilometer with a calibration cylinder and, if necessary, adjust to read 2.00 ± 0.05 turns (see section 8.9.1).
(d) Adjust the stabilometer base so that the distance from the bottom of the upper tapered ring to the top of the base is 89 mm (3.5 in.).
(e) Every effort should be made to fabricate test specimens with an overall height between 61 mm (2.40 in.) and 66 mm (2.60 in.); however, if the height is outside of this range, the stabilometer value should be corrected as indicated in **Figure 8.14**.
(f) Remove the mold with its specimen from the oven and place on top of stabilometer. Using the plunger, hand lever and fulcrum, force the specimen from the mold into the stabilometer. Take care that the specimen goes in straight and is firmly seated. Instead of using the hand lever and fulcrum, this can be done with the compression machine.
(g) Place follower on top of specimen and position the entire assembly in compression machine for testing.
(h) Using the displacement pump, raise the pressure in the stabilometer system until the test gauge (horizontal pressure) reads exactly 34.5 kPa (5 psi). (Tap test gauge lightly to assure an accurate reading.)
(i) Close displacement pump valve, taking care not to disturb the 34.5 kPa (5 psi) initial pressure. (This step is omitted on

Height correction should be made using the table and chart below.

Example: Overall height of 69 mm (2.75 in.) selects correction curve "B." Stabilometer value uncorrected = 35
Stabilometer value corrected = 38.

Overall Specimen Ht.	Correction Curve
71 mm to 76 mm (2.80 in. to 3.00 in.)	A
66 mm to 70 mm (2.60 in. to 2.79 in.)	B
61 mm to 65 mm (2.40 in. to 2.59 in.)	C
56 mm to 60 mm (2.20 in. to 2.39 in.)	D
51 mm to 55 mm (2.00 in. to 2.19 in.)	E

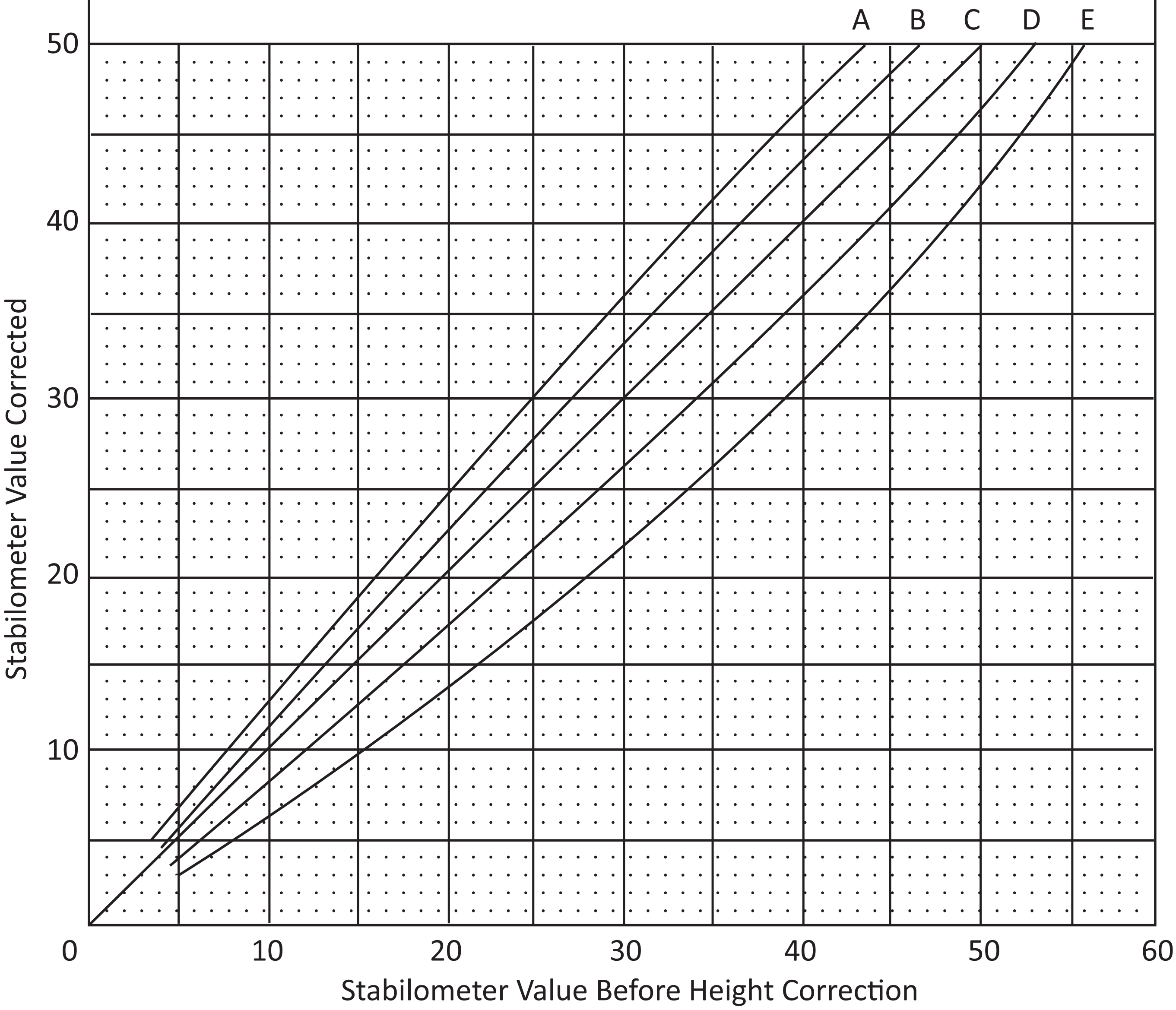


FIGURE 8.14 ***Chart for Correcting Stabilometer Values to Effective Specimen Height of 64 mm (2.50 in.)***

stabilometers that are not provided with a displacement pump valve.)

(j) Apply test loads with the compression machine using a head speed of 1.3 mm/min. (0.05 in./min.). Record readings of the stabilometer test gauge at vertical test loads of 13.4, 22.3 and 26.7 kN (3,000, 5,000 and 6,000 lbs.). Warning: Do not continue loading if the horizontal pressure gauge exceeds 125 psi.

(k) Immediately after recording the horizontal pressure reading, under maximum vertical load (26.69 kN [6,000 lb.]), reduce total load on specimen to 4.45 kN (1,000 lb.).

(l) Open the displacement pump angle valve and by means of the displacement pump adjust test gauge to 34.5 kPa (5 psi). (This will result in a reduction in the applied vertical load that is normal and no compensation is necessary.)

(m) Adjust dial gauge on pump to zero by means of small thumbscrew.

(n) Turn the displacement pump handle smoothly and rapidly (two turns per second) and to the right (clockwise) until a pressure of 690 kPa (100 psi) is recorded on the test gauge. [During this operation, the load registered on the testing press will increase and in some cases exceed the initial 4.45 kN (1,000 lb.) load. This change in load is normal and no adjustment or compensation is required.] Record the exact number of turns required to increase the test gauge reading from 34.5 kPa (5 psi) to 690 kPa (100 psi) as the displacement on specimen (2.5 mm [0.1 in.] dial reading is equivalent to one-turn displacement).

(o) After recording the displacement, first remove the test load and reduce pressure on the test gauge to zero by means of the displacement pump; then reverse the displacement pump an additional three turns and remove specimen from stabilometer chamber.

8.5.4 Bulk density determination

The bulk density test is performed on these specimens after the completion of the stabilometer tests as soon as the specimens have cooled to room temperature. The procedure for this test is presented in ASTM D1188, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens" or ASTM D2726, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface Dry Specimens."

8.5.5 Swell test

These steps outline the swell test procedure.

(a) Allow compacted swell test specimen to stand at room temperature for at least one hour. (This is done to permit rebound after compaction.)

(b) Place the mold and specimen in 190 mm (7 ½ in.) diameter by 64 mm (2 ½ in.) deep aluminum pan (see **Figure 8.11**).

(c) Place the perforated bronze disk on the specimen, position the tripod with the dial gauge on the mold and set the adjustable stem to give a reading of 2.54 mm (0.10 in.) on the dial gauge (see **Figure 8.11**).

(d) Introduce 500 ml of water into the mold on top of the specimen and measure the distance from the top of the mold to the water surface with the graduated scale.

(e) After 24 hours, read the dial gauge to the nearest 0.025 mm (0.001 in.) and record the change as swell. Also, measure the distance from the top of the mold to the water surface with the graduated scale and record the change as permeability or the amount of water in ml that percolated into and/or through the test specimen.

8.6

Interpretation of test data

8.6.1 Calculations

There are no calculations required for the swell test since the results are reported directly as differences.

The stabilometer measures the lateral pressure resulting from the dilation of the specimen caused by the applied vertical load. The ratio of a given applied unit compressive stress (P_v) to the transmitted lateral or horizontal pressure (P_h) is used to determine the S-value), on a scale ranging from 0 to 100, of the ability of the material to resist deformation. The higher the S-value, the higher the resistance to deformation. The S-values for Hveem compacted asphalt mixture specimens are typically a minimum of 35.

The remainder of the calculations are:

(a) Stabilometer value (S); calculate as:

$$S = \frac{22.2}{\frac{P_h D}{P_v - P_h} + 0.222}$$

Where,

S = stabilometer value

D = the number of turns of the air displacement pump handle needed to increase the lateral pressure from 5 psi to 100 psi

P_v = vertical pressure (typically 2.76 Mpa [400 psi] = 22.24 kN [5,000 lb.] total load)

P_h = the horizontal pressure (stabilometer pressure gauge) reading taken at the instant the vertical pressure P_v is 2.76 MPa or 400 psi (= at 5,000 lbs. vertical load)

(b) Density and voids analysis. Using the bulk specific gravity of the test specimens and the maximum specific gravity of the paving mixture determined using ASTM D2041, compute the percent air voids as illustrated in **Figure 8.15** and more fully described in chapter 5. These values may be plotted as a function of asphalt content as shown in **Figure 8.16**, similar to the Marshall procedure, to assist in design.

8.7

Design criteria

The suitability of the hot mix design by the traditional Hveem method is determined on the basis of whether the asphalt content and aggregate grading will satisfy the requirements in **Table 8.2**:

Calculate the air voids content of each specimen.

In applying these requirements, **the design asphalt content should be the highest percentage of asphalt the mix will accommodate without reducing stability or void content below minimum values.** The design asphalt content is determined from observations of surface flushing or bleeding of specimens after compaction, the stabilometer values and percent air voids.

8.8

Determination of Optimum Binder Content (OBC)

(a) Using **Figure 8.17**, insert in Step 1 of the pyramid the asphalt contents used for preparing the series of mix design specimens. Insert the asphalt contents in order of increasing amounts from left to right, with the maximum asphalt content used in the square on the right.

(b) Plot percent air voids versus percent asphalt content, and determine by interpolation the percent asphalt content that has 4.0 percent air voids.

(c) Select from Step 1 the three highest asphalt contents that do not exhibit moderate or heavy surface flushing and record on Step 2 of the pyramid. Surface flushing and/or bleeding is considered "Slight" (acceptable) if the surface has only a slight sheen. It is considered "Moderate" (unacceptable) if sufficient free asphalt is apparent to cause paper to stick to the surface but no distortion is noted. Surface flushing is considered "Heavy" (unacceptable) if there is sufficient free asphalt to cause surface puddling or specimen distortion after compaction.

(d) Select from Step 2 the two highest asphalt contents that provide the specified minimum stabilometer value and enter them in Step 3 of the pyramid.

(e) From the plot in (b), determine the theoretical asphalt content that has 4 percent air voids and is within the asphalt range listed in Step 3 of the pyramid. Always stay as close to 4 percent air voids as possible. The selected percent asphalt content is entered in Step 4, and is the optimum binder content (OBC) for the job mix formula (JMF).

(f) The asphalt content in Step 4 is the design asphalt content. However, if the maximum asphalt content used in the design set (Step 1) is the asphalt content entered on Step 4 of the pyramid, additional specimens must be prepared with increased asphalt contents in 0.5 percent increments and a new design asphalt content determination should be made.

FIGURE 8.15 ***Suggested Test Report Form Showing Test Data For A Typical Mix Design by the Hveem Method***

Sp. Gr. Asp. Cem. 1.012 Asp. Cem. AC-10
Avg. Bulk Sp. Gr. Agg. = 2.760

Lab. No. for ASP Cem. Used: 53-0741
Lab. Nos. for Agg. Used: 53-1252; 53-1253

Gradation, CKE and Percent Asphalt

Sieve Size, mm (in. or No.)	19.0 (3/4)	12.5 (1/2)	9.5 (3/8)	4.75 (4)	2.36 (8)	1.18 (16)	0.60 (30)	0.30 (50)	0.15 (100)	0.075 (200)
Specification Limits	100	100 80	90 70	70 50	50 35		29 18		16 8	10 4
Percent Passing	100	91	76	60	42	32	23	16	12	6
S.A. Factors			.41	.41	.82	1.64	2.87	6.14	12.29	32.77
Surface Area, m^2/kg*			.41	0.25	0.34	0.53	0.66	0.98	1.48	1.97

CKE: FA = 2.8; CA = 2.8; K_f = 1.0 ; K_c = 1.3 ; K_m = 1.0 ; Total Surface Area = 6.62 m^2/kg (32.3 $ft^2/lb.$)
Estimated Percent Asphalt Cement by Weight of Aggregate using CKE Tests only = 5.5
Recommended Percent Asphalt Cement by Weight of Aggregate using Mix Design Criteria = 5.0

Specimen Identification	A	B	C	D
Percent Asphalt Cement By Weight of Aggregate	5.0	5.5	6.0	6.5
Percent Asphalt Cement By Weight of Mix	4.76	5.21	5.66	6.10
Weight in Air, grams	1211.0	1223.3	1230.8	1235.9
Weight in Water, grams	714.9	723.8	727.6	733.3
Bulk Volume, cc	496.1	499.5	503.2	502.6
Bulk Specific Gravity	2.441	2.449	2.446	2.459
Maximum Specific Gravity	2.559	2.540	2.522	2.504
Air Voids, Percent	4.6	3.6	3.0	1.8
Unit Weight (kg/m^3), pcf	2.439 (152.3)	2.448 (152.8)	2.446 (152.6)	2.457 (153.4)

Total Load		Unit Load		Stabilometer			
kN	lbs.	MPa	psi				
2.22	500	0.28	40	9	9	9	10
4.45	1000	0.55	80	12	12	15	16
8.90	2000	1.10	160	15	16	24	26
13.34	3000	1.65	240	21	22	30	38
17.79	4000	2.21	320	28	30	42	55
22.24	5000	2.76	400	36	39	55	83
26.69	6000	3.31	480	50	52	62	105

	A	B	C	D
Displacement, turns	2.40	2.50	2.46	2.50
Stability Value	48	45	36	25

Jones
Inspector

* Surface Area: 1 m^2/kg = 4.8824 $ft^2/lb.$

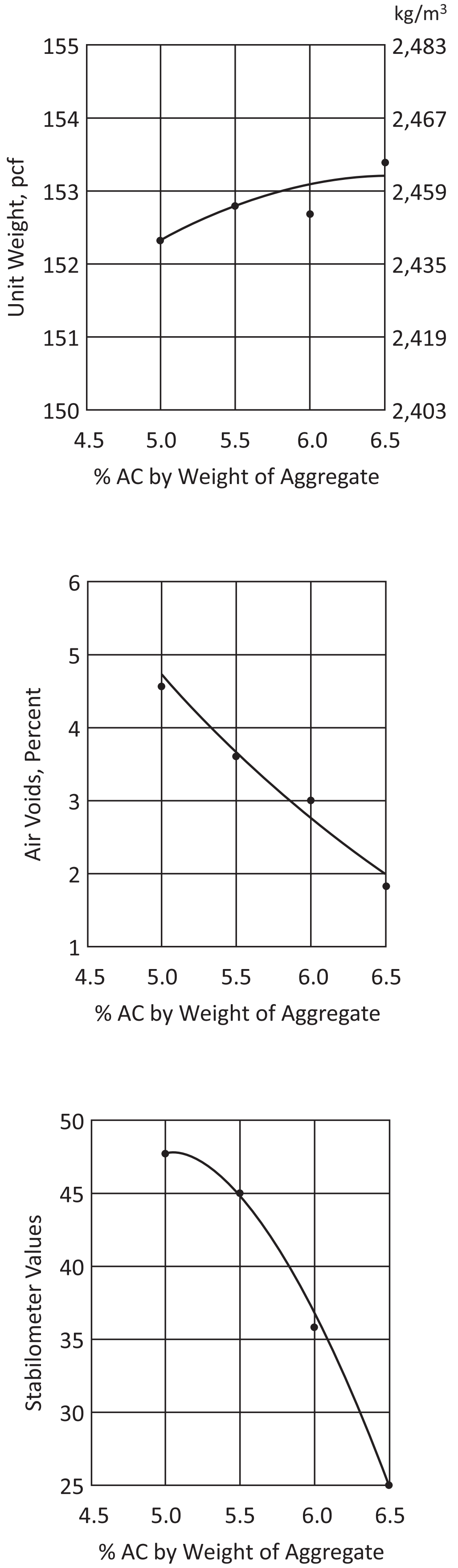


FIGURE 8.16 ***Test Property Curves for Hot Mix Design Data by the Hveem Method***

Quality Characteristic	Traffic Category		
	heavy	medium	light
Air voids content (%)	**4.0**	**4.0**	**4.0**
Voids in mineral aggregate (% min.)			
4.75-mm grading	17.0	17.0	17.0
9.5-mm grading	15.0	15.0	15.0
12.5-mm grading	14.0	14.0	14.0
19-mm grading	13.0	13.0	13.0
Stabilometer value (min.)			
12.5-mm and 19-mm gradings	37	35	32
Swell	less than 0.762 mm (0.030 in.)		

Notes:

1. Although not a routine part of this design method, an effort is made to provide a minimum percent of air voids of approximately 4 percent.
2. All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.
3. Hot mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed at 100 mm (4 in.) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.
4. Traffic classifications:
   Light: Traffic conditions resulting in a Design EAL <10^4.
   Medium: Traffic conditions resulting in a Design EAL between 10^4 and 10^6.
   Heavy: Traffic conditions resulting in a Design EAL >10^6.

TABLE 8.2 ***Hveem Mix Design Criteria***

(g) At the final OBC, compact three additional Hveem specimens and determine the final values for Hveem stability and volumetric properties as the average of each of the three test values.

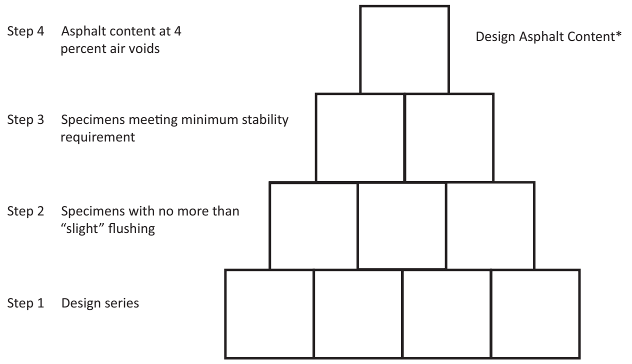
Additional mix design voids criteria are contained in chapter 5 as well as some factors to consider when selecting the final design asphalt content.

8.8.1 Example of Hveem mix design

Figure 8.18 illustrates determination of the design asphalt content for a heavy-traffic category pavement using typical mix design test data.

(a) Enter on Step 1 of the pyramid the percent asphalt contents 5.0, 5.5, 6.0 and 6.5.
(b) The plot of percent asphalt versus percent air voids (see **Figure 8.16**) shows that the Hveem compacted specimens have 4.0 percent air voids at 5.3 percent binder content.
(c) Specimen D having 6.5 percent asphalt content exhibits moderate (unacceptable) surface flushing. Enter 5.0, 5.5 and 6.0 on Step 2 of the pyramid.
(d) Specimens A and B are the highest asphalt content specimens from Step 2 that provide stabilometer values of 37 or more. Enter 5.0 and 5.5 on Step 3 of the pyramid.
(e) The asphalt content of 5.3 percent determined in (b) has 4.0 percent air voids and is within the asphalt range of specimens A and B listed in Step 3 of the pyramid. Enter 5.3 in Step 4 of the pyramid as the optimum binder content for the job mix formula.
(f) Because the OBC of 5.3 percent asphalt content reported in Step 4 is within the range of tested binder contents, no additional specimens are required.
(g) At the OBC of 5.3, compact three additional Hveem specimens and determine the final values for Hveem stability and volumetric properties as the average of each of the three test values.

FIGURE 8.17 ***Procedures for Selecting Design Asphalt Content, Hveem Method of Design***



*Design asphalt content is not valid if the maximum asphalt content used in the design series (Step 1) is the asphalt content arrived at in Step 4. In this event, additional specimens should be prepared with increased asphalt content in 0.5 percent increments and a new analysis performed.

8.9

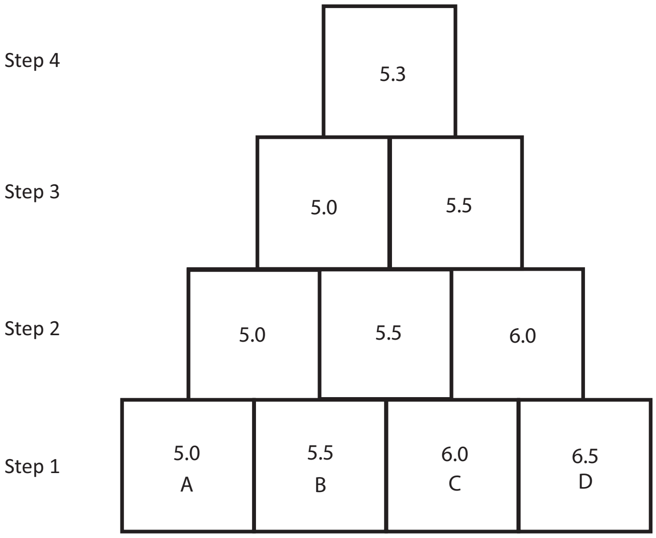
Daily calibration of the Hveem stabilometer

8.9.1 Adjusting the exact quantity of air in the system

The stabilometer reading is influenced by mixture displacement in the stabilometer, i.e., the lateral expansion of the specimen during the test. In order for the Hveem stabilometer readings to be correct and the S-value to be representative of the resistance to deformation of the mixture, it is very important for the Hveem stabilometer to contain the exact quantity of air required, as well as no air bubbles in the oil reservoir behind the rubber bladder. Therefore, frequent checks and adjustment should be made to assure the exact quantity of air is present in the stabilometer. The displacement (when testing the incompressible metal specimen) must be maintained at a constant value of 2.00 ± 0.05 turns (6.6 cm^3 [0.4 in^3]) by using this procedure. Variations in displacement are caused by entrained air and the failure of the diaphragm to fit tightly against the specimen after the initial pressure of 34.5 kPa (5 psi) has been applied. To check the displacement in the stabilometer, follow these steps:

(a) On models with a valve, check displacement pump by closing pump valve and turning the hand-wheel until tight. If more than one-eighth turn is required to bring the hand-wheel tight,

FIGURE 8.18 ***Determination of the Design Asphalt Content for a Heavy-Traffic Category Pavement***



the pump contains air that must be removed. (Late model stabilometers do not have an angle valve.)

(b) Place the stabilometer on the adjustable stage and insert a calibration cylinder of steel or brass, that's 101.6 mm (4.0 in.) in diameter by 139.7 mm (5-½ in.) high.

(c) Tighten clamp at base of stabilometer, place stabilometer on compression machine and apply a confining load of 0.45 kN (100 lb.) to top of the calibration cylinder.

(d) Open angle valve, turn displacement pump handle to right and force fluid into the cell until the stabilometer test gauge registers a horizontal pressure of 34.5 kPa (5 psi).

(e) Set the displacement pump dial gauge to zero with adjustment screw, and turn the displacement pump handle to the right (clockwise) at the rate of two turns per second until pressure on stabilometer test gauge reads 690 kPa (100 psi).

(f) Record displacement as the number of pump handle revolutions required to increase the stabilometer test gauge from 34.5 kPa (5 psi) to 690 kPa (100 psi). (2.5 mm [0.1 in.] dial reading is equivalent to one revolution.)

(g) The stabilometer test gauge should remain stationary at 690 kPa (100 psi). If it visibly decreases, there is a leak that must be located and sealed.

(h) Displacement volume in cm^3 (in^3) may be determined by multiplying the number of revolutions by 3.3 (0.2).

(i) If the displacement does not check within ± 0.05 turns of the required 2.00 (6.6 cm^3 [0.4 in^3]), an adjustment is necessary. Air is added to the system to increase the displacement or removed from the system to decrease the displacement.

(j) To adjust displacement to the required value, turn pump handle to the right (clockwise) until a pressure of 690 kPa (100 psi) is obtained on test gauge, then turn pump handle to the left (counter-clockwise) two turns. Either add or remove air until a pressure of 34.5 kPa (5 psi) is indicated on test gauge. Air may be added to air cell by means of the rubber bulb provided for this purpose. Continue to check and adjust until the required value of displacement is obtained.

(k) Release pressure and remove the calibration cylinder.

8.9.2 Diaphragm installation, refilling the liquid and calibration

For such matters as replacing the diaphragm, calibration of the stabilometer's pressure gauge and other parts, and maintenance, see the equipment users' manual and/or Caltrans Test Method CT 102.

CHAPTER 9

Moisture Sensitivity of the Mix

Moisture damage of asphalt pavement is a serious issue. Measurement of the sensitivity of a specific mix to moisture is a challenge, as are interpreting the results.

Treatment options vary with location, and success varies. Recent efforts have concentrated on developing better laboratory test methods to predict moisture damage problems in the field. To fully address the moisture damage problems, it is important to address both the chemical factors and the mechanical factors responsible for moisture damage. The chemical factors affect the interaction of asphalt molecules with the aggregate surface and how the interaction changes in the presence of water. The physical issues are mainly concerned with ways to reduce the access of water to the asphalt aggregate interface.

In addition to careful selection and quality control of materials, precautions should be taken in design and construction to keep moisture out of the pavement by providing proper drainage for water and good compaction of the asphalt mixture to minimize air voids and pavement permeability.

This chapter provides an overview of the mechanisms and offers some insight into the issue of moisture sensitivity as it applies to mix design.

For more information on moisture sensitivity, refer to the Asphalt Institute's MS-24 Moisture Sensitivity publication.

9.1

Possible mechanisms of moisture damage

The most predominant condition causing stripping is when repeated traffic loadings occur on a poorly compacted dense-graded pavement (with >8% air voids) with surface water (rain) present. The hydraulic pressure fluctuations in the interconnected voids will cause a scouring effect, in essence ripping the asphalt binder from the aggregate surface. While adhesion failure between the asphalt and aggregate (referred to as stripping) is the most commonly recognized mechanism of moisture damage, there are others. Other mechanisms include moisture-induced cohesion failures within the asphalt binder, cohesion failures within the aggregate, emulsification of the asphalt and freezing of entrapped water.

9.1.1 Adhesion failure between asphalt and aggregate (stripping)

Stripping is the process where the bond between the asphalt and aggregate is broken by the action of water. This process is based on a chemical phenomenon where water molecules alter the interactions between the asphalt and the aggregate surface.

Aggregates have a higher affinity for water than for asphalt and are more readily wetted by water than asphalt. Adhesion in asphalt and aggregate interaction is generally defined as the formation of a chemical bond between asphalt and the aggregate. Water will penetrate through asphalt by osmosis, and the rate will depend on the thickness of the asphalt film and the pressure difference across the asphalt film. In general, softer grades are more permeable to water because of lower viscosity.

9.1.2 Cohesion failure within asphalt

The bond between asphalt molecules is described as cohesion and it is possible that this bond can be broken. If water is trapped in air voids in the asphalt pavement and the temperature of the pavement increases, the evaporation of the water builds enough pressure to rupture the asphalt film, leading to the disruption of the bond between the asphalt molecules.

9.1.3 Cohesive failure within the aggregate

Some researchers propose that cohesive failure within the aggregate (breaking of the aggregate particle by the action of water) could cause moisture damage. Research suggests that the amount of moisture damage by this phenomenon is directly related to solubility of the aggregates.

9.1.4 Spontaneous emulsification

Spontaneous emulsification could possibly be a reason for stripping, especially when mineral clays or certain types of asphalt additives are present. This mechanism will weaken the asphalt-to-asphalt bond due to the presence of water in the asphalt. Because emulsification of asphalt requires high temperatures and high shear as found in a colloidal mill, it would be very unusual for this kind of emulsification to happen in a hot mix pavement.

9.1.5 Freezing

Another form of moisture damage is entrapped water that freezes in asphalt pavement and can expand nine percent. This expansion of water entrapped in the voids can significantly damage the asphalt pavement.

9.2

Influence of physical properties

In addition to chemical incompatibilities, there is a need to address the physical mixture properties of the ingredients and the final mixture.

9.2.1 Mix design

Mixture design is the process of determining the optimum proportions of aggregate and asphalt binder to meet the desired performance criteria. The stability and durability of the asphalt mixture are the two primary characteristics that are determined in the mixture design process. The components in the asphalt mixture and the design of the asphalt mixture contribute to the resistance of the mixture to moisture damage. Once the design is completed, the mixture is typically tested for moisture susceptibility.

If the mixture is determined to be susceptible to moisture, one or more of the following may be used:

- A new aggregate/asphalt binder combination may be selected.
- The component proportions can be changed.
- An antistripping treatment may be applied.

If any of these options are performed, then the mix design should be reverified. Changing any of the mixture components/proportions or adding ingredients/treatments may alter volumetric properties. The liquid additives are considered an ingredient in the asphalt binder, and hydrated lime or polymeric treatment is considered an ingredient in the aggregate. The moisture sensitivity testing should be repeated with the final components, proportions, ingredients and treatments to confirm that all requirements are met.

9.2.2 Effects of dust and clay

The presence of a dust coating on an aggregate can greatly interfere with the adhesion of the asphalt binder to the aggregate. The asphalt binder will coat and adhere to the dust and not adequately bond with the aggregate particles. The dust coating can act as a bond breaker, causing detachment of the asphalt binder film from the aggregate surface. The reduction or elimination of excessive dust or dirt on the aggregate will create an environment for proper asphalt binder adhesion.

In addition to the concerns that dust weakens asphalt-to-aggregate bonds (adhesion), there is also concern that clay promotes the inclusion of water within the mixture, leading to weakened asphalt-to-asphalt bonds (cohesion). The amount of plastic fines or clay is inferred from the Plasticity Index (PI). The PI is determined from the Atterberg

Limit tests (ASTM D4318 or AASHTO T 90) run on the combined blend's minus 40-sieve material. High quality mixes should contain nonplastic aggregates. Noncritical applications may use aggregates having a PI of less than 4 percent. The amount of clay present in an aggregate may be determined by the sand equivalent test (ASTM D2419 or AASHTO T 176). The sand particles are separated from the clay particles in a graduated cylinder with a calcium chloride solution. After settling, the height of the sand and the height of the clay are measured, and the sand equivalent value is the ratio of the sand height to the total settlement height. A sand equivalent ratio or value of 40 percent or greater is desired.

The methylene blue value is another test that is used for determining the clay content of an aggregate blend. The methylene blue value can be determined by following ASTM C837 or AASHTO T 330.

A solution of methylene blue dye is added to the minus 200-sieve portion of an aggregate until it is saturated. The sample is placed on a piece of filter paper. The dye spot on the filter paper establishes the visual determination of a saturation or end point. The methylene blue value is reported as the amount of methylene blue required per gram of minus 200 material to generate the end point. The higher the methylene blue value, the greater the amount of clay present in the sample.

9.3

Test methods for evaluating the moisture sensitivity of mixes

There are a number of methods for evaluating the moisture sensitivity of a mix. Different jurisdictions have used various methods with success. The effectiveness of a particular method is dependent on the local conditions and the local materials.

9.3.1 Indirect tensile tests

The most common test procedures for evaluating the moisture sensitivity of an asphalt mixture are the indirect tensile test procedures, AASHTO T 283 and ASTM D4867, commonly referred to as the modified Lottman and Root-Tunnicliff tests, respectively. In these tests, a minimum of six test specimens are compacted to a target percentage of air voids intended to simulate the expected in-place percentage of 7.0 percent air voids. The Asphalt Institute recommends AASHTO T 283 specimen preparation tolerances of ± 0.5 percent air voids rather than the ASTM tolerance of ± 1.0 percent. The compacted specimens are then separated into two subsets—a conditioned subset and an unconditioned, or control, subset. It is particularly important to assure that the two sets are equivalent in properties and that the range of air void contents within each set is kept to a minimum (typically ± 0.2 percent).

9.3.1.1 Conditioned subset

For the conditioned subset, three compacted specimens are partially saturated by submerging in water and applying a vacuum for a certain period of time. Several variations of conditioning procedures, such as vacuum levels, saturation levels and vacuum times are used by various agencies. Regardless of the initial saturation procedure, the ultimate purpose is to achieve a targeted final saturation level. The Asphalt Institute recommends the AASHTO T 283 specimen saturation range of 70 to 80 percent rather than the allowable range of 55 to 80 percent by ASTM. Following the initial saturation, the conditioned subset of specimens may be subjected to a freeze-thaw cycle (AASHTO T 283) by immediately freezing the specimens at –18°C for a minimum of 16 hours. After the freeze cycle, if used, the partially saturated specimens are placed in a water bath for 24 hours at 60°C. Finally, the partially saturated specimens are placed in a water bath for two hours at 25°C to bring the specimens to the test temperature.

9.3.1.2 Unconditioned subset

For the unconditioned subset, specimens are maintained at room temperature until immediately before the start of the indirect tensile strength test. Immediately before testing, the unconditioned subset of specimens is adjusted to 25°C by either an environmental chamber or a water bath.

9.3.1.3 Indirect tensile strength testing

Indirect tensile strength testing is performed on the conditioned and unconditioned specimens by loading the specimen (see **Figure 9.1**) at a displacement rate of 50 mm (2 in.) per minute. The indirect tensile strength is calculated for

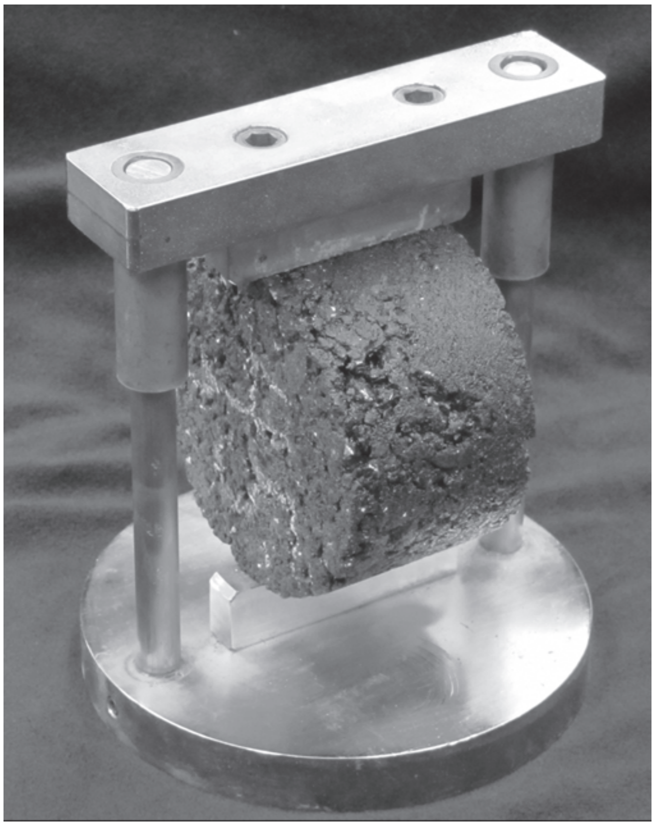


FIGURE 9.1 ***Sample Loaded for Tensile Strength Testing***

each specimen. A Tensile Strength Ratio (TSR) is calculated by dividing the average indirect tensile strength of the three conditioned (wet) specimens by the average indirect tensile strength of the three unconditioned (dry) specimens. According to the original research, a TSR value of 0.80 (sometimes identified as 80 percent) or greater is generally considered acceptable, indicating an asphalt mixture that is not susceptible to moisture damage. By contrast, a TSR value of 0.60 (or 60 percent) or less is generally considered unacceptable, indicating an asphalt mixture that is susceptible to moisture damage.

9.3.1.4 Acceptance criteria

Although a TSR value of 0.80 or greater is considered acceptable, some agencies have chosen to accept TSR values of 0.70 (70 percent) or greater based on their experience. In addition to the minimum TSR value, some agencies also specify a minimum average (wet or dry) indirect tensile strength of the specimens. For the agencies that use a minimum indirect tensile strength, the concern is that a mixture may have insufficient tensile strength to resist deformation under load even if the conditioning process does not result in a significant weakening of the mixture, as measured by the tensile strength of the unconditioned specimens. In other words, is an asphalt mixture with a TSR value of 0.80 (and a conditioned tensile strength of 280 kPa [40 psi]) equally acceptable to an asphalt mixture with a TSR value of 0.80 (and a conditioned tensile strength of 560 kPa [80 psi])? It can be argued that if the intent of TSR determination is simply to identify the moisture susceptibility of an asphalt mixture, then the answer to this question is yes. State highway agencies that have minimum tensile strength requirements are using the requirement to assure a minimum strength as a surrogate performance property that is not necessarily related to the moisture susceptibility of the asphalt mixture.

9.3.2 Boil test

Some agencies use the boil test (ASTM D3625) as a quick and easy visual indication of chemical incompatibility between the asphalt binder and aggregate. In this procedure, a sample of loose asphalt mixture is placed in boiling water for 10 minutes and then removed. The extent of retained asphalt coating on the aggregate is then evaluated relative to a nonconditioned sample.

The boil test is an example of a loose mix test that is used to characterize the bonding between the asphalt and aggregate (adhesion). Tests run on compacted samples are used to characterize not only adhesion properties but also the cohesion properties.

This test is often used as a quick screening test with new aggregate sources.

9.3.3 Hamburg Test

Wheel tracking tests, with the compacted sample submerged in water, have become more popular in recent years as a predictor of moisture susceptibility. As shown in **Figure 9.2**, the Hamburg Loaded Wheel Tester (AASHTO T 324) evaluates for rutting and moisture sensitivity in asphalt mixes by using a steel wheel that repeatedly tracks a specimen under water at a specified temperature, typically between 40 and 60°C. During the test, specimen deformation is recorded as a function of the number of loading cycles (see **Figure 9.3**). Many asphalt technologists believe this to be a very severe test for assessing moisture damage

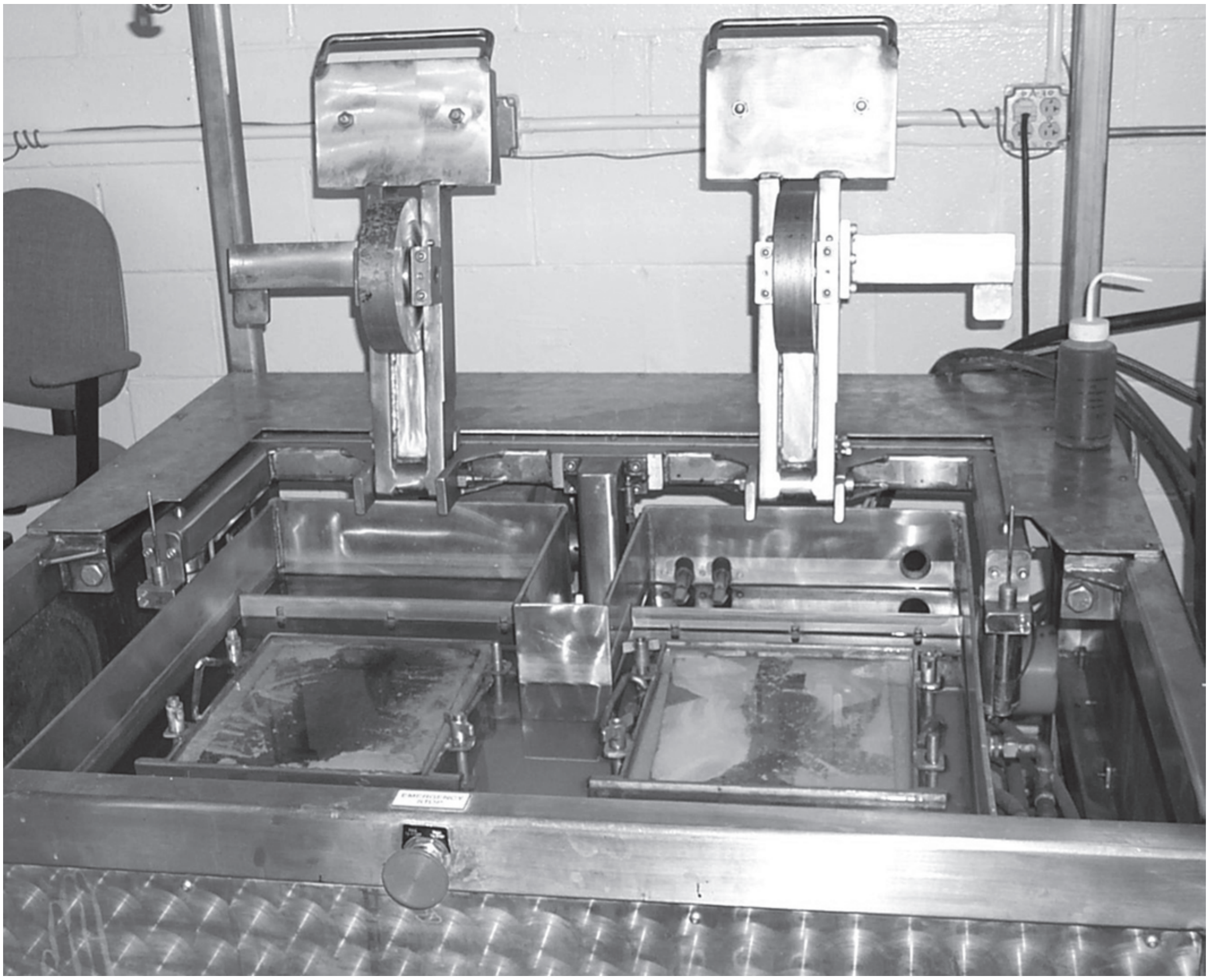


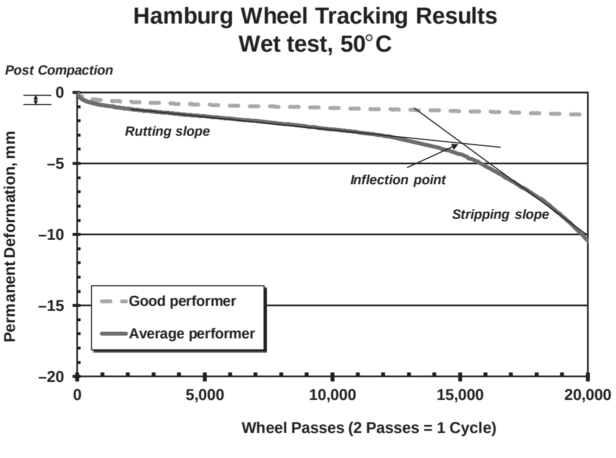
FIGURE 9.2 ***Hamburg Loaded Wheel Tester (Also see Figure 10.4)***

potential. The typical criterion applied by user agencies is a maximum deformation at a specified number of loading passes. Other technologists believe that the stripping slope and stripping inflection point provide some indication of the susceptibility of the mixture to moisture damage.

9.3.4 Immersion-compression test

One of the less common tests is the immersion-compression test (AASHTO T 165 and ASTM D1075). In the immersion-compression test, specimens are compacted to a targeted air void percentage and dimensions of 4 inches in height by 4 inches in diameter (100 mm by 100 mm). Six total test specimens are prepared and separated into two subsets—conditioned and unconditioned

FIGURE 9.3 ***Typical Results from Hamburg Wheel Tracking Test***



(or control). The conditioned subset specimens are placed in a water bath for 24 hours at 60°C. At the end of this conditioning period, the specimens are placed in a water bath for two hours at 25°C to bring them to the test temperature.

For the unconditioned subset, specimens are maintained at room temperature until immediately before the start of the compression test. Immediately before testing, the unconditioned subset of specimens is adjusted to 25°C by either an environmental chamber or a water bath.

Marshall stability testing is performed on the conditioned and unconditioned specimens by loading the specimens at a displacement rate of 5 mm/minute (0.2 in./minute). The ratio between the stability of the conditioned subset of specimens and the unconditioned subset of specimens is the index of retained stability.

9.4

Treatments to improve moisture sensitivity

If moisture susceptibility of the mix is an issue, there are several methods of improving performance.

9.4.1 Selecting an appropriate treatment

Three general methods to reduce the effects of moisture damage include adding liquid anti-strip (LAS) additives to the asphalt, adding hydrated lime to the aggregate and polymeric aggregate treatment. AASHTO T 283 or other designated moisture sensitivity test is utilized to ensure that acceptable improvement in resistance to moisture damage has been achieved.

In order to obtain representative results, it is critical that samples of the actual asphalt binder and the combined aggregate, along with the selected treatment, are used when performing laboratory verification testing.

In general, a loose mix-stripping test such as the boil test (ASTM D3625) can be used as a rapid way to screen the chemical compatibility between the selected treatment and the binder/aggregate combination used in the mix. Specifying agencies have different pass/fail criteria for the test, but in general, a retained coating of at least 95 percent is required. A loose mix-stripping test mostly indicates the chemical compatibility (adhesion) between asphalt and aggregate; whereas the compacted mix stripping tests such as AASHTO T 283 evaluate contributing factors that include adhesion, cohesion and other mixture properties.

9.4.2 Liquid anti-strip additives

Chemical additives known as liquid anti-stripping (LAS) additives have been used to treat moisture damage in asphalt pavements. Some of the chemical classes of amines that are used as LAS additives include fatty amines, amidoamines, imidazolines, etc. They all contain a hydrocarbon that has similar properties to asphalt and one or more amine groups. In addition to the chemistry, there are other properties that characterize LAS additives. These properties include heat stability and storage, concentration, viscosity and odor. LAS additives are often added at the refinery or suppliers terminal if requested. The method and rate of incorporation during the mix design should conform to the recommendations of the binder and LAS suppliers. Regardless of the type of LAS, the actual dosage rate should be determined by testing. There are cases where an increased dose has led to lower test results. For additional information on the various LAS additives and their properties (such as storage stability), see Asphalt Institute's MS-24 Moisture Sensitivity publication.

9.4.3 Hydrated lime added to the aggregate

Hydrated lime (Ca[OH]2), a highly alkaline inorganic powder that has many industrial and environmental applications, improves the adhesive bond between the aggregate and asphalt. In addition, hydrated lime can react with clay on or in the aggregates to minimize the moisture damage caused by the clay particles.

9.4.3.1 Types and dosages

Lime is available as either quick lime or hydrated lime. Because of safety and other concerns in using quick lime, hydrated lime is by far the most common lime used. The amount of hydrated lime needed to improve the moisture sensitivity of a hot mix asphalt generally ranges from 0.5 to 1.5 percent by dry weight of aggregate.

The addition of hydrated lime has been known to impact volumetric properties in the mixture. It is recommended that the manner of introducing lime during the mix design process be similar in nature to the method used in the field during plant mix production. It is also recommended that the mix designer optimize the amount of lime to be added to the mix. The amount of lime required will depend on the binder and aggregate combination utilized in the mix. The minimum amount of lime that enables the design to meet the moisture sensitivity requirements will minimize the impact of the lime on the final volumetric properties of the mixture.

9.4.3.2 *Methods of introducing hydrated lime*

Several methods are commonly used to introduce hydrated lime into the asphalt mixture. Below are descriptions of two methods commonly used throughout the United States during the mix design process.

9.4.3.2.1 *Dry lime on aggregate*

This method of adding dry hydrated lime is the simplest and generally the least costly during field production. A predetermined amount of lime is added to the aggregate prior to the introduction of the binder. Consequently, a portion of the lime reacts with the aggregate while a portion reacts with the asphalt.

9.4.3.2.2 *Lime slurry on aggregate*

Lime slurry is produced by mixing approximately 3 parts water to 1 part lime by mass and then mixing with the batched aggregates. The treated aggregate is either directly placed into the oven or allowed to "marinate" for a period of time to allow the lime to react with the aggregate surface and any clay that may be present on the aggregate.

9.4.4 *Polymeric aggregate treatments*

The Polymeric Aggregate Treatment (PAT) wets the aggregate with an SBR latex formulation, which dries to provide a water-resistant coating around the aggregate, bonding well with both the aggregate and the asphalt. PAT is an effective anti-strip agent because of its ability to adhere to a wide range of aggregates, while improving the aggregate's affinity for asphalt.

The polymer is specially selected to have compatibility with the asphalt and to enhance the aggregate coating and asphalt bonding. Designers utilizing PAT are encouraged to closely follow the manufacturer's guidelines for incorporation into the mixture.

9.5

Practices to minimize moisture damage

When constructing asphalt pavements, certain precautions will help minimize the moisture sensitivity of the finished product. The causes of moisture damage are not solely confined to the material and mix design process. Inadequate construction practices concerning stockpiles, asphalt plants, lay down operation and pavement drainage can also be the source of moisture damage. See Asphalt Institute MS-24 for more information. Sufficient compaction should be achieved to make the mat impervious.

CHAPTER 10

Mixture Performance Testing

Performance testing of HMA mixes is conducted to estimate the performance of HMA prior to use on a transportation facility and to evaluate new materials and design tools to improve the performance of HMA pavements. Different tests and loading configurations are used to address different distresses. The intent of this chapter is not to address every available performance test and test condition, but to provide a general overview of mixture performance tests that can be used to address specific pavement distresses.

10.1

Performance tests to address permanent deformation (rutting)

Permanent deformation or rutting of asphalt mixtures is a distress that occurs at high pavement temperatures under loaded conditions. As the pavement temperature increases, the asphalt mixture becomes softer and is more susceptible to movement under load. Permanent deformation occurs when the asphalt mixture deforms under load and then does not recover to its original, undeformed position. Over time, permanent deformation can lead to channelization or rutting (see **Figure 10.1**).

FIGURE 10.1 ***Rutting Shown at an Intersection***

Laboratory tests to assess rutting performance are conducted at high temperatures intended to represent the in-service temperature experienced by the asphalt mixture in the summer months. Simple tests that measure the stiffness or strength of the asphalt mixture have been used in the past—such as the Marshall stability and Hveem stabilometer tests (addressed in previous chapters). Each of these tests is conducted at 60°C (140°F)—a temperature generally representative of the maximum in-service pavement temperature—and is intended to provide an indication of the asphalt mixture's ability to resist deformation under load.

As the desire for mixture performance testing grew and technology advanced, other mixture performance tests became available and standardized for use by asphalt mix design technologists. Although mixture stiffness at high temperatures is considered important, most of the rutting tests commonly used involve some type of repeated loading. These tests are discussed in the following sections.

10.1.1 Loaded wheel tests

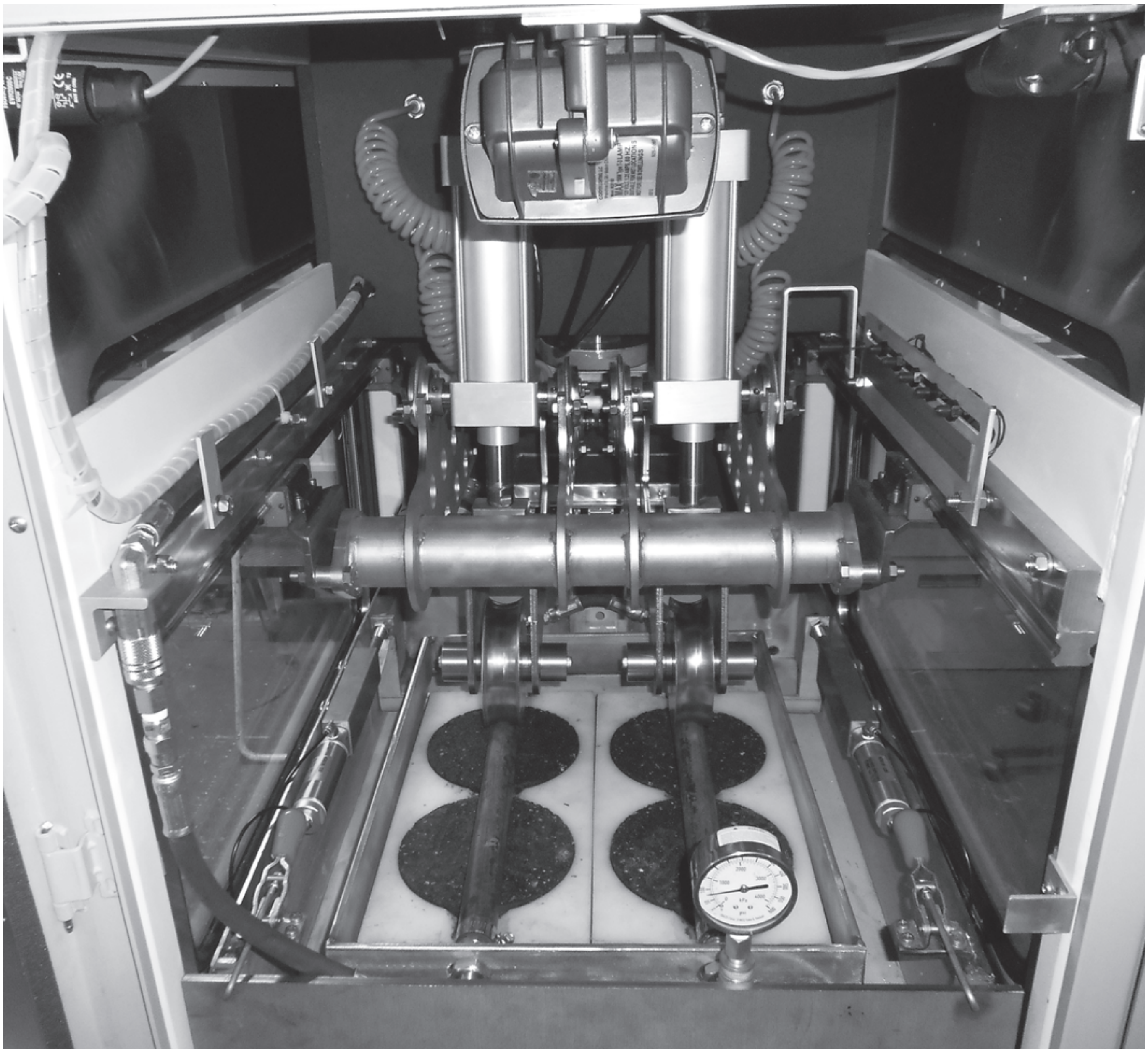
Loaded wheel testing is a common method used by some designers to evaluate the rutting susceptibility of an asphalt mixture. Although there are many different types of loaded wheel tests, with different loading configurations and test conditions, the principles of the test remain essentially the same.

In a loaded wheel test, a wheel runs over an asphalt mixture specimen at an elevated temperature in a reciprocating manner. With each load cycle, a certain amount of deformation is created in the asphalt mixture specimen. After a specified number of load cycles, the permanent deformation (rutting) in the asphalt mixture is determined and compared to established criteria to determine the rutting susceptibility of the asphalt mixture.

10.1.1.1 Asphalt Pavement Analyzer

A commonly used loaded wheel test is the Asphalt Pavement Analyzer (APA). The APA test equipment is a modification of the Georgia Loaded Wheel Tester (GLWT), which was developed in

FIGURE 10.2 ***Asphalt Pavement Analyzer (APA)***



Traffic (10^6 ESALs)	Maximum Rut Depth (mm)
<3	–
3 to <10	5
10 to <30	4
≥30	3

TABLE 10.1 ***Suggested APA Criteria (Standard Test, High PG Temperature)***

a cooperative study of the Georgia Department of Transportation and the Georgia Institute of Technology in 1985. While it has been used primarily to evaluate the rutting resistance of an HMA mixture, studies have been conducted to evaluate the use of the equipment for evaluating fatigue and moisture resistance of HMA mixtures. The APA is shown in **Figure 10.2**.

In normal operation, a loaded wheel is placed on a pressurized hose, which sits on the asphalt mixture test specimens, and then tracked back and forth to induce rutting. Samples can be tested dry or while submerged in water. For more details on the standard test procedure, refer to AASHTO T 340, "Determining Rutting Susceptibility of HMA Using the Asphalt Pavement Analyzer (APA)."

The test specimens for the APA can be either beam or cylindrical specimens. Beam samples are generally compacted to a target air void content of 7 percent using a rolling wheel, linear kneading or vibratory compactor. Cylindrical specimens are usually made with the Superpave gyratory compactor to a target air void content of 4 or 7 percent. In addition, cores from an in-service roadway can also be tested.

The test temperature that is used is selected based on the expected high temperature of the asphalt mixture in service or the high temperature grade of the "standard" asphalt binder used by the agency (i.e., the asphalt binder grade used for pavements with 3 million ESALs or more). A standard test temperature of 64°C is often used. Asphalt mixture specimens are tested to 8,000 cycles or another defined endpoint.

In the standard test, the wheel load is 100 pounds (445 Newtons) and the hose pressure is 100 psi (690 kPa). **Figure 10.3** shows typical results from the APA test. As can be seen, specimens deform rapidly at the beginning of the test, but then the amount of permanent deformation per cycle decreases and becomes quite steady after a certain number of load cycles.

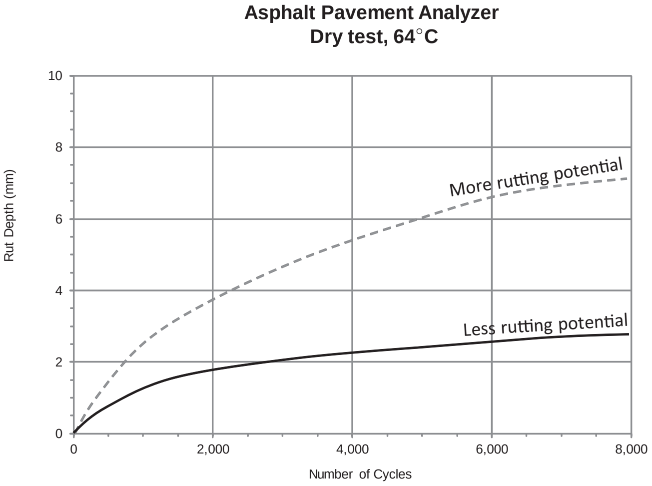


FIGURE 10.3 ***Typical Results from APA Test***

Although the criteria may vary depending on actual test conditions, acceptable criteria that have been established are shown in the above table (see **Table 10.1**) using standard test conditions (100-lb. load, 100 psi hose pressure, 8,000 cycles) with specimens having 7 ± 0.5 percent air voids.

10.1.1.2 Hamburg wheel tracking

Another commonly used loaded wheel test is the Hamburg wheel-tracking (HWT) test. The HWT equipment was developed in Germany in the 1970s to evaluate the rutting and stripping susceptibility of asphalt mixtures. During HWT testing, a loaded steel wheel rests directly on the asphalt mixture test specimens, and then is tracked back and forth to induce rutting. Samples can be slabs or cylindrical, cut from the field or compacted in the lab. Testing can be on dry samples or samples submerged in water. Many asphalt technologists consider wet testing to be an indicator of stripping susceptibility rather than rutting susceptibility. For an evaluation of only the rutting susceptibility of the mixture, the test should be performed under dry conditions. For more details on the standard test procedure, refer to AASHTO T 324, "Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)."

In the standard test using submerged specimens compacted to 7 percent air voids, a steel wheel (of specified dimensions) with an applied load of 705 Newtons (158 pounds) rolls across the surface

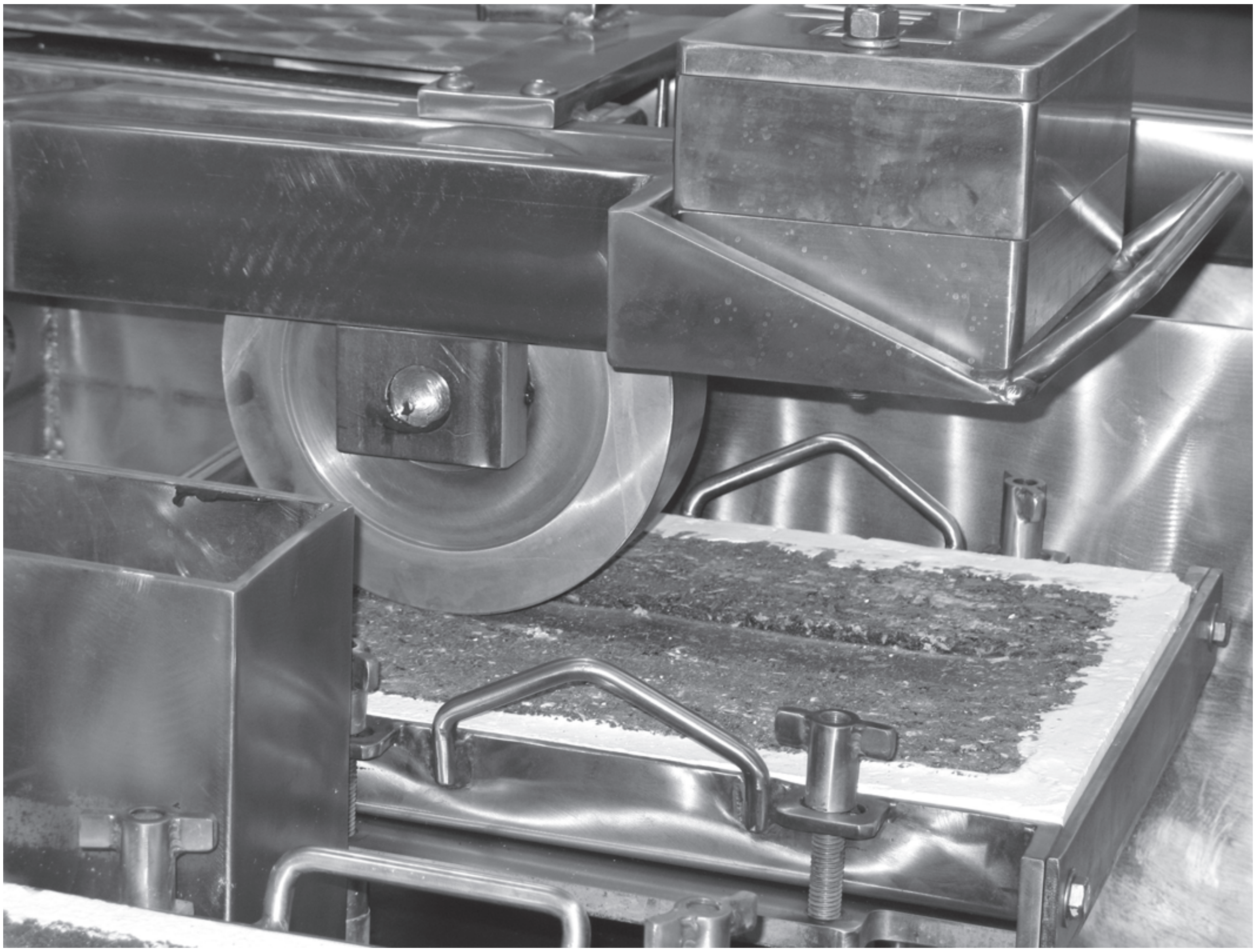
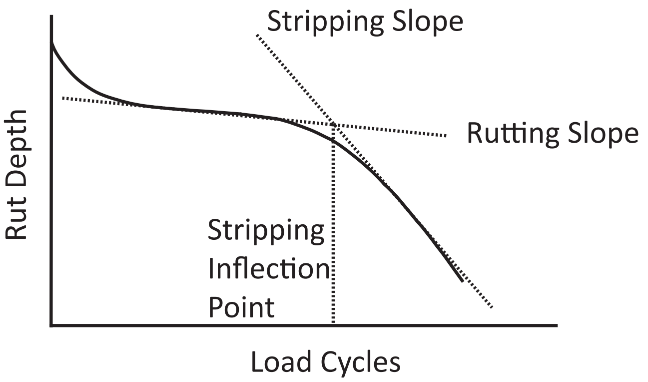


FIGURE 10.4 ***HWT Equipment***

of the asphalt mixture specimen. The standard test temperature is 50°C. Other test temperatures could be selected depending on project conditions. The test is run for 10,000 cycles (20,000 passes) or until a deformation of 20 millimeters is reached. The HWT equipment is shown in **Figure 10.4**.

Figure 10.5 is a sample of the typical data obtained from a submerged HWT test. Three parameters are identified on the curve: the rutting slope, the stripping slope and the stripping inflection point. The rutting slope occurs before the onset of stripping and is the inverse of the rate of deformation in the linear range of the deformation curve. This portion of the curve is where rutting will occur due to consolidation and plastic flow. The stripping slope is related to the severity of the damage due to moisture. It is the inverse of the rate of deformation in the linear region after the onset of the stripping to the end of the test. The stripping inflection point is related to the resistance of the asphalt mixture to the effects of the moisture. This point is determined at the number of passes at which the rutting slope and the stripping slope intersect.

FIGURE 10.5 ***Typical Submerged HWT Results***



Criteria differ widely among user agencies. A typical criterion for conventional dense-graded mixtures under standard test conditions is a maximum rut depth of 12.5 millimeters after 10,000 passes (5,000 cycles). It is not uncommon for users to modify the criterion to account for different traffic loads, binder types or environmental conditions. The rut depth to failure is usually held at 12.5 mm, but the test conditions may be altered. For example, for a given location comparing a surface course using a 64-22 or a 76-22 binder, the number of passes to failure may be doubled or the test temperature increased to account for the stiffer binder. Significant research may be needed to formulate a table of criteria for a particular location.

10.2

Repeated load creep tests on viscoelastic materials

Many asphalt mixture tests provide empirical test results, which are useful in differentiating between good and poor performance. Tests such as the Asphalt Mixture Performance Tester (AMPT) and the Repeated Shear Constant Height (RSCH) test can provide more fundamental engineering properties which can be used in pavement structural design.

Purely elastic materials deform under load and return to their original shape when the load is removed. Their fundamental engineering properties are the Young's modulus or modulus of elasticity (E) determined by axial loading of the material, and the shear modulus (G) determined by shear loading the material.

Viscoelastic materials such as asphalt, however, have a more complicated response to loading, including some non-recoverable permanent deformation. The moduli that characterize a viscoelastic material are therefore the complex modulus of elasticity E^* (pronounced E-star) and complex shear modulus G^* (pronounced G-star). The complex modulus E^* is defined as a complex number that relates stress to strain for a linear viscoelastic material subjected to a sinusoidal loading. G^* is defined in the same manner but with shear loads. To determine useful values for E^* and G^*, the tests are conducted at conditions of temperature and time of loading similar to conditions encountered by the pavement. Many performance tests use repeated step loading at a constant temperature. Depending on the particular test method, different names are given to these moduli, such as resilient modulus (M_r), dynamic modulus (E^*) and stiffness modulus (S).

The dynamic modulus test is specified in ASTM D3497, "Standard Test Method for Dynamic Modulus of Asphalt Mixtures." The dynamic modulus varies with the loading frequency. A frequency that most closely simulates the actual traffic should be selected for the test so that the dynamic modulus is equivalent to the resilient modulus for design purposes.

10.2.1 Asphalt Mixture Performance Tester

The Asphalt Mixture Performance Tester (AMPT) is a relatively new piece of laboratory equipment that can perform uniaxial testing of an asphalt mixture specimen. **Figure 10.6** shows the AMPT with the specimen loaded in the test chamber.

10.2.1.1 Flow number

One of the tests considered as a laboratory performance test for evaluating rutting potential is the AMPT flow number test. The flow number test is a repeated-load uniaxial test—or repeated load creep test—that is performed at temperatures representative of the high pavement temperature

FIGURE 10.6 ***AMPT (Environmental Chamber Raised, Instrumented Specimen Loaded)***

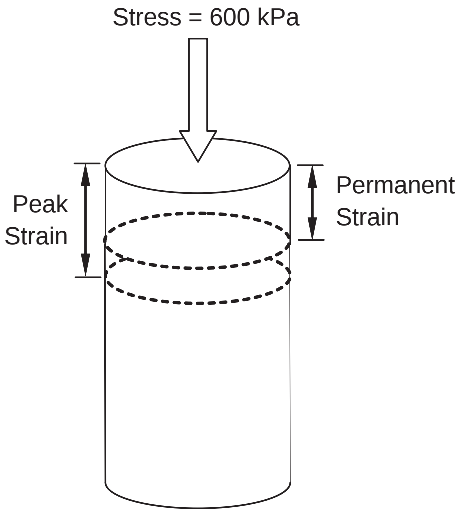


FIGURE 10.7 ***Graphic of a Core Loaded and What Strain is Measured***

expected at the project location where the asphalt mixture will be used. Although selection of the appropriate test temperature is subject to interpretation by users, a common guideline for a surface course is to choose the average 7-day maximum pavement temperature at a depth of 20 millimeters. This is the “50 percent reliability” high pavement temperature that can be determined using the LTPPBind software Version 3.1.

As shown in **Figure 10.7**, the test is conducted by applying an axial load of 600 kPa to an asphalt mixture specimen that is 100-mm diameter by 150-mm height for 0.1 seconds, followed by a 0.9-second rest period—which constitutes one load cycle. The loading cycle is repeated for a maximum of 10,000 cycles while measuring the permanent axial strain ε. In most cases, the test is performed without subjecting the asphalt mixture specimen to confining pressure. However, in some cases, confining pressure may be used. Further details can be found in AASHTO TP 79, “Determining the Dynamic Modulus and Flow Number for HMA Using the Asphalt Mixture Performance Tester (AMPT).” A typical test response is shown in **Figure 10.8**.

In **Figure 10.8**, each loading cycle results in a corresponding permanent axial strain, ε_n. By graphing the accumulated deformation as a function of loading cycles, it is possible to see the development of the permanent strain from the linear region until tertiary flow (failure) begins. This development is shown in **Figure 10.9**.

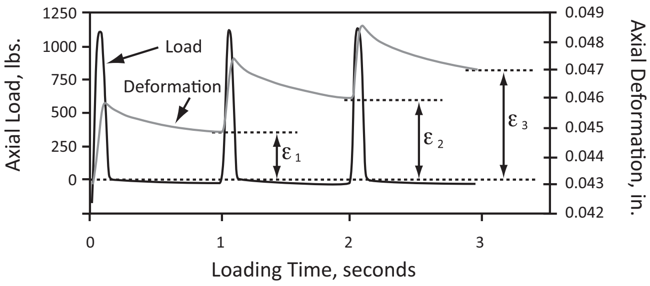


FIGURE 10.8 ***Repeated Load Axial Test***

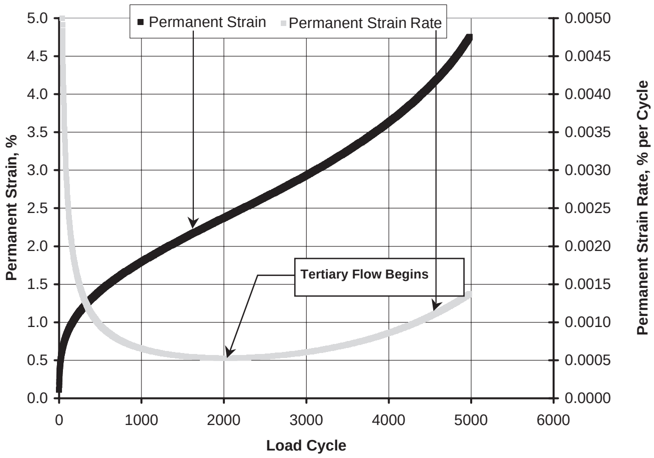


FIGURE 10.9 ***Development of Permanent Strain in a Repeated Load Test***

Traffic (10^6 ESALs)	Minimum Flow Number (Cycles)
<3	–
3 to <10	53
10 to <30	190
≥30	740

TABLE 10.2 ***Suggested AMPT Flow Number Criteria***

The onset of tertiary flow is considered the failure point and generally coincides with the flow number, which is determined as the number of cycles when the rate of change of permanent axial strain reaches a minimum (see **Figure 10.10**).

10.2.1.2 AMPT criteria

Although the criteria may vary depending on actual test conditions, recommended criteria for flow number are provided in **Table 10.2**. These criteria can be found in NCHRP Report 673, “A Manual for Design of Hot Mix Asphalt with Commentary” and have been established using standard test conditions (600 kPa axial stress, 0.1-second pulse loading, 0.9-second rest period, 10,000 cycles maximum) with specimens having 7 percent air voids.

10.2.2 Repeated shear test at constant height

The Repeated Shear Constant Height (RSCH) test was a test that was developed at the University of California-Berkeley as part of the SHRP project evaluating tests for asphalt mixture performance. The test is very similar in concept to the AMPT flow number test, in that it is a repeated-load test, but uses the Superpave Shear Tester (SST), shown in **Figure 10.11** and **Figure 10.12**. Testing is performed at temperatures representative of the high pavement temperature expected at the project location where the asphalt mixture will be used. Although selection of the appropriate test temperature is subject to interpretation by users, a commonly used guideline for a surface course is to choose the 7-day maximum pavement temperature at a depth of 20 millimeters as described in 10.2.1.1.

The test is conducted by applying a shear load of 69 kPa to an asphalt mixture specimen that is 150-mm diameter by 50-mm height for 0.1 seconds, followed by a 0.6-second rest period, thus constituting one load cycle. The loading cycle is repeated for 5,000 cycles while measuring the permanent shear strain (ε_p). Further details can be found in AASHTO T 320, "Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester" and ASTM D7312. The test response is very similar to the AMPT flow number test shown in **Figures 10.8** and **10.9**, except that the applied load is in shear, as is the resulting strain. In most cases, the asphalt mixture specimen does not reach the tertiary flow stage by 5,000 load cycles. As a consequence, the most common way of interpreting the data is to consider the maximum permanent shear strain (MPSS) at 5,000 cycles.

FIGURE 10.10 ***Determining Flow Number***

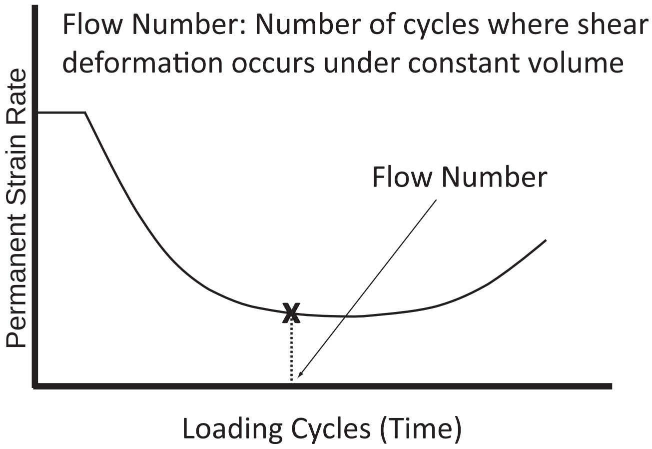


FIGURE 10.11 ***Superpave Shear Tester***

10.2.2.1 RSCH criteria

Recommended criteria for the MPSS from the RSCH test are provided in **Table 10.3**. These criteria can be found in NCHRP Report 673, "A Manual for Design of Hot Mix Asphalt with Commentary," and have been established using standard test conditions (69 kPa shear stress, 0.1-second pulse loading, 0.6-second rest period, 5,000 cycles maximum) with specimens having 3.0 ± 0.5 percent air voids.

10.3

Static creep tests

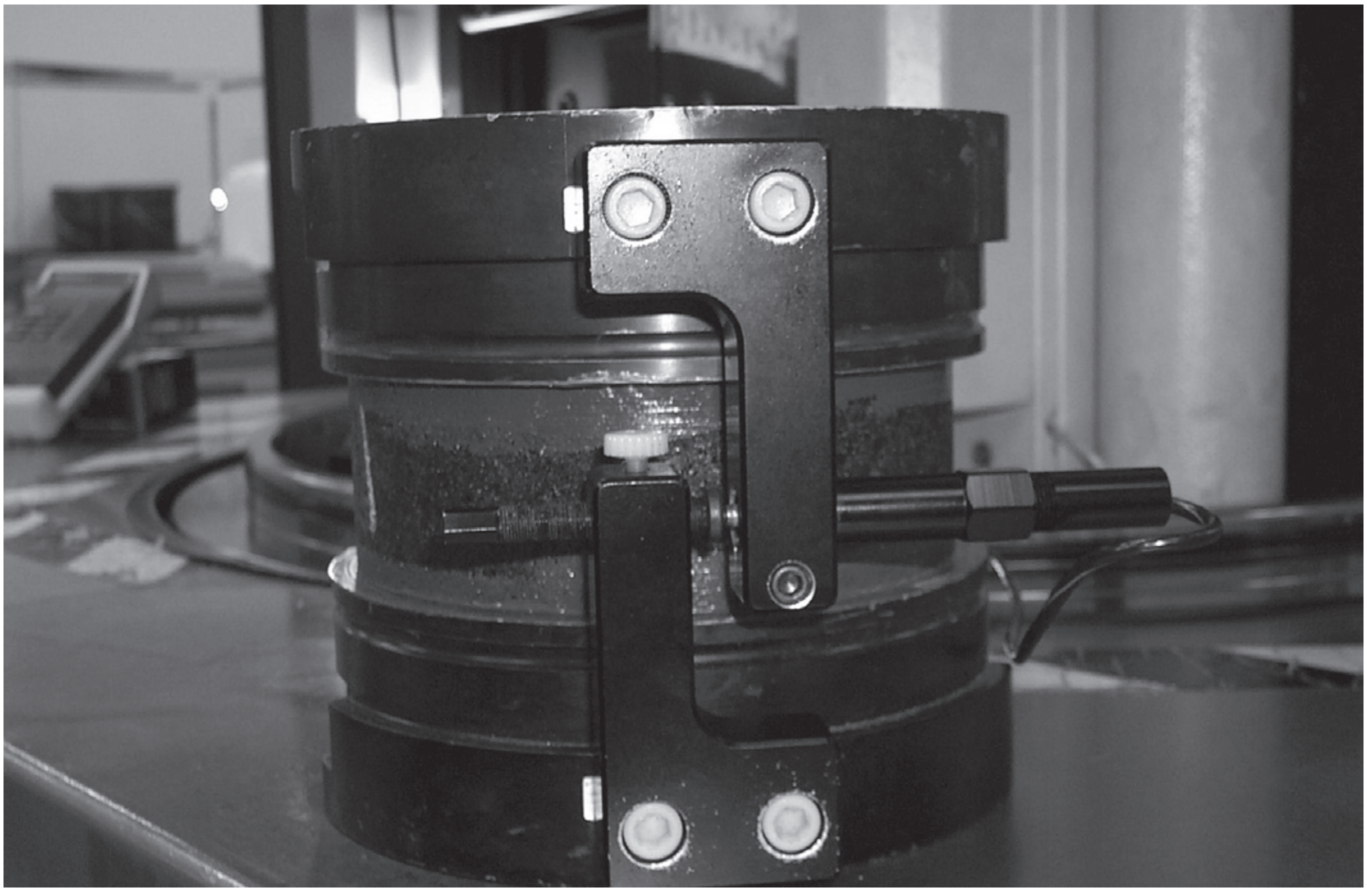
As with repeated load creep and modulus tests, both the AMPT and SST equipment can be used to perform static creep testing in uniaxial (AASHTO TP 79) and shear loading (AASHTO T 320). The AMPT test is referred to as the flow time test. The SST test is referred to as the simple shear test. In both tests, specimens (having the appropriate dimensions depending on mode of loading) are loaded with a single static load—either uniaxially (with AMPT) or in shear (with SST)—at a high temperature representative of the maximum in-service pavement temperature, and the total deformation is measured.

Traffic (10^6 ESALs)	Maximum MPSS (%)
<3	–
3 to <10	3.4
10 to <30	2.1
≥30	0.8

TABLE 10.3 ***Suggested RSCH MPSS Criteria***

In the AMPT flow time test, an axial load is rapidly applied (within 0.05 sec) to the test specimen until a desired axial stress is achieved (usually between 69 and 207 kPa). This axial stress is then maintained until the total axial strain reaches approximately 2 percent (20,000 microstrain) or tertiary flow occurs in the test specimen. The general shape of the axial strain curve is shown in **Figure 10.9**. Flow Time (FT) defines the time when shear deformation begins under constant volume.

FIGURE 10.12 ***Shear Test Specimen (Instrumented)***



Higher values of Flow Time (FT) from the axial static creep test are hypothesized to be related to better rutting resistance in the asphalt mixture.

Recommended criteria for the AMPT FT are provided in **Table 10.4**. These criteria can be found in NCHRP Report 673, "A Manual for Design of Hot Mix Asphalt with Commentary," and have been established using standard test conditions.

Traffic (10^6 ESALs)	Minimum Flow Time (seconds)
<3	–
3 to <10	20
10 to <30	72
≥30	280

TABLE 10.4 ***Suggested AMPT Flow Time Criteria***

10.4

Performance tests to address cracking

Cracking of asphalt mixtures can generally be separated into two categories: damage caused by repeated traffic loading ("load-associated") and damage principally caused by temperature changes ("non-load-associated").

Load-associated cracking is a distress that occurs at all pavement temperatures when the repeated loading of the asphalt mixture due to traffic causes tensile strains to develop that ultimately exceed the tensile strength of the mixture, leading eventually to crack formation and growth. Load-associated cracking is often referred to as "fatigue cracking." In classical fatigue cracking, tensile strains develop at the bottom of the pavement layer as it is loaded. It is there that the tensile strain is the greatest and there that the crack first forms. As repeated loading continues, the crack propagates to the surface.

Cracking may appear as longitudinal cracking (i.e., parallel to the centerline of the pavement), but may eventually interconnect with other cracks to form a distress known as "alligator cracking" (so called because of the appearance similar to the back of an alligator) as shown in **Figure 10.13**.

FIGURE 10.13 ***Load-Associated Cracking (Alligator Cracking)***

Unlike permanent deformation (rutting), asphalt mixtures are more susceptible to load-associated cracking as pavements age and are unable to resist the repeated deflections caused by loading. Although damage can happen at any temperature, classical fatigue cracking is usually modeled at intermediate temperature to represent damage that occurs throughout the year.

In recent years, asphalt technologists have come to recognize that the traditional model of fatigue cracking starting at the bottom of the layer and propagating to the surface is not always the mode of load-associated cracking. Top-down, load-associated cracking has also been noticed in thicker asphalt pavements where the cracks begin at the surface, in the wheel path, and propagate down. Bottom-up fatigue cracking is most often seen in thinner asphalt pavements constructed over a granular base (or other base layer). Top-down cracking, on the

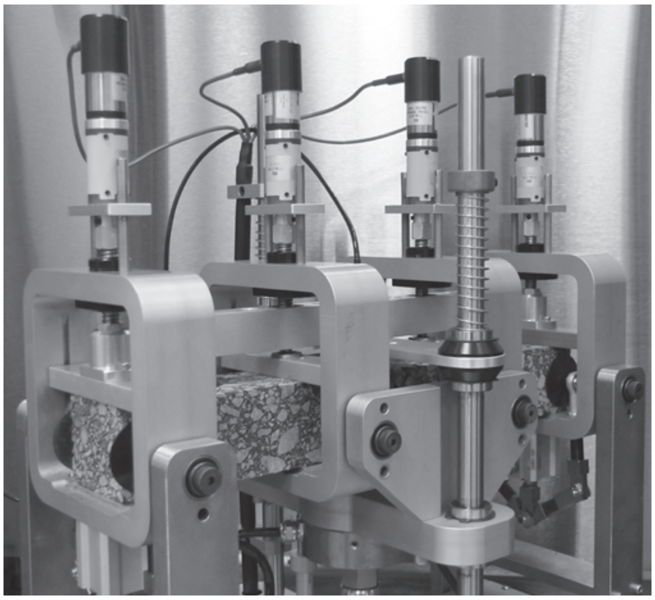


FIGURE 10.14 ***Flexural Beam Fatigue Test Equipment***

other hand, is most often seen in thicker asphalt pavements, or asphalt pavements constructed over a rigid base (i.e., an asphalt overlay of a Portland cement concrete pavement). Top-down cracking may also be associated with increasing stiffness of the asphalt mixture surface as it ages in-service.

Load-associated cracking can be affected by a number of variables other than the asphalt mixture. Pavement structure (thickness) and drainage can be key contributors to the cracking performance of the asphalt pavement, as can poor construction techniques such as segregation and poor compaction (in-place density). Nevertheless, the asphalt mixture can contribute to the success or failure of the asphalt pavement with respect to cracking.

Laboratory tests to assess load-associated cracking performance are usually conducted at intermediate temperatures intended to represent the in-service temperature experienced by the asphalt mixture throughout the year. Simple (and more complex) tests that measure the stiffness or strength of the asphalt mixture have been used in the past—such as the indirect tensile strength (simple) and resilient modulus (more complex). Each of these tests is typically conducted at 25°C (77°F)—a temperature generally representative of the average in-service pavement temperature—and is intended to provide an indication of the asphalt mixture's stiffness and strength under load.

Some tests to address load-associated cracking are discussed in the following sections.

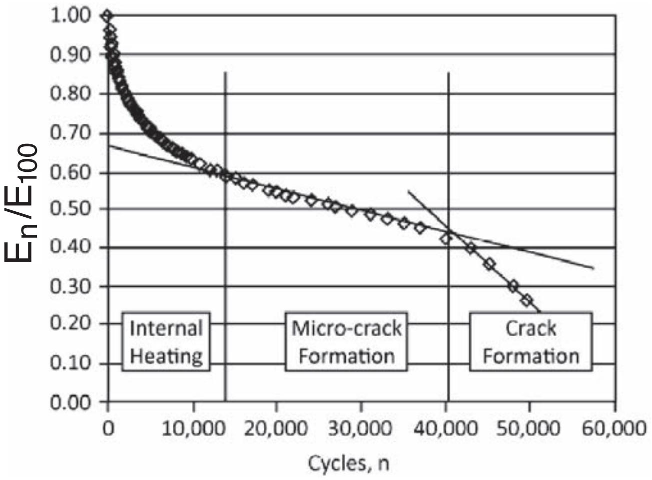
10.4.1 Flexural beam fatigue test

The flexural beam fatigue test is conducted to evaluate the fatigue characteristics of an HMA mixture, independent of other significant factors such as pavement structure (thickness), poor subgrade drainage and method of compaction and placement of the asphalt mix.

The flexural beam fatigue test is conducted by applying a repeated flexural strain to an asphalt beam specimen in four-point flexural loading at 10 Hz and a fixed temperature (usually 20°C) until the specimen stiffness (E_n)decreases to 50 percent of the initial stiffness (E_i). By this point, micro-cracks in the specimen are expected to have formed. The flexural beam fatigue equipment is shown in **Figure 10.14**. A typical test response is shown in **Figure 10.15**. Further details can be found in AASHTO T 321, "Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending" or ASTM D7460.

Unlike cylindrical specimens which are often produced using the Superpave gyratory compactor, beam specimens are most often produced using a rolling wheel or linear kneading compactor. After production, specimens are cut to the desired test dimensions of 63-millimeter (2.5-inch) width by 380-millimeter (15-inch) length by 50-millimeter (2-inch) thick. The target percentage of air voids in the test specimen is usually 7 ± 1 percent to represent the expected in-place density of the asphalt mixture. The asphalt mixture will likely be subjected to short-term conditioning following the procedures in AASHTO R 30 (loose mix conditioning at 135°C

FIGURE 10.15 ***Typical Fatigue Response of an Asphalt Mixture***



for four hours) prior to compaction. Long-term mix conditioning (as described in AASHTO R 30) may also be used.

In a single strain evaluation, two or more specimens are tested at a specified temperature and strain level and the average number of laboratory cycles to failure is compared to a selected criterion. If a range of strain levels is desired, specimens are prepared and tested at multiple strain levels (usually 3–5), likely varying from 250 to 750 µm. The output for this evaluation will look similar to **Figure 10.16**.

The flexural beam fatigue test has also been used to determine the fatigue endurance limit of an asphalt mixture. The endurance limit is a strain level below which the fatigue life of the asphalt mixture is infinite and the pavement will not experience bottom-up fatigue cracking. This is an important concept in the design of long life asphalt pavements. At the endurance limit, the fatigue response shown in **Figure 10.15** will be flat and not reach the crack formation phase.

Unlike some of the tests for assessing permanent deformation, there are no established, or widely recommended, criteria for the minimum number of cycles to failure to indicate good fatigue cracking resistance of an asphalt mixture at this time. Some agencies are specifying fatigue criteria based on their experience with local asphalt mixtures and pavement conditions.

The fatigue characteristics of the mix are input parameters used in more advanced structural pavement design.

10.4.2 Modulus (stiffness) tests

The Resilient Modulus, M_r, test is a repeated load indirect tension and is described in ASTM D7369, "Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test." See **Figure 10.17** for a photo of the test set-up.

The specimen used for the resilient modulus test is a cylindrical specimen that is 100-millimeter (4-inch) diameter by 63.5-millimeter (2.5-inch) thick. The asphalt mixture specimen is normally loaded to a stress level between 5 and 20 percent of indirect tensile strength using a repeated pulse load with subsequent rest period—usually 0.1-second loading with either 0.9, 1.9 or 2.9 seconds rest period. The strain is measured and used along with the applied stress to compute the resilient modulus of the mixture.

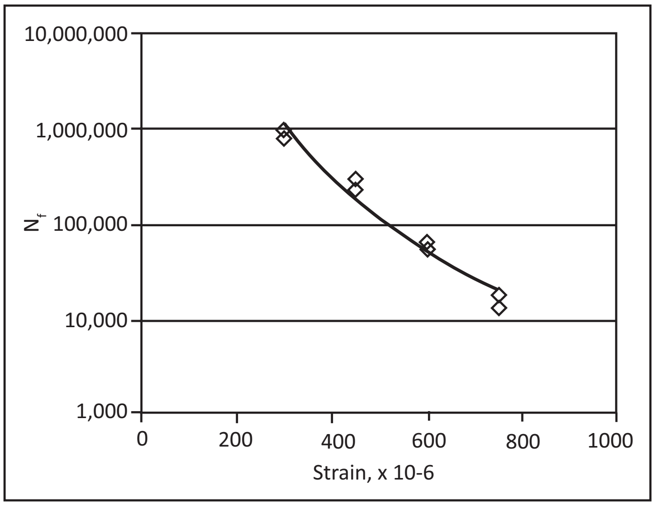
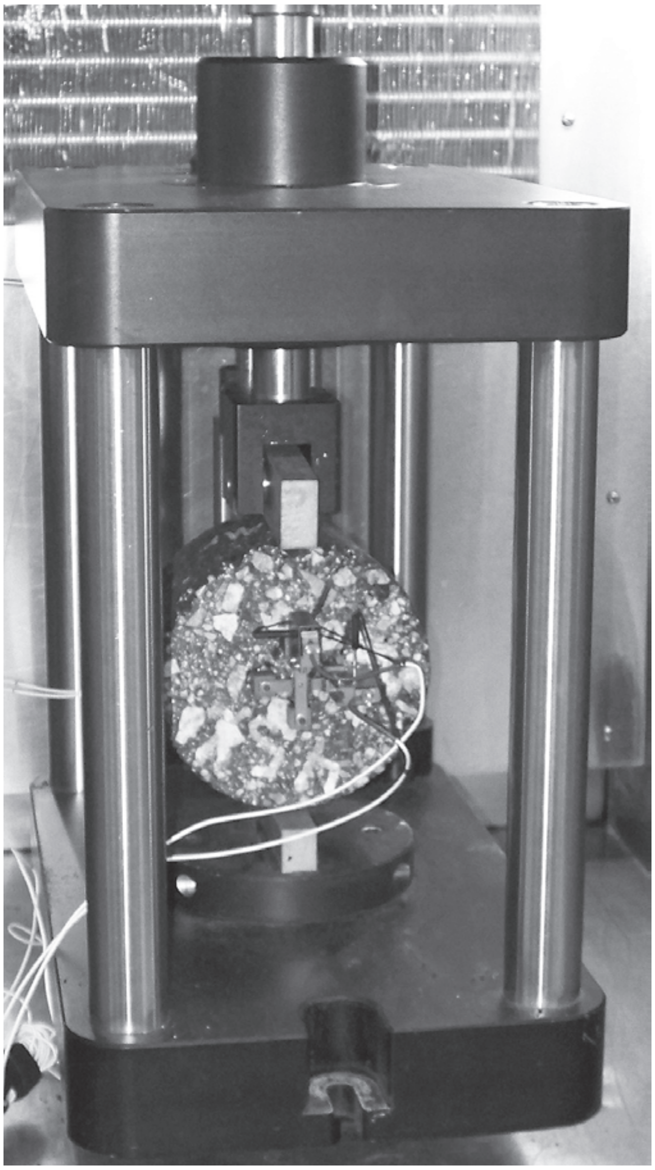


FIGURE 10.16 ***Effect of Strain on Fatigue Life Example***

FIGURE 10.17 ***Resilient Modulus Testing Configuration***



No performance criteria have been definitively established for the AMPT, SST or M_r tests. However, the $|E^*|$ procedure using the AMPT can be used as an input into the Pavement ME Design software. The results of the M_r test have been used in the past as input into layered elastic thickness design procedures and also to determine the structural coefficient for use in thickness design procedures.

10.4.3 Other load-associated cracking tests

Since most of the tests for load-associated cracking have not seen widespread use, there are no established correlations between lab and field performance. Some tests that are currently being evaluated for use include the Dissipated Creep Strain Energy test (University of Florida), Disk-Shaped Compact Tension test (University of Illinois), Semi-Circular Bend test (University of Minnesota and Louisiana State University) and the Texas Overlay Tester (Texas A&M University). Each test is conducted at low-intermediate temperatures and is intended to provide an indication of load-associated cracking resistance. The universities and their affiliated DOTs have, in many cases, established criteria based on the particular test and conditions common to their state, but there has usually been no widespread validation of the test and its criteria.

10.5

Non-load-associated cracking

Unlike load-associated cracking, which can be affected by a number of variables, non-load-associated cracking is affected much more by the asphalt mixture, and specifically, by the asphalt binder properties.

Aggregates, volume of asphalt in the mixture and compaction all play a role in the performance of the asphalt mixture, but for non-load-associated (or thermal) cracking, the asphalt binder properties are much more relevant. As a result, asphalt mixture tests to assess the performance of a mixture with respect to low temperature (non-load-associated) cracking are often not conducted.

FIGURE 10.18 ***Transverse (Thermal) Cracking***

Rather, the mix designer often simply relies on proper selection of the asphalt binder grade to mitigate the potential for low temperature cracking.

Thermal cracking is a distress that occurs at lower pavement temperatures when the increased tensile stress in the asphalt mixture created by decreasing temperature exceeds the tensile strength of the mixture, eventually leading to crack formation and growth. In contrast to classical fatigue cracking, thermal cracking begins at the pavement surface and often appears as transverse cracking (i.e., across the width of the pavement), as shown in **Figure 10.18**.

Although proper binder grade selection is the principal means to mitigate low temperature cracking of asphalt mixtures in the laboratory, there will be instances when mixture testing will be desired. For example, if an asphalt binder contains particles larger than 250 microns (0.25 millimeters)—as is the case for some Ground Tire Rubber modified asphalts—it may not be possible to characterize the asphalt binder properties using the procedures described in AASHTO M 320. In that case, low temperature performance could be evaluated using mix testing. If the mixture contains fibers, then asphalt binder test procedures are not appropriate and mix testing is recommended.

Laboratory tests to assess non-load-associated (thermal or low temperature) cracking performance are usually conducted at low temperatures intended to represent the minimum in-service temperature experienced by the asphalt mixture throughout its life.

Some tests to address low temperature cracking are discussed in the following sections.

10.5.1 Indirect tensile creep/strength test

Indirect Tensile Creep and Indirect Tensile Strength tests are used to analyze mixtures to assess their low temperature cracking performance. In both tests, the specimen is loaded in compression across the diametral plane to produce tension in the center of the specimen. The test is conducted in a temperature-controlled environmental chamber surrounding a servo-hydraulic load frame. The equipment is shown in **Figure 10.19**.

In the indirect tensile creep test (see **Figure 10.20**), a constant load is used to induce tensile creep in a cylindrical test specimen that is 150 millimeters in diameter and 50 millimeters thick over a loading period of 100 or 1,000 seconds. The vertical and horizontal deformation is measured as well as the creep load. Usually,

FIGURE 10.19 ***Uniaxial Servo-hydraulic Load Frame for ITC/ITS Tests***

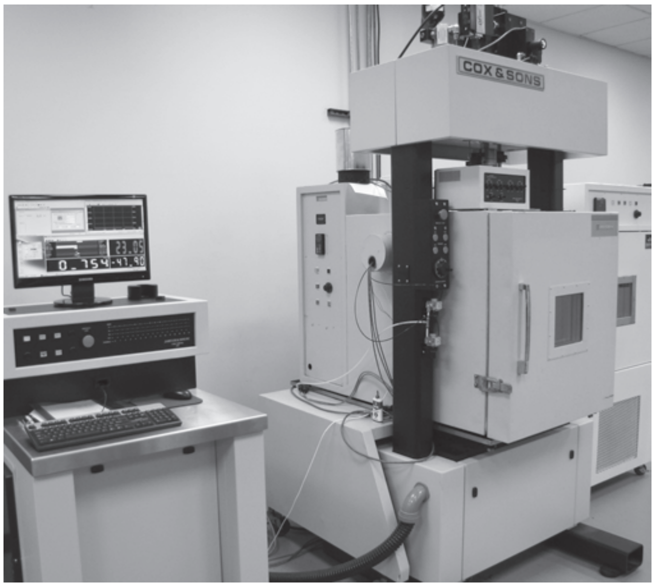
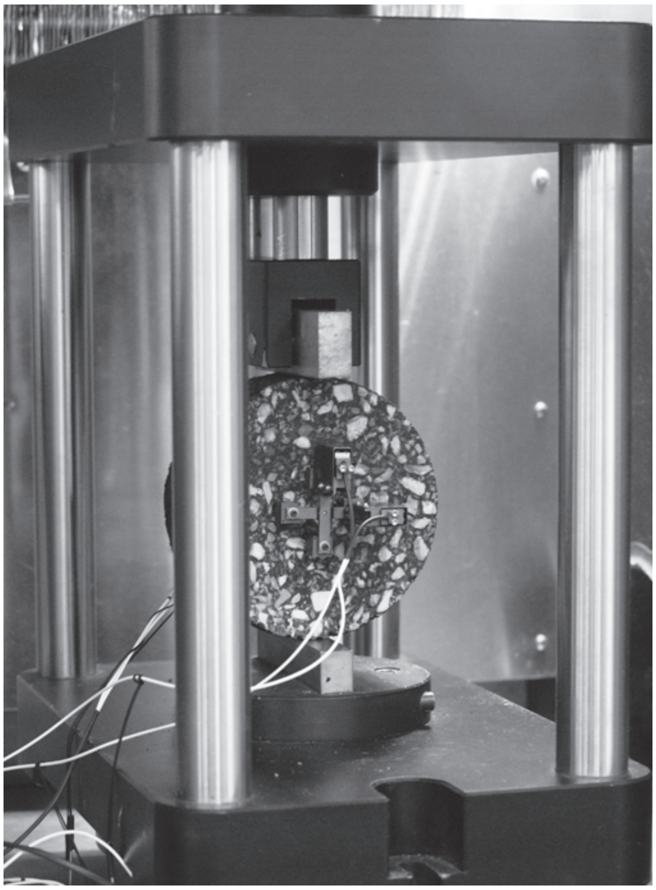


FIGURE 10.20 ***Indirect Tensile Creep/Strength Test Configuration***



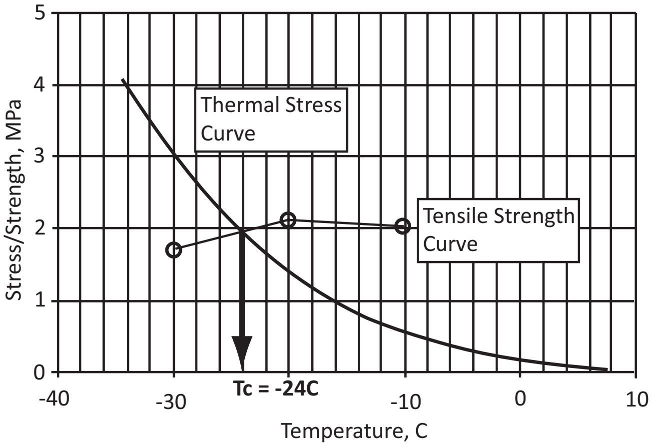


FIGURE 10.21 ***Determination of Critical Cracking Temperature of an Asphalt Mixture***

triplicate specimens are tested at multiple temperatures, between 10°C and –30°C depending on the PG grade, with the creep compliance determined as a function of time.

Following the completion of creep testing at a specific low temperature, the mix specimen is loaded to failure in the indirect tensile strength test. This test uses a constant displacement rate of 12.5 millimeters per minute to create a fracture in the specimen. Peak load is measured and fracture strength is calculated.

The most common, useful way of illustrating the data is to determine the critical cracking temperature of the asphalt mixture. In this procedure, creep data is used at multiple test temperatures to determine a master creep compliance curve that is then used to develop a thermal stress curve. The intersection of the thermal stress curve and the mixture tensile strength curve (from the indirect tensile strength test) is the critical cracking temperature. **Figure 10.21** illustrates the critical cracking temperature determination for an asphalt mixture.

Although no standard criteria exist, a simple means of evaluating the cracking performance of the mixture is to compare the critical cracking temperature to the expected low temperature at the project location where the mixture will be used. If the critical cracking temperature is lower (colder) than the expected low temperature at the project site, then the mixture would be expected to have acceptable low temperature cracking performance.

10.6

NCHRP Project 09-33 Performance Testing Recommendations

As part of National Cooperative Highway Research Program (NCHRP) Project 09-33, the research team at Advanced Asphalt Technologies conducted a thorough review of the performance testing currently available to supplement asphalt mix designs to ensure an adequate level of expected performance. Their findings are reported in NCHRP Report 673, “A Manual for Design of Hot Mix Asphalt with Commentary.” The Asphalt Institute concurs with these recommendations as summarized below.

1. Performance tests to address all forms of distress, permanent deformation (rutting), fatigue (load-associated) cracking and thermal (non-load-associated) cracking are typically reserved for higher priority pavements.
2. Of the three distresses, permanent deformation is the most affected by compositional factors such as aggregate angularity, asphalt binder grade (stiffness) and asphalt binder content. These factors could work in combination, in a manner difficult to estimate, to result in poorer-than-anticipated performance. For this reason, one or more tests to assess the laboratory permanent deformation performance of an asphalt mixture should be considered.
3. Laboratory testing to assess the permanent deformation performance of an asphalt mixture should be required when the expected in-service traffic over the life of the asphalt mixture is at least 3,000,000 ESALs.
4. NCHRP Report 673 suggests using either the |E*| AMPT Specification Criteria Program (described in NCHRP Report 580, “Specification Criteria for Simple Performance Tests for Rutting”) or the AMPT flow number test to evaluate permanent deformation performance.
5. Substantial research has shown that the laboratory fatigue cracking performance of asphalt mixtures is most strongly affected by the volume of effective asphalt binder in the mixture. Controlling the percentage of voids in the mineral aggregate (VMA) and percentage of air voids in the mix controls the volume of effective asphalt binder. Thus, as long as the

Traffic (10^6 ESALs)	Permanent Deformation Test	Fatigue Cracking Test	Thermal Cracking Test
<3	Not Required[1]	Not Required[1]	Not Required[1]
3 to <10	AMPT Flow Number (AASHTO TP 79) FN ≥ 53 cycles[2]	Not Required[1]	Not Required[1]
10 to <30	AMPT Flow Number (AASHTO TP 79) FN ≥ 190 cycles[2]	Not Required[1]	Not Required[1]
≥30	AMPT Flow Number (AASHTO TP 79) FN ≥ 740 cycles[2]	Not Required[1]	Not Required[1]

NOTES:

1. Although testing is not required, the user agency should consider whether additional testing is needed for asphalt mixtures that will be used in high traffic or other special conditions.
2. Test conducted on 100-mm diameter by 150-mm height cylindrical specimen having approximately 7 percent air voids using standard test conditions defined as follows:
   - Unconfined
   - 600 kPa axial stress
   - 0.1-second pulse loading with 0.9-second rest period constituting one cycle
   - Maximum of 10,000 load cycles
   - Test temperature equal to the average 7-day maximum pavement temperature at a depth of 20 millimeters for the project location (the "50 percent reliability" temperature in LTPPBind software Version 3.1)

TABLE 10.5 ***Recommended Performance Testing Requirements for Conventional Mixtures***

recommended volumetric mix design criteria are met for VMA and air voids, the expectation for conventional asphalt mixtures will be that they will have adequate laboratory fatigue cracking performance.

6. Research also has shown that thermal cracking performance of asphalt mixtures is most strongly affected by the asphalt binder properties. As long as the asphalt binder that is used in the mixture has the appropriate low temperature properties for the expected use, the expectation for conventional asphalt mixtures will be that they will have adequate laboratory thermal cracking performance.

10.6.1 Suggested performance testing for conventional mixes (Table 10.5)

The findings provided in NCHRP Report 673, as summarized in items 1–6 above, are for conventional asphalt mixtures. The term "conventional asphalt mixture" refers to a mixture that:

- uses coarse and fine aggregates that have been commonly used in asphalt mixtures to include natural (e.g., gravel), quarried (e.g., limestone, granite) and common synthetic (e.g., blast furnace or steel slag) aggregates;
- has a gradation that would be considered dense-graded and is bounded by the gradation limits shown in AASHTO M 323 or meets the gradation limits for an SMA mixture;
- uses an asphalt binder that can have its performance grade determined following AASHTO M 320 or M 332; and
- if using RAP, uses RAP that provides an asphalt binder replacement of 25 percent or less.

Although the AMPT flow number test is recommended, the user agency may elect to perform alternate rutting tests such as the asphalt pavement analyzer or Hamburg wheel tracking tests. Similarly, other cracking tests may be selected for use in place of the flexural beam fatigue and indirect tensile creep/strength tests. Using the guidelines provided in NCHRP Report 673, the Asphalt Institute recommends the following:

The AMPT flow number test is recommended as the permanent deformation (rutting) performance test for two reasons:

1. The test is most similar to the RSCH test that has been used successfully since SHRP to characterize the expected rutting performance of asphalt mixtures, but has the advantage of using simpler, less costly equipment; and
2. The development of permanent strain as a consequence of repeated loading has been shown to better characterize the rutting performance of modified asphalt binders than conventional modulus tests.

10.6.2 Suggested performance testing for unconventional asphalt mixtures (Table 10.6)

Unconventional asphalt mixtures are those asphalt mixtures that may use unconventional additives, modifiers, aggregates and/or recycled materials. This classification could include mixtures containing high levels of commonly used recycled materials such as RAP and/or RAS or less commonly used recycled materials. This classification could also include non-standard mixtures or mixtures where the standard volumetric requirements have not been met (e.g., low VMA mixes).

TABLE 10.6 ***Recommended Performance Testing Requirements for Unconventional Mixtures***

Traffic (10^6 ESALs)	Permanent Deformation Test	Fatigue Cracking Test	Thermal Cracking Test
<3	Not Required[1]	Not Required[1]	Not Required[1]
3 to <10	AMPT Flow Number (AASHTO TP 79) FN ≥ 53 cycles[2]	Flexural Beam Fatigue (AASHTO T 321)[3]	ITC/ITS (AASHTO T 322) $T_{cr} \leq T_{min,50\%}$[4]
10 to <30	AMPT Flow Number (AASHTO TP 79) FN ≥ 190 cycles[2]	Flexural Beam Fatigue (AASHTO T 321)[3]	ITC/ITS (AASHTO T 322) $T_{cr} \leq T_{min,98\%}$[5]
≥30	AMPT Flow Number (AASHTO TP 79) FN ≥ 740 cycles[2]	Flexural Beam Fatigue (AASHTO T 321)[3]	ITC/ITS (AASHTO T 322) $T_{cr} \leq T_{min,98\%}$[5]

NOTES:

1. Testing not required unless otherwise specified by the user agency.
2. Test conducted on 100-mm diameter by 150-mm height cylindrical specimen having approximately 7 percent air voids using standard test conditions defined as follows:
   - Unconfined
   - 600 kPa axial stress
   - 0.1-second pulse loading with 0.9-second rest period constituting one cycle
   - Maximum of 10,000 load cycles
   - Test temperature equal to the average 7-day maximum pavement temperature at a depth of 20 millimeters for the project location (the "50 percent reliability" temperature in LTPPBind software Version 3.1)
3. Testing conducted on beam specimens 375-mm length by 62.5-mm width by 50-mm height having approximately 7 percent air voids. Compare using standard test conditions to conventional asphalt mixture. Criteria for acceptability not determined, so testing could be waived at the discretion of the user agency.
4. Testing conducted on cylindrical specimens 150-mm diameter by 50-mm height having approximately 7 percent air voids using standard test conditions (three test temperatures). Critical cracking temperature (T_{cr}) to be determined. T_{cr} must be equal to or less than the average minimum pavement temperature determined at the surface for the project location (the "50 percent reliability" temperature in LTPPBind software Version 3.1).
5. Testing conducted on cylindrical specimens 150-mm diameter by 50-mm height having approximately 7 percent air voids using standard test conditions (three test temperatures). Critical cracking temperature (T_{cr}) to be determined. T_{cr} must be equal to or less than the minimum pavement temperature minus two standard deviations, determined at the surface for the project location (the "98 reliability" temperature in LTPPBind software Version 3.1).

CHAPTER 11

Recycled Asphalt Materials in the Mix Design Process

This chapter provides the designer with the best available information to design an asphalt paving mixture using recycled materials that meets all of the specified requirements for mixes produced with 100 percent virgin materials.

It is assembled to provide guidance enabling the designer to develop a mixture that preserves the original performance properties established in Superpave, including the recommended PG binder grades that have proven effective in resisting thermal cracking and permanent deformation. Extensive use of recycled materials that exceed the provisions included in this chapter has been utilized on many projects in the United States. For those mixtures that exceed the provisions of this chapter, the Asphalt Institute recommends the use of mixture performance tests such as those described in chapter 10 to assure sufficient long-term fatigue and low-temperature cracking performance on critical infrastructure projects.

11.1

Reclaimed Asphalt Pavement (RAP) general discussion

Reclaimed Asphalt Pavement (RAP) has been used as a valuable component of new asphalt mix for years. Most RAP is produced from milling. RAP is also produced from any existing asphalt pavement by processing it to an appropriate size to be used as a component of a new asphalt mixture. Since RAP consists of the same components as virgin HMA—aggregate and asphalt binder—it can readily be incorporated into a new mixture. Economically, there is a benefit to using RAP since these components can be reused, thereby lessening the need to purchase and use as much new (virgin) materials.

In addition to the economic benefits, the use of RAP in asphalt mixtures also has an environmental benefit. Reuse of a resource such as RAP lessens the depletion of nonrenewable natural resources, such as virgin aggregate and asphalt binder.

The source, stockpiling and variability of RAP are outside of the scope of this manual—as are the stockpiling and handling of virgin aggregates. (See MS-22 for more information.) However, a few words are necessary here to understand the issues with handling RAP during the mix design process.

Just as it is with virgin aggregates, the variability of a stockpile of RAP is important in both mix design and quality control during production. To effectively maximize the use of RAP in an asphalt mixture, the asphalt mixture producer should know the source of the RAP and, if practical, keep separate stockpiles of RAP from specific projects—or, at the least, keep RAP from one type of project (e.g., highways) separate from RAP of another type (e.g., parking lots). A RAP obtained from a neighborhood street or parking lot may have substantially different asphalt binder properties, asphalt binder content, aggregate physical properties and gradation than a RAP obtained from an urban highway.

Some contractors fractionate RAP by passing it over one or more screens to produce separate coarse and fine stockpiles for a more controlled gradation. These products are typically referred to as Fractionated Reclaimed Asphalt Pavement (FRAP). Separate stockpiles of FRAP can reduce the overall variability of the final mix using higher percentages of RAP. This has the potential to increase the available options to the mix designer because there are more stockpiles to work with. It should be noted that fractionation of the processed RAP tends to concentrate the binder in the finer fractions of FRAP and reduces the binder content in the coarse fractions—meaning FRAP stockpiles will typically have a higher or lower binder content than if the RAP were not fractionated.

In the past, agencies limited the total percentage of RAP in the mixture. This was an acceptable approach since the percentage of asphalt binder in the RAP was very similar to the total asphalt binder content of the mixture. With the increased use of fractionated RAP, most asphalt technologists and user agencies have recognized the need to consider the amount of RAP in the mixture in terms of the RAP Pavement Binder Ratio (RPBR)—the ratio of the RAP binder in the mix divided by the mixture's total binder content. The Asphalt Institute supports the RAP Pavement Binder Ratio approach. Further specifics on the use of RAP in mix design are provided after section 11.2.

11.2

Recycled Asphalt Shingles (RAS) general discussion

The use of Recycled (or Reclaimed) Asphalt Shingles (RAS) as a component in asphalt mixtures is appealing both economically and environmentally. RAS has a high percentage of asphalt binder—usually 20–30 percent—compared to RAP (which usually has an asphalt binder content of 4–8 percent). Since asphalt binder is generally the most expensive component of an asphalt mixture, using a recycled material with a high asphalt content can be very appealing to a mix designer. Unfortunately, the asphalt binder in RAS, which is manufactured differently than regular paving grade binders, is extremely stiff (some research has suggested that RAS binder may be a PG 142 or higher, with a softening point in excess of 120°C), making it difficult to reliably determine its physical properties using conventional binder testing equipment and procedures. This limits the ability to reliably predict asphalt binder performance in the field using the Performance-Graded (PG) Asphalt Binder Specification—a system that has a successful 20-year history of binder research and documented pavement performance.

In addition to the question about characterizing the physical properties of the RAS binder, many asphalt technologists question the degree of blending that actually occurs in an asphalt mixture between a virgin (softer) asphalt binder and the asphalt material contained in RAS. This is particularly a concern with warm mix asphalt mixtures.

Throughout its history, the Asphalt Institute has published established, well-proven techniques that promote the long-term quality performance of asphalt pavement. The Asphalt Institute supports, encourages and is actively engaged in research incorporating recycled binders. However, the difficulty in characterizing RAS binder properties and the uncertainty of RAS binder availability is the basis for our opinion that the technology is not sufficiently advanced at this time to include RAS in our standard mix design manual. Several recent research projects have also raised concerns that the addition of RAS reduces the cracking resistance of a mix. Those seeking to utilize RAS in asphalt pavement are encouraged to review the most recent research and literature available in order to maximize the probability of designing and constructing a long-lasting asphalt pavement. In addition, AASHTO's PP78 "Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures" provides detail regarding the effects of grind size and many of the other issues that can result in poor performance. If a mixture is going to utilize RAS, the Asphalt Institute also recommends the use of mixture performance testing described in chapter 10 to ensure fatigue and low-temperature cracking performance.

11.3

RAP properties

The amount of RAP that is used in the asphalt mixture will dictate what properties are needed to be known before the RAP can be used. For all levels of RAP, asphalt binder content and gradation will need to be known. At higher levels of RAP, the physical properties of the asphalt binder will also be needed so that blending charts can be used to select the appropriate grade of virgin, or newly added, asphalt binder.

11.3.1 Determining asphalt binder content and aggregate gradation

Regardless of the amount of RAP that is used in the mixture, the mix design technologist will need to know the asphalt binder content of the RAP so that the total asphalt binder content (RAP binder plus virgin binder) can be properly determined. There are two principal ways of determining the asphalt binder content of the RAP: ignition oven procedure and solvent extraction. The ignition oven procedure is detailed in AASHTO T 308, "Determining the Asphalt Binder Content of Hot-Mix Asphalt (HMA) by the Ignition Method." The solvent extraction procedure is detailed in AASHTO T 164, "Quantitative Extraction of Asphalt Binder from Hot-Mix Asphalt (HMA)." At the conclusion of the procedure, the recovered aggregate should be saved for determining the gradation using AASHTO T 30, "Mechanical Analysis of Extracted Aggregates."

Although either procedure can be used, there are disadvantages associated with each. With the ignition oven, degradation is possible with some aggregates, which could lead to a gradation that is finer than the actual gradation of the RAP. The ignition oven also requires a correction factor that may have to be estimated for the RAP and not measured. Experience with local aggregates can indicate if the ignition oven is an appropriate procedure to use. The biggest inconvenience with the second option using the extraction procedure is that it uses solvents.

Both procedures should be capable of producing a sufficient sample size after extraction so that the extracted aggregate can be tested for gradation. Additional extraction or ignition oven runs may be needed to complete other aggregate property tests as specified.

Although either procedure may be used for determining asphalt binder content and aggregate gradation, the solvent extraction procedure should be selected if asphalt binder properties are needed. In this case, the effluent from the extraction procedure is retained to be used in the selected recovery procedure as discussed in chapter 11, section 11.3.4.

11.3.2 RAP aggregate specific gravity

Before the use of the percentage of voids in the mineral aggregate, or VMA, as a mix design property, the gradation of the RAP aggregate was the principal aggregate property that was determined. Since VMA requires knowledge of the combined aggregate bulk specific gravity, it is important that an accurate determination of the RAP aggregate specific gravity ($G_{sb(RAP)}$) is made. In a mix using 25 percent RAP, an error of 0.04 in the specific gravity can affect the calculated VMA of the asphalt mixture by nearly 0.5 percent. This can be the difference between an acceptable and a marginal mix.

There are two main ways of determining the combined specific gravity of the RAP aggregate.

- *RAP Specific Gravity Method 1* uses the RAP aggregate that is obtained following a solvent extraction or ignition oven procedure. The aggregate is split into coarse and fine fractions, and individual specific gravity tests are performed. The combined RAP aggregate specific gravity is then determined.
- *RAP Specific Gravity Method 2* uses the RAP as is to determine the maximum theoretical specific gravity, G_{mm}, of the RAP. If the asphalt binder content of the RAP is known (or determined), then the effective specific gravity, G_{se}, of the RAP aggregate can be calculated. This value can then be used as a substitute for the bulk specific gravity, G_{sb}, of the RAP.
- *RAP Specific Gravity Method 2a* uses the same methodology as Method 2 with the exception that an asphalt absorption rate is determined or assumed and the G_{sb} of the RAP aggregate is calculated.

Table 11.1 lists some of the advantages and disadvantages of using each of the three methods.

Of the three methods, the Asphalt Institute recommends either *RAP Specific Gravity Method 2* or *RAP Specific Gravity Method 2a*. It is an easy step to measure the maximum theoretical specific gravity (G_{mm}) of the RAP and then calculate the effective specific gravity of the aggregate (G_{se}) using the predetermined asphalt binder content. The repeatability of G_{mm} (and hence G_{se}) measurements makes the use of G_{se} for recovered RAP aggregate the most attractive option. After that, an assumed

	RAP Specific Gravity Method 1	RAP Specific Gravity Method 2	RAP Specific Gravity Method 2a
Summary	Directly measure G_{sb}	Determine G_{mm}; Calculate G_{se}	Determine G_{mm}; Calculate G_{se}; Assume Absorption; Calculate G_{sb}
Advantage(s)	G_{sb} is determined like virgin aggregates	G_{se} is easy to determine; More repeatable than G_{sb} measurements	Same as Method 2; Assumption of absorption allows determination of G_{sb}
Disadvantage(s)	Requires coarse and fine G_{sb} determination; Extraction or ignition oven procedure may change aggregate properties	Use of G_{se} instead of G_{sb} artificially raises VMA, thus potentially decreasing effective asphalt and reducing durability	Absorption is an assumed value; Needs validation

TABLE 11.1 ***RAP Aggregate Specific Gravity Methods***

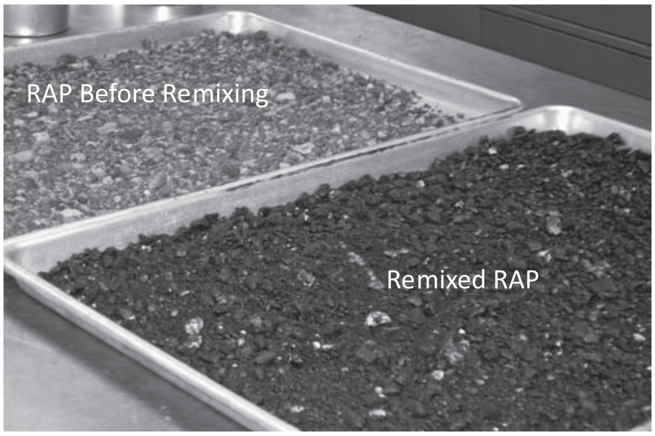
asphalt absorption may be used to get a more accurate measure of the G_{sb} of the recovered RAP aggregate.

11.3.2.1 Determining RAP G_{mm}

Before performing the G_{mm} test on RAP, it is important that the RAP is properly prepared, as described below:

1. Dry the test sample to constant mass in a forced draft oven at 110 + 5°C (230 + 9°F).
2. Break up the sample similar to a standard G_{mm} sample.
3. Mix the RAP sample thoroughly with a spatula to allow the old RAP binder to coat the uncoated aggregate particles. The difference between the remixed and unmixed RAP samples can be seen in **Figure 11.1.**

FIGURE 11.1 ***Preparing RAP Samples for G_{mm} Testing***
(photo courtesy of Tim Murphy)



Determining the maximum theoretical specific gravity on recycled asphalt or core samples can be confounded by the uncoated faces that result from crushing, milling or saw cutting if remixing of the sample is unable to provide sufficient coating of the mix. Uncoated faces may allow water to absorb into the samples and require the technician to institute the dry-back procedure contained in AASHTO T 209/ASTM D2041. The dry-back procedure is time consuming and introduces unnecessary error into the process. To prevent intrusion of water into the sample, add a measured quantity of new binder in the range of 1–3 percent of the total sample weight, as needed to fully coat all particles. Preheat the mix to 60–120°C as you would during a normal mix design, place it in the mixing bowl and add the additional binder to thoroughly coat the mixture and let it cool. When G_{mm} testing is completed, simply back out the added asphalt mass and volume from the original procedure using the following equation when the "Mass Determination in Air" from AASHTO T 209 is utilized:

$$\text{Theoretical Maximum Specific Gravity} = \frac{A - J}{(A + D) - (E + K)} \quad \text{(Eq. 11.1)}$$

Where:

A = mass of the oven-dry sample in air, g;
D = mass of the container filled with water at 25°C (77°F), g;
E = mass of the container filled with the sample and water at 25°C (77°F), g;
J = mass of the added asphalt binder in air, g;
K = volume of the added asphalt binder, cc or ml = $\frac{J}{G_b}$
where G_b = the specific gravity of the binder added.

Due to the variability of aggregate materials and their respective absorption rates, technicians may wish to experiment with the range of added asphalt. The desired goal is to thoroughly coat the sample, but not excessively to where drain-down can occur. The newly coated sample will resist water intrusion, minimize the presence of fugitive dust (cloudy water) and provide reliable, accurate results.

11.3.2.2 Calculating RAP G_{se} and G_{sb}

After determining the G_{mm}, the user can calculate the G_{se} (assuming the asphalt binder content is known) using the equation below:

$$G_{se(RAP)} = \frac{100 - P_{b(RAP)}}{\frac{100}{G_{mm(RAP)}} - \frac{P_{b(RAP)}}{G_{b(RAP)}}} \qquad \text{(Eq. 11.2)}$$

Where

$G_{se(RAP)}$ = effective specific gravity of the RAP
$G_{mm(RAP)}$ = measured maximum theoretical specific gravity of the RAP
$P_{b(RAP)}$ = asphalt binder content of the RAP
$G_{b(RAP)}$ = specific gravity of the RAP asphalt binder

The $G_{mm(RAP)}$ and $P_{b(RAP)}$ are determined from testing. While the $G_{b(RAP)}$ can be determined by testing, it would require that the asphalt binder be extracted and recovered. As such, it is recommended that an assumed $G_{b(RAP)}$ be used. In the absence of other guidance from the user agency, a $G_{b(RAP)}$ value of 1.04 is recommended.

Why use a $G_{b(RAP)}$ value of 1.04? Virgin paving grade asphalt binders usually have a specific gravity between 1.01 and 1.03, depending on several factors including the grade of the asphalt. As asphalt ages, the lighter fractions dissipate, leading to an increase in asphaltenes. This process increases the specific gravity of the asphalt binder. As such, it seems likely that the specific gravity of the RAP asphalt binder will be greater than 1.01 to 1.03. The selected $G_{b(RAP)}$ value of 1.04 accounts for the fact that the RAP asphalt binder is aged.

Lastly, the error caused by assuming a $G_{b(RAP)}$ value instead of actually measuring the value should be minimal. For an asphalt content of 5.5 percent and a G_{mm} of 2.500, an error of 0.02 in the $G_{b(RAP)}$ value will affect the calculated $G_{se(RAP)}$ by approximately 0.008. To put it in perspective, if 25 percent RAP is assumed to be used in the mix, then the $G_{se(RAP)}$ error of 0.008 will only change the combined specific gravity by 0.002—a difference that will cause less than a 0.1 percent error in VMA.

As an example, consider that the measured $G_{mm(RAP)}$ is 2.545 as determined using AASHTO T 209. The measured asphalt binder content of the RAP, $P_{b(RAP)}$, is 4.5 percent. Using an assumed $G_{b(RAP)}$ of 1.04, the $G_{se(RAP)}$ is calculated as follows:

$$G_{se(RAP)} = \frac{100 - 4.5}{\frac{100}{2.545} - \frac{4.5}{1.04}} = 2.731 \qquad \text{(Eq. 11.3)}$$

At this point, the user needs to decide if they are comfortable using the G_{se} of the RAP aggregate in VMA calculations or if they need to calculate the G_{sb} using an assumed value for asphalt absorption. The equation for determining G_{sb} of the RAP aggregate is shown below:

$$G_{sb(RAP)} = \frac{G_{se(RAP)}}{\frac{P_{ba(RAP)} \times G_{se(RAP)}}{100 \times G_{b(RAP)}} + 1} \qquad \text{(Eq. 11.4)}$$

Where

$G_{sb(RAP)}$ = bulk specific gravity of the RAP
$G_{se(RAP)}$ = effective specific gravity of the RAP
$P_{ba(RAP)}$ = asphalt binder absorption of the RAP
$G_{b(RAP)}$ = specific gravity of the RAP asphalt binder

In the preceding equation, the G_{mm} and asphalt binder content of the RAP are used to calculate $G_{se(RAP)}$, as shown earlier. As before, a $G_{b(RAP)}$ value of 1.04 is recommended as an estimated value. Mix design technologists can always measure the $G_{b(RAP)}$ if desired.

As an example, consider the previous example where the calculated $G_{se(RAP)}$ is 2.731 as determined using AASHTO T 209. Using an assumed $G_{b(RAP)}$ of 1.04 and an asphalt absorption of 1.2 percent (determined based on user experience with the RAP mixture), the $G_{sb(RAP)}$ is calculated as follows:

$$G_{sb(RAP)} = \frac{2.731}{\frac{1.2 \times 2.731}{100 \times 1.04} + 1} = 2.648 \qquad \text{(Eq. 11.5)}$$

There is uncertainty in the final G_{sb} of the RAP aggregate if using an assumed absorption, which would lead many users to simply stay with the G_{se} value as calculated and accept the slightly higher VMA values that result. However, if the asphalt absorption of the RAP and/or the RAP percentage increases, then the difference between the G_{sb} and G_{se} of the RAP aggregate can begin to have a significant effect on the calculated VMA.

Before determining specific gravity of the RAP aggregate, mix designers should check with the user agency to determine their preferences. In the absence of other guidance, the following is recommended:

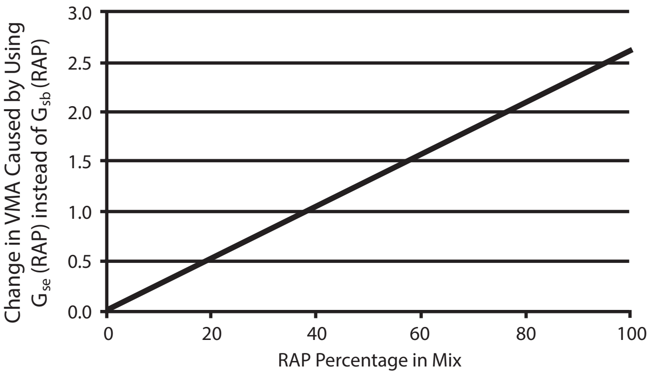
> If the “typical” aggregate absorption (water) is 2 percent or less (or the “typical” asphalt absorption is 1 percent or less) AND the RAP percentage is 20 percent or less, then use *RAP Specific Gravity Method 2*. Otherwise, use historical knowledge from past mix design experience to determine an appropriate absorption to assume and use *RAP Specific Gravity Method 2a*.

In this guidance, “typical” aggregate absorption means the absorption of the aggregates that would most likely have been used in the asphalt mixture that ultimately became RAP. Water absorption of 2 percent or less should result in asphalt absorption of 1 percent or less. Using a maximum of 1 percent asphalt absorption and 20 percent RAP, the error caused by using G_{se} instead of G_{sb} (G_{se} with an assumed absorption) should be approximately 0.014 for the specific gravity. This error could change VMA by as much as 0.45 percent.

In the previous examples, note that the assumed asphalt absorption is 1.2 percent. The difference between using $G_{se(RAP)}$ and $G_{sb(RAP)}$ in the VMA calculations is 0.083. At this difference, when combined with other aggregates, the effect on VMA increases as the RAP percentage increases as shown in **Figure 11.2**.

By using the data in the examples, we see in **Figure 11.2** that at 40 percent RAP, the effect of using $G_{se(RAP)}$ instead of $G_{sb(RAP)}$ is a +1 percent change in VMA. In other words, by using $G_{se(RAP)}$, the VMA will calculate to be higher by approximately 1 percent than actual. Unless properly considered, this error can lead to a mix with insufficient asphalt binder content.

FIGURE 11.2 ***Effect of RAP Aggregate Specific Gravity on Calculated VMA (using example data)***



11.3.3 Consensus aggregate properties

RAP aggregate, like virgin aggregate, must meet the consensus property requirements established in AASHTO M 323. It is important to remember that the consensus property requirements apply to the blend of aggregates, virgin and RAP, and not specifically to the individual aggregates. Some agencies allow mix design technologists to test individual aggregate components and mathematically combine them in the appropriate percentages to get a value for the combined aggregate.

To test RAP aggregate individually, it must first be separated into coarse and fine fractions by splitting the sample. Material retained on the 4.75-mm (#4) sieve is used for determining coarse aggregate angularity and flat/elongated particles. Material finer than the 2.36-mm (#8) sieve is used for the fine aggregate angularity test. The sand equivalent test is not conducted on RAP aggregate since it has already been coated with asphalt binder and the clay content, if any, will have been removed during the extraction or ignition oven procedure.

11.3.4 RAP asphalt binder properties

At higher levels of RAP, it may be necessary to know the properties of the RAP asphalt binder so that an appropriate virgin asphalt binder grade can be selected. If RAP binder properties are needed, then a solvent extraction/recovery procedure should be performed on a sample of the RAP.

Solvent extraction and recovery of asphalt binder may be performed in several ways, but the most commonly used procedure is to perform a solvent extraction following the procedures in AASHTO T 164, “Quantitative Extraction of Asphalt Binder from Hot-Mix Asphalt (HMA),” followed immediately by binder recovery following the procedures in ASTM D5404, “Recovery of Asphalt from Solution Using the Rotary Evaporator,” or ASTM D7906, “Recovery of Asphalt from Solution Using Toluene and the Rotary Evaporator.” AASHTO T 170, “Recovery of Asphalt from Solution by Abson Method,” is not recommended since research has shown that recovered binder properties using the Abson method may be significantly different than the properties determined using other recovery methods—most often as a result of solvent left in the binder at the presumed conclusion of the recovery procedure.

Another procedure that may be used is a combined extraction/recovery procedure that was developed during SHRP and modified as part of later research on RAP, AASHTO T 319, "Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures." Research indicated that this procedure would have less of an effect on the chemical and physical properties of the recovered asphalt binder than other extraction/recovery procedures.

Although either extraction/recovery procedure may be used, it is important to note that AASHTO T 319 is limited to approximately 1,000 grams of RAP (or asphalt mixture) to minimize the effects of solvent aging that could occur during the procedure. Assuming an asphalt binder content of 5.0 percent, this limitation will result in only 50 grams of recovered asphalt binder. While this may be enough for physical property testing using the PG specification, technologists who want to perform viscosity grading or other specification tests will either need to conduct more than one recovery using T 319 or use the T 164/D5404 procedures.

The following describes the steps needed to characterize the physical properties of the recovered RAP asphalt binder using the AASHTO M 320 specification. If a different binder specification is used, such as AASHTO M 226, then typically only absolute viscosity testing at 60°C is required to determine the viscosity of the recovered RAP asphalt binder.

Once the asphalt binder has been recovered, it should be tested using the procedures in AASHTO M 320 to determine the critical temperatures (sometimes called the "true grade" or "continuous grade" temperatures) where the specification criteria is exactly met. Determination of critical temperature requires testing at two or more temperatures, preferably bracketing the specification value. Recovered asphalt binder should be tested in accordance with the procedures in AASHTO M 320 with the following notes:

- Do not perform flash point (AASHTO T 48), rotational viscosity (AASHTO T 316) or original DSR testing (AASHTO T 315).
- Perform RTFO aging (AASHTO T 240), but do not determine mass loss.
- Test the RTFO-aged recovered RAP asphalt binder using AASHTO T 315 at two or more temperatures to determine the critical temperature where the $G^*/\sin\delta$ value exactly meets 2.20 kPa.
- Conduct PAV aging on the RTFO-aged recovered RAP asphalt binder.
- Test the PAV-aged recovered RAP asphalt binder using AASHTO T 315 at two or more temperatures to determine the critical temperature where the $G^*\sin\delta$ value exactly meets 5000 kPa.
- Test the PAV-aged recovered RAP asphalt binder using AASHTO T 313 at two or more temperatures to determine the critical temperature where the BBR Stiffness value exactly meets 300 MPa. Also determine the critical temperature where the BBR m-value exactly meets 0.300.

One question that usually arises with the preceding testing program is "Why conduct RTFO aging on a recovered RAP asphalt binder?" If the RTFO aging procedure is intended to simulate the aging that occurs when an asphalt mixture is produced in an HMA facility and RAP is asphalt that has already been mixed, placed and in-service for a number of years, then why conduct RTFO aging on recovered material? Research conducted as part of the NCHRP 9-12 project indicated that by RTFO aging the recovered RAP binder before testing, any residual solvent left from the recovery procedure would be removed and the linear blending chart approach better matched actual measured values.

Although research indicated that the low temperature properties of recovered RAP asphalt binder that was subjected to PAV aging were slightly different than those subjected to RTFO aging only, the researchers judged that the extra time and material required for PAV aging of recovered RAP asphalt binder didn't justify the slight changes it made in the estimated properties of the final blended binder. However, the Asphalt Institute believes that it is prudent to perform

RTFO and PAV aging on the recovered RAP asphalt binder to ensure that the most accurate properties are obtained when using blending charts.

To determine the critical high temperature where the G*/sin δ specification value is 2.20 kPa, testing is needed at two test temperatures bracketing the specification value. In the absence of other information, consider beginning the DSR testing at a temperature that is at least one grade above the standard grade that would normally be used. For example, begin testing at 70°C if a PG 64-22 asphalt binder would normally be used.

It is not absolutely necessary to test at temperatures bracketing the specification value, but it is preferable. By bracketing the specification value, the critical temperature can be interpolated. If the specification value is not bracketed, then the critical temperature may be extrapolated, which is inherently a more inaccurate method of estimating a final value.

To calculate the critical high temperature where the G*/sin δ value is 2.20 kPa, use the following equation:

$$T_c = T_1 + \frac{\log(2.20) - \log(G_1^* \sin\delta_1)}{\log(G_1^* \sin\delta_1) - \log(G_2^* \sin\delta_2)} \times (T_1 - T_2)$$

(Eq. 11.6)

Where

T_c = critical temperature (in this case, the critical high temperature)

T_1 = lower of the two test temperatures, °C, where the G*/sin δ value ≥ 2.20 kPa

T_2 = higher of the two test temperatures, °C, where the G*/sin δ value < 2.20 kPa

$G^*_1/\sin\delta_1$ = G*/sin δ value at temperature T_1, kPa

$G^*_2/\sin\delta_2$ = G*/sin δ value at temperature T_2, kPa

To determine the critical intermediate temperature where the G*sin δ specification value exactly meets 5000 kPa, testing is needed at two test temperatures bracketing the specification value. In the absence of other information, consider beginning the DSR testing at a temperature that is at least one grade above the standard grade that would normally be used. For example, begin testing at 28°C if a PG 64-22 asphalt binder would normally be used. As before, it is preferable but not absolutely necessary to test at temperatures bracketing the specification value.

To calculate the critical intermediate temperature where the G*sin δ value exactly meets 5000 kPa, use the following equation:

$$T_c = T_1 + \frac{\log(5000) - \log(G_1^* \sin\delta_1)}{\log(G_1^* \sin\delta_1) - \log(G_2^* \sin\delta_2)} \times (T_1 - T_2)$$

(Eq. 11.7)

Where

T_c = critical temperature (in this case, the critical intermediate temperature)

T_1 = higher of the two test temperatures, °C, where the G*/sin δ value ≤ 5000 kPa

T_2 = lower of the two test temperatures, °C, where the G*sin δ value > 5000 kPa

$G^*_1\sin\delta_1$ = G*sin δ value at temperature T_1, kPa

$G^*_2\sin\delta_2$ = G*sin δ value at temperature T_2, kPa

To determine the critical low temperature, two parameters are needed: (1) the temperature where the BBR Stiffness exactly meets 300 MPa ($T_{c,S}$); and (2) the temperature where the BBR m-value exactly meets 0.300 MPa ($T_{c,m}$). In the absence of other information, consider beginning the BBR testing at a temperature that matches the standard grade that would normally be used. For example, begin testing at –12°C if a PG 64-22 asphalt binder would normally be used. As before, it is preferable, but not absolutely necessary to test at temperatures bracketing the specification value.

To calculate the critical low temperature where the BBR Stiffness exactly meets 300 MPa, use the following equation:

$$T_c = T_1 + \frac{\log(300) - \log(S_1)}{\log(S_1) - \log(S_2)} \times (T_1 - T_2) \quad -10$$

(Eq. 11.8)

Where

T_c = critical temperature (in this case, the critical BBR Stiffness temperature)

T_1 = higher of the two test temperatures, °C, where the BBR Stiffness ≤ 300 MPa

T_2 = lower of the two test temperatures, °C, where the BBR Stiffness > 300 MPa

S_1 = BBR Stiffness at temperature T_1, kPa

S_2 = BBR Stiffness at temperature T_2, kPa

To calculate the critical low temperature where the BBR m-value exactly meets 0.300, use the following equation:

$$T_c = T_1 + \frac{0.300 - m_1}{m_1 - m_2} \times (T_1 - T_2) - 10$$

(Eq. 11.9)

Where

T_c = critical temperature (in this case, the critical BBR m-value temperature)

T_1 = higher of the two test temperatures, °C, where the BBR m-value ≥ 0.300

T_2 = lower of the two test temperatures, °C, where the BBR m-value < 0.300

m_1 = BBR m-value at temperature T_1, kPa

m_2 = BBR m-value at temperature T_2, kPa

As an example, assume a recovered RAP binder that is tested as described above. The RAP comes from a climate where a PG 64-22 asphalt binder would typically be used as a virgin asphalt binder. Based on this information:

- begin high temperature testing at 70°C and continue to failure;
- begin intermediate temperature testing at 28°C and continue to failure; and
- begin low temperature testing at –12°C and continue to failure.

Test data at multiple temperatures are shown in **Table 11.2**.

Using the example data in **Table 11.2**, the critical temperature can be calculated. For the critical high temperature, use the equation presented earlier in Eq. 11.6 (also shown below).

$$T_c = T_1 + \left[\frac{\log(2.20) - \log(G_1^* / \sin\delta_1)}{\log(G_1^* / \sin\delta_1) - \log(G_2^* / \sin\delta_2)} \times (T_1 - T_2) \right]$$

(Eq. 11.10)

To bracket the specification value, use the test data at 82 and 88°C as shown below.

$$T_c = 82 + \left[\frac{\log(2.20) - \log(3.50)}{\log(3.50) - \log(1.71)} \times (82 - 88) \right] = 85.9$$

(Eq. 11.11)

Using the data in **Table 11.2**, the critical temperature where $G^*/\sin\delta$ equals 2.20 kPa is 85.9°C. Because of potential errors in testing, calculation and ultimately blending, it is sufficient to round the critical temperature to the nearest degree where the criteria would be met. In this instance, the value would meet at 85°C, but not at 86°C, so the critical temperature is rounded down to 85°C.

Following this same approach, and using the appropriate equations, for the intermediate temperature DSR, BBR Stiffness and BBR m-value, the critical temperatures are as follows:

- $T_{c,high} = 85°C$ (calculated value = 85.9°C)
- $T_{c,intermediate} = 28°C$ (calculated value = 27.1°C)
- $T_{c,S} = -22°C$ (calculated value = –22.6°C)
- $T_{c,m} = -22°C$ (calculated value = –22.4°C)

The continuous or true grade of the recovered RAP binder is a PG 85-22. The above T_c values are rounded to the more conservative whole degree

TABLE 11.2 ***Recovered RAP Binder Properties (Example)***

Condition	Test Property	Temperature, °C	Test Value	Criteria
RTFO-aged	DSR G*/sin δ	70	15.97 kPa	≥ 2.20 kPa
		76	7.38 kPa	
		82	3.50 kPa	
		88	1.71 kPa	
	DSR G*sin δ	28	4492 kPa	≤ 5000 kPa
		25	6486 kPa	
	BBR Stiffness	–12	280 MPa	≤ 300 MPa
		–18	534 MPa	
	BBR m-value	–12	0.303	≥ 0.300
		–18	0.254	

in order to ensure compliance with the terms of AASHTO M 320 which results in a PG 82-22.

These critical temperatures can then be used to create linear blending charts to either select the appropriate virgin binder grade or the amount of RAP that can be used with a specific virgin asphalt binder.

11.3.4.1 Using RAP asphalt binder properties to select an appropriate virgin binder grade

Selection of an asphalt binder grade to use in a mixture is usually left up to the project designers. The selected binder grade is based on a number of factors such as climate and anticipated traffic loading at the project site. This is referred to as the project binder grade.

Because of the time and testing expense of determining RAP binder properties for blending, many asphalt mixture designers elect to use a percentage of RAP that will not require binder properties and blending charts and/or equations. At the time of this printing, AASHTO M 323, "Superpave Volumetric Mix Design," recommends the following:

Recommended Virgin Asphalt Binder Grade	RAP Percentage
No change in binder selection	< 15
Select virgin binder one grade softer than normal (e.g., select a PG 58-28 if a PG 64-22 would normally be used)	15 to 25
Follow recommendations from blending charts	> 25

TABLE 11.3 ***AASHTO M 323 Binder Selection Guideline for RAP Mixtures***

The data in **Table 11.3** was originally established by the Asphalt Mixtures Expert Task Group (ETG) and was validated as part of a Binder Effects experiment conducted during the NCHRP 9-12 research project. When 10 percent recovered RAP binder was blended with a virgin asphalt binder, there was no change in the binder grade. When 20 percent recovered RAP binder was blended with a virgin asphalt binder, the blended binder grade was approximately one grade stiffer. When 40 percent recovered RAP binder was blended with a virgin asphalt binder, the blended binder grade was sometimes only one grade stiffer, but sometimes two grades stiffer.

One important note about **Table 11.3**—the values were validated by binder blending and testing. Thus, even though the table refers to "RAP percentage," it is more correctly "RAP binder percentage." This is sometimes referred to as a percentage of asphalt binder replacement. When RAP having 4–6 percent asphalt binder content is used in an asphalt mixture with a design asphalt binder content of approximately the same values (4–6 percent), then the RAP percentage in the mixture is effectively the same as the RAP binder percentage (or binder replacement percentage). This simplification does not work if the RAP binder percentage is much higher or lower than the design asphalt binder content.

Based on the past research, the Asphalt Institute concurs with **Table 11.3** with the following notes:

- The RAP percentage is a RAP binder percentage—determined by knowing the percentage of RAP to be used in the mixture and the asphalt binder content of the RAP.
- **Table 11.3** was developed and validated using RAP only and is not intended for RAS or a combination of RAP and RAS. As discussed in section 11.2, RAS binder is significantly different than RAP binder.
- The use of blending equations and/or charts (see section 11.4) is the most robust and responsible approach when using RAP because it characterizes the RAP binder by sampling and testing, and then determines the properties of the blended RAP binder and virgin binder. It provides a continuous or true grade of the RAP binder, virgin binder and blended binder.
- For mixtures using 15–25 percent RAP, the standard recommendation is to select one grade softer (both high and low temperatures) than would normally be selected. A mix designer may choose to use blending equations or charts for any level of RAP, subject to agency approval. This could be important if, at 20 percent RAP, the mix designer finds from the blending analysis that a grade change is not needed.
- For projects requiring modified binder grades, the user agency should not choose an unmodified virgin binder grade if the reasons for specifying the modified grade are

other than rutting concerns. For instance, if the agency is specifying a modified binder for improved crack resistance, durability or overall service life, then they will want to ensure the virgin (newly added) binder grade using RAP is also a modified grade. This will generally be the case as long as any binder grade adjustments due to RAP are applied to both the high temperature side and low temperature side of the PG grade, and not just high temperature. As an example, if the project grade calls for a modified PG 76-22, and the mix designer wants to use 20 percent RAP in the mixture, they may do so by selecting a virgin binder that is a modified PG 70-28. While the total polymer content of the mixture is reduced by the recovered RAP binder, the requirement for the virgin binder to remain a modified grade helps ensure that the properties of the RAP mixture will be similar to those of a virgin mixture.

- At RAP binder percentages over 25 percent, blending charts or equations should be used. NCHRP 9-12 results showed that the predictability of linear blending theory become unstable as replaced binder contents approach and exceed 40 percent. The addition of unusually soft binders to compensate for elevated amounts of recycled binder can deviate from the predicted performance expected using blending analysis. Therefore, the Asphalt Institute recommends in no case should the selected virgin binder grade be more than two grades softer than the binder grade that would be used in a 100 percent virgin (0 percent RAP) mixture for that location and application. As before, if the project binder grade is intended to be a modified binder, then the selected virgin binder should also be a modified binder. If the only reason an agency is specifying the modified grade is for rutting concerns, a modified binder may not be necessary depending on the properties and quantity of the RAP being utilized.
- Historically, asphalt mixtures containing up to 20–25 percent RAP have performed successfully in service, when properly designed and constructed. More recently, research has been conducted to evaluate the design and performance of asphalt mixtures with high contents of RAP (defined as greater than 25 percent and up to, and exceeding, 50 percent). Regardless of the information provided by blending charts or equations, mix designers may wish to determine the expected performance of asphalt mixtures containing higher levels of RAP using appropriate laboratory performance-related tests to assess the expected rutting and cracking performance of the mixture (see chapter 10).
- The lower temperatures utilized in WMA technologies may directly impact binder grade selection due to the lower level of aging that occurs in the plant. The amount of binder blending that occurs when utilizing very stiff recycled binders at low plant mixing temperatures is difficult to quantify. Mix designers may wish to determine the expected performance of asphalt mixtures using appropriate laboratory performance-related tests to assess the expected rutting and cracking performance of the mixture (see chapter 10).

11.4

Using blending charts

11.4.1 How much blending really occurs (the black rock question)?

Most asphalt mixture technologists agree that when RAP is mixed with virgin aggregate and virgin asphalt binder in a mixing plant (or even in the lab), there is a certain amount of blending that occurs. In other words, RAP does not behave like a black rock—an aggregate that was previously coated with asphalt binder but does not contribute to the final asphalt binder physical properties. On the other hand, it seems very unlikely that the RAP binder will physically separate itself from the rock and blend completely with the virgin binder. The truth likely lies somewhere between these two extremes.

Blending equations and charts presume that complete blending occurs between the RAP and virgin binders. While this very likely does not occur, research has shown that the RAP mixture behaves as a composite as if the RAP and virgin binders were completely blended. What this means is that the RAP mixture will behave as if it were a virgin mixture blended with the final (blended) asphalt binder grade rather than just the virgin asphalt binder grade.

What is the risk if total blending does not occur and the properties of the virgin asphalt binder control the performance of the RAP mixture? Because the virgin asphalt binder grade is softer, the risk is that the RAP asphalt mixture will behave as if it were softer—leading to concerns about rutting. This is the reason for the recommendation that in no case should the selected virgin binder grade be more than two grades softer than the binder grade that would be used in a 100 percent virgin (0 percent RAP) mixture. If a mix designer is concerned about rutting susceptibility, then one or more performance-related tests (described in chapter 10) may be used to validate the performance of the mixture.

The following discussion on the use of blending equations and charts assumes complete blending, which again is unlikely. The extent of actual blending likely depends on many factors such as RAP binder stiffness, mixing temperature and storage temperature and time.

11.4.2 Blending analysis methods using PG grading system

When using blending charts or equations, there are three cases that will be encountered by a mix designer:

1. determining an appropriate virgin binder grade when the RAP binder percentage to be used in the mixture is known as well as the final blended binder grade (project binder grade);
2. determining the RAP binder percentage that may be used when a specific virgin binder grade is to be used and the final blended binder grade is known; and
3. determining the final blended binder grade when the RAP binder percentage is fixed and virgin asphalt binder grade is fixed.

Of these cases, the third case is least likely to be used and will not be discussed further. In most instances, the project (final blended) binder grade will be specified leaving only RAP binder percentage and virgin binder grade as variables. Section 11.4.2.1 provides an example of the first case using the PG system, while section 11.4.2.2 provides an example of the second case also using the PG system. Section 11.4.3 provides another example of the first case, but with using the viscosity grading system.

11.4.2.1 PG blending using a known RAP percentage

If the final blended binder grade, percentage of RAP and recovered RAP properties are known, then the properties of an appropriate virgin asphalt binder grade can be determined. Consider the following example:

- The specifying agency requires a blended binder grade of PG 64-22 or better.
- The RAP percentage in the mixture is 30 percent (RPBR = 0.30).
- The recovered RAP properties are as indicated in **Table 11.4**.

Using a linear blending equation, the critical temperatures of the virgin asphalt binder can be determined:

$$T_{Virgin} = \frac{T_{Blend} - (RPBR \times T_{RAP})}{(1 - RPBR)} \qquad \text{(Eq. 11.12)}$$

where:

T_{Virgin} = critical temperature of the virgin asphalt binder
T_{Blend} = critical temperature of the blended asphalt binder (final desired)
RPBR = RAP Pavement Binder Ratio
T_{RAP} = critical temperature of recovered RAP binder

Using the values in **Table 11.4** and the assumptions shown earlier, the critical high

TABLE 11.4 ***Example—Properties of Recovered RAP Binder***

Aging	Property	Critical Temperature, °C	
RTFO	DSR G*/sin δ	High	86
	DSR G*sin δ	Intermediate	31
	BBR S	Low	–14
	BBR m-value	Low	–11
	PG	Actual	PG 86-11
		M320	PG 82-10

temperature of the virgin asphalt binder is calculated as follows:

$$T_{\text{Virgin}} = \frac{64 - (0.30 \times 86)}{(1 - 0.30)} = 54.6 \qquad \text{(Eq. 11.13)}$$

Thus, the virgin asphalt binder would need to have a critical high temperature of 55°C or higher in order to produce a final binder grade of PG 64 using 30 percent of the RAP (for an RPBR equal to 0.30) with properties shown in **Table 11.4**. Note that the critical high temperature is rounded up in this instance because a temperature of 54°C would not produce a blended binder grade of PG 64.

This process is repeated for the critical intermediate temperature and the critical low temperatures for BBR Stiffness and m-value as shown below.

For the critical intermediate temperature:

$$T_{\text{Virgin}} = \frac{25 - (0.30 \times 31)}{(1 - 0.30)} = 22.4 \qquad \text{(Eq. 11.14)}$$

For the critical low temperature for BBR Stiffness:

$$T_{\text{Virgin}} = \frac{-22 - (0.30 \times -14)}{(1 - 0.30)} = -25.4 \qquad \text{(Eq. 11.15)}$$

For the critical low temperature for BBR m-value:

$$T_{\text{Virgin}} = \frac{-22 - (0.30 \times -11)}{(1 - 0.30)} = -26.7 \qquad \text{(Eq. 11.16)}$$

Using the equations for the high, intermediate and low critical temperatures, the properties of the virgin asphalt binder needed to satisfy the assumptions are determined as shown in **Table 11.5**.

Aging	Property	Critical Temperature, °C	
RTFO	DSR G*/sin δ	High	55
PAV	DSR G*sin δ	Intermediate	22
	BBR S	Low	−26
	BBR m-value	Low	−27
	PG	Actual	PG 55-27
		M320	PG 58-28

TABLE 11.5 ***Example—Properties Needed of Virgin Asphalt Binder***

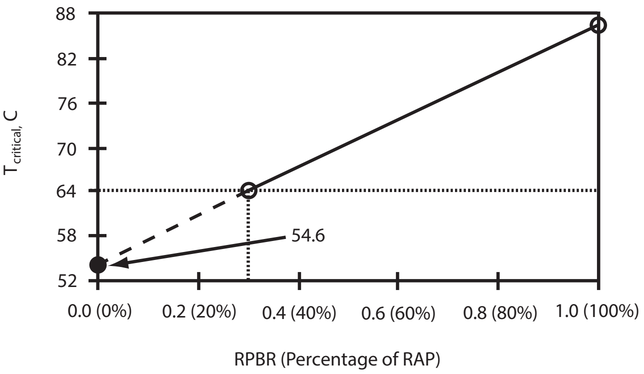


FIGURE 11.3 ***High Temperature Blending Chart (RAP Percentage Known)***

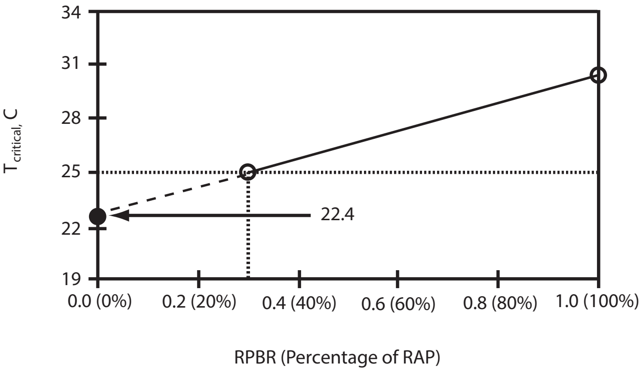


FIGURE 11.4 ***Intermediate Temperature Blending Chart (RAP Percentage Known)***

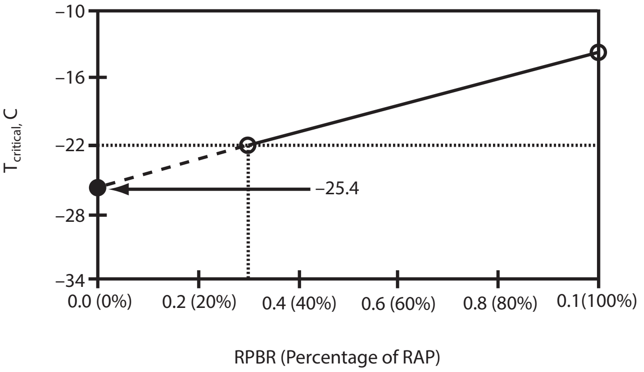


FIGURE 11.5 ***Low Temperature (S) Blending Chart (RAP Percentage Known)***

As indicated in **Table 11.5** and **Figure 11.3**, the minimum high temperature grade of the virgin asphalt binder should be 55°C (rounded to the nearest degree) to satisfy the requirements of the blended grade (PG 64-22) using the RAP in **Table 11.4** at 30 percent (RPBR = 0.30). This means that a PG 58-xx grade would be needed to ensure that the minimum required value of 55°C would be achieved.

Table 11.5 and **Figures 11.5** and **11.6** indicate that the minimum low temperature grade of the virgin asphalt binder should be −27°C (rounded to the nearest degree) to satisfy the requirements of the blended grade (PG 64-22) using the RAP in **Table 11.4** at 30 percent (RPBR = 0.30). This means that a

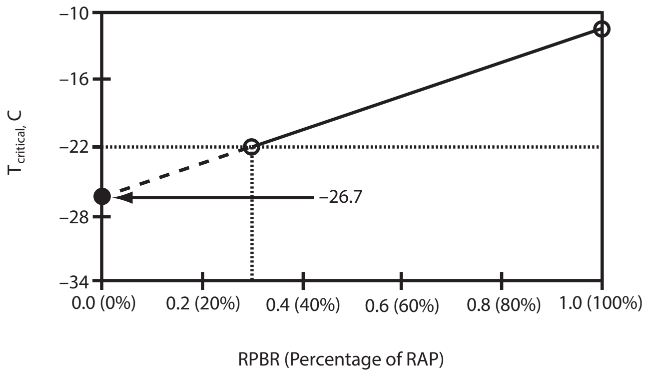


FIGURE 11.6 ***Low Temperature (m-value) Blending Chart (RAP Percentage Known)***

PG xx-28 grade would be needed to ensure that the minimum required value of −27°C would be achieved.

Thus, a PG 58-28 asphalt binder would be selected as the virgin asphalt binder for use in a mixture using 30 percent of the RAP (RPBR = 0.30) described in **Table 11.4**.

To meet the intermediate temperature grade (G*sin δ) in **Figure 11.4**, the virgin asphalt binder would need to have a critical intermediate temperature no higher than 22°C. Since the maximum critical intermediate temperature in AASHTO M 320 for a PG 58-28 binder is 19°C, the selected binder should easily meet all blended binder requirements.

11.4.2.2 PG blending using a known virgin binder grade

If the final blended binder grade, virgin asphalt binder grade and recovered RAP binder properties are known, then the amount of RAP that may be used can be determined. Consider the following example:

- The specifying agency requires a blended binder grade of PG 64-22 or better.
- The virgin binder grade is a PG 58-28 (critical temperatures in **Table 11.6**).
- The recovered RAP is a PG 82-10 (critical temperatures in **Table 11.6**).

By rearranging the equations described earlier, the percentage of RAP can be determined:

$$RPBR = \frac{T_{Blend} - T_{Virgin}}{T_{RAP} - T_{Virgin}} \qquad \text{(Eq. 11.17)}$$

where:

T_{Virgin} = critical temperature of the virgin asphalt binder

T_{Blend} = critical temperature of the blended asphalt binder (final desired)

RPBR = RAP Pavement Binder Ratio

T_{RAP} = critical temperature of recovered RAP binder

Using these equations for the high, intermediate and low critical temperatures, the RPBR (or percentage of RAP) needed to satisfy the assumptions can be determined. The calculation is shown below for the critical high temperature, using an assumed minimum value of 64°C for the final (blended) binder grade.

$$RPBR = \frac{64-61}{86-61} = \frac{3}{25} = 0.12 \qquad \text{(Eq. 11.18)}$$

Note that the RPBR of 0.12 (or RAP percentage of 12 percent) represents the minimum ratio of RAP binder that would need to be used with the RAP and virgin binder properties shown in **Table 11.6** to get a final blended binder that would be at least a PG 64-xx. Note also that the original DSR is not used since the RAP binder is considered

TABLE 11.6 ***Example—Properties of Virgin and Recovered RAP Binders***

		Critical Temperature, °C		
Aging	**Property**	**Temp. Range**	**Virgin Binder**	**RAP Binder**
Original	DSR G*/sin δ	High	61	n/a
RTFO	DSR G*/sin δ	High	61	86
PAV	DSR G*sin δ	Intermediate	15	31
	BBR S	Low	−32	−14
	BBR m-value	Low	−29	−11
	PG	Actual	PG 61-29	PG 86-11
		M320	PG 58-28	PG 82-10

			RPBR to Achieve:			
Aging	**Property**	**Temp.**	**PG 64-xx**	**PG 70-xx**	**PG xx-22**	**PG xx-28**
RTFO	DSR G*/sin δ	High	≥0.12	≤0.32	—	—
PAV	DSR G*sin δ	Intermediate	—	—	≤0.62	—
	BBR S	Low	—	—	≤0.55	≥0.28
	BBR m-value	Low	—	—	≤0.38	≥0.11

TABLE 11.7 ***Example—Estimated RPBR to Achieve Final Blended Grade***

to have already been aged to at least the level of an RTFO-aged material.

Although the ratio of 0.12 is the minimum to achieve a PG 64-xx grade, the mix designer may also want to ensure that the blended binder is not stiffer than a PG 64-xx grade. In this case, the same calculation is conducted, but with T_{Blend} set to one degree lower than the next highest grade (PG 70-xx, in this instance) as shown below:

$$RPBR = \frac{69-61}{86-61} = \frac{8}{25} = 0.32 \qquad \text{(Eq. 11.19)}$$

In this case, to reach a PG 64-xx grade, but not a PG 70-xx grade, the maximum RPBR should be 0.32.

The same approach was used for intermediate and low critical temperatures as shown below. Final values are indicated in **Table 11.7** and **Figures 11.7–11.10**.

For intermediate temperature (where T_{Blend} = 25°C for a PG 64-22 binder):

$$RPBR = \frac{25-15}{31-15} = \frac{10}{16} = 0.625 = 0.62 \qquad \text{(Eq. 11.20)}$$

FIGURE 11.7 ***High Temperature Blending Chart (RAP Percentage Unknown)***

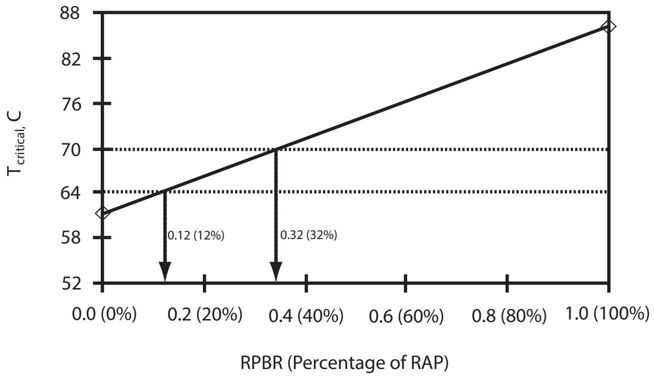


FIGURE 11.8 ***Intermediate Temperature Blending Chart (RAP Percentage Unknown)***

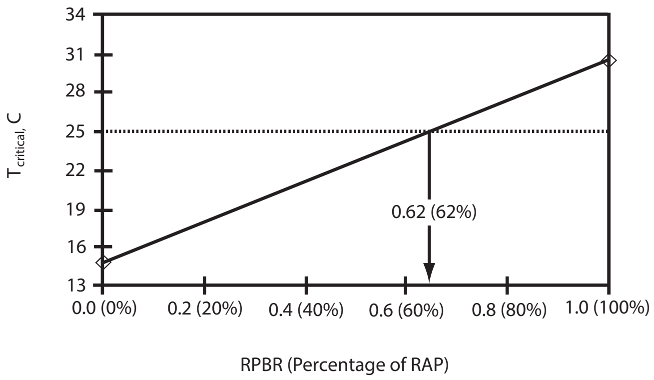


FIGURE 11.9 ***Low Temperature (S) Blending Chart (RAP Percentage Unknown)***

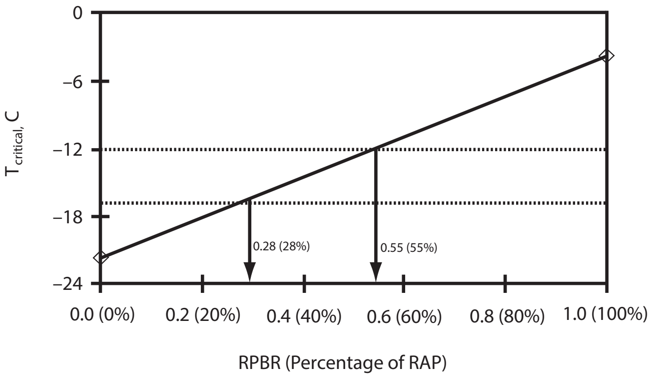
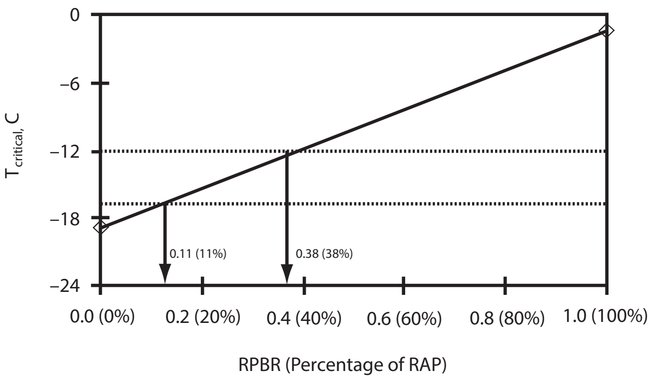


FIGURE 11.10 ***Low Temperature (m-value) Blending Chart (RAP Percentage Unknown)***



Note that for the intermediate temperature calculation, the RPBR is a maximum ratio instead of a minimum ratio. As such, the calculated value is rounded down from 0.625 to 0.62. It would not be likely that a mix designer would be interested in the minimum percentage of RAP binder needed to avoid having the blend be the next lower grade. If that was needed, then the same calculations can be performed, but with T_{Blend} equal to 23°C (one degree higher than the next lower grade).

For low temperature, S (where $T_{Blend} = -22°C$ for a PG 64-22 binder):

$$RPBR = \frac{-22-(-32)}{-14-(-32)} = \frac{10}{18} = 0.556 = 0.55 \quad \text{(Eq. 11.21)}$$

For low temperature, m-value (where $T_{Blend} = -22°C$ for a PG 64-22 binder):

$$RPBR = \frac{-22-(-29)}{-11-(-29)} = \frac{7}{18} = 0.389 = 0.38 \quad \text{(Eq. 11.22)}$$

As with the intermediate temperature calculation, the RPBR is a maximum ratio instead of a minimum ratio. As with the intermediate temperature calculations, the RPBR is rounded down for both low temperature parameters (S and m-value). In this case, the lower of the two calculated maximum ratios would be used, 0.38. Unlike the intermediate temperature, the mix designer might be interested in the minimum RPBR needed to avoid having a blend with too soft of a lower grade. As with the high critical temperature calculations, we simply use T_{Blend} set to one degree lower than the next lower grade (PG xx-28, in this instance) as shown below:

For low temperature, S (where $T_{Blend} = -27°C$ for a PG 64-22 binder):

$$RPBR = \frac{-27-(-32)}{-14-(-32)} = \frac{5}{18} = 0.278 = 0.28 \quad \text{(Eq. 11.23)}$$

For low temperature, m-value (where $T_{Blend} = -27°C$ for a PG 64-22 binder):

$$RPBR = \frac{-27-(-29)}{-11-(-29)} = \frac{2}{18} = 0.111 = 0.11 \quad \text{(Eq. 11.24)}$$

In this case, to reach a PG xx-22 grade, but not a PG xx-28 grade, the minimum RPBR should be 0.11 (controlled by m-value).

As indicated in **Table 11.7** and **Figure 11.7**, an RPBR between 0.12 and 0.32 should satisfy the high temperature requirements of the blended grade (PG 64-22) using the RAP and virgin asphalt binders in **Table 11.6**.

Table 11.7 and **Figure 11.10** indicate that an RPBR between 0.11 and 0.38 should satisfy the low temperature requirements of the blended grade (PG 64-22) using the RAP and virgin asphalt binders in **Table 11.6**. The data shown in **Figure 11.9** are less restrictive than the data in **Figure 11.10** and are therefore not used.

From **Table 11.7** and **Figures 11.7** and **11.10**, an RPBR between 0.12 and 0.32 would satisfy all the requirements of a blended PG 64-22 binder. If the maximum high temperature grade was not a concern, the RPBR could be increased to 0.38 without changing the desired low temperature grade of the blended asphalt binder.

To meet the intermediate temperature grade (G*sin δ) in **Figure 11.8**, the RPBR would need to be less than 0.62.

11.4.3 Viscosity blending using a known RAP percentage

After determining the absolute viscosity of the recovered RAP binder at 60°C, the mix designer can determine the virgin binder grade to be used, assuming the percentage of RAP binder that will be used in the mix is known. This can be done mathematically or graphically. **Figure 11.11** illustrates a blank semi-logarithmic viscosity blending chart.

To begin, a target viscosity of the final blended binder is selected. Commonly, the selected target viscosity is at the midrange of the final blended binder grade. For an AC-20 binder, the target viscosity would typically be 2,000 poises; for an AC-10 binder, 1,000 poises.

If performing the determination graphically, plot the viscosity of the recovered RAP binder on the left vertical axis (100 percent RAP binder or 0 percent virgin binder) of a semi-logarithmic viscosity blending chart. Then plot a data point using the target viscosity and the percentage of RAP binder to be used in the blend. Draw a line between the two data points and extrapolate the line to the right vertical axis (0 percent RAP binder or 100 percent virgin binder). The intersection of the line with the right vertical axis yields the required viscosity of the virgin asphalt binder that should be used with the recovered RAP binder at

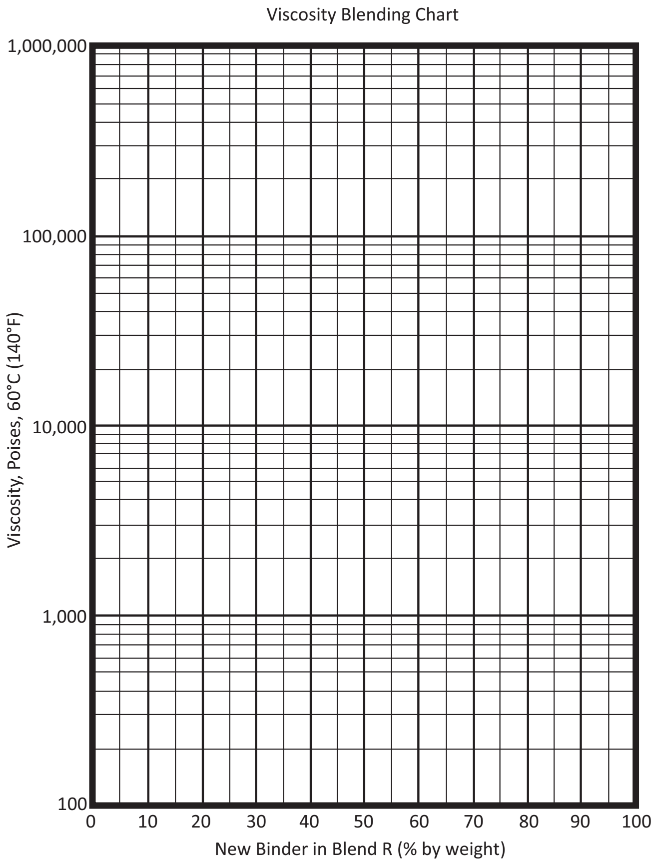


FIGURE 11.11 ***Blank Blending Chart***

the selected percentage of RAP binder to achieve the target blended binder viscosity.

Consider the following example:

- The absolute viscosity of the recovered RAP binder at 60°C is 75,000 poises.
- The specifying agency requires a blended binder grade of AC-20.
- The RAP binder percentage in the mixture is 30 percent.

As shown in **Figure 11.12**, the recovered RAP binder viscosity (75,000 poises) is plotted on the left vertical axis representing 100 percent RAP binder. The target blended binder viscosity is plotted at coordinates corresponding to 30 percent RAP binder percentage and 2,000 poises. A line is then connected between the two data points and extrapolated to the right vertical axis, representing 100 percent virgin binder. The viscosity of the virgin asphalt binder can then be read as approximately 450 poises. Thus, an AC-5 (400–600 poises) could be used as the virgin asphalt binder grade for the conditions shown in this example.

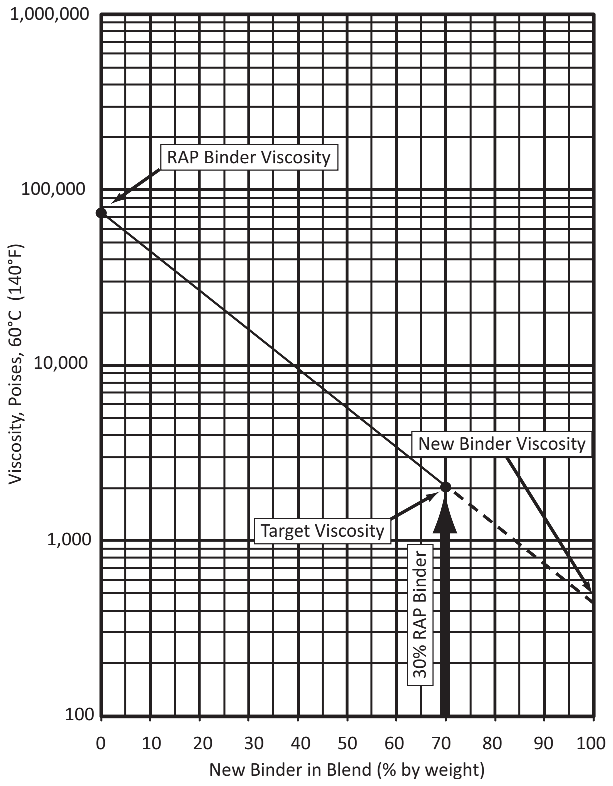


FIGURE 11.12 ***Blending Chart Example***

11.4.4 Testing reliability issues

Variability in test results typically can come from one of three sources: materials, sampling and testing. Often, variability in testing is attributed to the material being tested when, in reality, sampling or testing errors may have contributed to the variability in the test results.

Good sampling practices can effectively minimize variability in test results caused by sampling. Adherence to the proper test methods may minimize testing variability, but it still will be present. If sampling variability can be reduced by good sampling practices and testing variability can be properly accounted for, then material variability can be quantified.

The variability of recovered RAP asphalt binder properties can occur in either the recovery procedure or in the binder test procedure (i.e., DSR, BBR tests). Variability due to the combined effects of the recovery procedure and high temperature DSR testing is indicated in **Table 11.8** for two RAP samples tested in the NCHRP 9-12 research project (NCHRP Web Document 30).

The data in **Table 11.8** indicate that the test results from three separate recovery procedures

	$G^*/\sin\delta$, kPa							
	Kentucky				Florida			
	64°C	70°C	76°C	T_c	64°C	70°C	76°C	T_c
Rep 1	21.76	9.23	4.06	**86.0**	7.10	3.10	1.42	**78.6**
Rep 2	24.24	9.60	4.01	**85.3**	7.11	3.07	1.39	**78.4**
Rep 3	27.30	11.55	5.01	**87.4**	7.79	3.37	1.51	**79.0**
Average	24.43	10.13	4.36	**86.2**	7.33	3.18	1.44	**78.7**
σ (1s)	2.27	1.02	0.46	**0.9**	0.32	0.13	0.05	**0.2**
CV (1s%)	9%	10%	11%	**1%**	4%	4%	4%	**0.3%**
d2s	6.41	2.88	1.30	**2.5**	0.91	0.38	0.14	**0.7**
d2s%	26%	28%	30%	**3%**	12%	12%	10%	**1%**

TABLE 11.8 ***Testing Variability of AASHTO T 319 Method (with Toluene/Ethanol)***

indicated a change in the critical high temperature by as much as 2.1°C. The d2s limit defining the acceptable range of two test results (95 percent confidence limit) was 2.5°C for the Kentucky RAP and 0.7°C for the Florida RAP.

Applying a tolerance of 2.5°C to the critical high temperature of the RAP binder in **Tables 11.4** and **11.6** changes the value from 86°C to 88.5°C. The effect on the blending would change the virgin binder critical high temperature from 54.6°C (55°C) to 53.5°C (54°C) in Case 1—a change of approximately 1°C, but no change in the virgin binder grade. The effect on the blending would also change the minimum RPBR from 0.12 to 0.11 percent in Case 2. In either instance, because the RAP is being blended with virgin asphalt binder at percentages of (typically) 40 percent or less, variability in test results due to the recovery procedure and subsequent testing is decreased.

It should be noted that **Table 11.8** provides an indication of single laboratory testing variability associated with the modified AASHTO T 319 procedure. Multi-laboratory variability has yet to be determined. It should also be noted that no low temperature variability (single laboratory) was determined from the two RAP sources.

Other sources describe multi-laboratory testing variability associated with the PG binder tests. This information is readily available from the AASHTO Material Reference Laboratory Program. However, this testing variability is based on samples that are typically taken from the asphalt binder tanks, not recovered from an asphalt mixture sample. It is expected that the testing variability will increase as the binder is subjected to the recovery procedure.

Until testing variability can be sufficiently established for recovered asphalt binders, the mix designer may wish to add a factor of safety to ensure that the final blended asphalt binder grade is achieved. Based on the 2.5°C change in the critical high temperature of the recovered RAP binder (Kentucky), an increase/decrease of no more than 2.0°C in the critical temperatures of the desired binder grade should be sufficient. Therefore, a mix designer trying to achieve a PG 64-22 blended asphalt binder would fix the critical high temperature at 66°C instead of 64°C. The critical low temperature of the blended binder would be –24°C rather than –22°C. These adjustments may or may not result in a change in the virgin asphalt binder grade required or the percentage of RAP used in the mixture.

11.5

Developing the mix design

Once the appropriate virgin asphalt binder grade and percentage of RAP (or percentage of RAP binder) has been selected, the asphalt mixture can proceed through a normal mixture design process, with a couple of clarifications and minor changes.

11.5.1 Determining combined aggregate gradation

RAP should be treated like any other aggregate when determining the combined aggregate gradation—except that the mix designer must remember that the RAP contains aggregate and asphalt binder. Unlike virgin aggregates, the percentage of aggregate in the RAP will always be less than the total percentage of RAP added to the mixture.

To determine the combined gradation, the RAP_{Blend} percentage (i.e., how much RAP total will be used) is multiplied by the RAP aggregate percentage to get a stockpile percentage for determining combined mixture gradation. The equation is shown below:

$$RAP_{stockpile} = RAP_{Blend} \times \; 1 - \frac{P_{b,\,RAP}}{100} \qquad \text{(Eq. 11.25)}$$

where

$RAP_{stockpile}$ = the stockpile percentage of the RAP used in aggregate blending calculations

RAP_{Blend} = the total amount of RAP used in the mixture, in percent

$P_{b,RAP}$ = asphalt binder content of the RAP, expressed as a percent

11.5.1.1 Example

For example, consider that a mix designer wants to use 25 percent RAP with an asphalt content of 6.0 percent in a mixture. After finding the gradation of the RAP aggregate, the mix designer uses a blending program to determine the combined gradation. The stockpile percentage to use is shown as follows:

$$RAP_{stockpile} = 25 \times \; 1 - \frac{6.0}{100} \; = 23.5\%$$

The next step is to adjust the stockpile percentages of the virgin aggregates. This is done by first dividing the amount of each virgin aggregate used by the total amount of virgin aggregate in the mix and then proportioning that percentage as a function of the difference between the $RAP_{stockpile}$ and RAP_{Blend}. The equation is as follows:

$$VirginAgg_{stockpile} = VirginAgg_{Blend,n} + \frac{VirginAgg_{Blend}}{\left(\sum_1^n VirginAgg_{Blend,n}\right)} \times (RAP_{Blend} - RAP_{stockpile}) \qquad \text{(Eq. 11.26)}$$

where

$VirginAgg_{stockpile}$ = the stockpile percentage of a virgin aggregate used in blending calculations

$VirginAgg_{Blend,n}$ = the amount of a virgin aggregate (n) used in the mixture, in percent (where n = 1 is Stockpile #1, n = 2 is Stockpile #2, etc.)

RAP_{Blend} = the total amount of RAP used in the mixture, in percent

$RAP_{stockpile}$ = the stockpile percentage of the RAP used in aggregate blending calculations

For example, consider that a mix designer wants to use 25 percent RAP with 20 percent #57 coarse aggregate, 30 percent #8 coarse aggregate and 25 percent fine aggregate. The stockpile percentages to use are shown as follows:

For #57 coarse aggregate:

$$VirginAgg_{stockpile} = 20 + \frac{20}{(20+30+25)} \times (25 - 23.5)$$

$$= 20 + \frac{20}{75} \times 1.5$$

$$VirginAgg_{stockpile} = 20 + \frac{30}{75} \; = 20 + 0.4 = 20.4\%$$

For #8 coarse aggregate:

$$VirginAgg_{stockpile} = 30 + \frac{30}{(20+30+25)} \times (25 - 23.5)$$

$$= 30 + \frac{30}{75} \times 1.5$$

$$VirginAgg_{stockpile} = 30 + \frac{45}{75} \; = 30 + 0.6 = 30.6\%$$

For fine aggregate:

$$VirginAgg_{stockpile} = 25 + \frac{25}{(20+30+25)} \times (25 - 23.5)$$

$$= 25 + \frac{25}{75} \times 1.5$$

$$VirginAgg_{stockpile} = 25 + \frac{37.5}{75} \; = 25 + 0.5 = 25.5\%$$

Thus, for calculating the combined gradation, the stockpile percentages shown in **Table 11.9** would be used:

In addition to the combined gradation, the stockpile percentages should be used when calculating consensus aggregate properties and combined specific gravity.

11.5.2 RAP batching and handling in the lab

When preparing to conduct a mix design, the virgin aggregates are batched according to their stockpile percentages and blended into a combined batch that is then preheated at the mixing temperature (actually slightly above mixing temperature, as discussed earlier, to account for heat loss during weighing) prior to beginning the mix design process. When using RAP, the RAP aggregate and asphalt binder are batched together and must be kept separate and not heated like the virgin aggregates.

11.5.2.1 Example

As an example, consider that the mix designer wants to use a 5,000-gram batch of aggregates for preparation of a mixture. Using the stockpile percentages in **Table 11.9**, the amount of aggregate batched for each material is shown below:

#57 Coarse Aggregate	$20.4\% \times 5000$	$= 1020$ grams
#8 Coarse Aggregate	$30.6\% \times 5000$	$= 1530$ grams
Fine Aggregate	$25.5\% \times 5000$	$= 1275$ grams
RAP	$23.5\% \times 5000$	$= 1175$ grams
TOTAL		$= 5000$ grams

TABLE 11.9 ***Calculating Stockpile Percentages for Combined Gradation***

Material	Blend Percentage	Stockpile Percentage for Determining Combined Gradation
#57 Coarse Agg	20%	20.4%
#8 Coarse Agg	30%	30.6%
Fine Agg	25%	25.5%
RAP	25%	23.5%

The total amount of RAP that will need to be batched is calculated by multiplying the RAP aggregate weight by the $\text{RAP}_{\text{Blend}}$ divided by the $\text{RAP}_{\text{stockpile}}$ as shown below:

$$\text{RAP Batch Weight} = \text{RAP Aggregate Weight} \times \frac{\text{RAP}_{\text{Blend}}}{\text{RAP}_{\text{stockpile}}} \qquad \text{(Eq. 11.27)}$$

where

RAP Batch Weight = total weight of RAP to batch

RAP Aggregate Weight = weight of RAP aggregate in the blend

$\text{RAP}_{\text{stockpile}}$ = the stockpile percentage of RAP used in blending calculations

$\text{RAP}_{\text{Blend}}$ = the total amount of RAP used in the mixture, in percent

Using the example data above, the RAP Batch Weight is calculated as:

$$\text{RAP Batch Weight} = 1175 \times \frac{25}{23.5} = 1250 \text{ grams}$$

Thus for each 5,000-gram specimen to be made, 1,250 grams of RAP is batched separately in a pan.

11.5.3 Heating of RAP and virgin aggregates in the mix design process

Normally, virgin aggregates are batched, combined and placed in an oven overnight at the appropriate mixing temperature. Unfortunately, since RAP contains asphalt binder, it cannot be heated in the same manner without affecting its properties. For RAP mixtures, it is recommended that the RAP be batched separately in no more than 1- to 2-kilogram (1,000 to 2,000 gram) batches and heated for no more than two hours at 110°C (230°F). This is a sufficient time to heat the RAP and remove any extraneous surface moisture. Higher temperatures and longer heating times have been shown to change the properties of some RAP materials.

To compensate for the introduction of a lower temperature material, the virgin aggregate should be heated above the mixing temperature by a certain amount. Although actual mix temperatures vary, a good rule of thumb is to increase the temperature of the virgin aggregates by 0.5°C

(0.9°F) for every percent of RAP used in the mix (RAP_{Blend}). Thus for a mixture with RAP_{Blend} = 25 percent, the mixing temperature of the virgin aggregates should be raised by 12.5°C (22.5°F).

The mixing temperature of the virgin asphalt binder should not be adjusted.

To mix each sample, the pre-batched virgin aggregate is added to a mixing bowl (or bucket mixer) and the heated RAP is added and quickly mixed with the virgin aggregates. A crater is formed just as in the normal mixing process, and the virgin asphalt binder is added in the appropriate amount and the mixing process is started.

11.5.4 Determining new virgin binder quantity

Because the pre-batched RAP sample contains asphalt binder in addition to aggregate, the weight of the RAP binder must be accounted for so that the total asphalt binder content is correct. This is done by calculating the total weight of asphalt binder to be used for a specified asphalt binder content and then subtracting the RAP binder to get the weight of the virgin asphalt binder to be added. This is shown in the equation below:

$$\text{Virgin Binder Weight} = \text{Total Agg Weight} \times \frac{P_b}{P_s} - (\text{RAP Batch Weight} - \text{RAP Agg Weight}) \qquad \text{(Eq. 11.28)}$$

where

Virgin Binder Weight = the weight of the virgin binder to be added

Total Agg Weight = the total weight of virgin and RAP aggregates

P_b = desired asphalt binder content, percent

P_s = aggregate content, percent ($100 - P_b$)

RAP Batch Weight = total weight of RAP that was batched

RAP Aggregate Weight = weight of RAP aggregate in the blend

Using the data in the previous example, and assuming that a total asphalt binder content of 5.0 percent is desired, the virgin binder weight can be calculated as shown:

$$\text{Virgin Binder Weight} = \left(5000 \times \frac{5.0}{95.0}\right) - (1250 - 1175)$$

$$\text{Virgin Binder Weight} = (263) - (75) = 188$$

As shown above, to get a total asphalt binder content of 5.0 percent in the mixture, we need 263 grams of asphalt binder added to an aggregate weight of 5,000 grams. However, since the pre-batched RAP (1,250 grams) contains some asphalt binder, we need to account for it to calculate the amount of virgin asphalt binder to use. In this example, 75 grams of RAP asphalt binder exists in the mix, leaving 188 grams of virgin asphalt binder to be added to get a total asphalt binder content of 5.0 percent.

Once the batching and mixing processes are complete, the mix design process is unchanged by the addition of RAP.

CHAPTER 12

Specialty Mixes

12.1

Introduction

In this chapter, a number of specialty mixes will be described. Mix design procedures and parameters will be discussed in an effort to assist those asked to design the mixes covered. In previous chapters in this manual, the discussion has centered on general mix design procedures that are used for mostly dense-graded mix types, including those designed using Superpave, Marshall and Hveem mix design systems. The mixes described here are designed for a very specific purpose and/or application and therefore sometimes require mix design test procedures and criteria that are not typical for dense-graded materials. However, the standard test procedures already discussed are applicable and widely utilized in evaluating and designing many of these mixtures.

12.2

Airfield mixes

Hot mix asphalt (HMA) is used extensively for airfield pavements. HMA can be used for virtually all types of pavements and loading conditions that exist on the smallest to the largest airfields in the world. As in all HMA applications, a mix design must be performed to produce a blend of aggregates and binder that efficiently provides long-lasting performance when placed and compacted.

HMA materials that will perform well in airfield applications are different than those used for highways, roadways and streets. Even though portions of runways and taxiways have sufficient strength to accommodate extremely heavy loading, much of the pavement carries virtually no or infrequent loads. Airfield mixes must be designed to stay flexible and durable over a long period of time and avoid premature aging on areas that do not receive traffic.

The Federal Aviation Administration (FAA) has a set of specifications called Advisory Circulars (ACs). Asphalt pavement specifications are in AC 150/5370-10, Part 5—Flexible Surface Courses. To locate the most recent FAA specifications for asphalt pavements, go to:

www.faa.gov/airports/engineering/construction_standards/.

The FAA's dense-graded mix type is specified as P-401 or P-403. P-403 is similar to P-401 but without percent within limit calculations. Both specifications provide for a tailoring option for the design engineer to select either the Superpave Gyratory Compactor or Marshall Hammer.

Both P-401 and P-403 specifications directly refer to this manual.

The FAA in 2014 adopted an innovative asphalt mixture that reduces pavement rutting and shoving while increasing resistance to damage from fuel spills. The specification for this fuel-resistant mixture is P-601. The FAA's guidance states that P-601 mix should only be used as a surface course, 1 to 2 inches thick, to provide a fuel-resistant surface where pavements are subject to fuel spills. The mix uses a fine aggregate gradation and a highly polymer-modified asphalt binder. Key to the fuel resistance is not only the high polymer content but also making the mix extremely impermeable, accomplished by the fine gradation and designing at 2.5 percent air voids versus the normal 3.5 percent.

Regarding military airfield mixes, the specifications can be found in the "Unified Facilities Guide Specifications (UFGS)" at www.wbdg.org.

The predominant dense-graded asphalt mixture is UFGS-32 12 15-13 Hot-Mix Asphalt Airfield Paving. Similar to the FAA's P-401, this specification provides a tailoring option for the design engineer to select either the Superpave Gyrator Compactor or Marshall Hammer.

12.3

Open-graded mixes

Open-graded mixes are designed with uniformly sized aggregates to facilitate drainage, either through the base or surface of the pavement structure. Base mixes are typically composed of ¾ to 1 inch (19 to 25 mm) nominal maximum aggregate size (NMAS) aggregate chips, while surface mixes are typically composed of ⅜ to ½ inch (9.5 to 12.5 mm) NMAS aggregate chips. Open-graded surface mixes are being specified more and more for their safety and environmental characteristics.

12.3.1 Open-graded base mixes

Open-graded bases are sometimes referenced by the term Asphalt Treated Permeable Base (ATPB). Their purpose is to provide a means to drain water that enters the pavement structure, either from the surface or the subgrade. It is imperative that this layer is used in conjunction with some type of pavement drainage system to allow the water a means of egress from the system. The system can range from something simple, like shouldering with coarse aggregate, to something more complex, such as a pavement edge drain system. The main point is to avoid building a layer into the pavement structure that acts as a reservoir, which could rapidly increase the degradation of the pavement structure.

ATPB mixes are typically simple mixtures, often designed using only one aggregate size with predetermined binder content, usually in the range of 1.5 to 3.0 percent by weight of the total mix. Since the base is not intended to be used as a driving surface except during construction, aggregates with a high skid resistance are not required. However, the aggregate should not be so soft as to degrade significantly during production, creating fines that fill the voids necessary for proper drainage. The combination of large NMAS and low binder content makes these one of the most cost-effective mix types.

A neat (unmodified) binder is generally sufficient for these types of bases. If an ATPB mix is specified in a pavement structure, there is typically an ample thickness of pavement above it, which reduces the stress in the open-graded base. **Table 12.1** shows a typical gradation and binder content specified for open-graded bases.

At the mixing plant, this type of mix is often produced at a lower temperature to prevent drain-down. Although some modified binders or fibers can be incorporated to allow increased mixing temperatures without drain-down, they are usually reserved for open-graded surface mixes due to their additional cost and the location in the pavement structure.

TABLE 12.1 ***Typical open-graded base/ATPB mixture specifications***

Sieve Size	Typical	Alternative
2" (50 mm)	100	Use either an AASHTO #57 or #67 aggregate, with final gradation and binder targets based on source
1 ½" (37.5 mm)	75–100	
¾" (19 mm)	50–90	
½" (12.5 mm)	25–65	
No. 4 (4.75 mm)	0–20	
No. 200 (0.075 mm)	0–5	
Target % Binder	1.5–3.0	1.5–3.0

Coarse Aggregate Quality Requirements			
Test	**Method**	**Min (%)**	**Max (%)**
L.A. Abrasion	AASHTO T 96	—	30
Flat & Elongated	ASTM D4791 (5:1) ASTM D4791 (2:1)	— —	5 20
Fractured Faces (ff)	ASTM D5821 (2 ff) ASTM D5821 (1 ff)	90 100	— —
Soundness (5 cycles)	AASHTO T 104 *Sodium Sulfate*	—	10
	AASHTO T 104 *Magnesium Sulfate*	—	15
Uncompacted Voids (CAA)	AASHTO T 326 Method A	45	—

TABLE 12.2 ***Typical coarse aggregate properties of open-graded surface mixes***

Mixing time is often reduced from what is typically used for dense-graded mixes due to the smaller surface area of this type of mix and the desire to minimize degradation of the aggregate in the drum.

Some construction practices must be altered when placing open-graded bases. Do not tack the vertical faces of this mix, because it may impede its ability to drain properly. Use only two or three passes of a steel-wheel roller in static mode to properly seat the mix, but avoid over-densification or breakdown of the aggregates. ATPBs have historically been sufficiently durable enough to withstand the heavy vehicles associated with construction traffic.

12.3.2 Open-graded surface mixes

Open-graded surface mixes are referenced by several different terms, including Open Graded Friction Course (OGFC) and Permeable (or sometimes *Porous*) Friction Course (PFC). Their purpose is to provide drainage on the surface of an HMA pavement under wet conditions. The water drains vertically through the OGFC to an impermeable underlying layer and then laterally to the day-lighted edge of the OGFC. An OGFC layer will provide many benefits, including:

- reduced splash and spray;
- reduced potential for hydroplaning;
- high skidding resistance on wet pavement;
- enhanced visibility of pavement markings;
- reduced nighttime surface glare in wet weather; and
- reduced tire-pavement noise.

Many agencies have their own methods of designing open-graded surface mixes. The following section summarizes typical aggregate quality requirements for open-graded surface mixtures. Additional information can be found in NCHRP Report 640, "Construction and Maintenance Practices for Permeable Friction Courses."

In general, the design process includes selection of aggregates and binder meeting established criteria, the blending and mixing of materials, selection of optimum binder content and the testing of both loose and compacted mixes for properties such as abrasion, drain-down and moisture susceptibility.

12.3.2.1 Materials selection

Table 12.2 presents the coarse aggregate criteria for open-graded surface mixtures.

The aggregate structures of open-graded surface mixtures are often made up solely of coarse aggregates to maximize the void space. In some instances, a fine aggregate such as manufactured sand is included to enhance the mix durability. For those cases, typical fine aggregate criteria are shown below (see **Table 12.3**):

TABLE 12.3 ***Typical fine aggregate properties of open-graded surface mixes***

Fine Aggregate Quality Requirements			
Test	**Method**	**Min (%)**	**Max (%)**
Uncompacted Voids (FAA)	AASHTO T 304 Method A	45	—
Soundness (5 cycles)	AASHTO T 104 Sodium Sulfate	—	10
	AASHTO T 104 Magnesium Sulfate	—	15
Sand Equivalent	AASHTO T 176	50	—

Asphalt binders used in open-graded surface mixes are generally stiffer than those used for standard environmental and traffic conditions and are usually modified. Using the Superpave binder classification system, the high-temperature grades are typically increased by two grades over the standard asphalt binder. Roadways with medium traffic and fast-moving traffic may only need a one-grade bump in the high-temperature grade.

Stabilizing additives such as cellulose or mineral fibers are typically used in open-graded surface mixes to prevent drain-down during storage, transportation and placement of the mix. Mineral fibers are usually specified at 0.4 percent of the mix mass, while cellulose fibers are typically specified at 0.3 percent because their irregular shape tends to hold more binder than the straight-shaped mineral fibers. A combination of modified binders, production temperatures under 300°F and relatively short-haul distances have allowed for the elimination of fibers on some projects.

The aggregate gradation is a critical factor to ensure the high level of permeability desired in open-graded mixes. As with other mixes, if blending a high or low specific gravity aggregate with standard aggregates, the blend should then be evaluated by volume, not by mass. The following table (see **Table 12.4**) shows typical gradation bands for open-graded surface mixes.

TABLE 12.4 ***Example gradation specifications of open-graded surface mixes***

Sieve Size	NCHRP 640 1/2"	NCHRP 640 3/8"	Oregon 1/2"	Georgia 1/2" PEM
1 ½" (37.5 mm)				
1" (25 mm)				
¾" (19 mm)	100		99–100	100
½" (12.5 mm)	80–100	100	90–98	80–100
⅜" (9.5 mm)	35–60	85–100	—	35–60
No. 4 (4.75 mm)	10–25	20–30	18–32	10–25
No. 8 (2.36 mm)	5–10	5–15	3–15	5–10
No. 16 (1.18 mm)	—	—	—	—
No. 30 (0.60 mm)	—	—	—	—
No. 50 (0.30 mm)	—	—	—	—
No. 100 (0.15 mm)	—	—	—	—
No. 200 (0.075 mm)	0–4	0–4	1–5	1–4

12.3.2.2 Trial gradation selection and evaluation

It is mandatory to have a coarse aggregate skeleton with stone-on-stone contact to avoid rutting. This condition can be verified by ensuring that the voids in the coarse aggregate (VCA) fraction of the compacted mix (VCA_{mix}) are less than the VCA of the coarse aggregate in a dry-rodded condition (VCA_{DRC}). When the dry-rodded density of the coarse aggregate fraction has been determined according to AASHTO T 19, the VCA_{DRC} can be calculated using the following equation:

$$\text{VCA}_{\text{DRC}} = \frac{G_{\text{ca}}\gamma_{\text{w}} - \gamma_{\text{s}}}{G_{ca}\gamma_{\text{w}}} \times 100$$

where:

G_{ca} = bulk specific gravity of the coarse aggregate (AASHTO T 85)

γ_{w} = unit weight of water

γ_{s} = unit weight of the coarse aggregate fraction in the dry-rodded condition

The VCA_{mix} can be calculated using information from compacted mix specimens using the following equation:

$$\text{VCA}_{mix} = 100 - \left(\frac{G_{mb}}{G_{ca}}\right) \times P_{ca}$$

where:

G_{mb} = bulk specific gravity of the compacted mixture

G_{ca} = bulk specific gravity of the coarse aggregate (AASHTO T 85)

P_{ca} = percentage of the coarse aggregate in the total mixture

The VCA_{DRC} is a constant, as determined by AASHTO T 19, for each coarse aggregate blend. As the mix is compacted, the VCA_{mix} gets smaller. Stone-on-stone contact is assured when the voids in the compacted coarse aggregate fraction are less than when the coarse aggregate is in the dry-rodded condition while maintaining a minimum of 18 percent air voids. **Figure 12.1** shows the concept of VCA_{DRC} versus VCA_{mix}.

It is recommended to compact specimens using the Superpave Gyratory Compactor (SGC) at 50 gyrations to avoid aggregate breakdown. The binder content is typically in the 6.0 to 7.0 percent

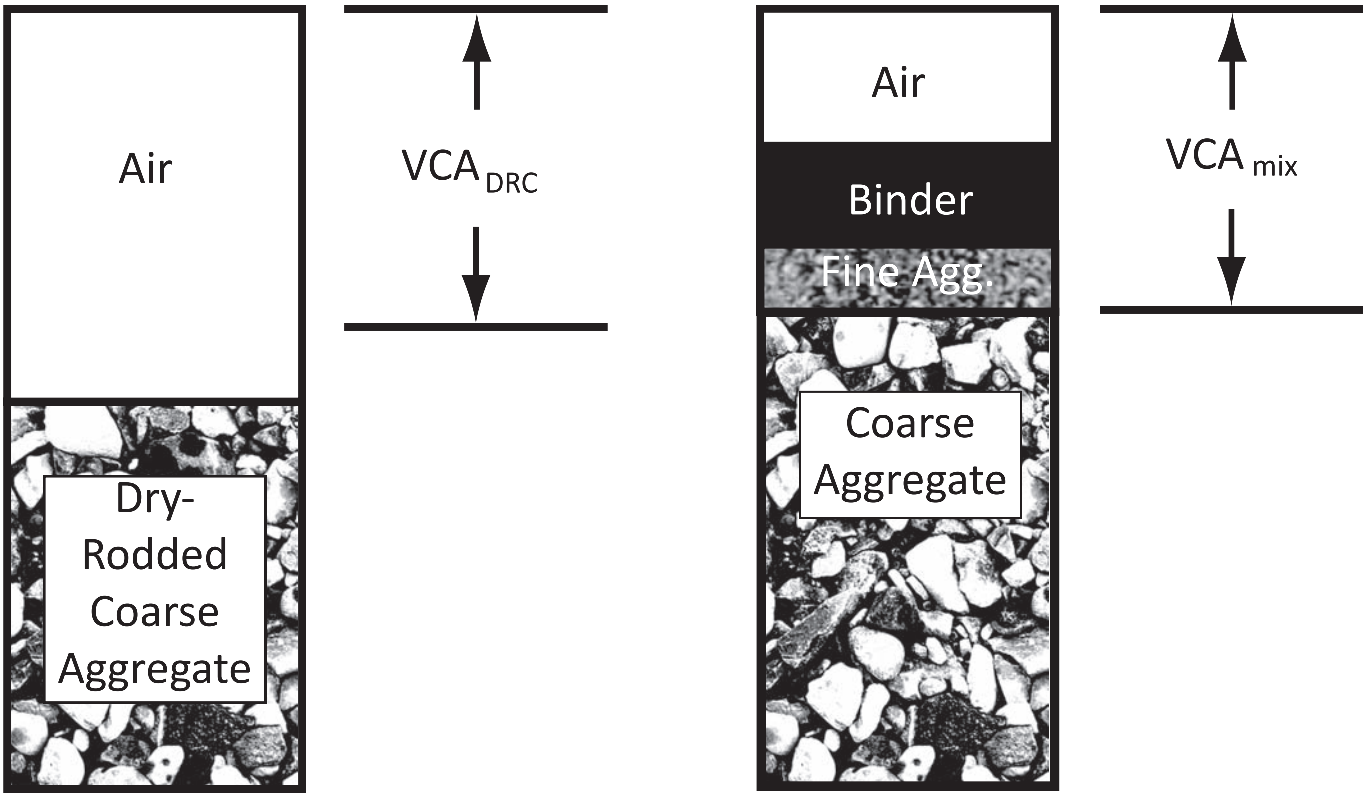


FIGURE 12.1 ***Conceptual difference between VCA_{DRC} and VCA_{mix}***

range. The NCHRP procedure suggests decreasing minimum binder contents for combined aggregate bulk specific gravities above 2.75 and increasing them for combined aggregate bulk specific gravities below 2.55. The mixing and compaction temperatures should be the same as those used for regular dense-graded mixes.

It is important to note that when determining G_{mb} on compacted open-graded specimens, a CoreLok® or comparable method must be used. The specimen saturated surface dry (SSD) weight cannot be properly measured using AASHTO T 166 because the water in the void spaces will drain away, resulting in a volume calculation that will be much smaller than the actual volume, and therefore a G_{mb} that is much greater than the actual bulk specific gravity.

Drain-down tests should be conducted on loose mix according to AASHTO T 305 (ASTM D6390) at a temperature 15°C higher than the anticipated production temperature.

Some agencies also run the Cantabro abrasion test, which uses compacted mix specimens in an L.A. Abrasion machine. The Cantabro abrasion test measures the breakdown of the compacted specimen after the completion of a specified number of revolutions. The percent of weight loss is said to be an indication of durability and is related to the quantity and quality of binder utilized. At the time of this writing, there were no AASHTO or ASTM procedures for the Cantabro test, but other test methods like the Texas DOT TEX-245-F can be found online.

The asphalt content that meets the following criteria (see **Table 12.5**) is selected.

If moisture susceptibility testing is desired, consideration should be given to the boil test or conducting a submerged Hamburg or submerged APA test.

TABLE 12.5 ***Open-graded surface mixture specifications***

NCHRP 640 Open-Graded Surface Mixture Specifications		
Test	**Method**	**Requirement**
Air Voids	AASHTO T 331	18%–22%
Cantabro Abrasion	TEX-245-F	15% max.
Drain-down	AASHTO T 305	0.3% max.

FIGURE 12.2 ***Drain-down basket***

12.3.2.3 Evaluation of drain-down

Due to the open-graded nature of an OGFC mixture, the possibility exists that the asphalt binder will drain off the aggregate during the construction process. To prevent this, a fiber is added to the mix.

The drain-down test is used to determine the correct amount of fiber needed for the mix. The test measures the potential for asphalt binder to drain from the coarse aggregate structure while the mix is held at an elevated temperature. The test is performed in accordance with ASTM D6390.

To run this test, a sample is prepared in the laboratory (during mix design) or obtained from field production. The sample is placed in a wire basket (see **Figure 12.2**) that is put onto a suitable container of known mass (generally a paper plate).

The sample, basket and container are then placed into a forced-draft oven for one hour at or above the anticipated production temperature. At the end of one hour, the mass of asphalt binder draining from the sample that is retained in the container is determined and the amount of drain-down calculated. If more than 0.3 percent drain-down is measured, adjustments in the type or percentage of fiber should be made and the drain-down test should be repeated.

12.3.2.4 Mixing at the plant

The main modification required to a standard asphalt mixing plant is the addition of a fiber-feeding device when required. The fibers must be fed homogeneously throughout the mix, in a manner which avoids either clumps of fibers or areas without fibers. The fibers are usually introduced either loose or in pelletized form. It is sometimes desirable to increase the mixing time to ensure a thorough dispersion of fibers and a uniform coating of binder on the aggregate.

Be aware that overflows in the hot bin screening of batch plants may occur with open-graded mixes due to the large proportion of similar-sized aggregate. Due to the potential drain-down issues of open-graded mixes, they should only be stored in silos or surge bins for very short periods of time.

12.3.2.5 Hauling, placement and compaction

Because open-graded surface mixes are usually made with some type of modified binder, extra attention must be given to treating truck beds with release agents to prevent the mix from sticking. Tarping of the mix in the haul truck is important to prevent excessive crusting, which could cause mat problems.

The surface on which open-graded mixes are laid must be impermeable and well tacked. The open-graded mix must be daylighted so that rainwater can flow freely out of the pavement. Avoid tacking the vertical faces of adjacent lanes to allow the free flow of water. Material transfer devices are highly recommended for the placement of open-graded surface mixes and little or no handworking of the mix should be done. Screeds must be maintained hot to avoid tearing of the mat.

Steel-wheel rollers in static mode are recommended for this type of mix. Pneumatic (rubber tire) rollers should not be used because they may seal the surface more, preventing the full flow of water through the mix and may also cause mat pick-up problems. Vibratory mode can more easily crush aggregate due to the stone-on-stone contact of open-graded surface mixes. One or two passes with the static steel-wheel roller are usually sufficient to seat the mix in place without over-densification.

12.4

Stone matrix asphalt

Stone matrix asphalt (SMA) mixtures have been used in the United States since 1991. These mixtures have performed well in Europe for many years, and experience in the United States is similar. Stone matrix asphalt is an HMA consisting of two parts: a coarse aggregate skeleton and a rich asphalt binder mortar.

The mixture must have an aggregate skeleton with stone-on-stone contact. The coarse aggregate is generally considered to be that fraction of the aggregate retained on the 4.75-mm (No. 4) sieve but may be designated as other sizes, depending on the nominal maximum aggregate size of the mix. The mix relies on this stone-on-stone contact to provide its internal strength. The rich binder mortar is a mixture of mineral filler, stabilizing fibers and binder. The purpose of the mortar is to partially fill the voids in the coarse aggregate skeleton and also to prevent drain-down of the asphalt binder.

The mix uses a high asphalt binder content of 6 to 7 percent or more. This high binder content provides improved durability. In most situations, the asphalt binder is polymer-modified. The in-place air voids are kept low—to less than 6 percent to ensure that the mix is impermeable to water.

SMA mixes can be used as either intermediate (binder) or surface pavement layers. Typical nominal maximum size of SMA surface mixes are ½ inch (12.5 mm) and ⅜ inch (9.5 mm), while SMA mixes used in intermediate or binder courses are ¾ inch (19.0 mm), as shown in **Table 12.8.** SMA is routinely used in many states as the surface of choice for high-volume roadways and intersections. SMA mixes are generally placed at compacted thicknesses that vary from 1.5 inches (37.5 mm) to 3.0 inches (75 mm).

12.4.1 Mix design characteristics and procedures

The mix design procedure presented is based on AASHTO R 46, “Designing Stone Matrix Asphalt (SMA).” This specification covers the design of SMA using the SGC. The SMA design is based on the volumetric properties of the SMA in terms of air voids (P_a), the voids in mineral aggregate (VMA) and the presence of stone-on-stone contact.

The following mix design process consists of these five steps:

1. Select proper materials (aggregate, asphalt binder, mineral filler and fibers).
2. Determine the aggregate gradation that yields stone-on-stone contact.
3. Ensure that the selected gradation provides the minimum required VMA.
4. Choose an asphalt binder content that provides the desired air voids.
5. Evaluate the mix for moisture susceptibility.

12.4.2 Materials selection

SMA mixtures require the use of a high-quality aggregate. AASHTO specifications require that 100 percent of the coarse aggregate have at least one crushed face and at least 90 percent must have two or more crushed faces. The strength of an SMA mixture is achieved through stone-on-stone contact. Therefore, most agencies have placed more stringent requirements on aggregate used in SMA. AASHTO requires a Los Angeles Abrasion loss of no more than 30 percent. But some states—because of a lack of high-quality aggregates—have built successfully performing projects with a less-stringent requirement.

Tables 12.6 and **12.7** provide the quality requirements for coarse and fine aggregate.

The gradation for different SMA mixtures is shown in **Table 12.8**. Note that for the 19 mm, 12.5 mm and 9.5 mm mixes, the mixtures have a low percentage passing the No. 4 sieve and a high percentage passing the No. 200 sieve. This is required to provide a mix with stone-on-stone contact and to provide a high design VMA (generally 17 percent or higher). The high percentage passing the No. 200 sieve is required to provide a stiff mix that will be rut-resistant and to ensure that drain-down does not occur during construction. The high percentage of minus No. 200 material usually requires that as much as 5 to 7 percent mineral filler is added to the SMA mix.

The asphalt binder shall be a performance grade which is appropriate for the climate and traffic-loading conditions at the site of the paving project. The asphalt binder for SMA mixtures has typically been a stiffer binder than is usually specified for dense-graded mixtures and are typically polymer-

Test	Criteria	Method	Minimum	Maximum
L.A. Abrasion, % loss		T 96	—	30[a]
Flat and Elongated, %[b]	3 to 1	D4791	—	20
	5 to 1	D4791	—	5
Absorption, %		T 85	—	2.0
Soundness (5 cycles), %		T 104		
	Sodium Sulfate			15
	Magnesium Sulfate			20
Crushed Content, %[c]		D5821		
	One Face		100	—
	Two Face		90	—

NOTES:

a Aggregates with higher L.A. Abrasion values have been used successfully to produce SMA mixes. However, when the L.A. Abrasion exceeds 30 percent loss, excessive breakdown may occur in the laboratory compaction process or during in-place compaction in the field.

b Flat and Elongated criteria apply to the design aggregate blend.

c Sodium sulfate or magnesium sulfate may be used. It is not a requirement to perform both methods.

TABLE 12.6 ***Coarse aggregate quality requirements***

modified. The use of stiffer binders will reduce the drain-down during construction and improve the rut resistance.

A stabilizer is typically used in the SMA mixture to prevent drain-down of the asphalt binder during construction. Two types of fibers—cellulose fibers and mineral fibers—have been used. Both types of fibers have been used successfully. The mineral fiber is usually heavier than cellulose fiber and is therefore added at a higher percentage (by weight) so that a sufficient volume of fibers is added.

Mineral filler shall consist of finely divided mineral matter such as crusher fines, fly ash or other approved products. At the time of use, it should be sufficiently dry to flow freely and be essentially free from agglomerations. Filler shall be free from organic impurities and have a plasticity index not greater than four.

12.4.3 Selection of desired gradation

Once satisfactory materials have been identified, the optimum aggregate gradation and binder content must be determined. This is accomplished by first selecting an appropriate aggregate blend. This blended gradation should fall within the specified gradation tolerances, provide an aggregate skeleton with stone-on-stone contact and furnish a mixture that meets

Test	Criteria	Method	Minimum	Maximum
Soundness (5 cycles), %[a]		T 104		
	Sodium Sulfate		—	15
	Magnesium Sulfate		—	20
Plasticity Index, %		T 90	Nonplastic	

NOTE:

a Sodium sulfate or magnesium sulfate testing can be used. It is not a requirement to use both.

TABLE 12.7 ***Fine aggregate quality requirements***

Sieve Size	Mixture Designation		
	19 mm	12.5 mm	9.5 mm
1 inch (25 mm)	100		
¾ inch (19 mm)	90–100	100	
½ inch (12.5 mm)	50–88	90–100	100
⅜ inch (9.5 mm)	25–60	50–80	70–95
No. 4 (4.75 mm)	20–28	20–35	30–50
No. 8 (2.36 mm)	16–24	16–24	20–30
No. 16 (1.18 mm)	—		21 max.
No. 30 (0.60 mm)	—		18 max.
No. 50 (0.30 mm)	—		15 max.
No. 200 (0.075 mm)	8–11	8–11	8–12

TABLE 12.8 ***Typical gradations for SMA mixes (percent passing by volume)***

or exceeds the minimum voids in mineral aggregate.

The condition of stone-on-stone contact is defined as the point at which the voids in the coarse aggregate fraction of the blend of the compacted SMA mixture (VCA_{mix}) is less than the VCA of the coarse aggregate fraction of the aggregate blend alone (VCA_{drc}) in the dry-rodded test (AASHTO T 19 or ASTM C29) as previously discussed in section 12.3.2.2.

12.4.4 Evaluation of mixture volumetrics

After the trial gradation samples have been compacted and allowed to cool, they are removed from the molds and tested to determine bulk specific gravity in accordance with AASHTO T 166. The volumetric properties of each compacted sample are then determined. The following are the volumetric calculations of interest:

$$P_a,\ \% = 100 \times \left(1 - \frac{G_{mb}}{G_{mm}}\right)$$

$$VCA_{mix},\% = 100 - \left(\frac{G_{mb}}{G_{ca}} \times P_{ca}\right)$$

$$VMA,\ \% = 100 - \left(\frac{G_{mb}}{G_{sb}} \times P_s\right)$$

where:

P_a = percentage of air voids in compacted mixture (by volume)
VCA_{mix} = voids in coarse aggregate fraction within compacted mixture
VMA = voids in mineral aggregate
G_{mb} = bulk specific gravity of compacted mixture
G_{mm} = theoretical maximum specific gravity (AASHTO T 209 or ASTM D2041)
P_s = percentage of aggregate in the total mixture (by weight)
P_{ca} = percentage of coarse aggregate in the total mixture (by weight)
G_{sb} = combined bulk specific gravity of the aggregates
G_{ca} = combined bulk specific gravity of the coarse aggregate fraction

Of the trial blends evaluated, the one that meets or exceeds the minimum VMA requirement and has a VCA_{mix} less than VCA_{DRC} should be selected as the design gradation. When an SMA mix has a VCA_{mix} less than VCA_{DRC}, it has achieved stone-on-stone contact.

Once a trial blend has been selected as the design gradation, it may be necessary to raise or lower the asphalt binder content to obtain the proper amount of air voids. Additional samples are prepared using the design gradation, and the binder content is varied.

The final chosen mix should meet the requirements shown in **Table 12.9**.

SMA mixture specifications refer to specimens compacted in accordance with AASHTO T 312 at 100 gyrations. Research has indicated that 100 gyrations may cause significant degradation of the aggregate structure in the compactor that will not occur in the field during mix placement. Many highway agencies in the U.S. have reduced the laboratory compaction requirements during SMA mix design. Compaction levels as low as 50 gyrations have been reported. Designers are encouraged to analyze laboratory-compacted SMA specimens for excessive aggregate degradation by conducting extraction or ignition oven testing. Mixtures that primarily consist of aggregate greater than the #4 sieve will develop a “lock up” point which may occur at compaction levels well below 100 gyrations. The Asphalt Institute recommends that a laboratory compaction level be selected at 75 gyrations or less. The chosen gyration level should be representative of field compaction to prevent aggregate degradation.

12.4.5 Evaluation of drain-down

Due to the gap-graded nature of an SMA mixture, the possibility exists that the asphalt binder will drain off the aggregate during the construction process. To prevent this, a fiber is added to the mix and it is evaluated as discussed in section 12.3.2.3.

12.4.6 Evaluation of moisture susceptibility

It is important to perform laboratory testing to determine the moisture susceptibility of the SMA mixture. This will help to identify mixtures that may tend to strip (lose the bond between aggregate and binder) during their service life. The test is performed in accordance with AASHTO T 283, and the recommended minimum tensile strength ratio (TSR) is 0.80 for mix design.

Property	Requirement
Air Voids, %	4.0[a]
VMA, %	17.0 min.
VCA_{mix}, %	Less than VCA_{DRC}[b]
Tensile Strength Ratio, AASHTO T 283	0.80 min.
Drain-down at Production Temperature, %	0.3 max.
Asphalt Binder Content, %	6.0 min.[c]

NOTES:

a For low-traffic-volume roadways or colder climates, design air voids less than 4.0 percent should be considered, but not less than 3.0 percent.

b See AASHTO R 46 for instructions on calculating VCA_{mix} and VCA_{DRC}.

c Experience has shown that binder contents should be from 6.0 to 7.0 percent. Lowering the binder content below 6.0 percent can detrimentally affect the durability of the SMA. When an SMA mix cannot be designed within the minimum binder content requirement using the available aggregates, refer to guidance in AASHTO R 46.

TABLE 12.9 ***SMA mixture specifications for Superpave Gyratory Compactor***

12.5

Rich bottom layer mixes

12.5.1 General description

A Rich Bottom Layer (RBL) is designed as a fatigue resistant layer in a Perpetual Pavement (PP) design. Its fatigue resistance is achieved by having the dense-graded HMA designed with higher asphalt content and lower air voids content.

Perpetual Pavements are designed and built to last longer than 50 years, without major structural rehabilitation or reconstruction. Only the surface is periodically renewed.

A Perpetual Pavement is achieved by assuring that the tensile strain at the bottom of the lowest HMA layer does not exceed the “endurance limit,” below which no material fatigue occurs. According to the most recent NCHRP reports (762), there is no single value of the endurance limit valid for all conditions. The magnitude varies depending on mix parameters, temperature and rest period between load applications. Recycled materials that may be included in the mix such as RAP and RAS may have a negative impact on endurance limit values. Documented values range from 20 to 300 microstrain, with values from 70 to 125 the most commonly reported.

12.5.2 Material characteristics

Rich Bottom Layer mixes can be designed to include either neat or modified asphalt binders, depending on the agency’s philosophy for binder selection in base layers.

The NMAS of the aggregate blend should be less than one-fourth the compacted thickness of the RBL to maximize compactibility. RBLs are typically designed to be 3 to 4 inches thick and laid in one lift.

12.5.3 Mix design procedures

Mix design is conducted using conventional mix design methodology. The RBL mixture is a standard dense-graded mix but with higher asphalt content to achieve lower air voids. This is typically achieved by designing at 2 to 2.5 percent air voids. In the past, this has also been achieved by designing at 4 percent air voids and arbitrarily adding an additional 0.5 percent binder. The RBL in the field should be compacted to less than 4 percent in-place air voids.

Some agencies conduct additional testing during mix design, including beam fatigue testing to verify that the mixture has improved fatigue properties. In the future, expect an evolution in mix design practices for RBL as their use becomes more widespread.

12.6

Hot In-Place Recycling (HIPR)

There are a number of different basic types of asphalt pavement hot in-place recycling processes. HIPR is performed on-site, in-place and the pavement typically is processed to a depth of ¾ to 2 inches (20 to 50 mm). The asphalt pavement is heated, softened and scarified to the depth specified. There are three specific hot in-place recycling methods including:

- **surface recycling or heater-scarification** a process where the surface is heated, loosened, modified, remixed and relaid as a new wearing surface;
- **remixing** a process where new HMA mixture is combined directly with the recycled mixture to significantly improve the recycled mixture before being placed; and
- **repaving** a process where the recycling surface is overlaid immediately with a new HMA mixture to form a thermal bond between the newly recycled mixture and the new HMA wearing surface.

Typically, HIPR is used on roadways with low to moderate anticipated traffic and loading conditions.

The philosophy of the HIPR mix designs has been to restore the properties of the existing, aged asphalt pavement to what would be expected of a virgin HMA. This approach attempts to account for the inevitable changes that have been made to mix properties in the existing HMA due to time, traffic and the HIPR process itself.

Prior to mix design, a distress survey and evaluation of the in-place properties of the existing asphalt pavement will be helpful in identifying specific problems that can be addressed in mix design. For example, if the predesign evaluation shows that the asphalt binder properties are inadequate for the anticipated climatic and loading conditions, the grade and amount of virgin binder/recycling agent can be selected to address this concern.

HIPR mix designs are more complex, with many more variables than standard virgin HMA mixes. This is due to the fact that old, existing materials are combined with new virgin materials in a typical HIPR project. Since a detailed testing protocol is necessary to determine the makeup and properties of the existing asphalt pavement materials, this adds a number of extra steps to a standard mix design using 100 percent virgin binder and aggregate. The variables that should be considered in an HIPR mix design are:

- properties of the existing roadway materials, including binder and aggregate fractions;
- percentage and type of virgin materials that need to be blended with existing materials; and
- an evaluation of the properties of the combined existing and virgin materials.

12.6.1 Mix design recommendations and procedures

There are many different levels of mix design for HIPR mix design. In some cases, especially where traffic and loading conditions are minimal, the owner or contractor may elect to do a very basic mix design procedure. In this case, the mix design may only include an effort to determine the grade and amount of new recycling agent or virgin binder that will be used in the HIPR process.

However, the Asphalt Institute recommends that a full mix design be performed for all projects using Marshall, Hveem or Superpave design procedures, including a detailed look at both existing and new materials and evaluating the blend of these materials. Therefore, it is recommended that the mix design process includes the following steps:

- evaluate the existing HMA including asphalt binder, aggregates and mix properties;
- determine if existing asphalt binder needs rejuvenation;
- determine the type, grade and amount of virgin liquid binder (recycling agent, asphalt binder);
- determine the amount of admix (new mixture) including aggregate gradation and the type, grade and amount of soft new asphalt binder;
- prepare and test laboratory specimens with various combinations of existing and virgin aggregate and binder;
- evaluate test results and determine the optimum combination of virgin and existing materials; and
- select the HIPR mixture with optimum physical properties and acceptable economics (based on knowledge of prior HIPR projects and mix design criteria specifications).

Those involved with HIPR are strongly encouraged to review the applicable standards, methods and procedures covering HIPR in the

Asphalt Recycling & Reclaiming Association's (ARRA) NP90, Basic Asphalt Recycling Manual.

12.7

Cold mix

Cold mix asphalt is a blend of either emulsified or cutback asphalt binders and aggregate that are mixed and placed at much lower temperatures than conventional hot mix asphalt mixtures.

Cold mixes use less energy and produce fewer emissions than hot mixes. Cold mix plants are lower cost, simpler and more mobile than hot mix plants, and emulsion mixes lend themselves to on-site and in-place manufacturing. Cold mix asphalt design, production and construction are generally less complicated than hot mix asphalt.

Cold mix operations can range from in situ cold mix preparation with a distributor and a motor-grader operation to the same quality control as that found in hot mix asphalt preparation when prepared in a plant-mix operation.

12.7.1 Description

Cold-laid asphalt mixes may be used for surface, base or sub-base courses if the pavement structure is properly designed. Cold-laid surface courses are suitable for light and medium traffic.

When used in the base or sub-base, they may be suitable for all types of traffic. Also, when used on large jobs, cold-laid paving mixtures reduce construction costs, but not necessarily the strength or quality of the pavement structure.

Asphalt pavements made by cold-laid methods can be designed and built to meet current traffic requirements but typically require a wearing surface of some type, such as a surface treatment or thin lift HMA surface.

12.7.2 Materials selection

A wide variety of aggregates and soil-aggregate combinations can be used in cold mixes.

The type and grade of asphalt selected for use in cold-laid pavement construction depends a great deal on the aggregate, mixing method and climatic conditions. Recommended asphalts for different types of cold-laid pavement mixtures are given in **Table 12.10**. Cold-laid asphalt mixtures prepared for maintenance use and stockpiled over a period of time are usually prepared with a lower-viscosity grade of MC or SC cutback asphalt.

12.7.3 Mix design methods

There is no universally accepted mix design method for cold mixes. Nearly all are modifications of the Hveem, Marshall or Superpave mix design methods. The Asphalt Institute's MS-19 Basic Asphalt Emulsion Manual, chapter 7, provides a standardized design method for dense-graded emulsion mixes as well as one for open-graded emulsion mixes. The design method for dense-graded mixes includes determining a starting point for emulsion content, coating and adhesion testing, preparing mix samples at varying emulsion contents and testing those samples for volumetrics and stability. The design method for open-graded mixes includes preparing trial mixtures at varying emulsion contents and evaluating drain-down.

While there are many templates for cold mix design reflecting local materials and conditions, it is essential that a laboratory-mix design be performed that includes trial mixes to determine proper emulsion or cutback grade for the aggregates used, emulsion or cutback design content and mix properties such as of workability, stability and susceptibility to water damage.

For the various in-place asphalt recycling methods and their appropriate mix design methods, an excellent reference is NP-90, the Basic Asphalt Recycling Manual.

12.8

Crumb rubber modified asphalt mix design

The use of crumb rubber modified (CRM) asphalt in mixtures falls into three broad categories:

- **Dry Process**—Any method that mixes crumb rubber modifier with the aggregate before the mixture is charged with asphalt binder at the hot plant. Particle size is typically larger than 10 mesh.
- **Wet Process**—A method that blends crumb rubber modifier with the asphalt cement prior to incorporating the binder into the aggregate, typically at the HMA plant site

Type of Construction	Emulsified Asphalts													Cutback Asphalts						
	Anionic									Cationic				Medium Curing (MC)				Slow Curing (SC)		
	MS-1	MS-2	MS-2h	HFMS-1	HFMS-2	HFMS-2h	HFMS-2s	SS-1	SS-1h	CMS-2	CMS-2h	CSS-1	CSS-1h	70	250	800	3000	250	800	3000
Cold-Plant Mix																				
Pavement Base & Surface																				
Open-Graded Aggregate	X	X	X	X	X	X				X	X					X				
Dense-Graded Aggregate							X	X	X						X	X	X	X	X	X
Patching, Immediate Use								X	X			X	X		X	X			X	
Patching, Stockpile												X	X		X	X		X	X	
Mixed-in-Place (Road Mix)																				
Pavement Base & Surfaces																				
Open-Graded Aggregate	X	X	X	X	X	X	X			X	X					X	X		X	X
Dense-Graded Aggregate							X	X	X			X	X		X	X		X	X	
Sand							X	X	X			X	X	X	X	X				
Sandy Soil							X	X	X			X	X		X	X				
Patching, Immediate Use							X	X	X			X	X		X	X			X	
Patching, Stockpile															X	X		X	X	

TABLE 12.10 ***Guide for asphalt type and grade***

using a portable blending system. Particle size is typically smaller than 10 mesh.

- **Terminal Blend**—A method that blends crumb rubber modifier with the asphalt binder at the asphalt terminal. There are two general types of terminally blended binder products. One type fully digests the rubber into the binder, while the other type intentionally retains small rubber particles in the blended binder. Particle size is typically smaller than 30 mesh.

Asphalt mixtures using terminally blended CRM with particles fully digested into the binder require no deviation from normal design procedure for any mix type. Designing asphalt mixtures using other types of CRM requires a few changes from the typical design process, which is the topic of discussion for the remainder of this section.

The discrete rubber particles may affect the aggregate's ability to fit together. If the rubber particles force the aggregate apart to any degree, VMA will be increased. If the air void content is kept constant, that means the required binder content will necessarily increase. CRM mixes incorporating larger rubber particle sizes typically require more adjustments than those with smaller particles. Because of the extra volume occupied by rubber particles, their use with open-graded and gap-graded mixes typically works best. However, dense-graded mixes can be successfully designed if the particle size is small enough, normally No. 30 (0.6 mm) or smaller. Adjustments to the aggregate gradation are often required to make space for the CRM, typically by reducing the amount of screenings of like-size material.

CRM mixtures may swell, or rebound, after laboratory compaction, affecting the mixture volumetrics. Acceptable mixes typically show negligible changes in specimen height after compaction. If the specimen exhibits noticeable rebound after compaction, it should be considered an indicator of potential mix volumetric issues. The composite aggregate gradation should then be adjusted to better accommodate the rubber particles.

Mix the CRM in the design lab in the same manner that it will be done in the field. For example, if the dry process is to be used, mix the rubber particles in with heated aggregate just before mixing with the binder. Handle wet process or terminal blend binder in a manner consistent with their use in the field.

12.9

Hydraulic mixes

12.9.1 Introduction

Asphalt materials have been used extensively in construction of hydraulic structures such as water reservoirs, irrigation canals, erosion control, shore protection, sewage lagoons, leaching pads and landfill liners. It has been found that asphalt materials perform well to waterproof a structure, to protect a structure or to reinforce a structure.

Various forms of asphalt applications are used for hydraulic structures, such as:

- impermeable asphalt mixes;
- porous asphalt mixes;
- asphalt mastics;
- asphalt cement; and
- prefabricated asphalt materials.

The most common way of using asphalt to make a hydraulic structure is to pave the surface with impermeable hot mix asphalt, which forms a water-tight lining when compacted. HMA mixes used for hydraulic linings are similar to mixes used for roadway paving, but with a higher dust and asphalt content to produce a mix that has very low air voids. Only neat (unmodified) binders should be used for drinking water reservoirs. The aggregates used should be sound and have good adhesion characteristics in the presence of water.

Besides the impervious mixes, two types of porous mixes are used, permeable and open-graded. Permeable asphalt mixes have at least 18 percent air voids and are used to line reservoirs that are made watertight by compacted layers of clay. The permeable asphalt lining protects the earth embankments from erosion caused by wave action. At the same time, they permit the free flow of water through the lining, eliminating the danger of hydrostatic back-pressures developing under the lining.

Open-graded mixes (with 20 to 22 percent air voids) are used as drainage layers beneath the impermeable hot mix asphalt layer. They are used to collect any leakage through the impervious lining, or seepage coming up from the underlying embankment. Open-graded layers add to the stability and overall strength of the impermeable hot mix asphalt lining.

12.9.2 General mix design procedures

While standard mix design equipment and tests can be used during the mix design of impervious hydraulic mixes, there are some special considerations. To ensure impermeability (having a coefficient of permeability “k” of less than 1×10^{-7} centimeters per second), the in-place air void content of an asphalt concrete lining should not exceed 3 percent. Increased asphalt content helps satisfy this design requirement. However, too high an asphalt content results in decreased stability. Although this mix does not need the high stability usually associated with highway paving mixes, it should be stable enough to support its own weight on relatively steep slopes while hot. It should not creep after the lining has been compacted and should be able to support some vehicles and equipment when the reservoir is emptied for maintenance.

For more detailed information on materials selection, mix design procedures, design and construction of hydraulic structures, reference the Asphalt Institute’s manual MS-12, Asphalt in Hydraulics.

12.10

Railway track bed mixes

12.10.1 Introduction

The most common design used on U.S. railroads is known as HMA underlayment, in which the HMA replaces the sub-ballast layer and is placed directly on select subgrade or old roadbed.

The typical layer of ballast is subsequently placed between the HMA layer and the ties. This design requires little change from normal track bed designs because the HMA layer merely replaces the typical granular sub-ballast layer. The HMA underlayer is typically 5 to 8 inches (125 to 200 mm) thick and 12 to 14 feet (3.7 to 4.3 meters) wide. It is similar to a paved lane of a highway, but is contained within the track structure to form a structural layer or hardpan between the ballast and subgrade or existing roadbed.

12.10.2 Mix composition and design

The HMA mix, which is considered to impart the ideal properties to the track structure, is a low modulus (plastic) mix, having design air voids of 1 to 3 percent. This mix will easily compact to less than 5 percent air voids in the field. This has been accomplished by specifying traditional, local dense-graded highway base mix having a maximum aggregate size no greater than 1.0 to 1.5 inches (25 to 37 mm).

Ideally, the asphalt binder content can be increased by about 0.5 to 1.0 percent above optimum for highway applications because rutting and bleeding are not concerns in the insulated track bed environment. Caution is advised on excessively stiff mixtures. One of the key aspects of HMA underlayment is the ability of the lower portions of the ballast to “key” into the plastic mix, which assists the track bed structure in developing an ideal track bed modulus.

Long-term monitoring and testing of in-service track beds indicate that this low-voids, impermeable mix undergoes minimal oxidation from the effects of air and water. It is also isolated from extreme temperature fluctuation within the insulated track bed environment. This isolation provides a layer with reasonably consistent stiffness in hot weather but slightly resilient in cold weather, thus eliminating any chance for the mix to rut and bleed in hot weather or crack in cold weather, which results in very durable separation between the track bed and subgrade. See Asphalt Institute Information Series IS-137, Hot Mix Asphalt For Quality Transit and Track Beds, for more information.

12.11

Ultra-thin bonded wearing course

12.11.1 General

Ultra-Thin Bonded Wearing Course (UTBWC) is a gap-graded HMA mixture placed in a thin layer onto a warm, polymer-modified emulsion membrane. These mixes are primarily used in high-traffic areas as a surface treatment over HMA and PCC surfaces.

The mix is generally placed and compacted to a thickness that is one and a half times as thick as the largest stone in the gradation. However, it may be placed thicker to correct minor surface irregularities. The UTBWC is typically placed in compacted thickness of ⅝ to ¾ inches (9.5 mm to 19 mm). The UTBWC treatment offers a very durable, skid-resistant surface that will protect the overlaid pavement for a number of years and is a good inhibitor for reflective cracking.

The purpose of using a gap grading in a UTBWC is to provide improved stone-to-stone contact similar to an SMA but with less mineral filler and fiber content. A key component of UTBWC is that an emulsion is sprayed onto the existing pavement immediately before applying the hot mix asphalt using a “spray paver” that is equipped with an emulsified asphalt tank and distributor spray bar. This allows the emulsion to be sprayed uniformly and directly ahead of the placement of the HMA layer.

The UTBWC polymer-modified asphalt emulsion membrane seals the existing pavement and bonds the gap-graded mix to the surface. The thick nature of the membrane allows it to migrate upward into the mix, filling voids in the aggregate and creating an interlayer of high cohesion. The gap-graded HMA layer provides an open surface texture. A UTBWC can be applied and opened to traffic quickly, usually within 15 minutes, without sanding or tracking. The grade of the asphalt binder is typically modified and is chosen based on the climate and the traffic conditions for the project. The binder content generally ranges from 5 to 6 percent, depending on the traffic, climate and condition of the existing pavement.

12.11.2 Materials selection

The mix design for a UTBWC includes specifications and tests for aggregate gradation, aggregate properties, a PG binder grade and requirements for the polymer-modified emulsion tack coat.

12.11.3 Gradation

Typical UTBWC aggregate gradations are shown in **Table 12.11**.

Mixture Requirements			
Sieves	**Composition by weight percentages**		
	Design General Limits, % Passing		
ASTM	**4.75-mm Mixture**	**9.5-mm Mixture**	**12.5-mm Mixture**
19 mm[6]	—	—	100
12.5 mm	—	100	75–100
9.5 mm	100	75–100	50–80
4.75 mm	40–55	25–38	25–38
2.36 mm	22–32	19–27	19–27
1.18 mm	15–25	15–23	15–23
0.600 mm	10–18	10–18	10–18
0.300 mm	8–13	8–13	8–13
0.150 mm	6–10	6–10	6–10
0.075 mm	4–6	4–6	4–6
Asphalt Content, %	5.0–6.2	4.8–6.2	4.6–6.2

NOTES

6 A target of 100% passing the 19-mm sieve is recommended. Mixtures containing 19 mm aggregate size will require greater paving thickness.

TABLE 12.11 ***Typical UTBWC aggregate gradations***

12.11.4 Polymer-modified asphalt emulsion

The asphalt emulsion used in the membrane for a bonded wearing course is specially formulated to provide high flexibility and bonding in the range of climatic conditions in which bonded wearing courses are placed. See the manufacturers' recommendations.

12.12

Bridge deck mixes

Asphalt mixture surfaces are desirable for bridge deck overlay for many reasons. First, they are highly skid-resistant and are not affected by salts used for deicing. If properly constructed, they may waterproof the bridge deck, thus preventing water and salts from damaging the concrete and reinforcing steel in the deck.

Test	Test Method	Requirement
Film Thickness, microns[a]	Asphalt Institute MS-2 Table 8.1	10 Min.
Drain-down[b]	AASHTO T 305	0.10% Max.
Moisture Sensitivity[c]	AASHTO T 283	80% Min.

NOTES:

a The film thickness shall be a minimum of 10 microns when calculated using the effective asphalt content in conjunction with the surface area for the aggregates in the JMF.

b Drain-down from the loose mixture shall not exceed 0.10 percent when tested in accordance with AASHTO T 305. The drain-down shall be tested at optimum asphalt content plus 0.5 percent. The temperature shall be the mixing temperature plus 15°C. The temperature shall not exceed 180°C.

c The tensile strength ratio shall meet or exceed 80 percent when tested in accordance with AASHTO T 283. Specimens for AASHTO T 283 shall be 100 millimeters in diameter and compacted in accordance with AASHTO T 312 to 100 gyrations. One freeze-thaw cycle shall be done prior to testing.

TABLE 12.12 ***UTBWC mixture requirements***

12.12.1 Mix Composition and Design

Where standard HMA is used, typical mix design equipment, tests and criteria are utilized. However, there are some proprietary modified asphalt additives for bridge deck overlay applications. The manufacturers' mix design recommendations and procedures should be utilized on proprietary mixtures.

In areas where chemicals are expected to be used to deice the pavement, aggregates that do not react with deicing chemicals should be selected. Lightweight or manufactured aggregates may be desirable to reduce the weight of the asphalt pavement on a bridge structure.

Steel bridge decks often utilize HMA as a driving surface. Steel bridge decks create unique design parameters that require in-depth engineering and experience to successfully accomplish. Highly modified binders along with low design air voids are typically utilized on steel decks. Proprietary materials and systems exist to construct asphalt surfaces on steel bridge decks. A mix designer is encouraged to follow manufacturers' recommendations when designing these specialty mixtures.

12.13

Sand-asphalt mixes

Appropriately graded, manufactured sand or natural sand or a combination of both can be used effectively as either a base or a surface mixture. The primary difference between a base and surface sand-asphalt mix would be in the amount of asphalt cement and minus No. 200 (75 μm) material that may be specified. Also known in some areas as a plant-mix seal or as sheet asphalt, sand-asphalt mixes do not normally have the high stability associated with larger-size aggregate mixtures. These types of mixtures are not recommended in heavy traffic-load areas.

Sand-asphalt mixes produce the tightest surface texture of any HMA and with proper selection of aggregate type (hardness and shape) can also produce a highly skid-resistant mixture. An additional advantage of sand mixes is that they can be placed in thicknesses as thin as 0.60 inch (15 mm). For this reason, sand mixes can be used as a thin leveling course prior to an HMA overlay.

12.14

Warm mix asphalt

Warm mix asphalt (WMA) is a generic term for various products and technologies that are incorporated into plant-produced hot mix asphalt to reduce the plant mixing, roadway paving and compaction temperatures while maintaining workability. WMA can also be used as a compaction aid at hot mix temperatures. The goal with WMA is to produce mixtures with similar strength, durability and performance characteristics as HMA using substantially reduced production temperatures. There are a number of WMA technologies and products that have been proposed, and it is likely that additional innovative approaches will be developed in the future. Further information can be obtained at the WMA website www.warmmixasphalt.com.

Traditionally, HMA is produced in either batch or drum plants at a discharge temperature of between 280° and 320°F. Mixes using WMA technologies are produced and placed at temperatures in the range of 0° (only as a compaction aid) up to 100°F lower than typical hot mix. Research and field studies have shown that there can be a number of significant advantages to the use of WMA, including reduced fuel consumption, reduced emissions and increased compactibility.

12.14.1 WMA technologies

WMA technologies can be broken down into four basic groups. They are:

- plant foaming;
- organic additives;
- chemical additives; and
- hybrids.

12.14.1.1 Plant foaming

In each of the foaming technologies, a small amount of water is placed in contact with hot bitumen which vaporizes and creates foam, i.e. vapor bubbles, surrounded by a thin film of bitumen. The duration of this foam is short but sufficient to coat the fine elements of materials.

There are several suppliers of water-based foaming technology, where cold water is injected into hot asphalt binder which foams the asphalt binder.

12.14.1.2 Organic additives

Organic or wax additives are used to achieve the temperature reduction by reducing the viscosity of the binder. The processes show a decrease of viscosity above the melting point of the additive, making it possible to produce asphalt mixes at lower temperatures. Crystallization of the additives occurs as the mat cools, which then increases the stiffness of the asphalt binder.

12.14.1.3 Chemical additives

Chemical additives include a combination of emulsification agents, surfactants, polymers and additives to improve coating, mixture workability and compaction, as well as adhesion promoters (antistripping agents). The chemical additive package is used either in the form of an emulsion or added to the bitumen in the mix production process prior to being mixed with hot aggregate. Relatively minor modifications are needed to the asphalt plant or to the mix design process.

12.14.1.4 Hybrids

There have also been reports of various combinations of WMA technologies being used. Waxes have been used in conjunction with either chemical additives or foaming additives. Other combinations are possible.

12.14.2 Mix design procedures

It is good practice to conduct a laboratory mix design that includes the WMA additive and takes the specific process of producing the WMA with that additive into consideration. The mix design should mimic the dosage of additive, mix and compaction temperatures, and the aging time that will occur during production, storage and hauling to the roadway paving site.

For most WMA projects, the warm mix technology has been introduced into an existing HMA mixture design with no change to the job mix formula.

The objective of NCHRP Project 9-43, "Mix Design Practices for Warm Mix Asphalt," was to develop a mix design method for WMA in the form of a draft AASHTO-recommended practice. This method was: (1) to be based on Superpave mix design methodology; (2) to include a suite of performance tests to assess whether a WMA mix design will provide satisfactory field service; and (3) to be applicable to any WMA technology. The final report, NCHRP Report 691, "Mix Design Practices for Warm Mix Asphalt," indicates that a stand-alone WMA mix design method distinct from that for HMA is not warranted. Instead, an Appendix X2 to AASHTO R 35, "Standard Practice for Superpave Volumetric Design for Hot Mix Asphalt (HMA)," was developed, titled NCHRP Report 714, "Special Mixture Design Considerations and Methods for Warm Mix Asphalt."

12.14.2.1 Mixture design considerations

Binders from WMA mixtures have improved low-temperature properties due to the lower amount of aging that occurs during production. Although the improvement in low-temperature properties is not large enough to warrant changing the low-temperature grade, it is large enough to affect the acceptable amount of RAP that can be added to a mixture. For RAP mixes, reduced virgin binder aging at very low WMA plant temperatures can impact the virgin binder grade required. See chapter 11 for more information on blending charts and RAP mix design.

12.14.2.2 Moisture sensitivity

Moisture sensitivity is evaluated as part of the WMA mix design process. The method and criteria are the same as for HMA, using AASHTO T 283. The tensile strength ratio values for WMA mixes are often lower than for HMA. The use of lime or liquid anti-strip is typically used to improve TSR values. Agencies are encouraged to correlate laboratory results with field performance to evaluate the specified criteria.

12.14.2.3 Rutting resistance

It is recommended to conduct some type of rutting resistance test to evaluate stability of the proposed design. Due to lesser aging occurring at lower plant-mix temperatures, it is not uncommon for WMA rutting resistance test results to be lower than traditional HMA. Additional guidance is available to designers in NCHRP Reports 691 and 714 along with the appendix to AASHTO R 35.

12.14.3 Ongoing research

It is recommended that designers use AASHTO standards when conducting WMA mix design including Appendix A in AASHTO R 35, "Standard Practice for Superpave Volumetric Design for HMA." It is likely that the mix design practices for WMA may evolve over the coming years. Therefore, it is recommended that designers stay up to date on the most recent specifications and guidance related to WMA mix design from AASHTO and owner/agencies.

CHAPTER 13

Field Verification of Asphalt Mixtures

A thorough understanding of the concepts discussed in chapter 5 regarding volumetric properties is essential in understanding variability that can occur when field-produced mixtures are compared to laboratory mix design results. Issues such as absorption, mixture conditioning and gradation have significant impact on the volumetric results of field-produced mixtures.

13.1

Laboratory design versus field production

Good performance of asphalt pavements is achieved by using high-quality materials, precise process control during mix production and best placement practices during construction. The life of a pavement and the cost of maintenance can be very sensitive to seemingly minor variations in materials properties, such as aggregate gradation, asphalt content, mixture volumetrics and compaction.

The goal of mix design is to establish mix formulation and process control targets known as the job mix formula (JMF). A JMF should contain the following criteria along with appropriate production tolerances:

- binder content;
- gradation of each specified sieve size;
- cold feed and/or hot bin proportions;
- aggregate properties including bulk specific gravities;
- volumetric properties including P_a, VMA, VFA, and DP;
- bulk and maximum specific gravities of the mixture; and
- stability, flow or other performance parameters specified.

Field verification of the asphalt mix involves testing and analyzing the field-produced mixture to ensure that the JMF criteria specified above are met. Significant differences exist between the small-scale operation of the laboratory mixing bowl and a 500-ton-per-hour HMA plant.

HMA produced in the field will often display different volumetric properties when compared to results in the lab. Laboratory mix and field-produced mix have different physical handling of the aggregate before and after blending of materials, varied use of particulate control and a differing environment in which absorption can occur. All of these, along with other variables, can produce changes in the VMA and air voids of the field-produced mixtures when compared to the laboratory mix design.

A mix design technician or engineer should account for anticipated differences between the lab and field when they have experience with the proposed materials, especially the aggregate, and the plant that will produce the mix.

Field verification of an asphalt mix is necessary to measure what differences exist and what corrective measures, if any, need to be made. Out-of-date mixture designs are often a source of production problems. If the mix design was performed some time (maybe months) ahead of paving, or the materials used during the design are not representative of the materials used on construction, there may be an unacceptable difference between the plant-produced mix and the JMF established in the mix design phase. Plant adjustments may bring the plant-produced mix within the gradation specifications, but may have a hard time complying with the volumetric and performance requirements.

Reconciliation of the differences that are found between the JMF and field-produced materials should be approached from a systematic perspective. First, recognition of what can or cannot be readily changed to meet the project requirements during production needs to be established. An example of where changes may not be allowed without a complete redesign of the mix includes the failure of aggregate materials to meet source property requirements.

Many adjustments can be made during production to maintain adequate results. Aggregate proportions and asphalt content are two items that are often adjusted during production. Adjustment to only one item at a time is recommended. Multiple, concurrent adjustments can cancel each other out to produce a net result of no change or a drastic change to the mixture.

13.2

Quality control tests and calculations

13.2.1 Asphalt content

Several methods can be used to determine binder content. The most frequently used methods are the extraction test (AASHTO T 164 or ASTM D2172) and the ignition oven test (AASHTO T 308). "Automatic recordation" can be used to calculate asphalt content if the asphalt mixing facility makes detailed measurements of the materials used during production. Properly calibrated nuclear asphalt content gauges can also provide measurements on the produced mixture (AASHTO T 287 or ASTM D4125).

13.2.2 Aggregate gradation

Various methods also exist to determine aggregate gradation. Gradation results can vary depending on the sample location. Samples are often obtained from the cold feed belt or hot bins prior to the introduction of binder into the mixture. Extraction or ignition oven testing of the plant-mixed material is considered to be a more accurate measurement for the aggregate gradation of the final mixture. There can be a significant difference between a cold feed gradation and extracted gradation caused by the physical breakdown of the material as it is processed through the plant. Gradations should always be washed to ensure complete accuracy of the amount of minus No. 200 (75 μm) material.

13.2.3 Maximum specific gravity of the mix

The theoretical maximum specific gravity, G_{mm}, of the HMA paving mixture is a key measurement during both laboratory mix design and field verification. Also called the "Rice" specific gravity after its developer, G_{mm} is the specific gravity of an asphalt mixture with air voids excluded. Using a vacuum procedure to remove entrapped air from the mixture, the test determines the volume of the asphalt mix in a voidless state. Multiplying G_{mm} by the density of water, ρ_w, will yield the theoretical maximum density of an asphalt mixture. This G_{mm} is used to determine the air voids of compacted mixtures as well as a reference typically used to determine the in-place density of the HMA. The following example demonstrates this process.

$$
\begin{aligned}
\text{Maximum specific gravity} &= G_{mm} = 2.438 \\
\text{Density of water} &= \rho_w = 1{,}000 \text{ kg/m}^3 \\
&\quad (62.4 \text{ lb/ft}^3) \\
\text{Maximum density} &= G_{mm} \times \rho_w \\
&= 2.438 \times 1{,}000\ (62.4) \\
&= 2{,}438 \text{ kg/m}^3 \\
&\quad (152.1 \text{ lb/ft}^3)
\end{aligned}
$$

13.2.4 Bulk specific gravity of the mix

The bulk specific gravity of the mix (G_{mb}) can be determined on any compacted HMA. This includes material compacted in the lab or material compacted on the roadway. Laboratory compacted samples of plant-produced mix must be compacted using the same procedure used in the mix design (SGC, Hveem or Marshall). Multiplying G_{mb} by the density of water will yield the bulk density of the compacted HMA sample. The following example demonstrates this process.

Bulk specific gravity, $G_{mb} = 2.344$
Density of water, $\rho_w = 1{,}000\ \text{kg/m}^3\ (62.4\ \text{lb/ft}^3)$
Bulk density $= G_{mb} \times \rho_w$
$= 2.344 \times 1{,}000\ (62.4)$
$= 2{,}344\ \text{kg/m}^3$
$(146.3\ \text{lb/ft}^3)$

When sampling field mix for bulk specific gravity determination, laboratory compaction without reheating is recommended. If reheating of cold or stored samples cannot be avoided, a correlation should be made to adjust the compactive effort on the reheated mix to match the volumetric properties (such as VMA and percent air voids) of field mix which was not reheated.

13.2.5 Air voids

Since G_{mb} is measured on a compacted sample, the measurement includes air contained in the mix. The percent air voids, P_a, of the compacted mixture is calculated using the bulk and theoretical maximum specific gravities in this equation.

$$P_a = 100 \times \frac{G_{mm} - G_{mb}}{G_{mm}}$$

The following is a demonstration of this equation in use.

$$G_{mm} = 2.438 \qquad G_{mb} = 2.344$$

$$P_a = 100 \times \frac{2.438 - 2.344}{2.438} = 3.9\%$$

13.2.6 Stability (Marshall and Hveem) and flow (Marshall)

These properties can be measured on the laboratory compacted samples of field-produced material. Some agencies include them in field verification tests. These tests have traditionally been utilized as a measure to indicate the rutting resistance and durability of an HMA. However, these values are affected by many different aggregate and asphalt properties and are less reliable than volumetric property tests for predicting pavement performance. If the volumetric properties (air voids, VMA, asphalt content) and the aggregate quality and gradation of the mixture are properly controlled, then the stability and flow will normally meet the appropriate specifications.

13.3

Job mix formula verification and daily mix verification

Field verification involves two different levels of analysis performed on the HMA. The first involves analyzing the mixture on the first day of full production to compare the mixture to the job mix formula (JMF). The second uses day-to-day field verification tests to determine if the mixture properties continue to meet specifications.

13.3.1 Job mix formula verification

Asphalt content, gradation, void analysis and other specified tests are performed to compare field-produced mixture properties with the JMF. These tests will indicate if the aggregate characteristics have varied since the mix design, and may indicate if problems exist from changes in the aggregate after processing through the asphalt mixing plant.

Field verification results may show that changes are necessary to meet the JMF and specification's applied tolerances. For example, minor changes in the asphalt content may bring air voids back into compliance. Alternatively, if the mixture is meeting agency specifications but not the JMF and applied tolerances, the JMF can be adjusted to the average results of the plant-produced mix. Significant differences between the laboratory design and field-produced mixture may necessitate a new mix design using the actual production materials. Once the JMF has been verified or adjusted, the final JMF is established and payment is measured from these values. This final JMF is often referred to as the adjusted JMF or AJMF.

Adjusting the JMF at start-up is common to maintain mix volumetrics. However, continuous changes to the JMF are not acceptable and indicative of poor production processes.

13.3.2 Daily mix verification

Daily testing can provide an early warning by indicating if the mixture properties deviate from the specifications. This daily verification is part of plant process control that can identify potential acceptance problems before a large amount of mix is placed.

Daily field verification tests are typically performed on random samples taken from a set quantity of material called a lot. A lot is typically a day's production or a given tonnage of material. It is important to use random sampling techniques so an unbiased evaluation of the material can be made (see Asphalt Institute's MS-22, Principles of Construction of Hot Mix Asphalt Pavements, for further information).

13.3.2.1 Control charts

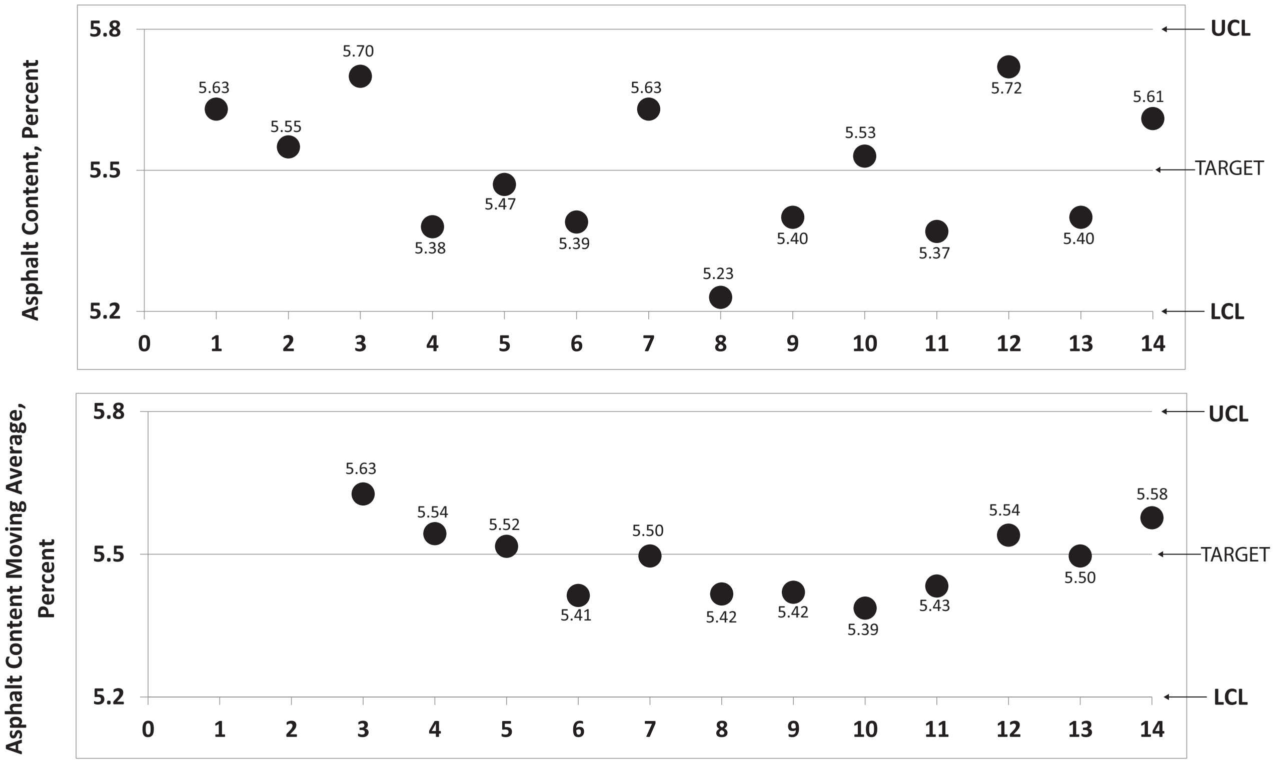
Values from daily field verification tests are plotted on control charts. Continuous plots of mix data for percent air voids, binder content and aggregate percentages passing certain sieves such as 4.75 mm (No. 4), 600 µm (No. 30) and 75 µm (No. 200) provide a graphic representation of the production process. Target values and tolerance limits for each material property are drawn on the chart (usually appearing as parallel lines) and then production values are plotted in relation to these limits.

Figure 13.1 shows a set of control charts for binder content during production with Upper Control Limit (UCL), target value and Lower Control Limit (LCL). The top chart shows the value of each binder content test result. The bottom chart shows the running or moving average of the binder content data. The running average is calculated from typically three to five previous values per subgroup or sublot. After each test is performed, the new test value replaces the oldest test value in the subgroup to calculate the new running average.

When analyzing field verification data, it is important to recognize sources of variation in the data. These sources include variation in the testing and sampling procedures, normal variations in the materials and production process, and variations due to problems in production. Following the testing and sampling procedures exactly as specified will minimize these variations. Strict adherence to sampling and testing protocol will also help reduce testing variability between the individuals responsible for process control, as well as allow valid comparisons of quality control test results with agency tests done for acceptance. Adjusting the production process on the basis of a single test result is not desirable. Before plant changes are made, an out-of-spec test result should be verified as soon as possible.

Control charts can help differentiate between variations inherent in the material and production variation, providing early signs of problems that need attention. The data should be dispersed randomly about the target value and between the

FIGURE 13.1 ***Typical mixture production quality control charts***



*The UCL and LCL come from the specification tolerances.

control limits. A few possible indications of existing or upcoming problems are:

- values consistently higher or lower than the target value;
- gradual or erratic shifts in the data; and
- systematic cycling of the data.

Based on the present knowledge of plant production and pavement behavior, field verification must be utilized to manage the process of asphalt mixture production. This will minimize the variability between the mix design goals set in the laboratory and actual mix results achieved in the plant.

13.4

Volumetric adjustments

The most common problem encountered in plant-produced mix is the failure to meet VMA and air voids volumetric parameters. These properties are related in that a failing air voids test is usually the result of a changing VMA—assuming the binder content is correct. The change in VMA is most often explained by inconsistent mixture curing during testing or a change in the gradation of the aggregate.

As discussed previously in chapter 4, mixture conditioning of samples is essential in providing accurate volumetric properties. Changes in air voids or VMA are often encountered and go unexplained. Careful observation of material conditioning times can often explain fluctuating volumetric test results. The time and temperature of mixture conditioning can greatly affect the amount of asphalt absorbed in the aggregate, thus changing the theoretical maximum specific gravity (G_{mm}) and, to a lesser extent, bulk specific gravity (G_{mb}) test results. For example, G_{mm} samples taken from a truck at the plant and allowed to immediately cool can have significantly lower results (lowered absorbed asphalt) than if the same material were hauled to the roadway, sampled behind the paver, and returned to the plant lab in an insulated specimen container, and then allowed to cool. It is common for neither of these scenarios to match the curing time performed in the laboratory during the mix design process. In order for field-produced mixtures to match laboratory design values, mix samples should be cured at similar temperatures for a similar length of time. Highly absorptive aggregates will greatly magnify the importance of matching the curing time and temperature in the lab to that in the field.

It is important to remember that VMA measured on a lab-compacted specimen is the result of the amount of aggregate packing that occurs in the mold when placed in the compactor. Anything that changes the amount of aggregate that can be compacted into the specimen will affect the resulting VMA. A change on one aggregate sieve can alter the compaction characteristics of the mix and change the way the entire aggregate structure "fits" together. A common scenario when going from design in the lab to production in the field is that the aggregate experiences further "breakdown" in the plant relative to the mixing bowl. This breakdown can create a higher percentage of minus No. 200 material (dust) that will decrease air voids and decrease (collapse) VMA in a compacted sample.

When QC/QA plans only require the monitoring of the air voids in a mixture, and when VMA unknowingly decreases, it is a common adjustment to simply reduce the amount of binder being added to the mixture to restore the specified air voids level. Caution should be exercised here as the real reason for the lower air voids is the collapse of the available VMA in the mix or the binder coating the aggregate has not had sufficient time to be absorbed. Simply reducing the binder content may correct the air voids deviation but leave the mixture with an insufficient amount of binder to provide a durable, fatigue-resistant pavement. Other options exist to restore the VMA in a mixture, including but not limited to the following suggestions:

- Evaluate sampling and testing procedures to assure a standardization of curing parameters;
- Make gradation changes that generate additional VMA;
- Increase the fracture content of the aggregate;
- Reduce natural sand components and increase the usage of manufactured sand;
- Introduce highly fractured, durable, intermediate-sized "chips" into the aggregate structure;
- Reduce the dust in the mixture by increasing the fine aggregates that contain less material passing the No. 200 sieve, or by not returning

all of the material from the dust collection system; and
- Wash the aggregates to reduce the dust.

Powerful, advanced techniques exist today that can be highly useful in guiding plant operators to the most efficient adjustment for their situation. Perhaps the most widely recognized and refined of these is the Bailey Method. With tools like the Bailey Method, mathematical predictions of how a proposed gradation change will affect VMA, and thus other volumetric properties, can be made.

A quality control manager can investigate various scenarios, typically via a spreadsheet, to determine which bin adjustments will likely best reconcile their mix to the correct volumetric properties. The Bailey Method explains why very small changes in gradation can have a dramatic effect on the resulting volumetric properties depending on the mixture. More information on the Bailey Method can be found on the Asphalt Institute's website.

Absent of Bailey Method-like tools, trial-and-error plant adjustments based on experience can be made to achieve compliance with the JMF. The complex dynamics of a mix does not allow for this manual to fully address the unique circumstances of individual situations. It must be emphasized that regardless of mixture adjustment techniques utilized, uniformity of stockpiled materials are essential in producing quality asphalt mix. It is very difficult, if not impossible, to predict the results of a change in bin proportions when dealing with highly variable stockpiles.

13.5

Density specifications

The goal of compaction is to achieve a smooth, uniform surface at optimum air voids content that ultimately determines whether the pavement will perform as expected. The in-place air voids of HMA after compaction is a very important factor that affects performance of the mixture throughout the life of the pavement. Achieving compliance with compaction specifications is the final step in the quality management of the HMA construction procedures and must be accomplished to produce a quality asphalt pavement.

There are four primary methods for specifying the compaction of in-place HMA pavements:

- method specifications;
- control strip specified density;
- bulk specified density; and
- theoretical maximum specified density.

13.5.1 Method specifications

A "method" specification has no reference density against which the in-place density and air voids are compared. This type of specification lists such items as number, type and size of rollers to be used, number of passes each roller makes, use of temperature measurements, descriptions such as "surface is rolled until free of roller marks" and so forth. Judgment is the primary decision tool for determining optimum compaction when using this type of specification. Method specifications are generally only applicable for smaller projects with light traffic, or thin lift construction (1 inch or less), such as leveling courses and thin or nonuniform HMA overlays. In these cases, cost and the inability to obtain meaningful data may preclude the use of a reference density specification.

Specified density methods compare the in-place density of the pavement to a reference density. The pavement density is specified as a percentage of the reference density. One of the following three reference densities is typically used in density specifications.

13.5.2 Control strip specified density

This process calls for the construction of a pavement control strip of a minimum length or volume of mix at the start of each lift being constructed. After compaction is completed, a specified number of tests are measured from random locations within the control strip. The average density obtained is calculated and becomes the reference density.

The reference control strip density must then be compared to either the laboratory or theoretical maximum density of the field-produced HMA to determine if densification is adequate and accepted. Once an acceptable control strip has been obtained, the test strip density becomes the reference density typically specified during construction. Values over 102 percent indicate that something has changed and a new control strip should be considered.

Control strip specified density requirements are the least effective in assuring optimum pavement performance as control strip compaction conditions are highly variable. Temperature conditions, subgrade condition, roller ballast, tire pressure, operator inconsistencies, along with many other variables can significantly affect a reference control strip density.

13.5.3 Bulk specified density

This method compares in-place pavement density to a laboratory compacted sample of field-produced mix. Field-produced HMA is compacted using the same effort used during the mix design. The laboratory density is measured using the bulk specific gravity test.

In terms of specification compliance, an agency compares the in-place pavement density to the reference density in the form of a ratio:

$$\text{Percent of Bulk Density} = \frac{\text{In-Place Density} \times 100}{\text{Laboratory Bulk Density}}$$

When it has been verified that the field-produced mix matches the mix design properties, the laboratory compacted samples provide the same air voids content as determined in the mix design, typically 4 percent. If an in-place air voids content of 8 percent is desired, the in-place density should be 96 percent of the laboratory bulk density.

Minimum bulk density specifications typically range from 96 to 100 percent of the laboratory compacted bulk density. Values over 100 percent may be indicative of the following:

- The materials have changed.
- There are problems with the plant equipment.
- Poor sampling or testing techniques may have been used.
- There may have been problems with the laboratory equipment.
- Aggregate is being crushed under the roller.

13.5.4 Theoretical maximum specified density

Theoretical maximum density testing determines a voidless unit weight of the mix, as if it were compacted to a zero air voids condition. Using the Rice test method, the theoretical maximum density of the field-produced mixture is determined as the reference density.

The relative density of the in-place pavement is again calculated as the ratio of the in-place density to theoretical maximum density.

$$\begin{array}{l}\text{Percent of Theoretical}\\ \text{Maximum Density}\end{array} = \frac{\text{In-Place Density} \times 100}{\text{Theoretical Max. Density}}$$

The theoretical maximum density represents a voidless mixture. When the in-place air voids content is 8 percent, the in-place density is 92 percent of the reference theoretical maximum density. The theoretical maximum specified density is the most widely used method of specifying pavement compaction. Minimum compaction requirements typically range from 92 to 96 percent of the theoretical maximum density. Values over 97 percent may be indicative of problems in the following areas:

- The materials have changed.
- There are problems with the plant equipment.
- Poor sampling or testing techniques may have been used.
- There may have been problems with the laboratory equipment.

Test results over 100 percent of G_{mm} are theoretically impossible.

13.5.5 Summary of the different specified density methods

The relationship between the reference density measurements and air voids of the in-place pavement is shown in **Figure 13.2**. Laboratory compacted and in-place air voids benchmarks equal to 4 and 8 percent are depicted against each type of reference density.

It should be noted that, while the comparison between theoretical maximum density and in-place air voids content is constant, the relationship between the other two reference density types and in-place air voids will shift up or down depending on the actual mix design and compaction criteria used in this specification. For example, if the mix design air voids is at 5 percent, then 100 percent of laboratory density would be at 5 percent air voids. If the same compaction criteria of 96 percent of laboratory density were used, this would yield an in-place air voids content of 9 percent (96% × 0.95), not 8 percent.

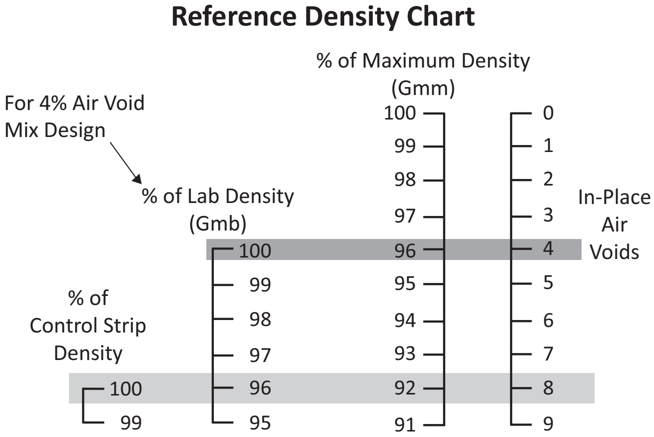


FIGURE 13.2 ***Relationships between the reference density measurements and the air voids***

Each of the reference density specification procedures has additional considerations that may make one more favorable than another on a particular project. These considerations include the traffic volume, subgrade support, size of the project, construction and testing schedules and any lift thickness variation.

A higher degree of compaction monitoring is recommended in the initial stages of the construction process, regardless of which density specification is used, to ensure optimum results from the compaction process.

In addition to maintaining minimum compaction, it is also necessary to investigate excessive compaction. There are many causes for this occurrence, including:

- excessive moisture in the plant-produced mix;
- improper baghouse operations;
- variable absorption rates of aggregate materials; and
- poor quality aggregate that degrades during compaction, often associated with thin lifts.

Whenever abnormally high density (below 3 percent air voids) occurs in a compacted mix, the cause should be determined and corrected. This problem may also require that the mix be adjusted or redesigned.