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SUPERPAVE for Local Governments

Participant Manual

National Highway Institute

Contents

I: Superpave: The Future of Asphalt

COURSE OBJECTIVES

Superpave is an acronym for Superior Performing Asphalt Pavements. Superpave is a new, comprehensive asphalt mix design and analysis system, a product of the Strategic Highway Research Program. Congress established SHRP in 1987 as a five-year, $150 million research program to improve the performance and durability of United States roads and to make those roads safer for both motorists and highway workers. $50 million of the SHRP research funds were used for the development of performance based asphalt material specifications to relate laboratory analysis with field performance.

Since the completion of the SHRP research in 1993, the asphalt industry and most highway agencies have been focusing great effort in implementing the Superpave system in their highway design and construction practices. Much of the implementation effort has involved training personnel in the proper use of Superpave technology, from introductory courses on how Superpave works to detailed laboratory courses for providing hands-on instruction with the new Superpave materials testing equipment.

This course is another step toward informing highway industry personnel of the benefits of Superpave. The intended audience for this course is local government personnel and contractors involved in the specifying, design and construction of hot mix asphalt pavements, including contractors, agency personnel, and consulting engineers. The primary goals of this course are to describe the Superpave components, the critical requirements, why they are needed, and how this new system could impact the production and construction procedures for hot mix asphalt.

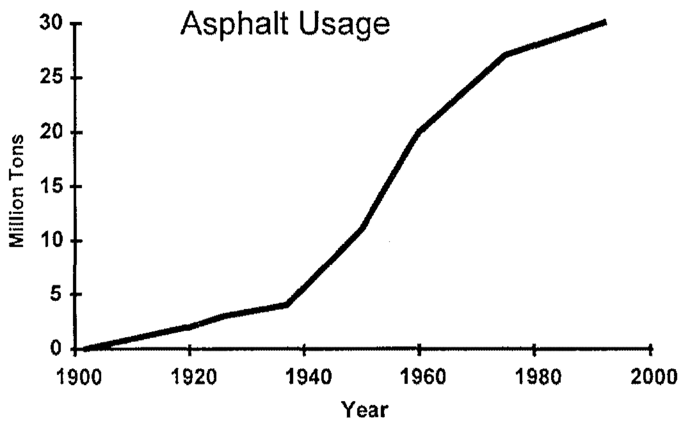
This course begins with an introduction to the research that produced Superpave. Then the ways in which Superpave can improve pavement performance are investigated, including a review of the behavior of hot mix asphalt materials. An overview of the Superpave material tests is next, followed by a discussion of how materials (asphalt, aggregate, and total mixture) are selected in a Superpave mix design. There is a discussion of the possible handling differences that Superpave requirements could bring about during hot mix asphalt production, placement and compaction. The course concludes with an update of the ongoing activities involved with implementing the Superpave system, both nationwide and in your state.

To benefit as much as possible from this course, participants are encouraged to ask questions and share experience, especially those related to their job activities. In order to have a comprehensive reference when you leave, you are encouraged to take notes directly in this book.

WHY SUPERPAVE?

To fully understand the evolution of the Superpave system, it may help to review a bit of the history of the development of highways and the asphalt industry.

Since the development of the gas engine and the discovery of the petroleum asphalt refining process, asphalt has seen increasingly widespread use in pavement applications. From road oiling of local roads to heavy duty airfield applications, the versatility of asphalt materials has provided the pavement engineer with a valuable material resource.



The design of asphalt mixtures evolved with its increasing use. The Hubbard-Field method was originally developed in the 1920s for sheet asphalt mixtures with 100 percent passing the 4.75 mm sieve, and later modified to cover the design of coarser asphalt mixtures. The Hubbard-Field Stability test measured the strength of the asphalt mixture with a punching-type shear load.

Hveem Mix Design was developed by the California Department of Highways Materials and Design Engineer in the 1930s. The Hveem stabilometer measures an asphalt mixture's ability to resist lateral movement under a vertical load. Hveem mix design is still used in California and other western states.

Marshall mix design was originally developed by a Mississippi State Highway Department Engineer and later refined in the 1940s by the Corps of Engineers for designing asphalt mixtures for airfield pavements. The primary features of Marshall mix design are a density/voids analysis and the stability/flow test. Prior to Superpave, Marshall mix design was widely used in the United States, and is by far the most commonly used mix design procedure worldwide.

Refinements to the concepts of asphalt mix design procedures came about not only with the increasing use of asphalt, but also with the increasing demand placed on the mixtures by increases in traffic volume and loading. The authorization of the Interstate Highway System in 1956 set the cornerstone for the United States reliance on highway transportation for its primary mode of transporting goods and people.

The AASHO Road Test, conducted from 1958 to 1962, set the standard for pavement structural design, and the data that the Road Test produced is still the basis for the majority of pavement design procedures. The researchers were aware that the Road Test was limited to one set of soils and climatic conditions, and other studies were planned to extend their findings to other geographic areas. Generally, these studies were not conducted, and the AASHO Road Test results were extrapolated to fit other design conditions.

The growth of the Interstate system was matched by the increase in trucking as a mode for shipping goods -- vehicles-miles traveled increased 75 percent between 1973 and 1993. Provided with an infrastructure to transport the goods, the trucking industry pushed for increased productivity, and the legal load limit was raised from 73,280 to 80,000 lb. in 1982. This seemingly small increase actually increases the stress to the pavement 40 to 50 percent for a given structural design. The advent of more economical radial tires also increased the stress to the pavement.

As the transportation industry grew, the use of hot mix asphalt in heavy-duty pavement applications grew, and the results were not always favorable. Many theories were suggested to explain the reduction in performance of asphalt pavements: since the 1973 oil embargo, the oil companies have taken the "goodies" out of the asphalt to make more gasoline; the increased use of reclaimed asphalt pavement (RAP) has led to weaker mixtures; drum mixers don't make as good a mixture as batch plants.

Although none of these theories was found to have any basis, in truth the states were finding an increasingly fine line developing between mixtures that performed well and mixtures that performed poorly. The materials were the same, but the increases in traffic load and volume were pushing the need for a better understanding of asphalt materials and pavement performance.

STRATEGIC HIGHWAY RESEARCH PROGRAM

Against this background of declining performance and diminishing research funding, SHRP was approved by Congress in 1987 as a five year, $150 million research program to improve the performance and durability of United States roads and to make those roads safer for both motorists and highway workers. One third of the SHRP research funds were directed for the development of performance based asphalt material specifications to more closely relate laboratory measurements with field performance.

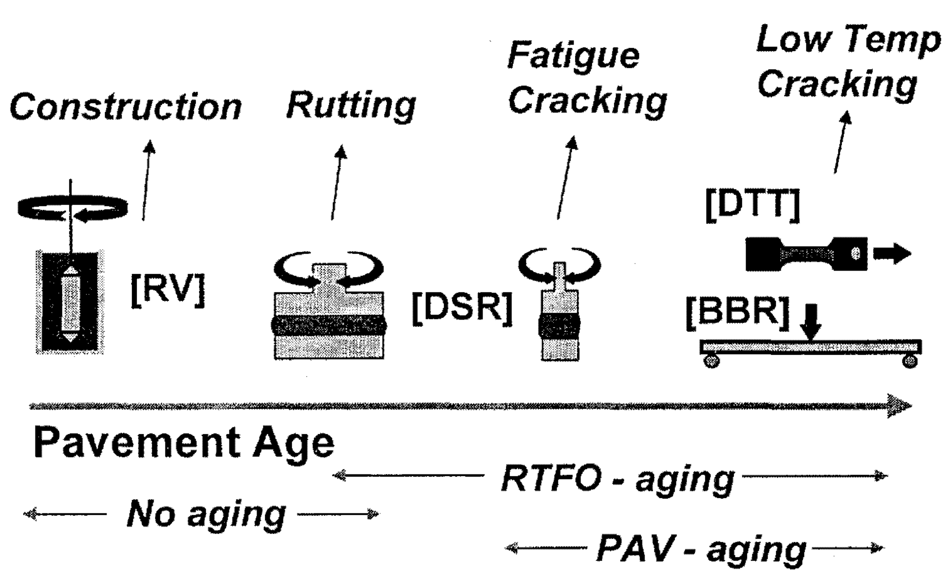
SHRP was originally proposed in Transportation Research Board Special Report No. 202, "America's Highways: Accelerating the Search for Innovation." This report outlined the need for a concentrated effort to produce major innovations for increasing the productivity of the nation's highways. Various problems in areas of highway performance and safety had been hampering the highway industry, and this report called for a renewed effort to solve these problems. However, this report did not just call for funding of research in these areas, but also emphasized the need for conducting the research with implementation in mind. A "program designed without taking into account obstacles on implementation of research will fail" noted the report, and this statement continues to guide the highway industry now that the SHRP research has been completed and its products are being evaluated and implemented.

The goal of the SHRP asphalt research was the development of a system that would relate the material characteristics of hot mix asphalt to pavement performance. Asphalt materials have typically been tested and designed with empirical laboratory procedures, meaning that field experience was still required to determine if the laboratory analysis implied good pavement performance. However, even with proper adherence to these procedures and the development of mix design criteria, asphalt technologists have had various degrees of success in overcoming the three main asphalt pavement distresses: permanent deformation or rutting; fatigue cracking, which leads to alligator cracking; and low temperature cracking.

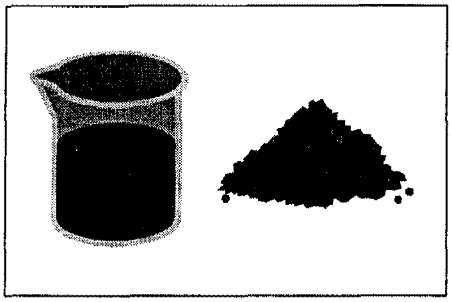
The opinions of what issues needed to be resolved by the SHRP asphalt research varied. Some industry personnel felt that a chemical based specification would provide the answer to developing a more "robust" asphalt cement to ensure better pavement performance in light of increased traffic and higher wheel loads. Other engineers believed that poor pavement performance was a combination of inadequate mix design procedures and poor construction practices, and that focusing solely on the asphalt cement would be unproductive. Consequently, SHRP researchers set out to develop a chemically based asphalt binder specification and investigate improved methods of mix design.

A final product of the SHRP asphalt research is the Superpave asphalt mixture design and analysis system. Superpave is an acronym for Superior Performing Asphalt Pavements. Superpave represents an improved, performance-based system for specifying asphalt binders and mineral aggregates, performing asphalt mixture design, and analyzing pavement performance. The system includes an asphalt binder specification that uses new binder physical property tests; a series of aggregate tests and specifications; a hot mix asphalt (HMA) design and analysis system; and computer software to integrate the system components. As with any design process, field control measurements are still necessary to ensure the field produced mixtures match the laboratory design. The Superpave binder specification and mix design procedures incorporate various test equipment, test methods, and design criteria.

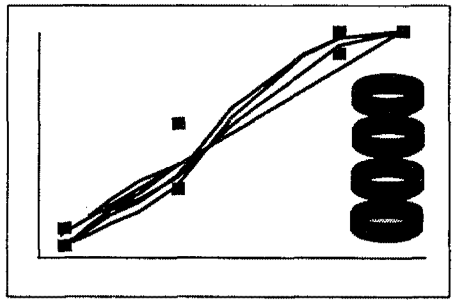
A unique feature of the Superpave system is that its tests are performed at temperatures and aging conditions that more realistically represent those encountered by in-service pavements. If the pavement distresses addressed by Superpave (rutting, fatigue cracking, and low temperature cracking) do occur in the pavement, they do so at relatively typical stages in a pavement's life and under relatively common temperature conditions. The Superpave performance graded (PG) binder specification makes use of these tendencies to test the asphalt under a project's expected climatic and aging conditions to help reduce pavement distress. SHRP researchers developed new equipment standards as well as incorporated equipment used by other industries to develop the binder tests.



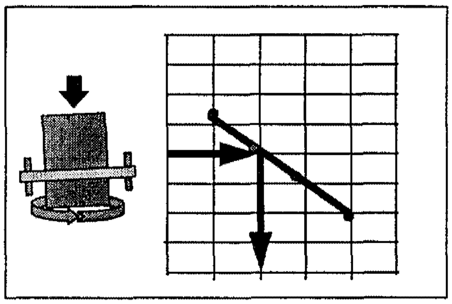
The Superpave mixture design and analysis system uses increasingly rigorous degrees of testing and analysis to provide a well performing mixture for a given pavement project. The Superpave mix design procedure involves careful material selection and volumetric proportioning as a first approach in producing a mix that will perform successfully. The four basic steps of Superpave asphalt mix design are materials selection, selection of the design aggregate structure, selection of the design asphalt binder content, and evaluation of the mixture for moisture sensitivity.



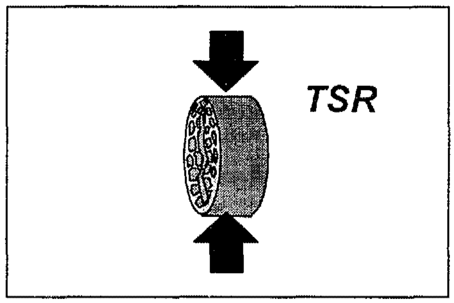
1. Materials Selection



2. Design Aggregate Structure



3. Design Binder Content



4. Moisture Sensitivity

4 Steps of Superpave Mix Design

Asphalt mixes in more critical, higher traffic volume projects can be optimized for the actual project conditions using an analysis to estimate pavement performance. The analysis procedures, still under development, will use increasingly sophisticated and comprehensive testing and modeling of the design asphalt mixture, as desired and necessary to predict performance for the actual pavement structure, climate, and traffic.

SUPERPAVE IMPLEMENTATION

How far along are the asphalt industry and government agencies toward the routine use of Superpave? That question will be answered in detail in the final section of this course. At this point it is sufficient to note that the FHWA, the states and the asphalt industry are working together in the many on-going Superpave implementation and validation activities. Through AASHTO, the Superpave test methods and specifications are being standardized, which will further accelerate and facilitate the acceptance and use of this new and improved asphalt mix design and analysis system.

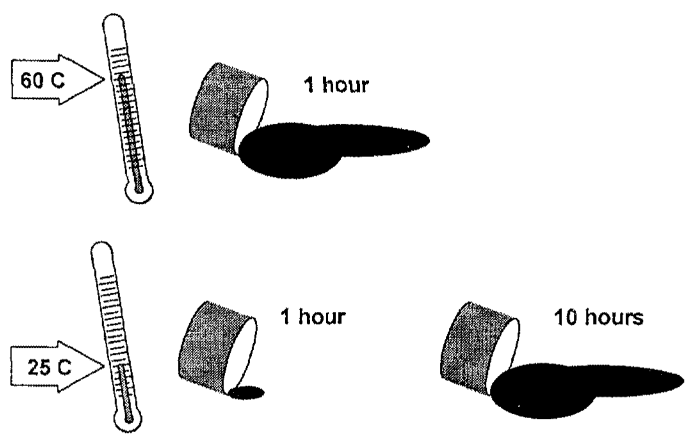
II. Improving HMA Performance with Superpave

To understand how the performance based specifications of Superpave are used to improve pavement performance requires an understanding of the characteristics of the individual materials that make up hot mix asphalt (HMA), and how they behave together as an asphalt mixture. Both the individual properties and their combination affect the pavement performance. Superpave uses these characteristics in ways that are new to the asphalt industry, as well as in ways that have been used for many years. A comparison between the old and the new helps bridge the understanding to the Superpave system.

The objectives of this session will be to describe the material properties of HMA, both of the individual components of HMA (asphalt and aggregate) and the HMA mixture itself. This description will include the tests and specifications that are used to characterize HMA materials, both prior to Superpave and under the new Superpave system. Most importantly, the session will describe how the Superpave system uses the tests and specifications to improve upon the three primary distresses in HMA pavements: permanent deformation, fatigue cracking and low temperature cracking.

HOW ASPHALT BEHAVES

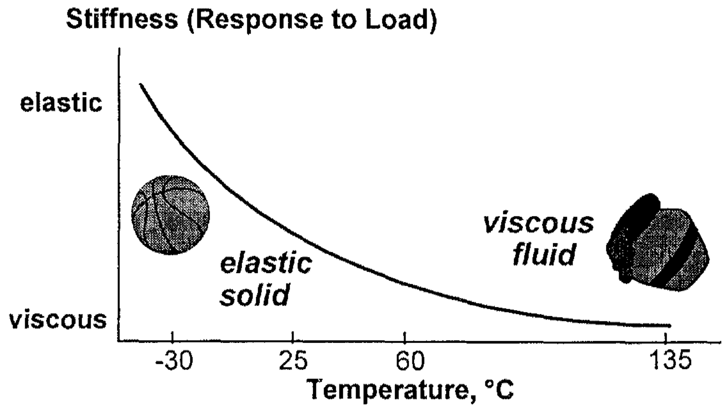
Asphalt is a *viscoelastic* material. This term means that asphalt has the properties of both a viscous material, such as motor oil, or more realistically, water, and an elastic material, such as a rubber. However, the property that asphalt exhibits, whether viscous, elastic, or most often, a combination of both, depends on *temperature* and *time of loading*. The flow behavior of an asphalt could be the same for one hour at 60°C or 10 hours at 25°C.



In other words, the effects of time and temperature are related; the behavior at high temperatures over short time periods is equivalent to what occurs at lower temperatures and longer times. This is often referred to as the time-temperature shift or superposition concept of asphalt cement.

High Temperature Behavior

In hot conditions (e.g., desert climate) or under sustained loads (e.g., slow moving trucks), asphalts cements behave like *viscous* liquids and flow. Viscosity is the material characteristic used to describe the resistance of liquids to flow.



Viscous liquids like hot asphalt are sometimes called *plastic* because once they start flowing, they do not return to their original position. This is why in hot weather, some asphalt pavements flow under repeated wheel loads and wheel path ruts form. However, rutting in asphalt pavements during hot weather is also influenced by aggregate properties and it is probably more correct to say that the asphalt *mixture* is behaving like a plastic.

Low Temperature Behavior

In cold climates (e.g., winter days) or under rapid loading (e.g., fast moving trucks), asphalt cement behaves like an *elastic* solid. Elastic solids are like rubber bands; when loaded they deform, and when unloaded, they return to their original shape. Any elastic deformation is completely recovered.

If too much load is applied, elastic solids may break. Even though asphalt is an elastic solid at low temperatures, it may become too brittle and crack when excessively loaded. This is the reason low temperature cracking sometimes occurs in asphalt pavements during cold weather. In these cases, loads are applied by internal stresses that accumulate in the pavement when it tries to shrink and is restrained (e.g., as when temperatures fall during and after a sudden cold front).

Intermediate Temperature Behavior

Most environmental conditions lie between the extreme hot and cold situations. In these climates, asphalt binders exhibit the characteristics of both viscous liquids and elastic solids. Because of this range of behavior, asphalt is an excellent adhesive material to use in paving, but an extremely complicated material to understand and explain. When heated, asphalt acts as a lubricant, allowing the aggregate to be mixed, coated, and tightly-compacted to form a smooth, dense surface. After cooling, the asphalt acts as the glue to hold the aggregate together in a solid matrix. In this finished state, the behavior of the asphalt is termed viscoelastic; it has both elastic and viscous characteristics, depending on the temperature and rate of loading.

Conceptually, this kind of response to load can be related to an automobile shock absorbing system. These systems contain a spring and a liquid filled cylinder. The spring is elastic and returns the car to the original position after hitting a bump. The viscous liquid within the cylinder dampens the force of the spring and its reaction to the bump. Any force exerted on the car causes a parallel reaction in both the spring and the cylinder. In hot mix asphalt, the spring represents the immediate elastic response of both the asphalt and the aggregate. The cylinder symbolizes the slower, viscous reaction of the asphalt, particularly in warmer temperatures. Most of the response is elastic or viscoelastic, (recoverable with time), while some of the response is plastic and non-recoverable.

Aging Behavior

Because asphalt cements are composed of organic molecules, they react with oxygen from the environment. This reaction is called oxidation and it changes the structure and composition of asphalt molecules. Oxidation causes the asphalt cement to become more brittle, generating the term oxidative hardening or age hardening.

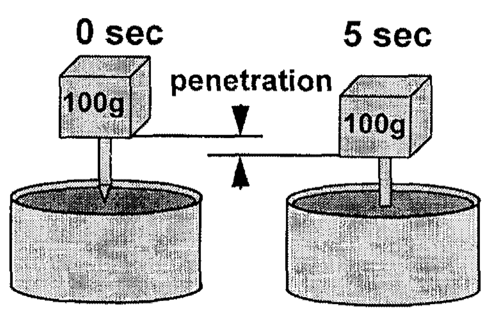
In practice, a considerable amount of oxidative hardening occurs before the asphalt is placed. At the hot mix facility, asphalt cement is added to the hot aggregate and the mixture is maintained at elevated temperatures for a period of time. Because the asphalt cement exists in thin films covering the aggregate, the oxidation reaction occurs at a much faster rate. "Short term aging" is used to describe the aging that occurs in this stage of the asphalt's "life".

Oxidative hardening also occurs during the life of the pavement, due to exposure to air and water. "Long term aging" happens at a relatively slow rate in a pavement, although it occurs faster in warmer climates and during warmer seasons. Because of this hardening, old asphalt pavements are more susceptible to cracking. Improperly compacted asphalt pavements may exhibit premature oxidative hardening. In this case, inadequate compaction leaves a higher percentage of interconnected air voids, which allows more air to penetrate into the asphalt mixture, leading to more oxidative hardening.

Other forms of hardening include volatilization and physical hardening. Volatilization occurs during hot mixing and construction, when volatile components tend to evaporate from the asphalt. Physical hardening occurs when asphalt cements have been exposed to low temperatures for long periods. When the temperature stabilizes at a constant low value, the asphalt cement continues to shrink and harden. Physical hardening is more pronounced at temperatures less than 0°C and must be considered when testing asphalt cements at very low temperatures.

PRE-SUPERPAVE ASPHALT PROPERTY MEASUREMENTS

Because of its chemical complexities, asphalt specifications have been developed around physical property tests, using such tests as penetration, viscosity, and ductility. These physical property tests are performed at standard test temperatures, and the test results are used to determine if the material meets the specification criteria. However, there are limitations in what these test procedures provide. Many of these tests are empirical, meaning that field experience is required before the test results yield meaningful information. Penetration is an example of this. The penetration test represents the stiffness of the asphalt, but any relationship between asphalt penetration and performance has to be gained by experience. An additional drawback of empiricism is that the relationship between the test and performance may not be very good.



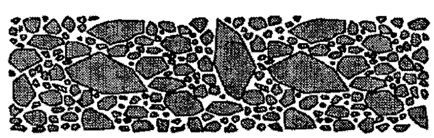
MINERAL AGGREGATE BEHAVIOR

A wide variety of mineral aggregates have been used to produce HMA. Some materials are referred to as *natural* aggregate because they are simply mined from river or glacial deposits and are used without further processing to manufacture HMA. These are often called "bank-run" or "pit-run" materials. *Processed* aggregate can include natural aggregate that has been separated into distinct size fractions, washed, crushed, or otherwise treated to enhance certain performance characteristics of the finished HMA. In most cases, the main processing consists of crushing and sizing.

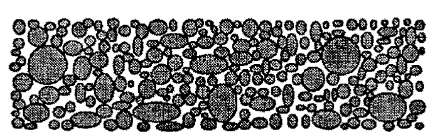
Synthetic aggregate consists of any material that is not mined or quarried and in many cases represents an industrial by-product. Blast furnace slag is one example. Occasionally, a synthetic aggregate will be produced to impart a desired performance characteristic to the HMA. For example, light-weight expanded clay or shale is sometimes used as a component to improve the skid resistance properties of HMA.

An existing pavement can be removed and reprocessed to produce new HMA. Reclaimed asphalt pavement or "RAP" is a growing and important source of aggregate for asphalt pavements.

Increasingly, waste products are used as aggregate or otherwise disposed of in asphalt pavements. Scrap tires and glass are the two most well known waste products that have been successfully "landfilled" in asphalt pavements. In some cases, waste products can actually be used to enhance certain performance characteristics of HMA. In other cases, it is considered sufficient that a solid waste disposal problem has been solved and no performance enhancing benefit from the waste material is expected. However, it is hoped that performance will not be sacrificed simply to eliminate a solid waste material.



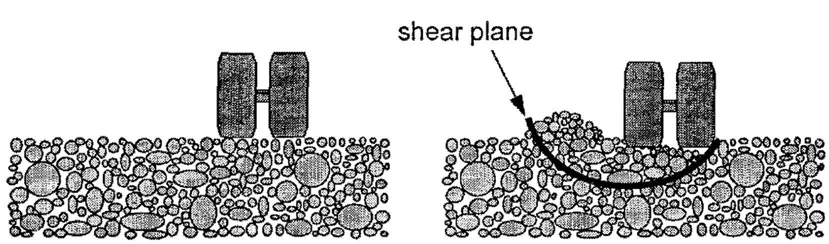
Cubical Aggregate



Rounded Aggregate

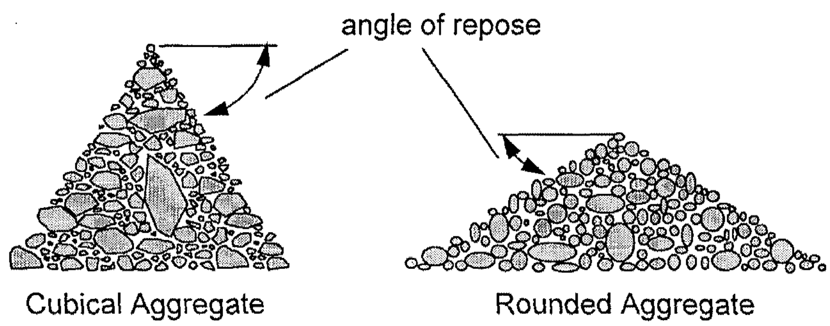
Regardless of the source, processing method, or mineralogy, aggregate is expected to provide a strong, stone skeleton to resist repeated load applications. Cubical, rough-textured aggregates provide more strength than rounded, smooth-textured aggregates Even though a cubical piece and rounded piece of aggregate may possess the same inherent strength, cubical aggregate particles tend to lock together resulting in a stronger mass of material. Instead of locking together, rounded aggregate particles tend to slide by each other.

When a mass of aggregate is loaded, there may occur within the mass a plane where aggregate particles begin to slide by or "shear" with respect to each other, which results in permanent deformation of the mass. It is at this plane where the "shear stress" exceeds the "shear strength" of the aggregate mass. Aggregate shear strength is of critical importance in HMA.



Before Load After Load

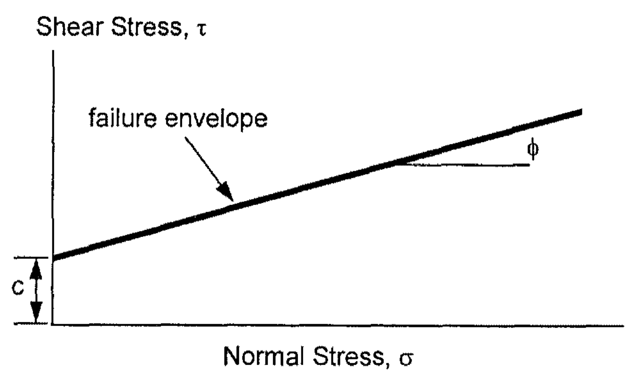
Contrasting aggregate shear strength behavior can easily be observed in aggregate stockpiles since crushed (i.e., mostly cubical) aggregates form steeper, more stable piles than rounded aggregates. The slope on stockpiles is the angle of repose. The angle of repose of a crushed aggregate stockpile is greater than that of an uncrushed aggregate stockpile.



Engineers explain the shearing behavior of aggregates and other materials using Mohr-Coulomb theory, named after the individuals who originated the concept. This theory declares that the shear strength of an aggregate mixture is dependent on how well the aggregate particles hold together in a mass (often called cohesion), the stress the aggregates may be under, and the internal friction of the aggregate. The Mohr-Coulomb equation used to express the shear strength of a material is:

$$\tau = c + \sigma \times \tan \phi$$

where, τ = shear strength of aggregate mixture,
c = cohesion of aggregate,
σ = normal stress to which the aggregate is subjected
ϕ = angle of internal friction.



A mass of aggregate has relatively little cohesion. Thus, the shear strength is primarily dependent on the resistance to movement provided by the aggregates. In addition, when loaded, the mass of aggregate tends to be stronger because the resulting stress tends to hold the aggregate more tightly together. In other words, shear strength is increased. The angle of internal friction indicates the ability of aggregate to interlock, and thus, create a mass of aggregate that is almost as strong as the individual pieces.

To ensure a strong aggregate blend for HMA, engineers typically have specified aggregate properties that enhance the internal friction portion of the overall shear strength. Normally, this is accomplished by specifying a certain percentage of crushed faces for the coarse portion of an aggregate blend. Because natural sands tend to be rounded, with poor internal friction, the amount of natural sand in a blend is often limited.

SUPERPAVE MINERAL AGGREGATE PROPERTY MEASUREMENTS

During the SHRP research, pavement experts were surveyed to ascertain which aggregate properties were most important. There was general agreement that aggregate properties played a central role in overcoming permanent deformation. Fatigue cracking and low temperature cracking were less affected by aggregate characteristics. SHRP researchers relied on the experience of these experts and their own to identify two categories of aggregate properties that needed to be used in the Superpave system: consensus properties and source properties. In addition, a new way of specifying aggregate gradation was developed. It is called the design aggregate structure.

Consensus Properties

It was the consensus of the pavement experts that certain aggregate characteristics were critical and needed to be achieved in all cases to arrive at well performing HMA. These characteristics were called "consensus properties" because there was wide agreement in their use and specified values. Those properties are:

- coarse aggregate angularity,
- fine aggregate angularity,
- flat, elongated particles, and
- clay content.

There are required standards for these aggregate properties. The consensus standards are not uniform. They 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 standards. They are 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. Each of these consensus property tests will be described in detail later in this text.

Source Properties

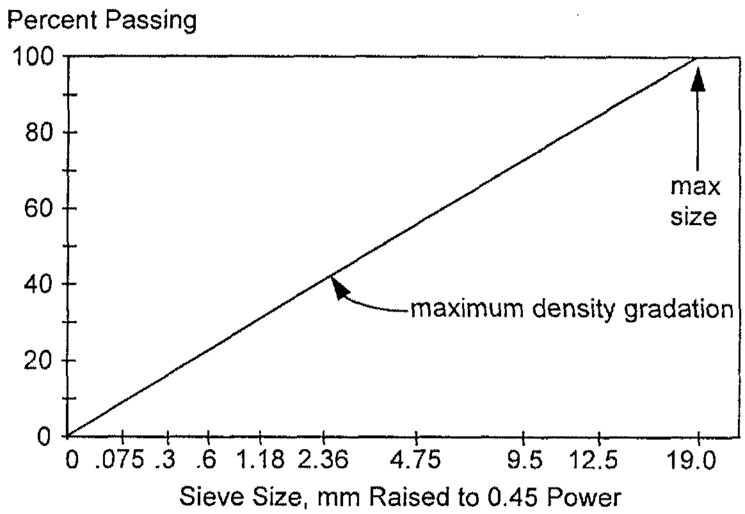
In addition to the consensus aggregate properties, pavement experts believed that certain other aggregate characteristics were critical. However, critical values of these properties could not be reached by consensus because needed values were source specific. Consequently, a set of "source properties" was recommended. Specified values are established by local agencies. While these properties are relevant during the mix design process, they may also be used as source acceptance control. Those properties are:

- toughness,
- soundness, and
- deleterious materials

Gradation

To specify gradation, Superpave uses a modification of an approach already used by some agencies. It uses the 0.45 power gradation chart to define a permissible gradation. An important feature of the 0.45 power chart is the maximum density gradation. This gradation plots as a straight line from the maximum aggregate size through the origin. Superpave uses a standard set of ASTM sieves and the following definitions with respect to aggregate size:

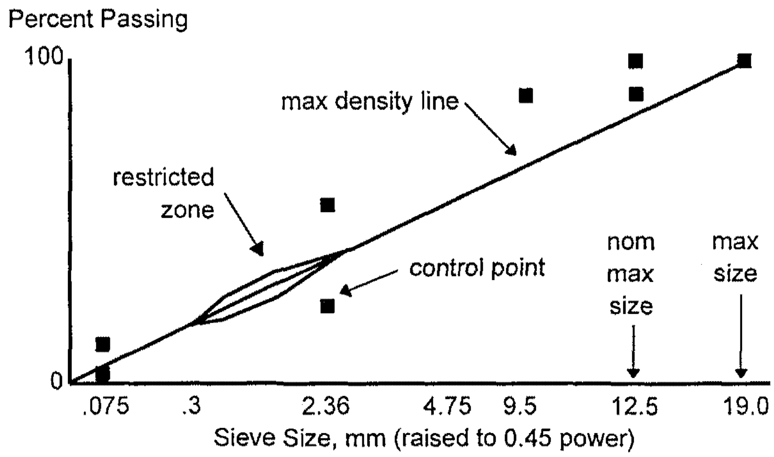
- Maximum Size: One sieve size larger than the nominal maximum size.
- Nominal Maximum Size: One sieve size larger than the first sieve to retain more than 10 percent.



The maximum density gradation represents a gradation in which the aggregate particles fit together in their densest possible arrangement. Clearly this is a gradation to avoid because there would be very little aggregate space within which to develop sufficiently thick asphalt films for a durable mixture. Shown is a 0.45 power gradation chart with a maximum density gradation for a 19 mm maximum aggregate size and 12.5 mm nominal maximum size.

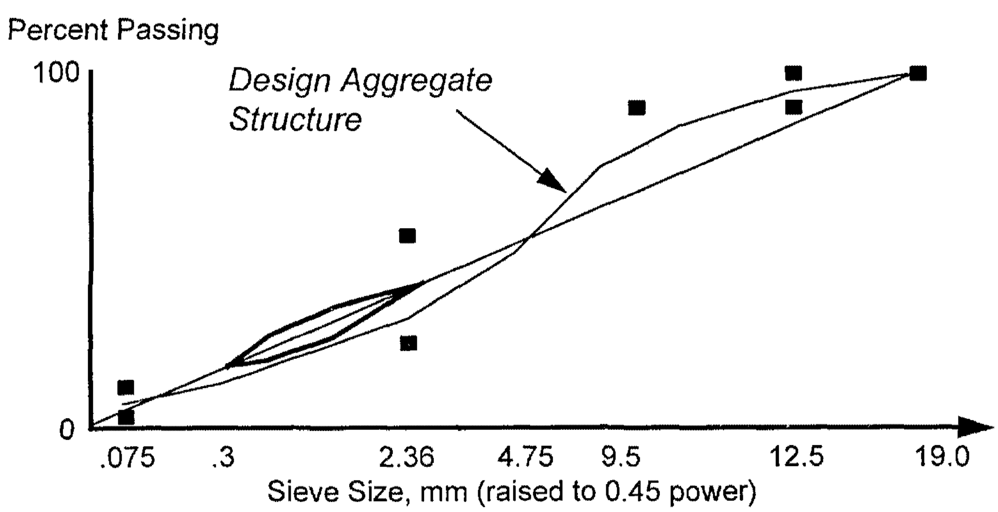
To specify aggregate gradation, two additional features are added to the 0.45 power chart: control points and a restricted zone. Control points function as master ranges through which gradations must pass. They are placed on the nominal maximum size, an intermediate size (2.36 mm), and the dust size (0.075 mm). Illustrated are the control points and restricted zone for a 12.5 mm Superpave mixture.

The restricted zone resides along the maximum density gradation between the intermediate size (either 4.75 or 2.36 mm) and the 0.3 mm size. It forms a band through which gradations should not pass. Gradations that pass through the restricted zone have often been called "humped gradations" because of the characteristic hump in the grading curve that passes through the restricted zone. In most cases, a humped gradation indicates a mixture that possesses too much fine sand in relation to total sand. This gradation practically always results in tender mix behavior, which is manifested by a mixture that is difficult to compact during construction and offers reduced resistance to permanent deformation during its performance life. Gradations that violate the restricted zone possess weak aggregate skeletons that depend too much on asphalt binder stiffness to achieve mixture shear strength. These mixtures are also very sensitive to asphalt content and can easily become plastic.



The term used to describe the cumulative frequency distribution of aggregate particle sizes is the *design aggregate structure.* A design aggregate structure that lies between the control points and avoids the restricted zone meets the requirements of Superpave with respect to gradation. Superpave defines five mixture types as defined by their nominal maximum aggregate size:

Superpave Mixtures		
Superpave Mixture Designation	Nominal Maximum Size, mm	Maximum Size, mm
37.5 mm	37.5	50
25 mm	25	37.5
19 mm	19	25
12.5 mm	12.5	19
9.5 mm	9.5	12.5



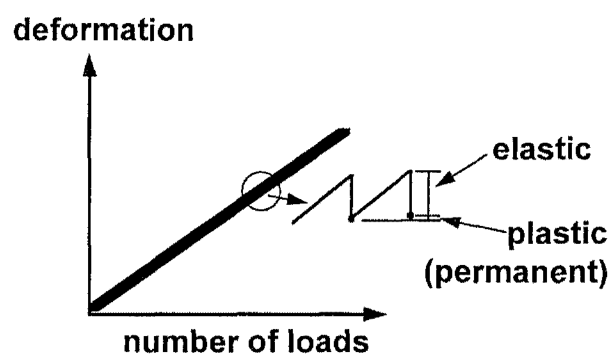
ASPHALT MIXTURE BEHAVIOR

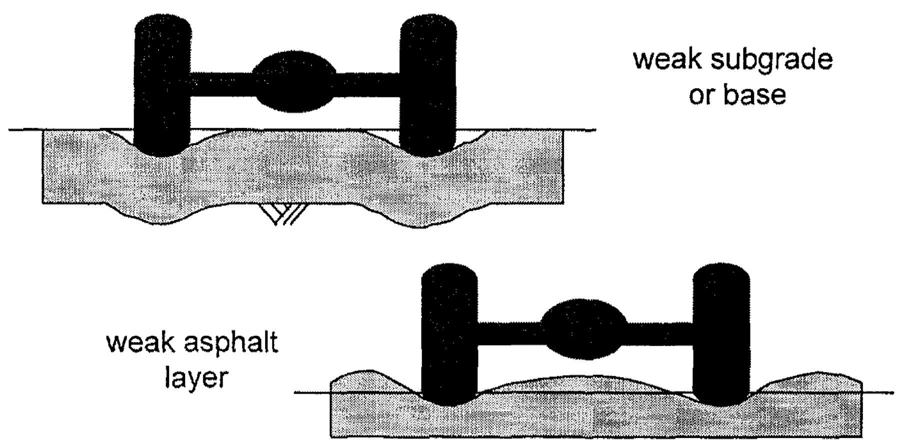
When a wheel load is applied to a pavement, two stresses are transmitted to the HMA: vertical compressive stress within the asphalt layer, and horizontal tensile stress at the bottom of the asphalt layer. The HMA must be internally strong and resilient to resist the compressive stresses and prevent permanent deformation within the mixture. In the same manner, the material must also have enough tensile strength to withstand the tensile stresses at the base of the asphalt layer, and also be resilient to withstand many load applications without fatigue cracking. The asphalt mixture must also resist the stresses imparted by rapidly decreasing temperatures and 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 in Superpave.

Permanent Deformation

Permanent deformation is the distress that is characterized by a surface cross section that is no longer in its design position. It is called "permanent" deformation because it represents an accumulation of small amounts of deformation that occurs each time a load is applied. This deformation cannot be recovered. Wheel path rutting is the most common form of permanent deformation. While rutting can have many sources (e.g., underlying HMA weakened by moisture damage, abrasion, and traffic densification), it has two principal causes.



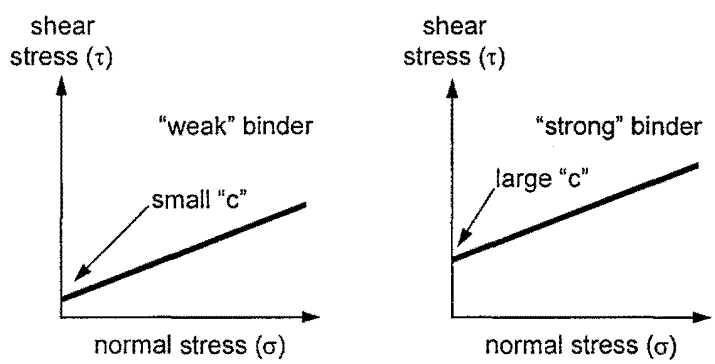


In one case, the rutting is caused by too much repeated stress being applied to the subgrade (or subbase or base) below the asphalt layer. Although stiffer paving materials will partially reduce this type of rutting, it is normally considered more of a structural problem rather than a materials problem. Essentially, there is not enough pavement strength or thickness to reduce the applied stresses to a tolerable level. A pavement layer that has been unexpectedly weakened by the intrusion of moisture may also cause it. The deformation occurs in the underlying layers rather than in the asphalt layers.

The type of rutting of most concern to asphalt designers is deformation in the asphalt layers. This rutting results from an asphalt mixture without enough shear strength to resist the repeated heavy loads. A weak mixture will accumulate small, but permanent, deformations with each truck pass, eventually forming a rut characterized by a downward and lateral movement of the mixture. The rutting may occur in the asphalt surface course, or the rutting that shows on the surface may be caused to a weak underlying asphalt course.

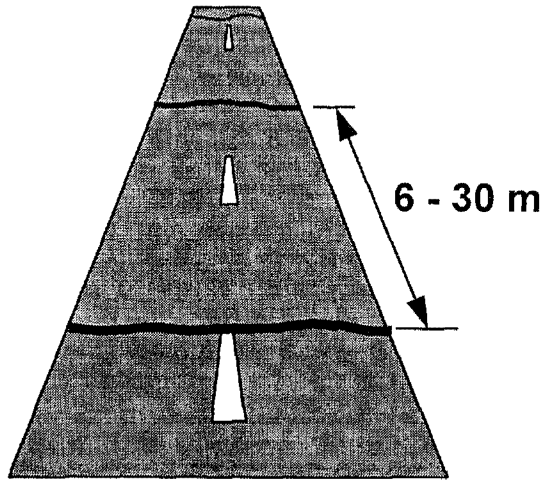
Rutting of a weak asphalt mixture typically occurs during the summer under higher pavement temperatures. While this might suggest that rutting is solely an asphalt cement problem, it is more correct to address rutting by considering the mineral aggregate and asphalt cement. In fact, the previously described Mohr-Coulomb equation ($\tau = c + \sigma \times \tan \phi$) can again be used to illustrate how both materials can affect rutting.

$$\underbrace{\tau}_{\textbf{shear strength}} = \underbrace{\mathbf{c}}_{\substack{\textbf{asphalt binder}\\ \textbf{contribution}}} + \underbrace{\sigma}_{\substack{\textbf{normal}\\ \textbf{stress}}}(\tan \underbrace{\phi}_{\substack{\textbf{aggregate}\\ \textbf{contribution}}})$$



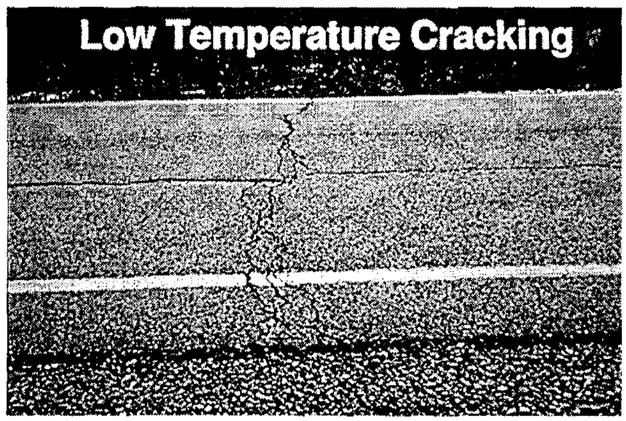
In this case, τ is considered the shear strength of the asphalt mixture. The cohesion (c) can be considered the portion of the overall mixture shear strength provided by the asphalt cement. Because rutting is an accumulation of very small permanent deformations, one way to ensure that asphalt cement provides its "fair share" of shear strength is to use an asphalt cement that is not only stiffer but also behaves more like an elastic solid at high pavement temperatures. That way, when a load is applied to the asphalt cement in the mixture, it tends to act more like a rubber band and spring back to its original position rather than stay deformed.

Low Temperature Cracking



Low temperature cracking is caused by adverse environmental conditions rather than by applied traffic loads. It is characterized by intermittent transverse cracks that occur at a surprisingly consistent spacing.

Low temperature cracks form when an asphalt pavement layer shrinks in cold weather. As the pavement shrinks, tensile stresses build within the layer. At some point along the pavement, the tensile stress exceeds the tensile strength and the asphalt layer cracks. Low temperature cracks occur primarily from a single cycle of low temperature, but can develop from repeated low temperature cycles.



The asphalt binder plays the key role in low temperature cracking. In general, hard asphalt binders are more prone to low temperature cracking than soft asphalt binders. Asphalt binders that are excessively aged, because they are unduly prone to oxidation and/or contained in a mixture constructed with too many air voids, are more prone to low temperature cracking. Thus, to overcome low temperature cracking engineers must use a soft binder that is not overly prone to aging, and control in-place air void content and pavement density so that the binder does not become excessively oxidized.

Two new sophisticated testing devices were developed: the Superpave Shear Tester (SST) and Indirect Tensile Tester (IDT). The test output from these devices can provide direct indications of mix behavior, or will eventually generate input to performance prediction models.

Using the mechanical properties of the HMA and these performance prediction models, mix design engineers will be able to estimate the combined effect of asphalt binders, aggregates, and mixture proportions. The models will take into account the structure, condition, and properties of the existing pavement (if applicable) and the amount of traffic to which the proposed mixture will be subjected over its performance life. The output of the models will be millimeters of rutting, percent area of fatigue cracking, and spacing (in meters) of low temperature cracks. By using this approach, the Superpave system will become the ultimate design procedure by linking material properties with pavement structural properties to predict actual pavement performance. When the pavement modeling is completed, the benefit (or detriment) of new materials, different mix designs, asphalt modifiers, and other products can be quantified in terms of cost versus predicted performance. This capability would reduce the dependency on field test sections for relative comparisons.

III. Superpave Binders

Superpave uses a completely new system for testing, specifying, and selecting asphalt binders. The objectives of this section will be to:

- describe the Superpave binder test equipment
- discuss where the tests fit into the range of material conditions (temperature and aging conditions) experienced by asphalt pavements
- explain the Superpave specification requirements and how they are used in preventing permanent deformation, fatigue cracking and low temperature cracking
- discuss how to select the performance grade (PG) binder for a project's climatic and traffic conditions

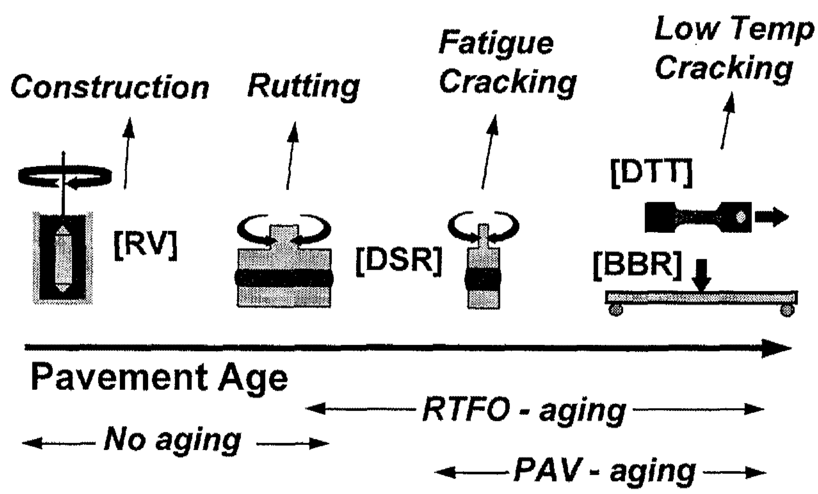
SUPERPAVE BINDER TESTS

Binder Aging Methods

A central theme of the Superpave binder specification is its reliance on testing asphalt binders in conditions that simulate critical stages during the binder's life. The three most critical stages are:

- during transport, storage, and handling,
- during mix production and construction, and
- after long periods in a pavement

Tests performed on unaged asphalt represent the first stage of transport, storage, and handling.



Aging the binder in a rolling thin film oven (RTFO) simulates the second stage, during mix production and construction. The RTFO aging technique was developed by the California Highway Department and is detailed in AASHTO T 240 (ASTM D 2872). This test exposes films of binder to heat and air and approximates the exposure of asphalt to these elements during hot mixing and handling.

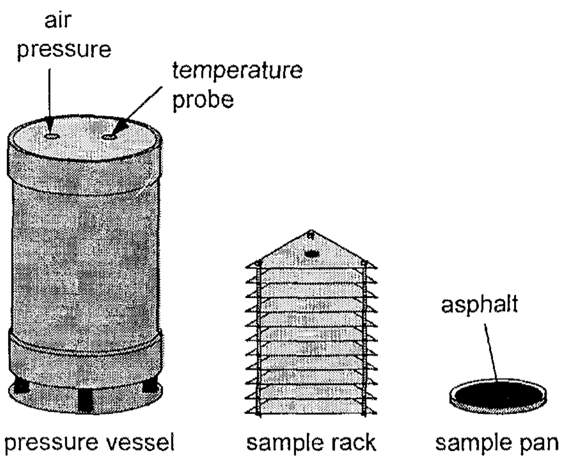
The third stage of binder aging occurs after a long period in a pavement. This stage is simulated by use of a pressure aging vessel (PAV). This test exposes binder samples to heat and pressure in order to simulate, in a matter of hours, years of in-service aging in a pavement.

It is important to note that for specification purposes, binder samples aged in the PAV have already been aged in the RTFO. Consequently, PAV residue represents binder that has been exposed to all the conditions to which binders are subjected during production and in-service.

PRESSURE AGING VESSEL

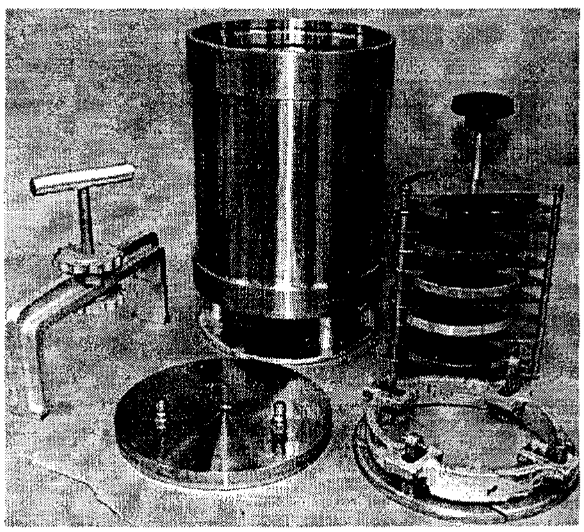
Test Equipment

Two types of pressure aging devices have been developed. The first type consisted of the stand-alone pressure aging vessel that was placed inside a temperature chamber. The second type consists of the pressure vessel built as part of the temperature chamber. The operating principles of the equipment are the same. Specific equipment details can be found in AASHTO PP1, *"Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)"*. For illustrative purposes, the separate vessel type is shown and described here.



The pressure vessel is fabricated from stainless steel and is designed to operate under the pressure and temperature conditions of the test. The vessel must hold at least 10 sample pans, using a sample rack that fits conveniently into the vessel. The vessel lid is prevents pressure loss.

Air pressure is provided by a cylinder of dry, clean compressed air with a pressure regulator, release valve, and a slow release bleed valve. The vessel lid is fitted with a pressure coupling and temperature transducer. The temperature transducer connects to a digital indicator that allows visual monitoring of internal vessel temperature throughout the aging period. Continuous monitoring of temperature is required during the test.



A forced draft oven is used as a temperature chamber. The oven should be able to control the test temperature to within ± 0.5° C for the duration of the test.

Specimen Preparation

To prepare for the PAV, RTFO residue is transferred to individual PAV pans. The sample should be heated only to the extent that it can be readily poured and stirred to ensure homogeneity. Each PAV sample should weigh 50 g. Residue from approximately two RTFO bottles is normally needed for one 50-g sample.

Overview of Procedure

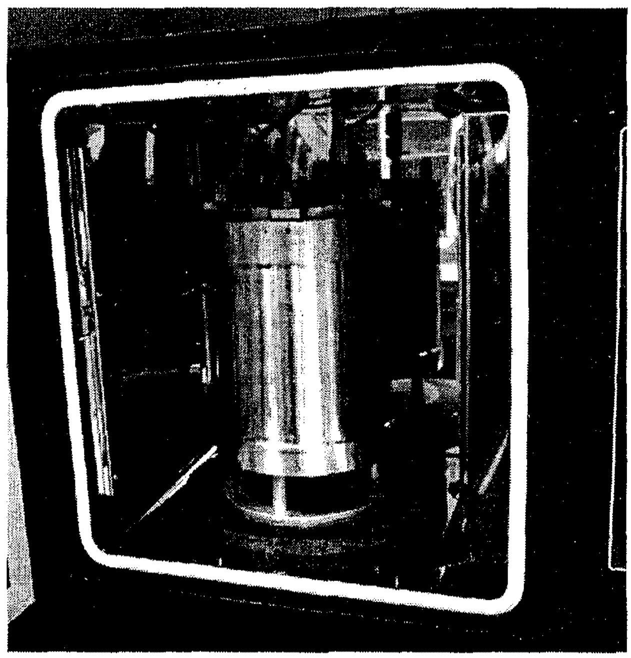
The temperature chamber (oven) is turned on and the vessel is placed in the chamber, unpressurized, and allowed to reach the desired test temperature. The PAV pans are placed in the sample rack. When the test temperature has been achieved the vessel is removed from the oven and the samples in the sample rack are placed in the hot vessel. The lid is then secured and the temperature and pressure are maintained for the 20-hour aging period.

Data Presentation

The sole purpose of the PAV procedure is the preparation of aged binder materials for further testing and evaluation with the Superpave binder tests. A report for the PAV procedure contains:

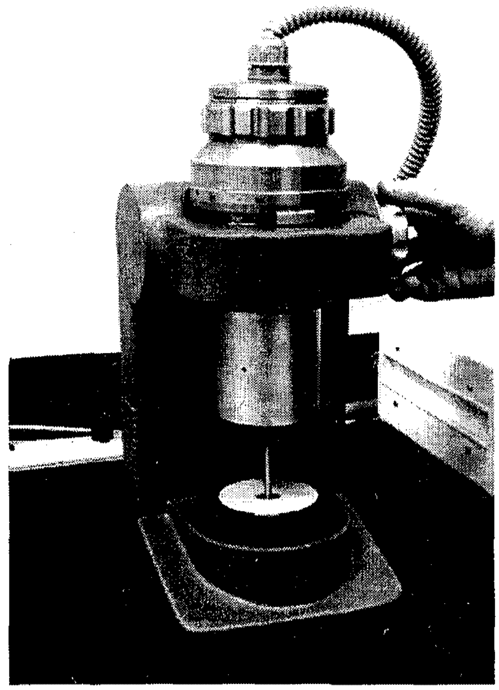
- sample identification,
- aging test temperature to the nearest 0.1° C,
- maximum and minimum aging temperature recorded to the nearest 0.1° C,
- total time during aging that temperature was outside the specified range to the nearest 0.1 min., and
- total aging time in hours and minutes.

Pressure Vessel Built into Oven



Pressure Aging Vessel Inside of Oven

Dynamic Shear Rheometer

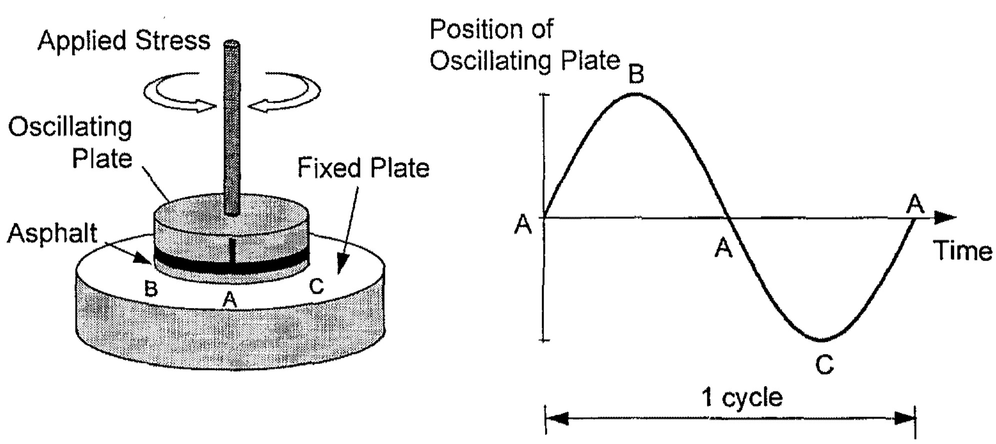


As discussed earlier, asphalt is a viscoelastic material, meaning that it simultaneously shows the behavior of an elastic material (e.g. rubber band) and a viscous material (e.g. molasses). The relationship between these two properties is used to measure the ability of the binder to resist permanent deformation and fatigue cracking. To resist rutting, a binder needs to be stiff and elastic; to resist fatigue cracking, the binder needs to be flexible and elastic. The balance between these two needs is a critical one.

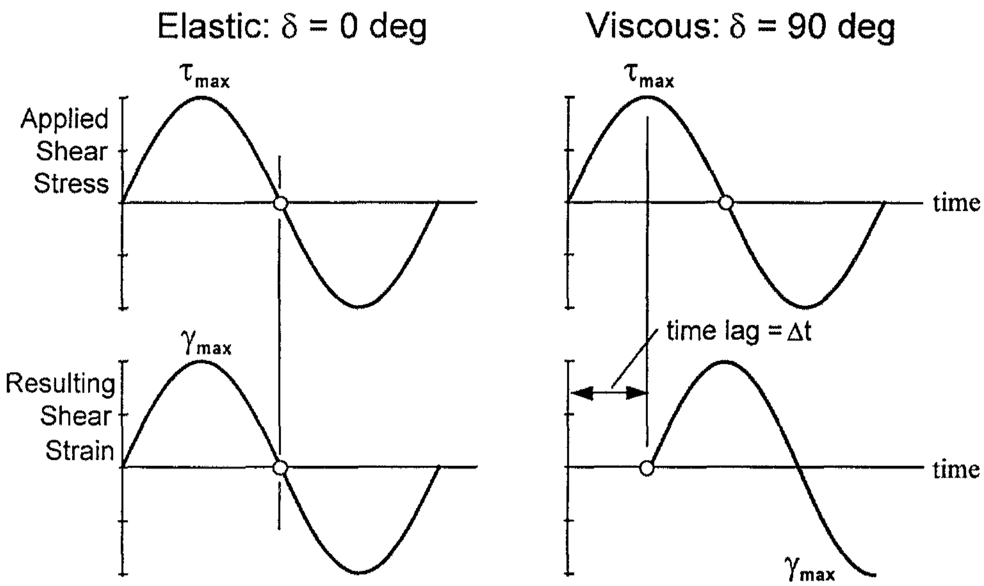
The Dynamic Shear Rheometer (DSR) is used to characterize the viscous and elastic behavior of asphalt binders. It does this by measuring the viscous and elastic properties of a thin asphalt binder sample sandwiched between an oscillating and a fixed plate. Operational details of the DSR can be found in AASHTO TP5 *"Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer."*

Test Equipment

The principle of operation of the DSR is straightforward. An asphalt sample is sandwiched between an oscillating spindle and the fixed base. The oscillating plate (often called a "spindle") starts at point A and moves to point B. From point B the oscillating plate moves back, passing point A on the way to point C. From point C the plate moves back to point A. This movement, from A to B to C and back to A comprises one cycle.

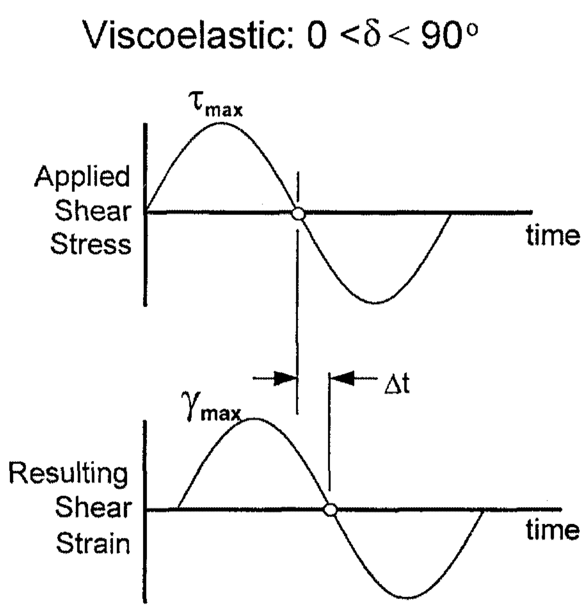


As the force (or shear stress, τ) is applied to the asphalt by the spindle, the DSR measures the response (or shear strain, γ) of the asphalt to the force. If the asphalt were a perfectly elastic material, the response would coincide immediately with the applied force, and the time lag between the two would be zero. A perfectly viscous material would have a large time lag between load and response. Very cold asphalt performs like an elastic material. Very hot asphalt performs like a viscous material.



At temperatures where most pavements carry traffic, asphalt behaves both like an elastic solid and a viscous liquid. The relationship between the applied stress and the resulting strain in the DSR quantifies both types of behavior, and provides information necessary to calculate two important asphalt binder properties: the complex shear modulus (G* -"G star") and phase angle (δ - "delta").

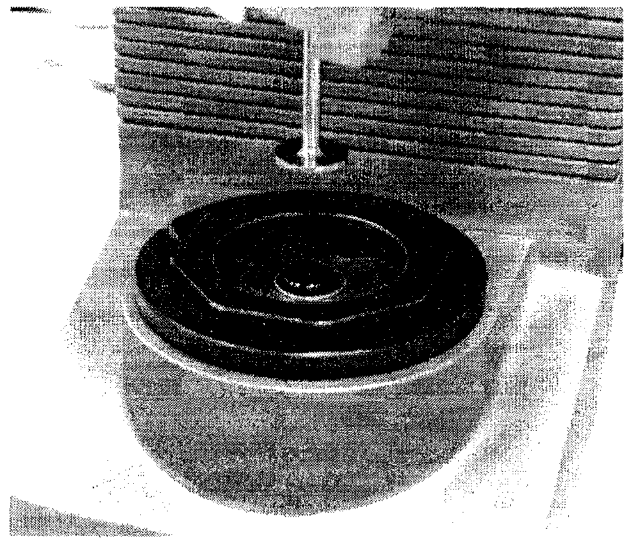
G* is the ratio of maximum shear stress (τ_{max}) to maximum shear strain (γ_{max}). The time lag between the applied stress and the resulting strain is the phase angle δ. For a perfectly elastic material, the phase angle, δ, is zero, and all of the deformation is temporary. For a viscous material (such as hot asphalt), the phase angle approaches 90 degrees, and all of the deformation is permanent. In the DSR, a viscoelastic material such as asphalt at normal service temperatures displays a stress-strain response between the two extremes, as shown below.



$$G^* = \frac{\tau_{max}}{\gamma_{max}}$$

Δt = time lag $\Longrightarrow \delta$

Because the properties of asphalt binders are so temperature dependent, rheometers must have a precise means of controlling the temperature of the sample. This is normally accomplished by means of a circulating fluid bath or forced air bath. Fluid baths normally use water to surround the sample. The water is circulated through a temperature controller that precisely adjusts and maintains the sample temperature uniformly at the desired value. Air baths operate in the same manner as water baths except that they surround the sample with heated air during testing. In either case, the temperature of the air or water must be controlled so that the temperature of the sample across the gap is uniform and varies by no more than 0.1°C.



Data Presentation

The DSR is capable of measuring asphalt response over a range of temperature, frequency, and strain levels. However, G* and δ are required for Superpave specification testing at specific conditions. The DSR software calculates G* and δ. Therefore, it is a simple matter of comparing results with requirements of the Superpave specification to determine compliance. A complete report includes:

- G* to the nearest three significant figures,
- δ to the nearest 0.1 degrees,
- test plate size to the nearest 0.1 mm and gap to nearest 1μm,
- test temperature to the nearest 0.1°C,
- test frequency to the nearest 0.1 rad/sec, and
- strain amplitude to the nearest 0.01 percent.

G* is divided by sin δ to develop a "high temperature stiffness" factor that addressed rutting; G* is multiplied by sin δ to develop an "intermediate temperature stiffness" factor that addresses fatigue cracking. The use of these parameters in discussed later in this section.

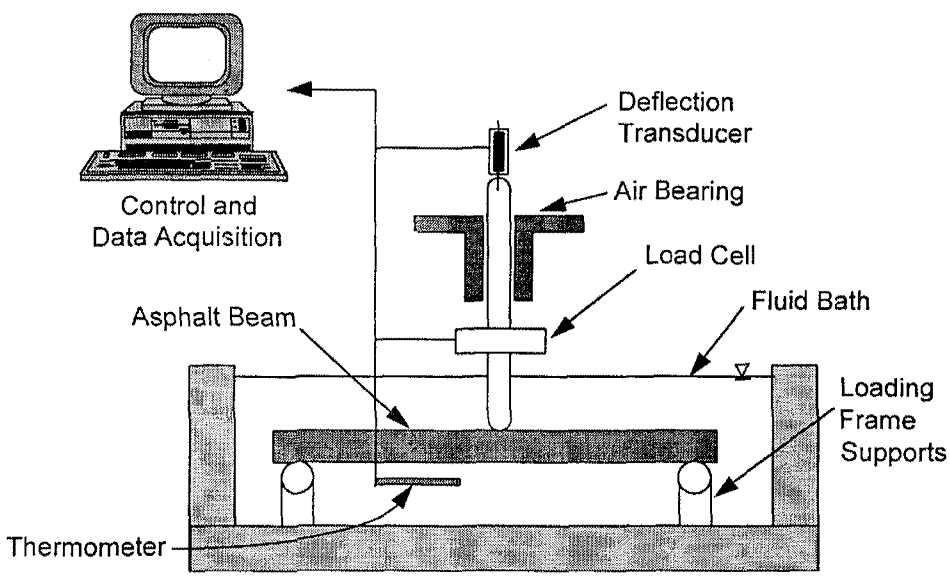
Bending Beam Rheometer

The Bending Beam Rheometer (BBR) is used to measure the stiffness of asphalts at very low temperatures. The test uses engineering beam theory to measure the stiffness of a small asphalt beam sample under a creep load. A creep load is used to simulate the stresses that gradually build up in a pavement when temperature drops. Two parameters are evaluated with the BBR. *Creep stiffness* is a measure of how the asphalt resists constant loading and the *m-value* is a measure of how the asphalt stiffness changes as loads are applied.

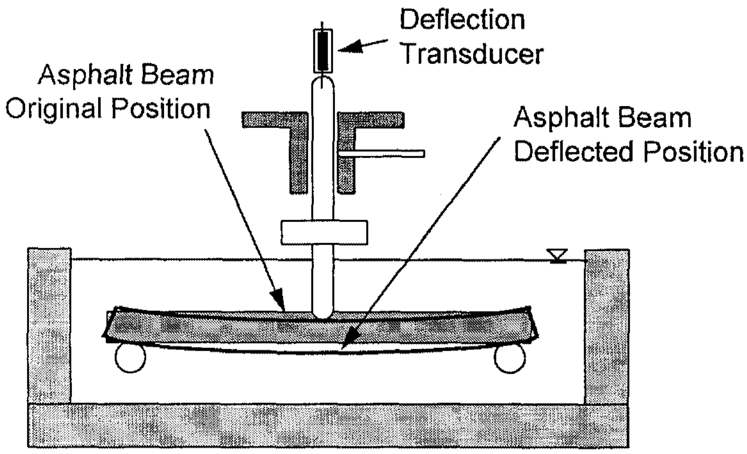
Details of the BBR test procedure can be found in AASHTO TP1 *"Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)."*

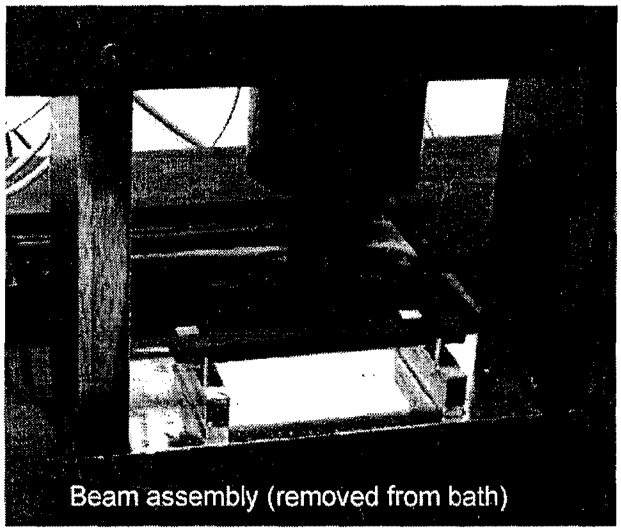
Test Equipment

The BBR gets its name from the test specimen geometry and loading method used during testing. The key elements of the BBR are a loading frame, controlled temperature fluid bath, computer control and data acquisition system, and test specimen. The BBR uses a blunt-nosed shaft to apply a midpoint load to the asphalt beam , which is supported at two locations. A load cell is mounted on the loading shaft, which is enclosed in an air bearing to eliminate any frictional resistance when applying load. A deflection measuring transducer is affixed to the shaft to monitor deflections. Loads are applied by pneumatic pressure and regulators are provided to adjust the load applied through the loading shaft. The fluid bath maintains the test temperature to within 0.1°C.



As the load bends the beam, the *deflection transducer monitors the* movement. This deflection is plotted against time to determine creep stiffness and m-value. During the test, load and deflection versus time plots are continuously generated on the computer screen for the operator to observe. At the end of the test period, the test load is automatically removed and the rheometer software calculates creep stiffness and m-value.




Beam assembly (removed from bath)

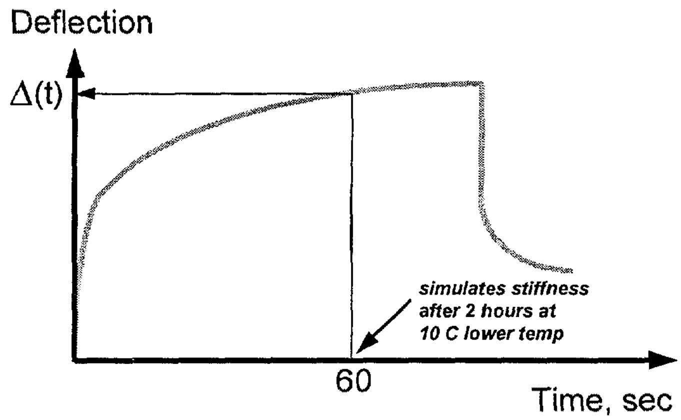
Data Presentation

Beam analysis theory is used to obtain creep stiffness of the asphalt in this test. The formula for calculating creep stiffness, S(t), is:

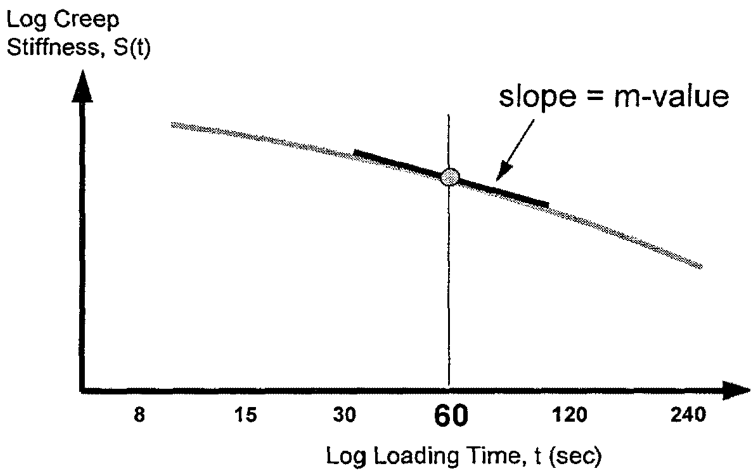
$$S(t) = \frac{PL^3}{4bh^3\Delta(t)}$$

where,
S(t) = creep stiffness at time, t = 60 seconds
P = applied constant load, 100 g (980 mN)
L = distance between beam supports, 102 mm
b = beam width, 12.5 mm
h = beam thickness, 6.25 mm
$\Delta(t)$ = deflection at time, t = 60 seconds

By using the equation for S(t) and the deflection from the graph, the stiffness at time, t=60 seconds can be obtained. Creep stiffness is desired at the minimum pavement design temperature after two hours of load. However, SHRP researchers discovered that by raising the test temperature 10° C, an equal stiffness is obtained after a 60 second loading. The obvious benefit is that a test result can be measured in a much shorter period of time.



The second parameter needed from the bending beam test is the m-value. The m-value represents the rate of change of the stiffness, S(t), versus time. This value also is calculated automatically by the bending beam computer. However, to check the results from the computer, the value for m is easily obtained. To obtain m-value, the stiffness is calculated at several loading times. These values are then plotted against time. The m-value is the slope of the log stiffness versus log time curve at any time, t.

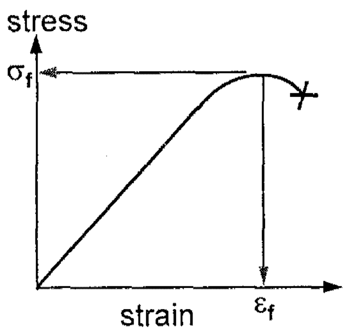


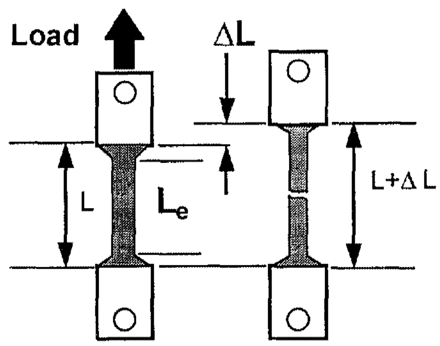
Computer-generated output for the bending beam test automatically reports all required reporting items. It includes plots of deflection and load versus time, actual load and deflection values at various times, test parameters, and operator information.

Direct Tension Tester

The direct tension test measures the low temperature ultimate tensile strain of an asphalt binder. The test is performed at relatively low temperatures ranging from 0° to -36°C, the temperature range within which asphalt exhibits brittle behavior. Furthermore, the test is performed on binders that have been aged in a rolling thin film oven and pressure aging vessel. Consequently, the test measures the performance characteristics of binders as if they had been exposed to hot mixing in a mixing facility and some in-service aging.

A small dog-bone shaped specimen is loaded in tension at a constant rate. The strain in the specimen at failure (ε_f) is the change in length (ΔL) divided by the effective gauge length (L).

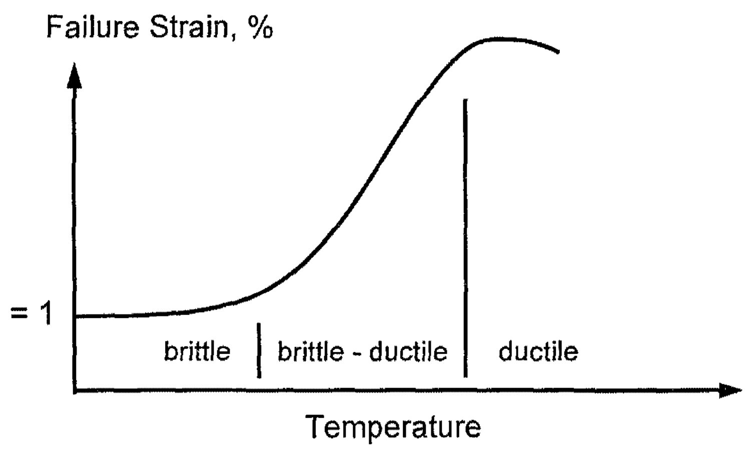




$$\textbf{failure strain } (\varepsilon_f) = \frac{\textbf{change in length } (\Delta L)}{\textbf{effective length } (L_e)}$$

In the direct tension test, failure is defined by the stress where the load on the specimen reaches its maximum value, and not necessarily the load when the specimen breaks. Failure stress (σ_f) is the failure load divided by the original cross section of the specimen (36 mm^2).

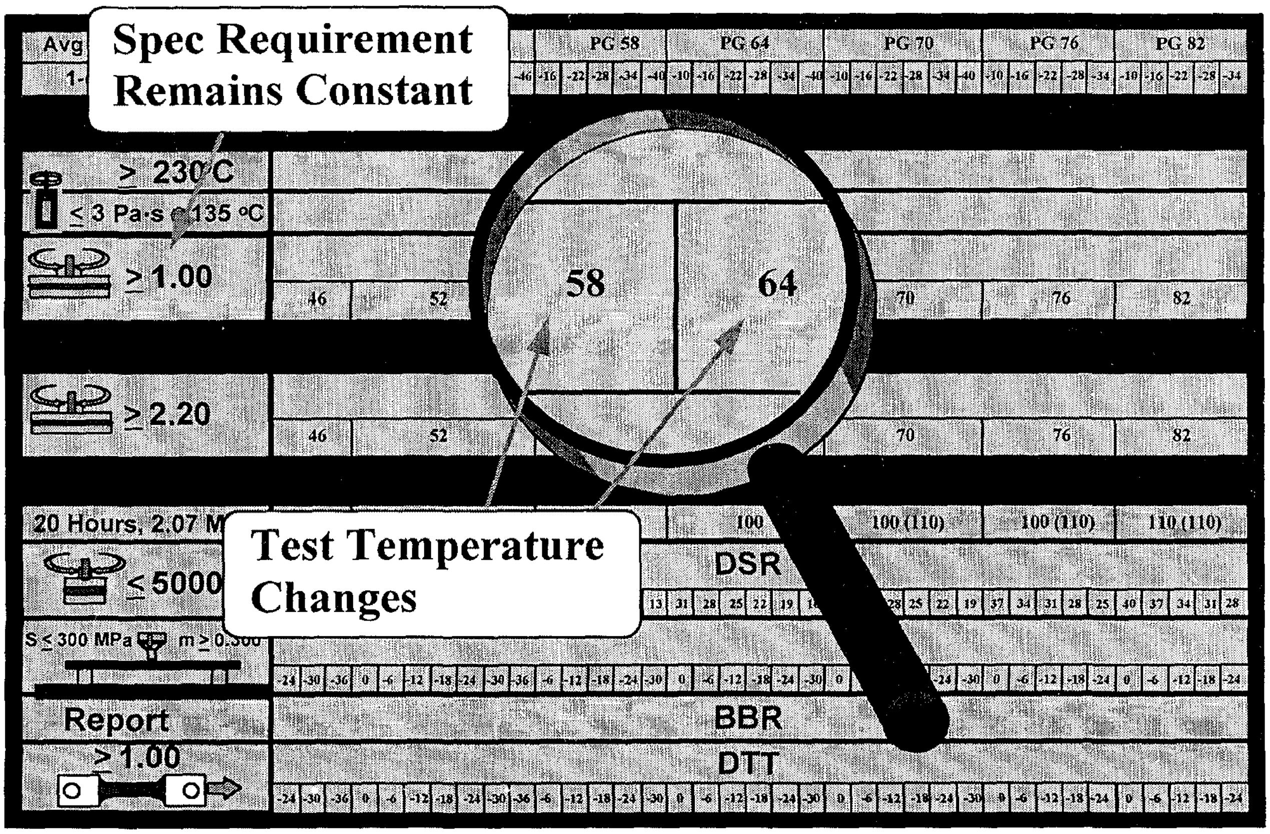
The stress-strain behavior of asphalt binders depends greatly on their temperature. If an asphalt were tested in the direct tension tester at many temperatures, it would exhibit the three types of tensile failure behavior: brittle, brittle-ductile, and ductile.



SUPERPAVE ASPHALT BINDER SPECIFICATION

The Superpave asphalt binder specification (the complete provisional specification is shown in Appendix A) is intended to improve performance by limiting the potential for the asphalt binder to contribute to permanent deformation, low temperature cracking and fatigue cracking in asphalt pavements. The specification provides for this improvement by designating various physical properties that are measured with the equipment described previously. This section will explain how each of the new test parameters relates to pavement performance, and how to select the asphalt binder for a specific project.

One important difference between the currently used asphalt specifications and the Superpave specification is the overall format of the requirements. The physical properties remain constant for all of the performance grades(PG). However, the temperatures at which these properties must be achieved vary depending on the climate in which the binder is expected to serve. As an example, this partial view of the specification shows that a PG 58-22 grade is designed to sustain the conditions of an environment where the average seven day maximum pavement temperature of 58°C and a minimum pavement design temperature is -22°C.



Permanent Deformation (Rutting)

As discussed earlier in the section describing the DSR, the total response of asphalt binders to load consists of two components: elastic (recoverable) and viscous (non-recoverable). Pavement rutting or permanent deformation is the accumulation of the non-recoverable component of the responses to load repetitions at high service temperatures. If permanent deformation occurs, it generally does so early on in the life of a pavement, so Superpave addresses rutting using unaged binder and binder aged in the RTFO.

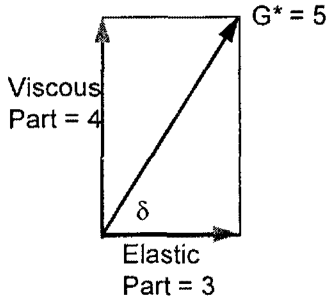
The Superpave specification defines and places requirements on a rutting factor, G*/sin δ, , that represents the high temperature viscous component of overall binder stiffness. This factor is called "G star over sine delta," or the high temperature stiffness. It is determined by dividing the complex modulus (G*) by the sine of the phase angle (δ), both measured by the DSR. G*/sin δ must be at least 1.00 kPa for the original asphalt binder and a minimum of 2.20 kPa after aging in the rolling thin film oven test. Binders with values below these may be too soft to resist permanent deformation.

Viscosity, ASTM D 4402:[b] Maximum, 3 Pa-s (3000 cP), Test Temp, C
Dynamic Shear,TP5:[c] **G*/sin δ, Minimum, 1.00 kPa** Test Temperature @ 10 rad/s, C
Rolling Thin Film Oven (T240)
Mass Loss, Maximum, %
Dynamic Shear, TP5: **G*/sin δ, Minimum, 2.20 kPa** Test Temp @ 10 rad/sec, C

Spec Requirements to Address Rutting

Higher values of G* and lower values of δ are considered desirable attributes from the standpoint of rutting resistance. For the two materials A and B shown there is a significant difference between the values for sin δ. Sin δ for Material A (4/5) is larger than sin δ for Material B (3/5). This means that when divided into G* (equal for both A and B), the value for G*/sin δ will be smaller for Material A (6.25) than Material B (8.33). Therefore, Material B should provide better rutting performance than Material A. This is sensible because Material B has a much smaller viscous part than Material A.

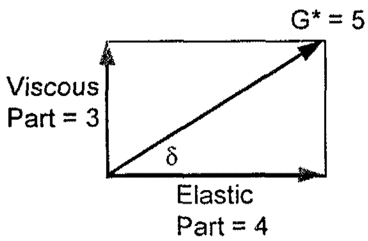
Material A



$$\sin\delta = \frac{\text{Viscous Part}}{G^*} = \frac{4}{5}$$

$$\frac{G^*}{\sin\delta} = \frac{5}{4/5} = 6.25$$

Material B



$$\sin\delta = \frac{\text{Viscous Part}}{G^*} = \frac{3}{5}$$

$$\frac{G^*}{\sin\delta} = \frac{5}{3/5} = 8.33$$

Larger value means behaves more like elastic solid

Fatigue Cracking

G* and δ are also used in the Superpave asphalt specification to help control fatigue in asphalt pavements. Since fatigue generally occurs at low to moderate pavement temperatures after the pavement has been in service for a period of time, the specification addresses these properties using binder aged in both the RTFO and PAV.

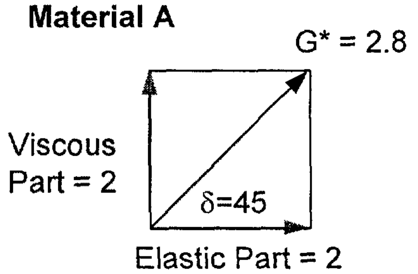
The DSR is again used to generate G* and sin δ. However, instead of dividing the two parameters, the two are multiplied to produce a factor related to fatigue. The fatigue cracking factor is G*sin δ, which is called "G star sine delta," or the intermediate temperature stiffness. It is the product of the complex modulus, G*, and the sine of the phase angle, δ. The Superpave binder specification places a maximum value of 5000 kPa on G* sin δ.

PAV Aging Temp, C
Dynamic Shear, TP5:
G*sin δ, Maximum, 5000 kPa
Test Temp @ 10 rad/sec, C
Physical Hardening [e]
Creep Stiffness, TP1:[f]
S, Maximum, 300 MPa
m-value, Minimum, 0.300
Test Temp, @60 sec, C
Direct Tension, TP3:[f]
Failure Strain, Minimum, 1.0%
Test Temp @ 1.0 mm/min, C

Specification requirement to address fatigue cracking

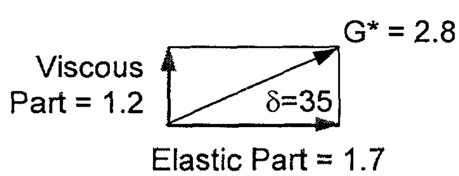
The ability to function as a soft elastic material and recover from many loadings is a desirable binder trait in resisting fatigue cracking. As shown below, for two materials with the same stiffness, the material with a smaller value of δ would be more elastic, and that would improve its fatigue properties. It is possible that a combination of G* and δ could result in a value for G*sin δ so large that the viscous and elastic parts would become too high and the binder would no longer be able to effectively resist fatigue cracking. This is why the specification places a maximum limit of 5000 kPa for G*sin δ.

Material A



$$\text{Viscous Part} = G^* \sin\delta = 2.8 \sin 45 = 2.8 \times 0.71 = 2.0$$

Material B



$$\text{Viscous Part} = G^* \sin\delta = 2.8 \sin 35 = 2.8 \times 0.57 = 1.6$$

Smaller value means softer elastic material

Low Temperature Cracking

When the pavement temperature decreases HMA shrinks. Since friction against the lower pavement layers prevents movement, tensile stresses build-up in the pavement. When these stresses exceed the tensile strength of the asphalt mix, a low temperature crack occurs -- a difficult distress to alleviate. The bending beam rheometer is used to apply a small creep load to the beam specimen and measure the creep stiffness -- the binder's resistance to load. If creep stiffness is too high, the asphalt will behave in a brittle manner, and cracking is more likely to occur. To prevent this cracking, creep stiffness has a maximum limit of 300 MPa.

PAV Aging Temp, C
Dynamic Shear, TP5:
G*sin δ, Maximum, 5000 kPa
Test Temp @ 10 rad/sec, C
Physical Hardening[e]
Creep Stiffness,TP1:[f]
S, Maximum, 300 MPa
m-value, Minimum, 0.300
Test Temp, @60 sec, C
Direct Tension, TP3:[f]
Failure Strain, Minimum, 1.0%
Test Temp @ 1.0 mm/min, C

Specification requirements to address low temperature cracking

The rate at which the binder stiffness changes with time at low temperatures is controlled using the m-value. A high m-value is desirable because as the temperature decreases and thermal stresses accumulate, the stiffness will change relatively fast. A relatively fast change in stiffness means that the binder will tend to shed stresses that would otherwise build up to a level where low temperature cracking would occur. A minimum m-value of 0.300 is required by the Superpave binder specification.

As the temperature of a pavement decreases, it shrinks. This shrinkage causes stresses to build in the pavement. When these stresses exceed the strength of the binder, a crack occurs. Studies have shown that if the binder can stretch to more than 1% of its original length during this shrinkage, cracks are less likely to occur. Therefore, the direct tension test is included in the Superpave specification. It is only applied to binders that have a creep stiffness between 300 and 600 MPa. If the creep stiffness is below 300 MPa, the direct tension test need not be performed, and the direct tension requirement does not apply. The test pulls an asphalt sample in tension at a very slow rate, that which simulates the condition in the pavement as shrinkage occurs. The amount of strain that occurs before the sample breaks is recorded and compared to the 1.0 percent minimum value allowed in the specification.

Miscellaneous Specification Criteria

Other binder requirements are contained in the specification. They are included to control handling and safety characteristics of asphalt binders.

The flash point test (AASHTO T 48) is used to address safety concerns. The minimum value for all grades is 230°C. This test is performed on unaged binders.

To ensure that binders can be pumped and handled at the hot mixing facility, the specification contains a maximum viscosity requirement on unaged binder. This value is 3 Pa·s (3000 cP on rotational viscometer) for all grades. Purchasing agencies may waive this requirement if the binder supplier warrants that the binder can be pumped and mixed at safe temperatures.

A mass loss requirement is specified to guard against a binder that would age excessively from volatilization during hot mixing and construction. The mass loss is calculated using the RTFO procedure and must not exceed 1.00 percent.

During storage or other stationary periods, particularly at low temperatures, physical hardening occurs in asphalt binders. Chemical association of asphalt molecules causes physical hardening. Because of this physical hardening phenomenon, the Superpave specification requires that physical hardening be quantified. To measure this hardening, the bending beam test is performed on pressure aged binder after it has been conditioned for 24 hours at the required test temperature. Therefore, two sets of beams are fabricated for creep stiffness and m-value measurements. One set is tested after one hour of conditioning, while the other set is tested after 24 hours of conditioning. The creep stiffness and m-value are reported for information purposes. Currently, no specified values must be achieved.

SELECTING ASPHALT BINDERS

Performance graded asphalt binders are selected based on the climate in which the pavement will serve. The distinction among the various binder grades is the specified minimum and maximum pavement temperatures at which the requirements must be met.

Appendix A provides a listing of the more common binder grades in the Superpave specification. However, the PG grades are not limited to those given classifications. In actuality, the specification temperatures are unlimited, extending unbounded in both directions. The high and low temperatures extend as far as necessary in the standard six-degree increments. For example, even though a PG 58-10 is not shown, it exists as a legitimate grade in the system.

A module in the Superpave software assists users in selecting binder grades. Superpave contains three methods by which the user can select an asphalt binder grade:

- By Geographic Area: An Agency would develop a map showing binder grade to be used by the designer based on weather and/or policy decisions.
- By Pavement Temperature: The designer would need to know design pavement temperature.
- By Air Temperature: The designer determines design air temperatures, which are converted to design pavement temperatures.

The Superpave software contains a database of weather information for 6092 reporting weather stations in the US and Canada that allows users to select binder grades for the climate at the project location. For each year that these weather stations have been in operation, the hottest seven-day period was determined and the average maximum air temperature for this seven-day period was calculated. SHRP researchers selected this seven-day average value as the optimum method to characterize the high temperature design condition. For all the years recorded, the mean and standard deviation of the **seven-day average maximum air temperature** have been computed. Similarly, the **one-day minimum air temperature** of each year was identified and the mean and standard deviation of all the years of record was calculated. Weather stations with less than 20 years of records were not used.

However, the design temperatures to be used for selecting asphalt binder grade are the pavement temperatures, not the air temperatures. Superpave defines the high pavement design temperature at a depth 20 mm below the pavement surface, and the low pavement design temperature at the pavement surface.

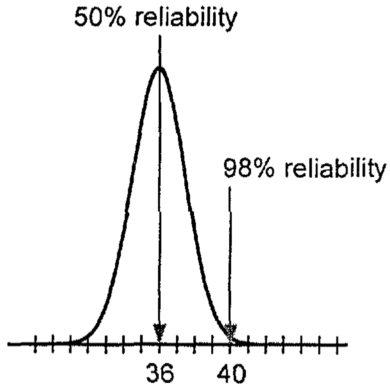
Using theoretical analyses of actual conditions performed with models for net heat flow and energy balance, and assuming typical values for solar absorption (0.90), radiation transmission through air (0.81), atmospheric radiation (0.70), and wind speed (4.5 m/sec), this equation was developed for the:

$$T_{20mm} = (T_{air} - 0.00618\, Lat^2 + 0.2289\, Lat + 42.2) (0.9545) - 17.78$$

where T_{20mm} = high pavement design temperature at a depth of 20 mm
T_{air} = seven-day average high air temperature
Lat = the geographical latitude of the project in degrees.

The low pavement design temperature at the pavement surface is defined as the low air temperature.

The Superpave system allows the designers to use reliability measurements to assign a degree of design risk to the high and low pavement temperatures used in selecting the binder grade. As defined in Superpave, reliability is the percent probability in a single year that the actual temperature (one-day low or seven-day average high) will not exceed the design temperatures.

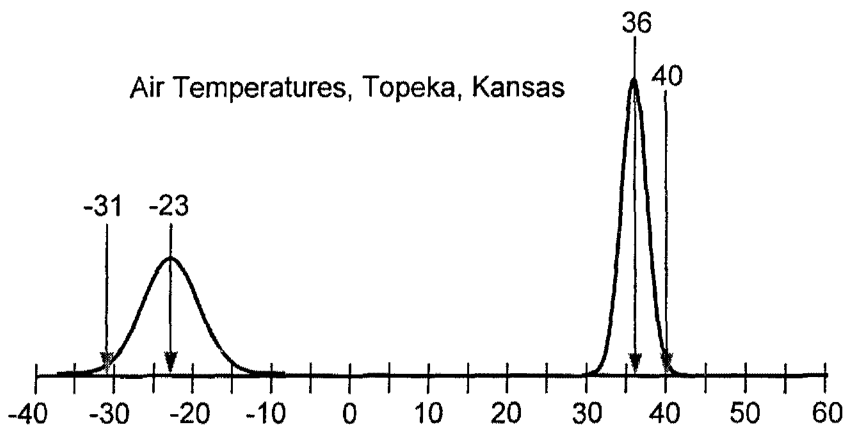


7-Day Maximum Air Temperature
Topeka, Kansas

Superpave binder selection is very flexible in that a different level of reliability can be assigned to high and low temperature grades. Consider summer air temperatures in Topeka, Kansas, which has a mean seven-day maximum of 36°C and a standard deviation of 2°C. In an average year there is a 50 percent chance the seven-day maximum air temperature will exceed 36°C. However, only a two percent chance exists that the temperature will exceed 40°C; hence, a design air temperature of 40°C will provide 98 percent reliability.

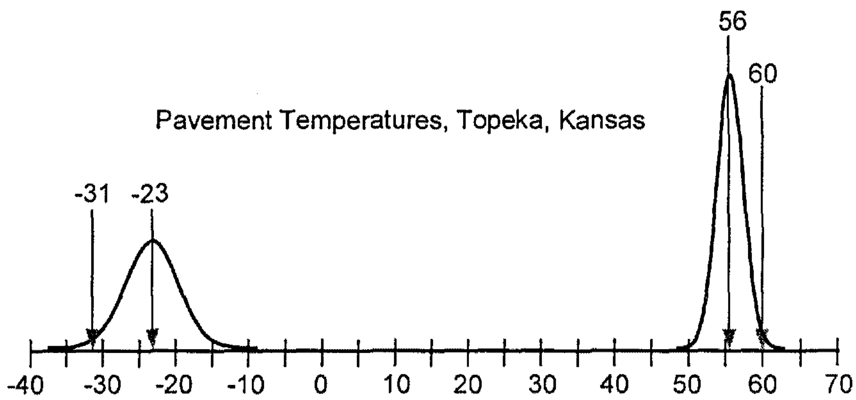
Start with Air Temperature

To see how the binder selection works assume that an asphalt mixture is designed for Topeka. In a normal summer, the average seven-day maximum air temperature is 36°C with a standard deviation of 2°C. In a normal winter, the average coldest temperature is -23°C. For a very cold winter the temperature is -31°C, with a standard deviation of 4°C.

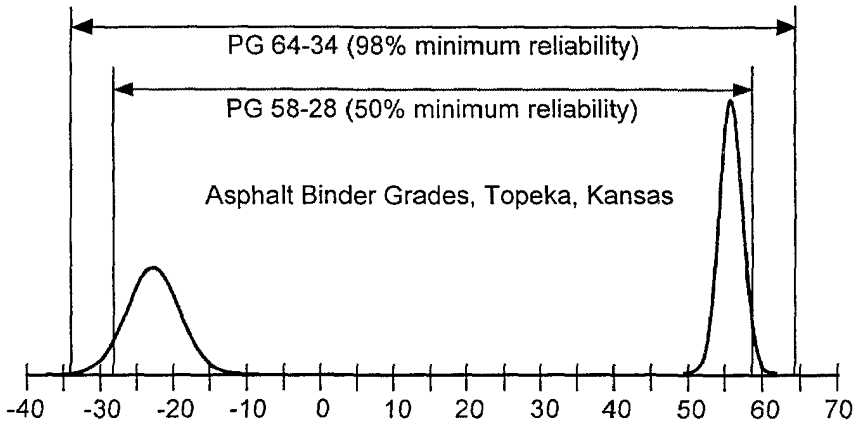


Convert to Pavement Temperature

Superpave software calculates high pavement temperature 20 mm below the pavement surface and low temperature at the pavement surface. For a wearing course at the top of a pavement section, the pavement temperatures in Topeka are 56°C and -23°C for 50 percent reliability and 60°C (56°C + 2 standard deviations) and -31°C for 98 percent reliability.



Select the Binder Grade



For a reliability of at least 50 percent, the high temperature grade for Topeka must be PG 58. Selecting a PG 58 would actually result in a higher level of reliability, about 85 percent, because of the "rounding up" to the next standard grade. The next lower grade only protects to 52°C, less than 50 percent reliability. The low temperature grade must be a PG -28. Likewise, rounding to this standard low temperature grade results in almost 90 percent reliability. For 98 percent reliability, the needed high temperature grade is PG 64; the low temperature grade is PG -34. Also, the reliabilities of the high and low temperature grades could be selected at different levels depending upon the needs of the design project. For instance, if low temperature cracking was more of a concern, the binder could be selected as a PG 58-34.

Manipulating temperature frequency distributions is not a task that the designer needs to worry about. Superpave software handles the calculations. For any site, the user can enter a minimum reliability and Superpave will calculate the required asphalt binder grade. Alternately the user can specify a desired asphalt binder grade and Superpave will calculate the reliability obtained.

Effect of Traffic Speed and Volume on Binder Selection

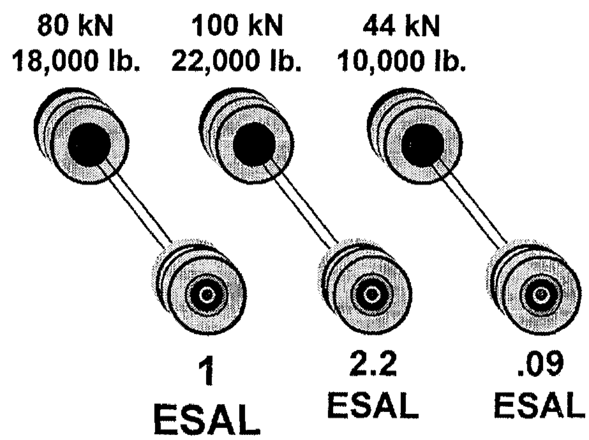
The Superpave binder selection procedure described is the basic procedure for typical highway traffic conditions. Under these conditions, it is assumed that the pavement is subjected to a design number of fast, transient loads. For the high temperature design situation, controlled by specified properties relating to permanent deformation, the traffic speed has an additional effect on performance. The AASHTO MP1 specification includes an additional shift in the selected high temperature binder grade for slow and standing traffic situations. Similar to the time-temperature shift described with the test temperature for the BBR (testing at 10°C higher temperature reduced the test duration from 2 hours to 60 seconds), higher maximum temperature grades are used to offset the effect of the slower traffic speed. For slow moving traffic loads, the binder should be selected one high temperature grade to the right (one grade "warmer"), such as a PG 64 instead of a PG 58. For standing traffic loads, the binder should be selected two high temperature grades to the right or two grades "warmer", such as a PG 70 instead of a PG 58.

Also, a shift is included for extraordinarily high numbers of heavy traffic loads. These are locations where the design lane traffic is expected to exceed 10,000,000 equivalent single axle loads (ESAL). If the design traffic is expected to be between 10,000,000 and 30,000,000 ESAL, then the engineer may consider selecting one high temperature binder grade higher than the selection based on climate. If the design traffic is expected to be greater than 30,000,000 ESAL, then the binder should be selected one high temperature grade higher.

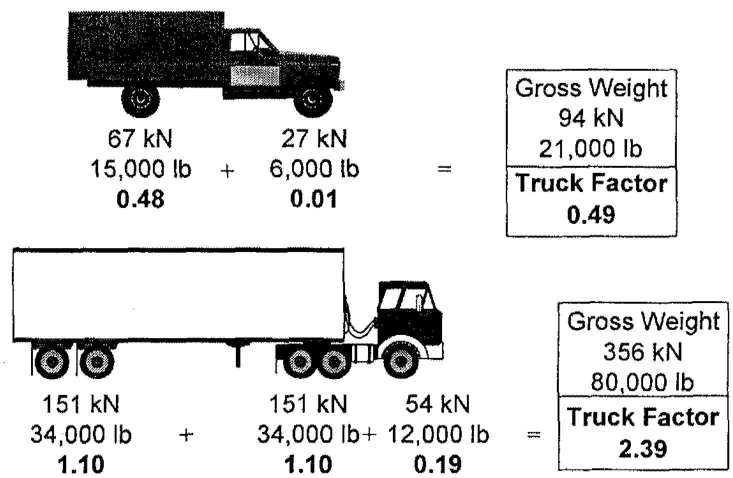
Traffic Analysis

Superpave material selection criteria are based on the traffic volume of the design project, expressed in equivalent single axle loads (ESAL). This brief synopsis describes the calculation of ESALs. For further information, see the Asphalt Institute's *Thickness Design -- Asphalt Pavements for Highways and Streets,* Manual Series No. 1.

An ESAL is defined as one 18,000-pound (80-kN) four-tired dual axle and is the unit used by most pavement thickness design procedures to quantify the various types of axle loadings into a single design traffic number. If an axle contains more or less weight, it is related to the ESAL using a *load equivalency factor.* The relationship between axle load and ESAL is not a one to one equivalency, but a fourth power relationship. If you double an 18,000 lb load, the ESAL is not 2, but almost the fourth power of two, (2^4) or about 14. As well, if axles are grouped together, such as in tandem or tridem axle arrangements, the total weight carried by the axle configuration determines its load equivalency factor.



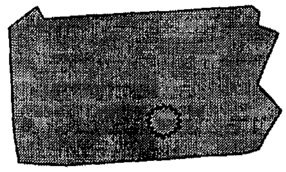
For a given vehicle the load equivalency factors are totaled to provide the *truck factor* for that vehicle. Truck factors can be calculated for any type of trucks or combination of truck types. Traffic count and classification data is then used in combination of the truck factor for each vehicle classification to determine the design traffic in ESAL.



The Superpave binder specification and tests are intended for both unmodified and modified binders. However, there are certain occasions, such as RTFO aging, binder selection and budgeting, when it would be helpful to know if the binder is modified. The difference between the high and low temperature grades can provide some indication whether the binder may be modified. A very general rule of thumb in the industry says if the difference is greater than 90, the binder may be modified, and the likelihood and quantity of modification increases as the difference increases. For instance, the difference between the high and low temperature grades of a PG 64-34 is 98. This grade will probably include a modifier in the binder. However, many factors affect the value (90) of this "rule", such as the viscosity of the binder and the crude oil source.

Class Example Binder Selection

- **Southcentral Pennsylvania**
  - **rural 4-lane access road**
  - **11 million ESAL**
  - **many traffic lights**



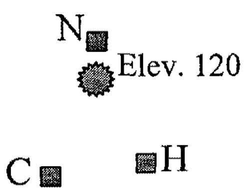
- **Budget for project : $29/ton mix**
- **Governor's mother's street needs repairs**
  - **milling & leveling equivalent of $6/ton**
  - **patching costs equivalent of $2/ton**

Which PG would you select ?

PG 52-16, 52-22, 58-22 : $23/ton
PG 64-22, 58-28 : $27/ton
PG 70-22, 64-28 : $29/ton
PG 70-28 : $43/ton
PG 76-28 : $52/ton

Class Example Binder Selection
Southcentral Pennsylvania Project

							Air Temp			
							Low Temp		High Temp	
ST	County	Dist	Station	Long	Lat	Elev	Avg	Std	Avg	Std
PA	Perry	8	Newport	77.13	40.48	116	-20	4	33	2
PA	York	30	Harrisburg	76.85	40.22	104	-17	3	33	2
PA	Cumb.	31	Carlisle	77.22	40.20	143	-19	4	34	2



50% Reliability						
TEMPERATURES				Binder Grade		
MaxAir	MaxPvt	MinAir	MinPvt	PG	HT	LT
33	53	-20	-20			
33	53	-17	-17	PG	?-?	
34	54	-19	-19			

98% Reliability						
TEMPERATURES				Binder Grade		
MaxAir	MaxPvt	MinAir	MinPvt	PG	HT	LT
35	57	-28	-28			
37	57	-23	-23	PG	?-?	
38	58	-27	-27			

IV. Superpave Aggregates

Superpave refined existing methods for testing and specifying aggregates for HMA. The objectives of this section will be to:

- describe the Superpave aggregate test procedures
- explain the Superpave aggregate specification requirements and how they are used in preventing permanent deformation and fatigue cracking
- discuss the Superpave aggregate gradation evaluation procedure

AGGREGATE TESTS AND SPECIFICATIONS

Consensus Properties

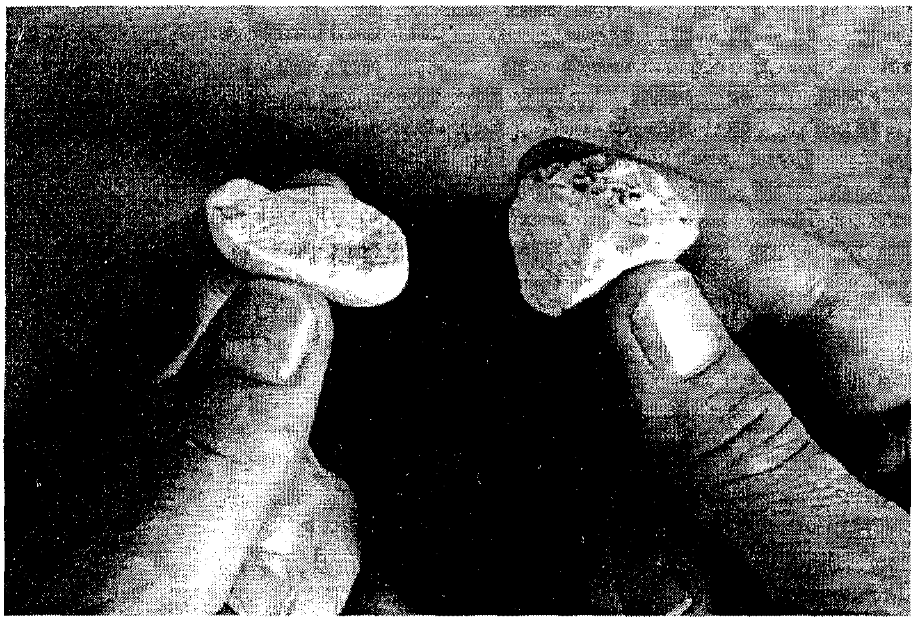
It was the consensus of the SHRP pavement researchers that certain aggregate characteristics were critical and needed to be achieved in all cases to arrive at well performing HMA. These characteristics were called "consensus properties" because there was wide agreement in their use and specified values. Those properties are:

- coarse aggregate angularity,
- fine aggregate angularity,
- flat, elongated particles, and
- clay content.

COARSE AGGREGATE ANGULARITY

This property ensures a high degree of aggregate internal friction and rutting resistance. It is defined as the percent by weight of aggregates larger than 4.75 mm with one or more fractured faces.

The test procedure for measuring coarse aggregate angularity is ASTM D 5821, *Standard Test Method for Determining the Percentage of Fractures Particles in Coarse Aggregate.* The procedure involves manually counting particles to determine fractured faces. A fractured face is defined as any fractured surface that occupies more than 25 percent of the area of the outline of the aggregate particle visible in that orientation.



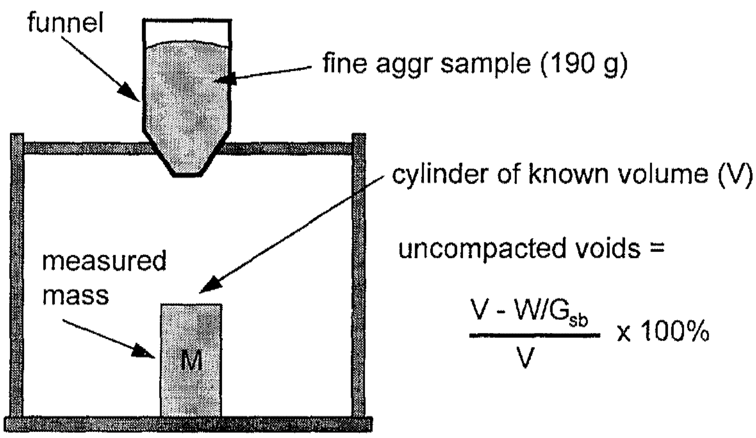
The required minimum values for coarse aggregate angularity are a function of traffic level and position within the pavement. These requirements apply to the final aggregate blend, although estimates can be made on the individual aggregate stockpiles.

Superpave Coarse Aggregate Angularity Requirements		
Traffic, million ESALs	Percent, Minimum	
	Depth from Surface	
	< 100 mm	> 100 mm
< 0.3	55/-	-/-
< 1	65/-	-/-
< 3	75/-	50/-
< 10	85/80	60/-
< 30	95/90	80/75
< 100	100/100	95/90
≥ 100	100/100	100/100
Note: "85/80" means that 85 % of the coarse aggregate has one fractured face and 80 % has two fractured faces.		

FINE AGGREGATE ANGULARITY

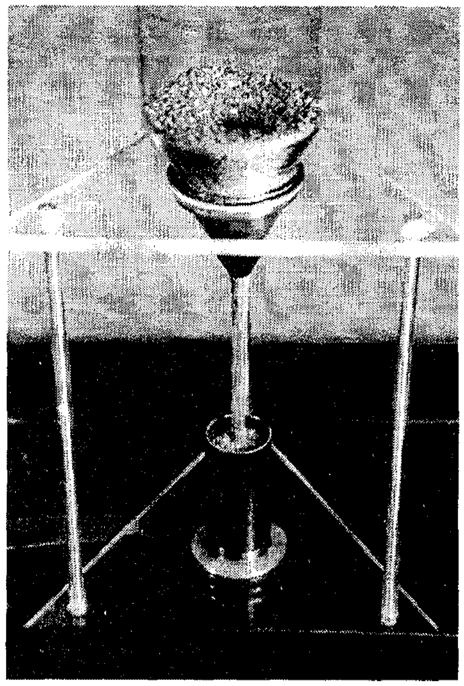
This property ensures a high degree of fine aggregate internal friction and rutting resistance. It is defined as the percent air voids present in loosely compacted aggregates smaller than 2.36 mm. Higher void contents mean more fractured faces.

The test procedure used to measure this property is AASHTO T 304 "*Uncompacted Void Content - Method A.*" In the test, a sample of fine aggregate is poured into a small calibrated cylinder by flowing through a standard funnel. By determining the weight of fine aggregate (W) in the filled cylinder of known volume (V), void content can be calculated as the difference between the cylinder volume and fine aggregate volume collected in the cylinder. The fine aggregate bulk specific gravity (G_{sb}) is used to compute fine aggregate volume.



uncompacted voids =

$$\frac{V - W/G_{sb}}{V} \times 100\%$$



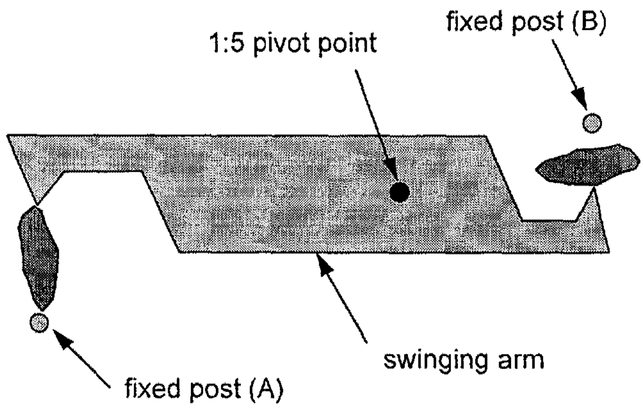
The required minimum values for fine aggregate angularity are a function of traffic level and position within pavement. These requirements apply to the final aggregate blend, although estimates can be made on the individual aggregate stockpiles.

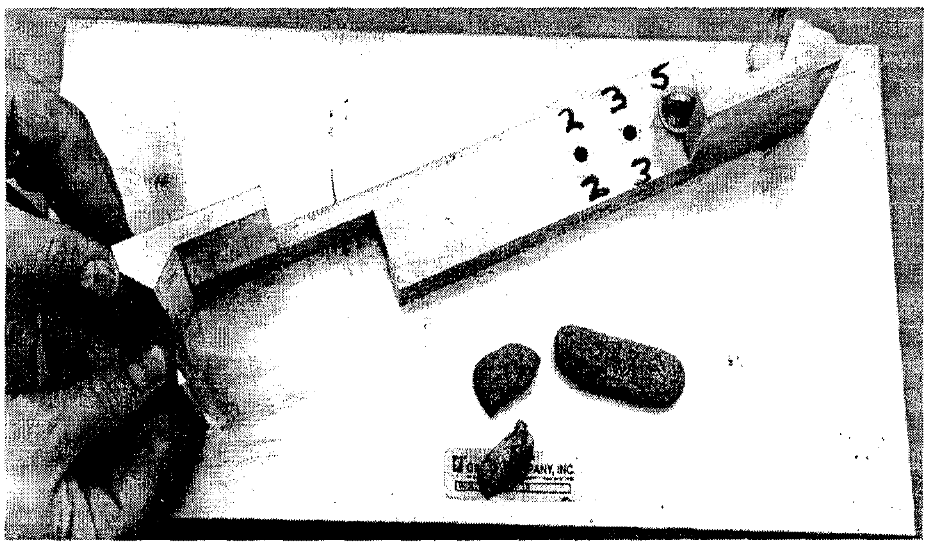
Superpave Fine Aggregate Angularity Requirements		
Traffic, million ESALs	Percent, Minimum	
	Depth from Surface	
	< 100 mm	> 100 mm
< 0.3	-	-
< 1	40	-
< 3	40	40
< 10	45	40
< 30	45	40
< 100	45	45
≥ 100	45	45
Note: Criteria are presented as percent air voids in loosely compacted fine aggregate.		

FLAT, ELONGATED PARTICLES

This characteristic is the percentage by weight of coarse aggregates that have a maximum to minimum dimension of greater than five. Elongated particles are undesirable because they have a tendency to break during construction and under traffic.

The test procedure used is ASTM D 4791, *Standard Test for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate* and it is performed on coarse aggregate larger than 4.75 mm. The procedure uses a proportional caliper device to measure the dimensional ratio of a representative sample of aggregate particles. 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 fixed post at position B. If the aggregate passes through this gap, then it is counted as a flat or elongated particle. The total flat, elongated, or flat and elongated particles are measured.





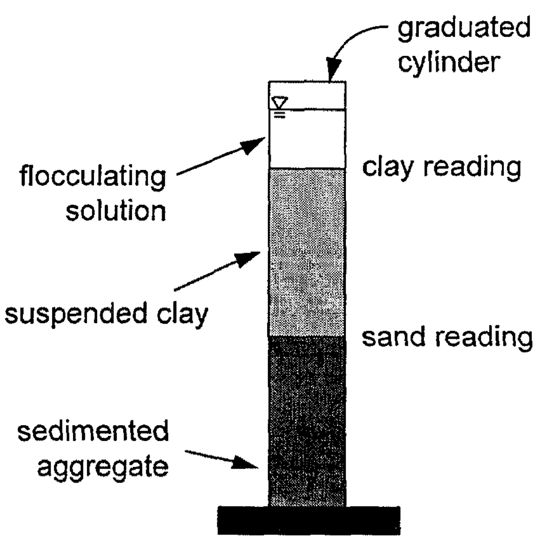
The required maximum values for flat, elongated particles in coarse aggregate are a function of traffic level. These requirements apply to the final aggregate blend, although estimates can be made on the individual aggregate stockpiles.

Superpave Flat, Elongated Particle Requirements	
Traffic, million ESALs	Percent, maximum
< 0.3	-
< 1	-
< 3	10
< 10	10
< 30	10
< 100	10
≥ 100	10
Note: Criteria are presented as maximum percent by weight of flat and elongated particles.	

CLAY CONTENT

Clay content is the percentage of clay material contained in the aggregate fraction that is finer than a 4.75 mm sieve. It is measured by AASHTO T 176, *Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test.*

In this test, a sample of fine aggregate is placed in a graduated cylinder with a flocculating solution and agitated to loosen clayey fines present in and coating the aggregate. The flocculating solution forces the clayey material into suspension above the granular aggregate. After a period that allows sedimentation, the cylinder height of suspended clay and sedimented sand is measured. The sand equivalent value is computed as a ratio of the sand to clay height readings expressed as a percentage.



The required clay content values for fine aggregate are expressed as a minimum sand equivalent and are a function of traffic level. These requirements apply to the final aggregate blend, although estimates can be made on the individual aggregate stockpiles

Superpave Clay Content Requirements	
Traffic, million ESALs	Sand Equivalent, minimum
< 0.3	40
< 1	40
< 3	40
< 10	45
< 30	45
< 100	50
≥ 100	50

Source Properties

In addition to the consensus aggregate properties, pavement experts believed that certain other aggregate characteristics were critical. However, critical values of these properties could not be reached by consensus because needed values were source specific. Consequently, a set of "source properties" were recommended. Specified values are established by local agencies. While these properties are relevant during the mix design process, they may also be used as source acceptance control. Those properties are:

- toughness,
- soundness, and
- deleterious materials.

TOUGHNESS

Toughness is the percent loss of materials from an aggregate blend during the Los Angeles Abrasion test. The procedure is stated in AASHTO T 96, "Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine." This test estimates the resistance of coarse aggregate to abrasion and mechanical degradation during handling, construction, and in-service. It is performed by subjecting the coarse aggregate, usually larger than 2.36 mm, to impact and grinding by steel spheres. The test result is percent loss, which is the weight percentage of coarse material lost during the test as a result of the mechanical degradation. Maximum loss values typically range from approximately 35 to 45 percent.

SOUNDNESS

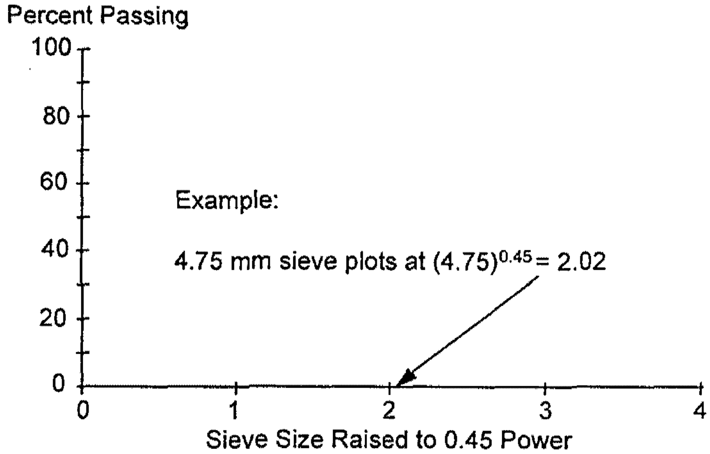
Soundness is the percent loss of materials from an aggregate blend during the sodium or magnesium sulfate soundness test. The procedure is stated in AASHTO T 104, "Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate." This test estimates the resistance of aggregate to weathering while in-service. It can be performed on both coarse and fine aggregate. The test is performed by alternately exposing an aggregate sample to repeated immersions in saturated solutions of sodium or magnesium sulfate each 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 re-hydrates 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 range from approximately 10 to 20 percent for five cycles.

DELETERIOUS MATERIALS

Deleterious materials are defined as the weight percentage of contaminants such as shale, wood, mica, and coal in the blended aggregate. This property is measured by AASHTO T 112, "Clay Lumps and Friable Particles in Aggregates." It can be performed on both coarse and fine aggregate. The test is performed by wet sieving aggregate size fractions over prescribed sieves. The weight 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 maximum permissible percentage of clay lumps and friable particles is evident. Values range from as little as 0.2 percent to as high as 10 percent, depending on the exact composition of the contaminant.

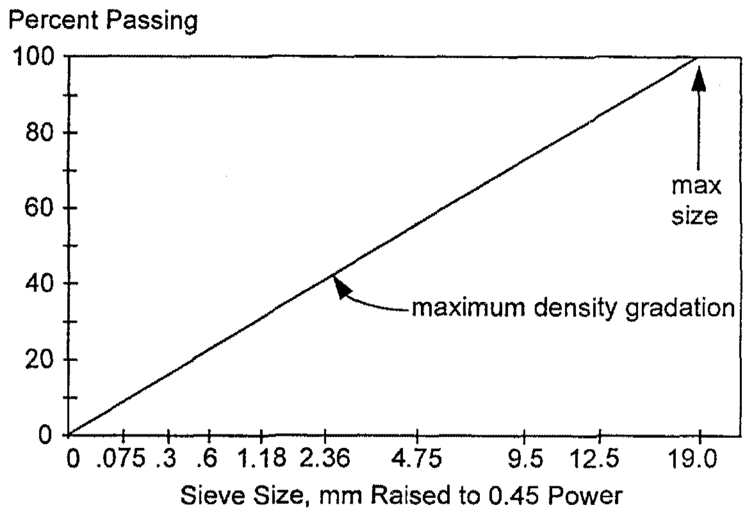
Gradation

Superpave uses the 0.45 power gradation chart to define a permissible gradation. This chart uses a unique graphing technique to judge the cumulative particle size distribution of a blend of aggregate. The ordinate of the chart is percent passing. The abscissa is an arithmetic scale of sieve size in millimeters, raised to the 0.45 power. As an example, the 4.75 mm sieve is plotted at 2.02 units to the right of the origin, This number, 2.02, is the sieve size, 4.75 mm, raised to 0.45 power. Normal 0.45 power charts do not show arithmetic abscissa labels arithmetically. Instead, the scale is annotated with the actual sieve size.



An important feature of this chart is the maximum density gradation. This gradation plots as a straight line from the maximum aggregate size through the origin. Superpave uses a standard set of ASTM sieves and these definitions with respect to aggregate size (Appendix B shows sieve sizes used by Superpave):

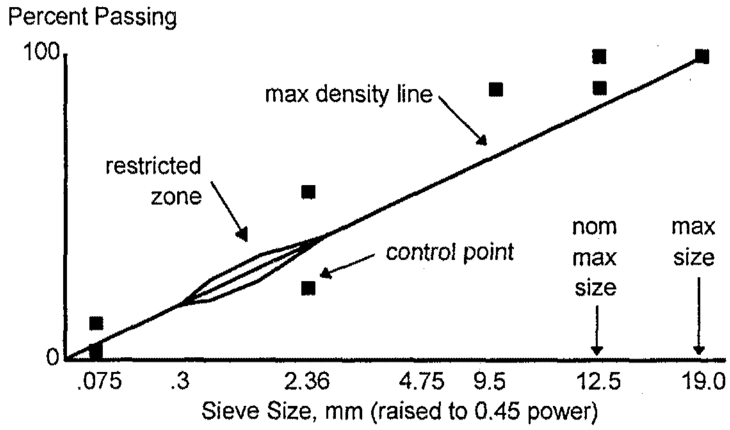
- Maximum Size: One sieve size larger than the nominal maximum size.
- Nominal Maximum Size: One sieve size larger than the first sieve to retain more than 10 percent.



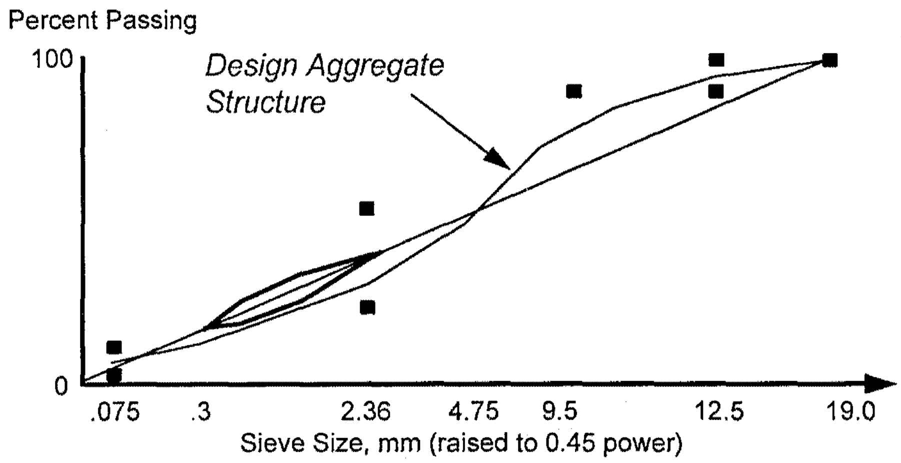
The maximum density gradation represents a gradation in which the aggregate particles fit together in their densest possible arrangement. This is a gradation to avoid because there would be very little aggregate space within which to develop sufficiently thick asphalt films for a durable mixture.

To specify aggregate gradation, two additional features are added to the 0.45 power chart: control points and a restricted zone. Control points function as master ranges through which gradations must pass. They are placed on the nominal maximum size, an intermediate size (2.36 mm), and the dust size (0.075 mm).

The restricted zone resides along the maximum density gradation between the intermediate size (either 4.75 or 2.36 mm, depending on the maximum size) and the 0.3 mm size. It forms a band through which gradations should not pass. Gradations that pass through the restricted zone have often been called "humped gradations" because of the characteristic hump in the grading curve that passes through the restricted zone. In most cases, a humped gradation indicates a mixture that possesses too much fine sand in relation to total sand. This gradation practically always results in tender mix behavior, which is manifested by a mixture that is difficult to compact during construction and offers reduced resistance to permanent deformation during its performance life. Gradations that violate the restricted zone possess weak aggregate skeletons that depend too much on asphalt binder stiffness to achieve mixture shear strength. These mixtures are also very sensitive to asphalt content and can easily become plastic.



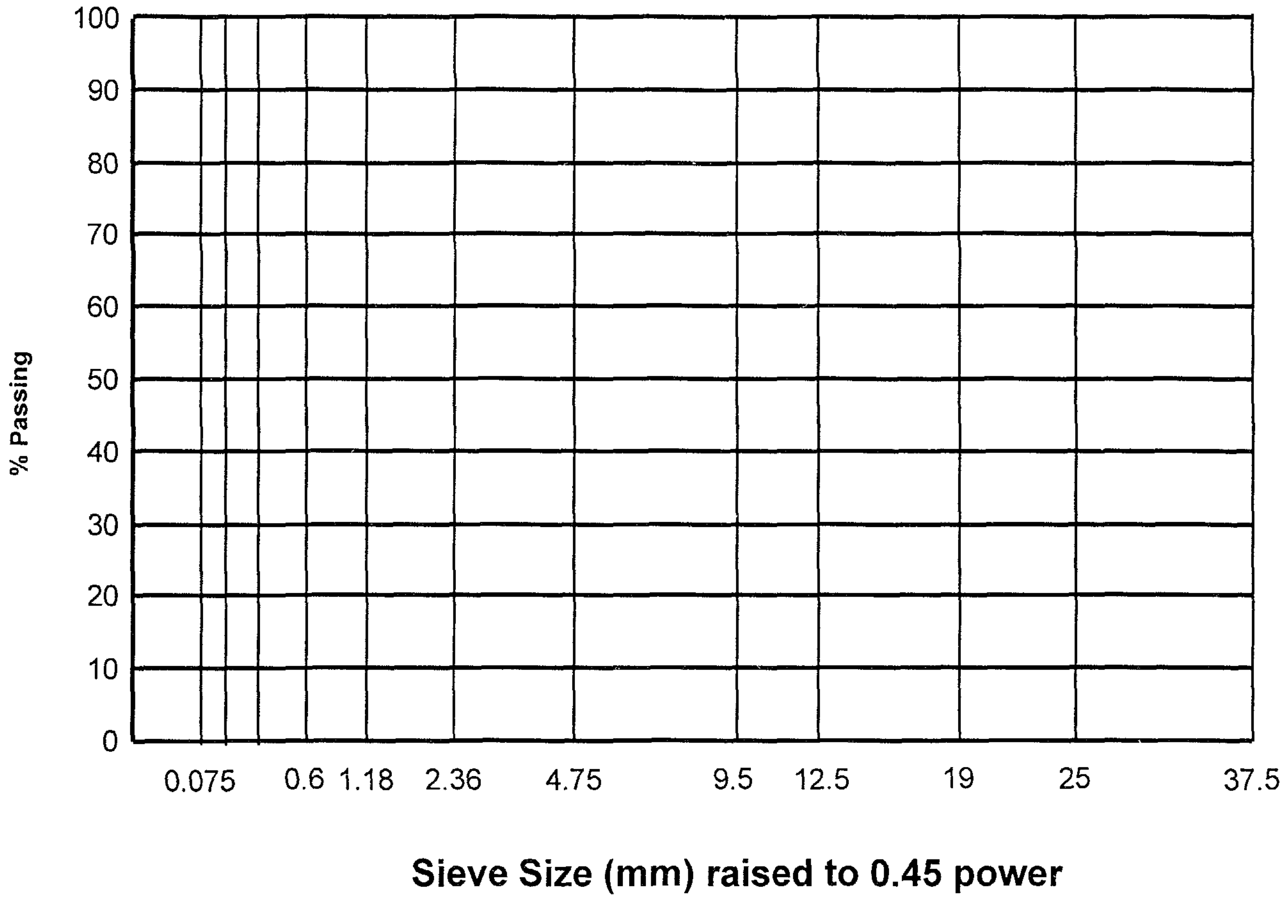
The term used to describe the cumulative distribution of aggregate particle sizes is the *design aggregate structure*. A design aggregate structure that lies between the control points and avoids the restricted zone meets the requirements of Superpave with respect to gradation. Superpave defines five mixture types as defined by their nominal maximum aggregate size. Appendix B shows the gradation limits for the five Superpave mixtures.



Superpave Mixtures		
Superpave Mixture Designation	Nominal Maximum Size, mm	Maximum Size, mm
37.5 mm	37.5	50
25 mm	25	37.5
19 mm	19	25
12.5 mm	12.5	19
9.5 mm	9.5	12.5

Superpave recommends, but does not require, mixtures to be graded below the restricted zone. It also recommends that as project traffic level increases, gradations move closer to the coarse (lower) control points. Furthermore, the Superpave gradation control requirements were not intended to be applied to special purpose mix types such as stone matrix asphalt or open graded mixtures.

Superpave Aggregate Gradation Example



Sieve	**Gradation**
25.0 mm	***100.0***
19.0 mm	***97.6***
12.5 mm	**89.5**
9.5 mm	**77.7**
4.75 mm	**44.3**
2.36 mm	***31.9***
1.18 mm	**22.2**
600 µm	**14.5**
300 µm	**7.9**
150 µm	**4.1**
75 µm	***3.5***

Nominal Maximum Size -

Maximum Size -

Draw Maximum Density Line

Plot Control Points

Plot Restricted Zone

Plot Gradation

Meet criteria?

V. Superpave Mixtures

Superpave asphalt mixture requirements were developed from both previously established criteria, and new criteria that were developed in conjunction with new compaction equipment. The objectives of this section will be to:

- describe the Superpave Gyratory Compactor
- review the Superpave mixture criteria, including mixture compaction requirements and mixture volumetric criteria
- describe the moisture sensitivity test and criteria

ASPHALT MIXTURE TESTS

Superpave Gyratory Compaction

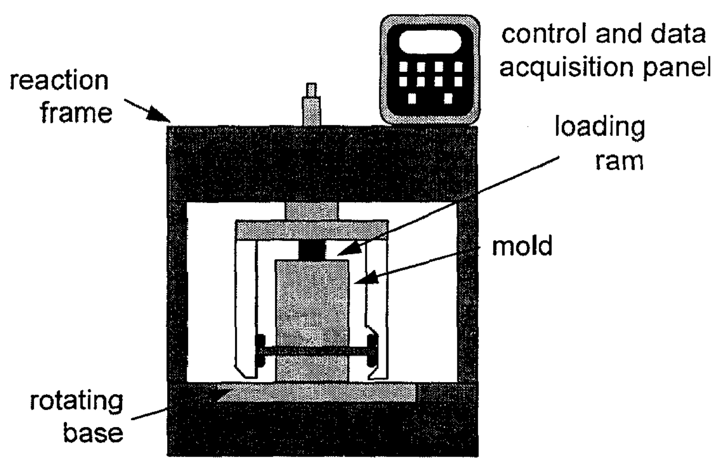
SHRP researchers had several goals in selecting a method of laboratory compaction. Most important, they desired a device that would realistically compact trial mix specimens to densities achieved under actual pavement climate and loading conditions. The device needed to be capable of accommodating large aggregates. Furthermore, it was desired that the device afford a measure of compactability so that potential tender mixture behavior and similar compaction problems could be identified. A high priority for SHRP researchers was a device that was well suited to mixing facility quality control and quality assurance operations. No compactor in current use achieved all these goals. Consequently, a new compactor was developed, the Superpave Gyratory Compactor (SGC).

The basis for the SGC was a large Texas gyratory compactor modified to use the compaction principles of a French gyratory compactor. The Texas device accomplished the goals of achieving realistic specimen densification and it was reasonably portable. Its 6-inch sample diameter (ultimately 150 mm on an SGC) could accommodate mixtures containing aggregate up to 50 mm maximum (37.5 nominal) size. SHRP researchers modified the Texas device by lowering its angle and speed of gyration and adding real time specimen height recordation.

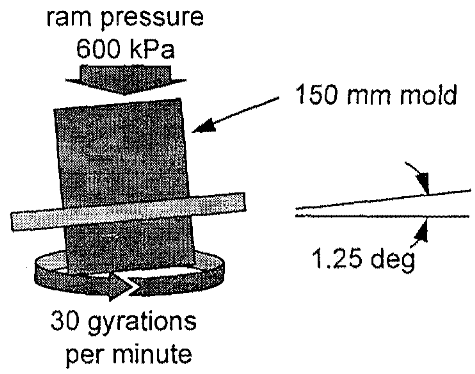
Test Equipment

The SGC is a mechanical device comprised of the following system of components:

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

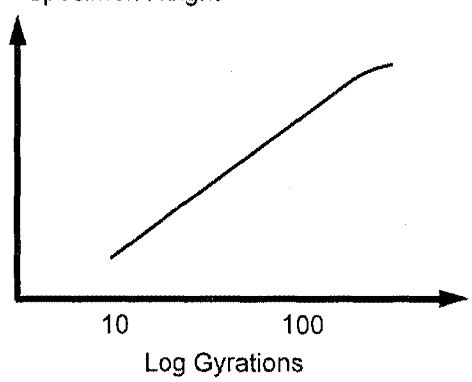


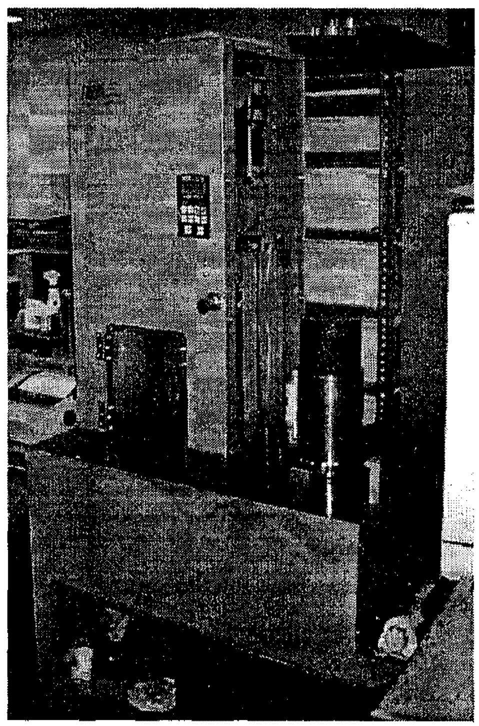
The reaction frame provides a stiff structure against which the loading ram can push when compacting specimens. The base of the SGC rotates and is affixed to the loading frame. It supports the mold while compaction occurs. The SGC uses a mold with an inside diameter of 150 mm and a nominal height of at least 250 mm. A base plate fits in the bottom of the mold to afford specimen confinement during compaction. Reaction bearings are used to position the mold at a compaction angle of 1.25 degrees, which is the compaction angle of the SGC. An electric motor drives the rotating base at a constant speed of 30 revolutions per minute.



A hydraulic or mechanical system applies a load to the loading ram, which imparts 600 kPa compaction pressure to the specimen. The loading ram diameter nominally matches the inside diameter of the mold, which is 150 mm. A pressure gauge measures the ram pressure during compaction. As the specimen densifies during compaction, the pressure gauge and loading ram maintain compaction pressure.

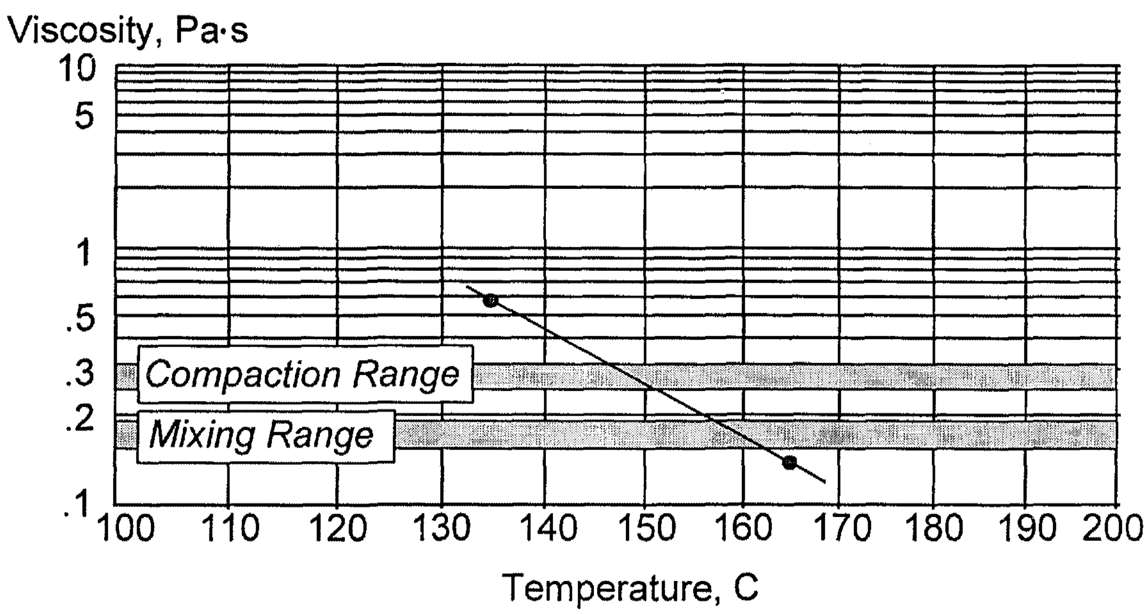
Specimen height measurement is an important function of the SGC. Using the mass of material placed in the mold, the diameter of the mold, and the specimen height, an estimate of specimen density can be made at any time throughout the compaction process. Specimen density is computed by dividing the mass by the volume of the specimen. The specimen volume is calculated as the volume of a smooth-sided cylinder with a diameter of 150 mm and the measured height. Height is recorded by measuring the position of the ram before and during the test. The vertical change in ram position equals the change in specimen height. The specimen height signal is connected to a personal computer, printer, or other device to record height (i.e., density) measurements throughout the compaction process. By this method, a compaction characteristic is developed as the specimen is compacted.





Specimen Preparation

To normalize the effect of the binder, compaction specimens require mixing and compaction under equiviscous temperature conditions corresponding to 0.170 ± 20 Pa·s and 0.280 ± 30 Pa·s, respectively, as determined from the temperature-viscosity characteristics for the asphalt binder. If the temperature-viscosity plot produces a mixing temperature higher than 170°C, it may indicate that the asphalt is modified. Because of their distinctive characteristics, modified asphalts can frequently be mixed and compacted at higher viscosities (lower temperatures) than the shaded ranges shown above. It should be noted that temperatures above 177°C may lead to binder thermal degradation and should not be used. The binder supplier should always be consulted for recommendations of the optimum laboratory and field mixing and compaction temperatures for modified binders. Users may defer to the manufacturer's recommendations for all PG binder grades.



Mixing is accomplished using a mechanical mixer. After mixing, loose test specimens are subjected to four hours of short term aging in a forced draft oven maintained at a constant 135°C. During short term aging, loose mix specimens are required to be spread into a thickness resulting in 21 to 22 kg per square meter and stirred every hour to ensure uniform aging. The compaction molds and base plates should also be placed in an oven at 135°C for at least 30 to 45 minutes prior to use.

Three specimen sizes are used. If specimens are to be used for volumetric determinations only, use sufficient mix to arrive at a specimen 115 mm ± 5 mm height. This requires approximately 4500 grams of mixture. In this case, the test specimen produced is tested without trimming. Alternatively, to produce specimens for performance testing, approximately 5500 grams of mixture is used to fabricate a specimen that is 150 mm in diameter by approximately 135 mm height. In this case, specimens will have to be trimmed to 50 mm before testing in the SST or IDT. At least one loose sample should remain uncompacted to obtain a maximum theoretical specific gravity using AASHTO T 209. For performing moisture sensitivity tests (AASHTO T283), test specimens are fabricated to a height of 95 mm, which requires approximately 3500 grams of mixture.

Overview of Procedure

After short term aging the loose test specimens are ready for compacting. The compactor is initiated by turning on its main power. The vertical pressure should be set at 600 kPa (± 18 kPa). The gyration counter should be zeroed and set to stop when the desired number of gyrations is achieved. Three gyration levels are of interest:

- design number of gyrations (N_{design} or N_{des}).
- initial number of gyrations ($N_{initial}$, or N_{ini}), and
- maximum number of gyrations ($N_{maximum}$ or N_{max}).

Test specimens are compacted using N_{max} gyrations. The relationship between N_{des}, N_{max}, and N_{ini} are:

$$\text{Log}_{10} N_{max} = 1.10 \times \text{Log}_{10} N_{des}$$

$$\text{Log}_{10} N_{ini} = 0.45 \times \text{Log}_{10} N_{des}$$

The design number of gyrations (N_{des}) ranges from 68 to 172 and is a function of the climate in which the mix will be placed and the traffic level. The average design high air temperature is provided by Superpave software and represents the average seven-day maximum air temperature for project conditions. The range of values for N_{des}, N_{max}, and N_{ini} are shown:

Superpave Design Gyratory Compactive Effort

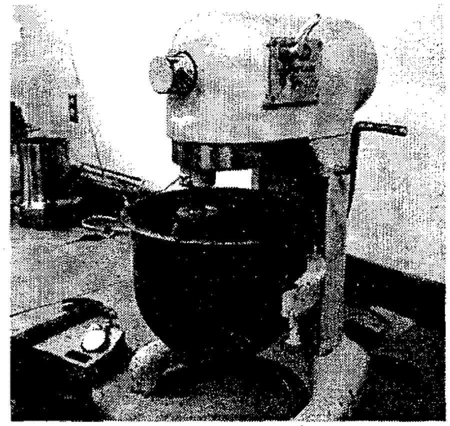
Design ESALs (millions)	Average Design High Air Temperature											
	<39°C			39 - 40°C			41 - 42°C			43 - 44°C		
	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}
< 0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3 - 1	7	76	117	7	83	129	7	88	138	8	93	146
1 - 3	7	86	134	8	95	150	8	100	158	8	105	167
3 - 10	8	96	152	8	106	169	8	113	181	9	119	192
10 - 30	8	109	174	9	121	195	9	128	208	9	135	220
30 - 100	9	126	204	9	139	228	9	146	240	10	153	253
> 100	9	142	233	10	158	262	10	165	275	10	172	288

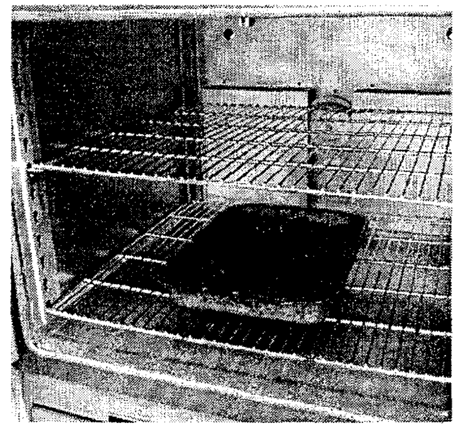
After the base plate is in place, a paper disk is placed on top of the plate and the mold is charged in a single lift. The top of the uncompacted specimen should be slightly rounded. A paper disk is placed on top of the mixture.

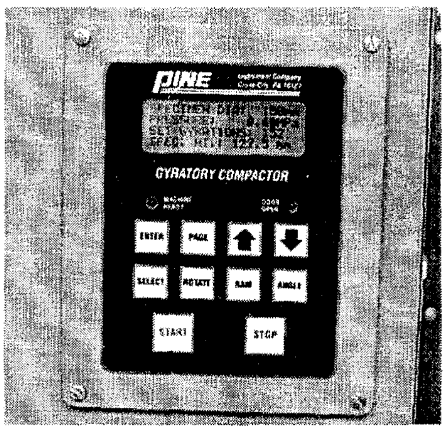
The mold is placed in the compactor and centered under the ram. The ram is then lowered until it contacts the mixture and the resisting pressure is 600 kPa (± 18 kPa). The angle of gyration (1.25° ± 0.02°) is then applied and the compaction process begins.

When N_{max} has been reached, the compactor automatically stops. After the angle and pressure are released, the mold containing the compacted specimen is then removed. After a suitable cooling period, the specimen is extruded from the mold.

The bulk specific gravity of test specimens should be measured using AASHTO T 166. Maximum theoretical specific gravity should be measured using AASHTO T 209.






PINE
GYRATORY COMPACTOR
ENTER
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START
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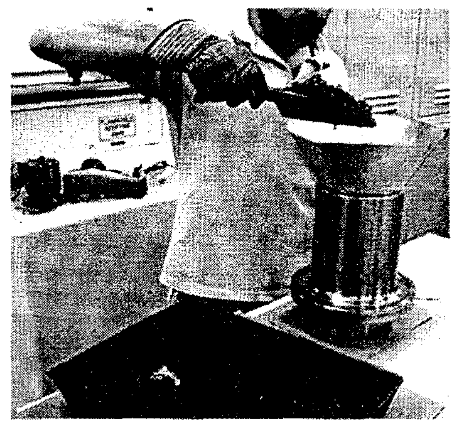
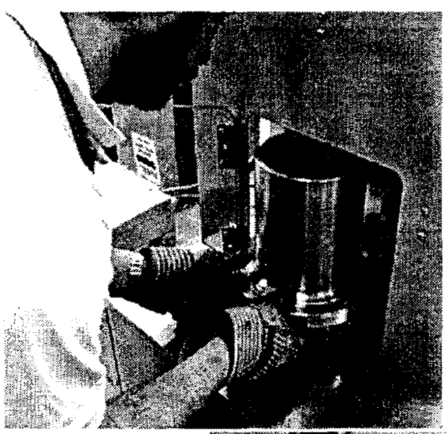
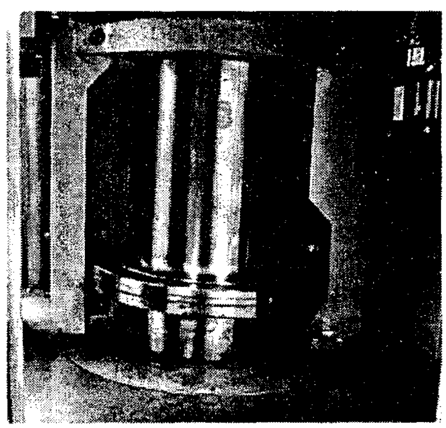
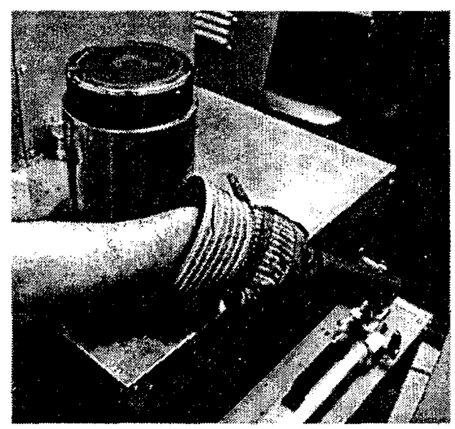
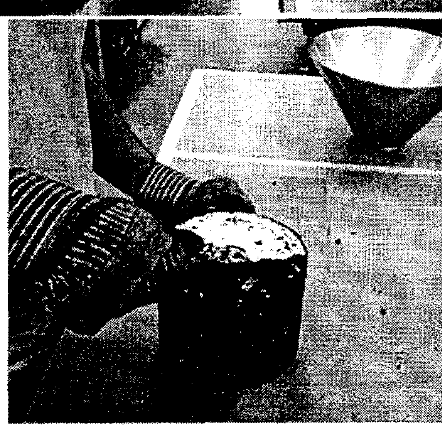
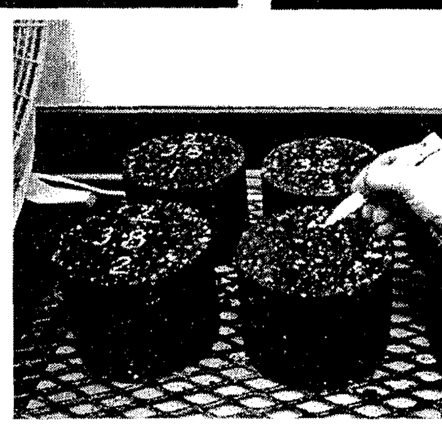








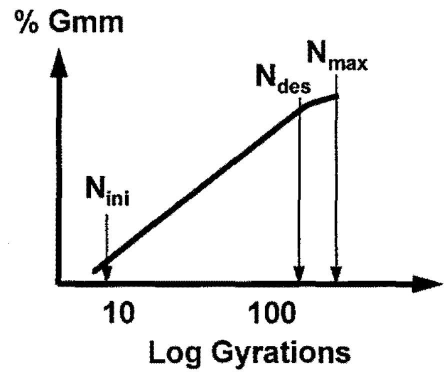




Data Presentation

Superpave gyratory compaction data is analyzed by computing the maximum theoretical specific gravity for each desired gyration. This example specimen compaction information illustrates this analysis:

Specimen No. 1: Total Mass = 4869 g		
No. of Gyrations	Height, mm	% G_{mm}
8 (N_{ini})	127.0	86.5
50	118.0	93.1
100	115.2	95.4
109 (N_{des})	114.9	95.6
150	113.6	96.7
174 (N_{max})	113.1	97.1
G_{mb}	2.489	
G_{mm}	2.563	



Project conditions for this mixture are such that N_{max} = 174, N_{des} = 109, and N_{ini} = 8 gyrations. During compaction, the height is measured after each gyration and recorded for the number of gyrations shown in the first column. After compaction, the specimen is removed from the cylinder and, after cooling, the G_{mb} is measured. G_{mb} is then divided by G_{mm} to determine the % G_{mm} @ N_{max}. The % G_{mm} at any number of gyrations (N_x) is then calculated by multiplying % G_{mm} @ N_{max} by the ratio of the heights at N_{max} and N_x. The calculations for this example are illustrated here:

G_{mb} = 2.489

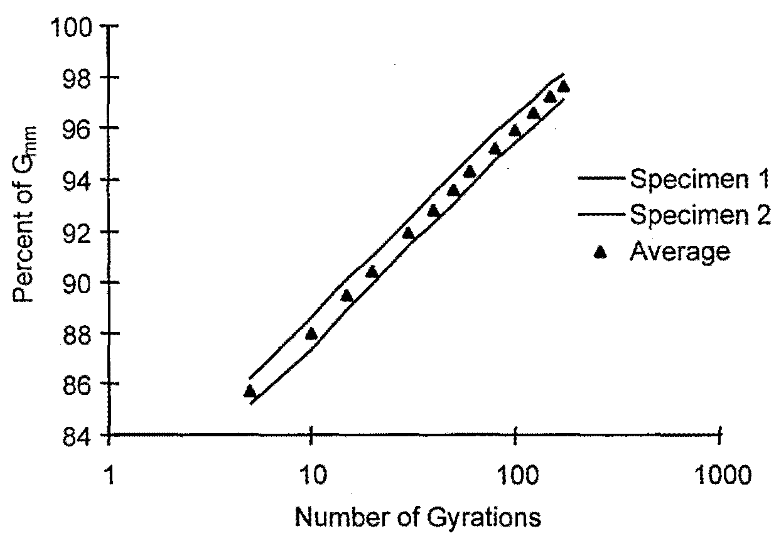
G_{mm} = 2.563

% G_{mm} @ N_{max} = G_{mb} ÷ G_{mm} = 2.489 ÷ 2.563 x 100% = 97.1%

% G_{mm} @ N_x = % G_{mm} @ N_{max} x (H_{max} ÷ H_x)

For N = 50, % G_{mm} @ N_{50} = % G_{mm} @ N_{max} x (H_{max} ÷ H_{50}) = 97.1 x (113.1 ÷ 118.0) = 93.1 %

If this example had been for a mix design, a companion specimen would have been compacted and average percent G_{mm} values resulting from the two specimens would have been used for further analysis. A compaction characteristic curve for this example showing two specimens and an average is shown:



ASPHALT MIXTURE REQUIREMENTS

Asphalt mixture design requirements in Superpave consist of:

- mixture volumetric requirements,
- dust proportion, and
- moisture susceptibility.

Specified values for these parameters are applied during mixture design.

Mixture Volumetric Requirements

Mixture volumetric requirements consist of air voids, voids in the mineral aggregate, voids filled with asphalt, and the mixture density during compaction at N_{ini} and N_{max}. Air void content is an important property because it is used as the basis for asphalt binder content selection. In Superpave, the **design air void content is four percent**.

VOIDS IN THE MINERAL AGGREGATE

Superpave defines voids in the mineral aggregate (VMA) as the sum of the volume of air voids and effective (i.e., unabsorbed) binder in a compacted sample. It represents the void space between aggregate particles. The goal is to furnish enough space for the asphalt binder so it can provide adequate adhesion to bind the aggregates, but without bleeding when the temperatures rise and the asphalt expands. Specified minimum values for VMA at the design air void content of four percent are a function of nominal maximum aggregate size.

Superpave VMA Requirements	
Nominal Maximum Aggregate Size	Minimum VMA, %
9.5 mm	15.0
12.5 mm	14.0
19 mm	13.0
25 mm	12.0
37.5 mm	11.0

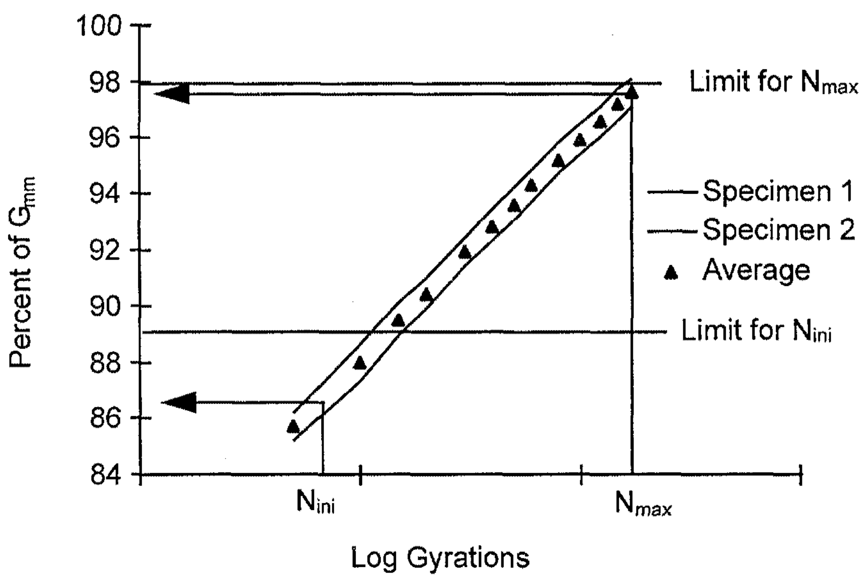
VOIDS FILLED WITH ASPHALT

Voids filled with asphalt (VFA) is defined as the percentage of the VMA containing asphalt binder. Consequently, VFA is the volume of effective asphalt binder expressed as a percentage of the VMA. Although VFA, VMA and air voids 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. The acceptable range of VFA at four percent air voids is a function of traffic level.

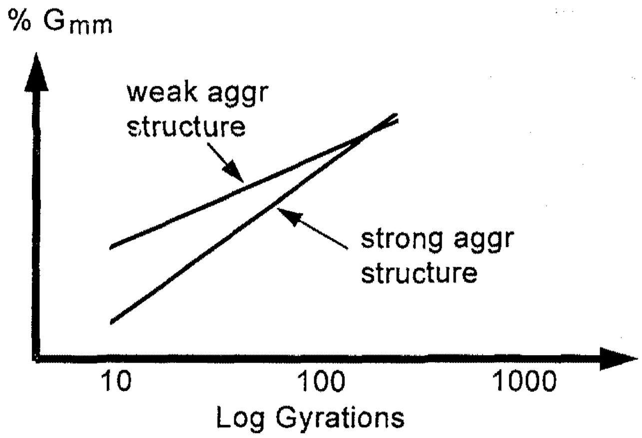
Superpave VFA Requirements	
Traffic, ESALs	Design VFA, %
$< 3 \times 10^5$	70 - 80
$< 1 \times 10^6$	65 - 78
$< 3 \times 10^6$	65 - 78
$< 1 \times 10^7$	65 - 75
$< 3 \times 10^7$	65 - 75
$< 1 \times 10^8$	65 - 75
$> 3 \times 10^8$	65 - 75

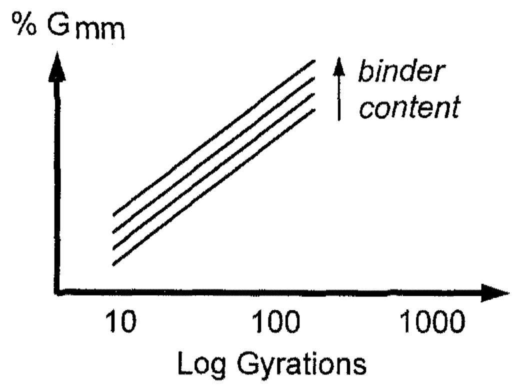
DENSITY REQUIREMENTS

Maximum specimen density criteria are specified at N_{ini} and N_{max}. The compaction characteristic curve developed during gyratory compaction illustrates the limits of 89 percent maximum density at N_{ini} and 98 percent maximum density at N_{max}.



The compaction characteristic curve developed during gyratory compaction provides information about the relative strength of aggregate structures and binder contents. At the same asphalt content, weaker aggregate structures will have flatter slopes and higher density than stronger aggregate structures. For the same aggregate structure, an increase in binder content will produce a mixture with increased density





Specifying a maximum value of percent density N_{ini} prevents design of a mixture that has a weak aggregate structure and low internal friction, indicators of a tender mix. Specifying a maximum value of percent density at N_{max} prevents design of a mixture that will not compact excessively under the design traffic, become plastic, and produce permanent deformation. Since N_{max} represents a compactive effort that would be equivalent to traffic much greater than the design traffic, excessive compaction under traffic will not occur.

Dust Proportion

Another mixture requirement is the dust proportion. This is computed as the ratio of the percentage by weight of aggregate finer than the 0.075 mm sieve to the effective asphalt content expressed as a percent by weight of total mix. Effective asphalt content is the total asphalt used in the mixture less the percentage of absorbed asphalt. Dust proportion is used during the mixture design phase as a design criterion. An acceptable dust proportion is in the range from 0.6 to 1.2, inclusive for all mixtures. Low dust proportion values are indicative of mixtures that may be unstable, and high dust proportion values indicate mixtures that lack sufficient durability.

Moisture Susceptibility

The adhesion between the asphalt and aggregate is an important, yet complex and not well understood, property that helps ensure good pavement performance. The loss of bond, or stripping, caused by the presence of moisture between the asphalt and aggregate is a problem in some areas and can be severe in some cases. Many factors such as aggregate characteristics, asphalt characteristics, environment, traffic, construction practices and drainage can contribute to stripping.

The moisture susceptibility test used to evaluate HMA for stripping is AASHTO T 283, *"Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage."* This test is not a performance based test but serves two purposes. First, it identifies whether a combination of asphalt binder and aggregate is moisture susceptible. Second, it measures the effectiveness of anti-stripping additives.

In the test, two subsets of test specimens are produced. Specimens are compacted have a specimen height of 95 mm and to achieve an air void content in the range from six to eight percent with a target value of seven percent. Test specimens should be sorted so that each subset has the same air void content. One subset is moisture conditioned by vacuum saturation to a constant degree of saturation in the range from 55 to 80 percent. This is followed by an optional freeze cycle. The final conditioning step is a hot water soak. After conditioning both subsets are tested for indirect tensile strength. The test result reported is the ratio of tensile strength of the conditioned subset to that of the unconditioned subset. This ratio is called the "tensile strength ratio" or TSR. This table outlines the current test parameters in AASHTO T 283:

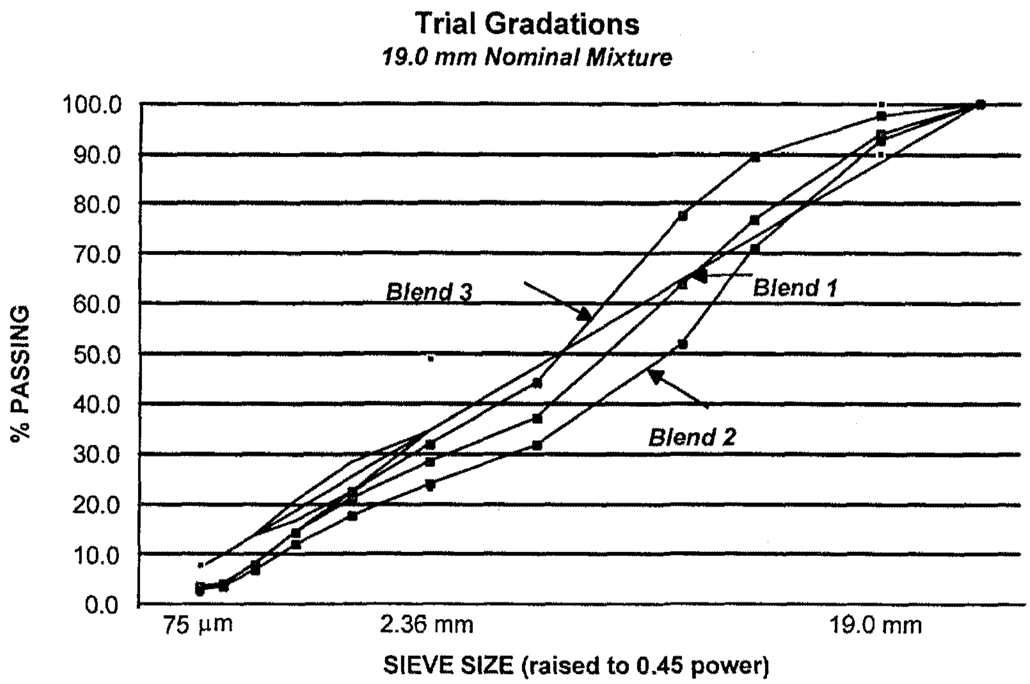
Test Parameter	Test Requirement
Short-Term Aging	Loose mix[1]: 16 hrs at 60°C Compacted mix: 72-96 hrs at 25°C
Air Voids Compacted Specimens	6 to 8 %
Sample Grouping	Average air voids of two subsets should be equal
Saturation	55 to 80 %
Swell Determination	None
Freeze	Minimum 16 hrs at -18°C (optional)
Hot Water Soak	24 hrs at 60°C
Strength Property	Indirect tensile strength
Loading Rate	51 mm/min at 25°C
Precision Statement	None
[1] Short-term aging protocol of AASHTO T 283 does not match short-term aging protocol of Superpave. Suggest using T283 procedure of 16 hours at 60°C.	

Superpave requires a minimum TSR of 80 percent. Lower values are indicative of mixtures that may exhibit stripping problems after construction.

SUPERPAVE MIX DESIGN OVERVIEW

Several publications provide the specific details of conducting a Superpave mix design. Participants are encouraged to seek out these references if they require more details than the overview provided here.

The first step in Superpave mix design is evaluating trial aggregate blends of the selected aggregates to determine a design aggregate structure. Typically, three blends are developed to cover the allowable ranges of the control points and restricted zone. Superpave does not require blends to go beneath the restricted zone, although it is recommended for mixes used in high traffic areas.



Once the trial blends are selected, a preliminary determination of the blended aggregate properties is necessary to ensure they will meet the aggregate criteria. These properties are estimated mathematically from the individual aggregate properties.

Estimated Aggregate Blend Properties				
Property	Criteria	Trial Blend 1	Trial Blend 2	Trial Blend 3
Coarse Ang.	95%/90% min.	96%/92%	95%/92%	97%/93%
Fine Ang.	45% min.	46%	46%	48%
Thin/Elongated	10% max.	0%	0%	0%
Sand Equivalent	45 min.	59	58	54
Combined G_{sb}	N/a	2.699	2.697	2.701
Combined G_{sa}	N/a	2.768	2.769	2.767

Using the asphalt binder selected for the project, each trial blend is mixed and compacted at an estimated asphalt binder content determined selectively for each trial blend. The volumetric properties and compaction characteristics of each trial blend are determined, and adjusted to reflect the properties at four percent air voids.

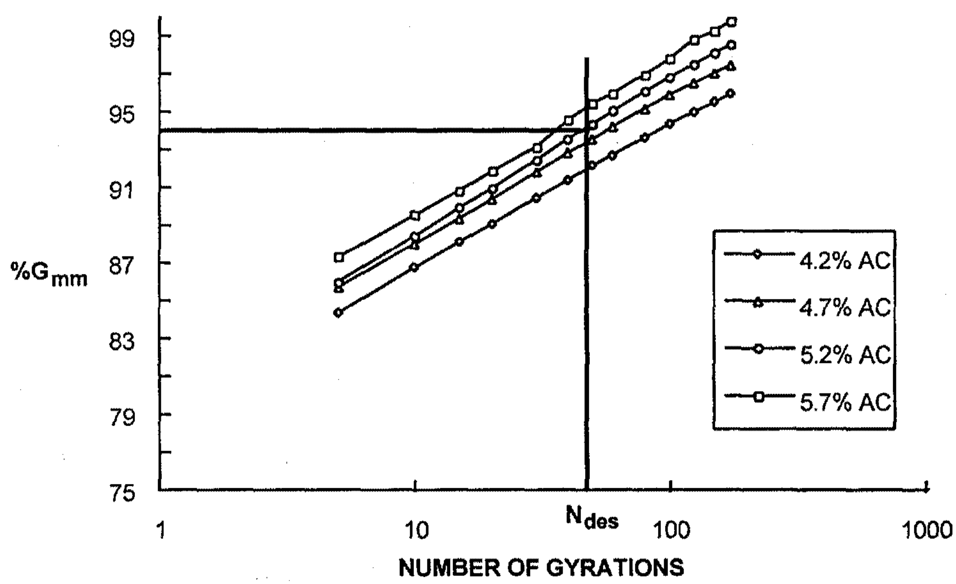
Estimated Mixture Volumetric Properties @ N_{des}						
Blend	Trial %AC	Est. %AC	%Air Voids	%VMA	%VFA	D.P.
1	4.4%	4.3%	4.0%	12.7%	68.5%	0.86
2	4.4%	4.5%	4.0%	13.0%	69.2%	0.78
3	4.4%	4.7%	4.0%	13.3%	70.1%	0.88

Estimated Mixture Compaction Properties				
Blend	Trial %AC	Est. %AC	$\%G_{mm}$ @ N=8	$\%G_{mm}$ @ N=174
1	4.4%	4.3%	86.9%	97.4%
2	4.4%	4.5%	85.9%	97.7%
3	4.4%	4.7%	87.1%	97.3%

After establishing all the estimated mixture properties, the designer compares the values to the mix design criteria and decides if one or more are acceptable, or if further trial blends need to be evaluated.

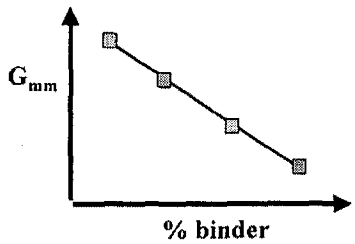
Once the design aggregate structure is selected, specimens are compacted at varying asphalt binder contents. The volumetric properties and compaction characteristics of the selected blend at the varying asphalt binder contents are then evaluated to determine the design binder content for the mixture. Moisture susceptibility is evaluated using AASHTO T283.

Trial Blend, 19.0 mm Nominal

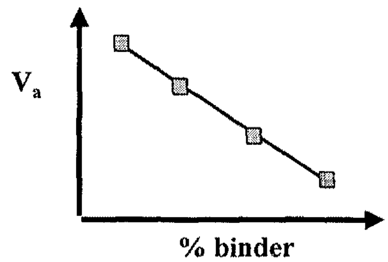


EFFECT OF CHANGING ASPHALT CONTENT ON VOLUMETRIC PROPERTIES

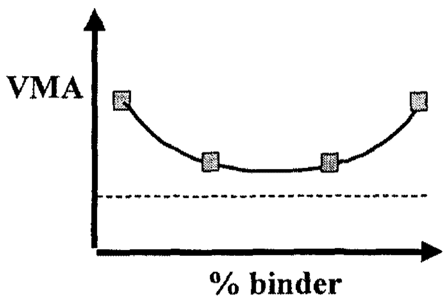
Maximum Theoretical Specific Gravity at Other Asphalt Contents



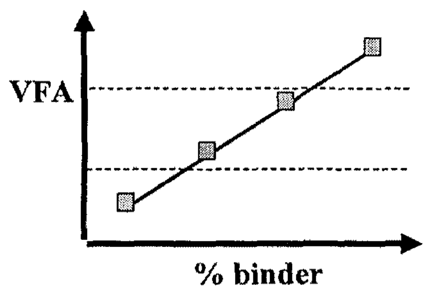
Air Void Content



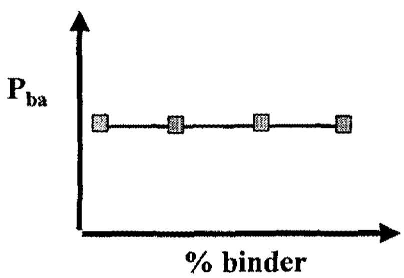
Voids in the Mineral Aggregate



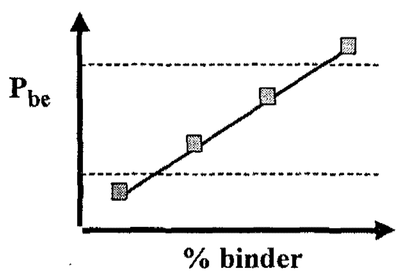
Voids Filled With Asphalt



Absorbed Asphalt Content



Effective Asphalt Content



VI: Impact of Superpave on HMA Construction

To meet the Superpave mix design requirements, personnel in the asphalt industry may begin working with materials that are slightly or even drastically different than those they have encountered previously. Although many current sources of materials can be used in Superpave, some may not be acceptable for every design situation and new sources of materials may be required. Binders with different handling characteristics may be specified. Different sizes and size distributions of aggregates from local sources may be required to create appropriate gradations for Superpave mixtures.

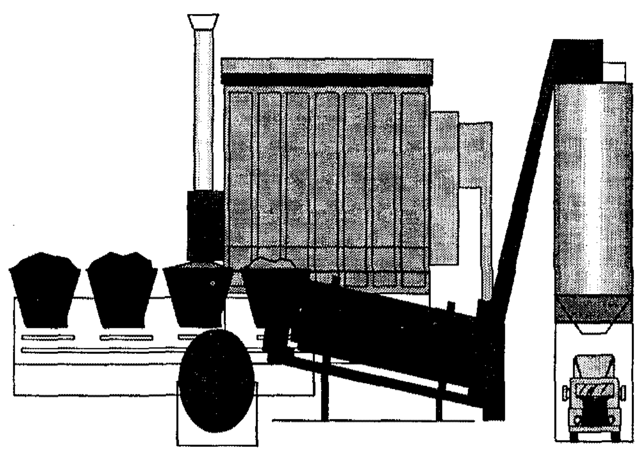
Using these new materials has the potential to affect the production and construction characteristics of the resulting mixture. The degree of difference encountered will depend on prior practice. If the commonly used mixes are fine-graded and contain appreciable quantities of rounded sands, more fines, and/or relatively high asphalt contents (which make paving "easy"), significant differences in the overall construction process will be noted. However, if the producer is already familiar with mixes having a high degree of stone-to-stone contact (such as SMA), the differences will not be as pronounced. Depending on climate and traffic, a modified asphalt binder may be selected for the project mixture. If the producer has no experience with modified asphalt binders, adjustments to the construction process may be required.

To foster a complete understanding of what is involved with Superpave mixes, it is strongly recommended that a prepaving conference be held with all involved parties. If expected differences in materials characteristics are identified and anticipated in the design process, as well as recognized and communicated in the construction phase, the impact need not be significant.

This section is intended to describe how Superpave mixtures *could* behave differently during some of the phases of construction as well as offer suggestions for modifying current handling operations.

MATERIALS HANDLING AND PLANT OPERATIONS

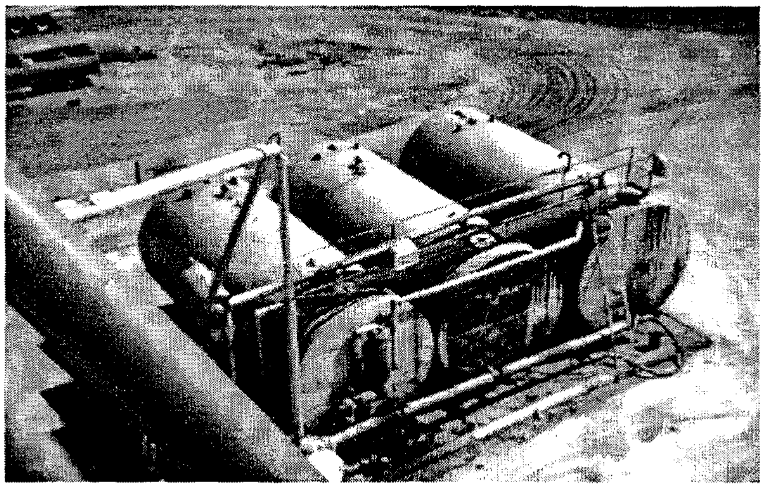
Superpave mixes may or may not contain components that are different from previous experience. The degree of difference will depend on the composition of current mixes. The asphalt binder, the aggregate sizes or blends, and the material sources may need to be changed. Drastically different materials may require different handling procedures from those routinely used with current ingredients.



Binders

Depending on the PG grade of the AC/AR/pen graded binders used until now, the new PG-graded binders could impact the construction process in several ways. If a mix producer is involved with multiple projects, the traffic volumes of the different jobs may require that more than one grade of binder be stored at the plant at a given time. Depending on current availability, this variation in binder may create the need for additional storage tanks to provide sufficient on-site production capacity. Even if enough tanks are available, precautions must be taken.

Any material remaining in a storage tank should be purged before adding different binders. Intermixing different grades of PG binders should be avoided. Since there are numerous ways of producing binders of the same PG grade, different formulations of even the same grade may not be compatible and should never be combined in the same tank. The asphalt supplier should be contacted when considering the intermixing of binders.



The storage temperature of the asphalt binder may need to be adjusted to facilitate pumping from the storage tank to the mixing chamber. Some PG binders will contain some type of modifier. Generally, less storage time is recommended for modified binders and storage temperatures up to 170°C may be necessary. If the binder is to be stored for more than a few days, some suppliers of modified binders recommend lowering the storage temperature after the first three or four days of storage to prevent thermal degradation of the modifier. Thermal degradation can result in the asphalt binder losing its performance benefits.

Similarly, some means of circulation or agitation of the asphalt within the storage tank may be needed to keep the binder homogenous. In-place horizontal tanks can be adapted with additional mixers in existing manholes. If additional tanks are considered, vertical tanks generally work more efficiently and have less "stagnant zones" in the circulatory flow pattern.

If allowed by the agency, an in-line blending process may sometimes produce PG binders. In-line blending will typically occur in a "mixing unit" installed in the asphalt supply line. Sampling valves should be located downstream of the mixing unit where blending occurs. For this situation, an understanding of the details regarding blending procedures, reaction time, sampling, testing, and acceptance is needed.

Overheating can be a problem for all asphalt binders; however, for some polymer-modified binders, contact with super-heated surfaces (greater than 200°C) should be avoided to prevent thermal degradation. Therefore, tanks with hot-oil heated coils are strongly recommended over tanks that use direct-fired burners.

Common asphalt cement additives such as silicone or liquid anti-stripping agents may change the performance characteristics of any binder. The incorporation of these additives may change the high-temperature portion of the PG classification of marginally graded binders enough to cause the resulting binder to "go out of grade". This becomes more of a consideration when the additives are introduced into the binder at the mixing plant. The blended asphalt may be used before it can be tested. It is important that the specifier and the contractor mutually agree on how these kinds of additions will be managed.

In any case, the binder supplier should be contacted for instructions regarding storage temperature and time, required circulation, introduction of additives, and any other product-specific needs.

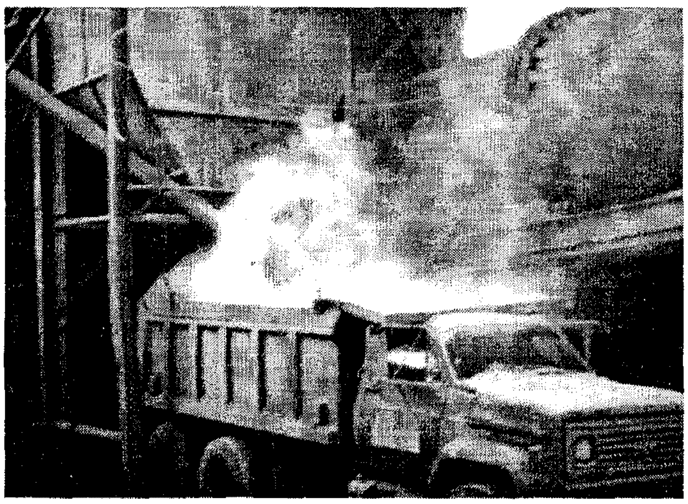
In order to reduce delays while PG binders are being tested for approval, many agencies have adopted procedures for binder suppliers to certify their material. AASHTO PP26, *Standard Practice for an Approved Supplier Certification System for Suppliers of Performance-Graded Binders*, contains standardized procedures developed by industry and agency personnel. It is recommended that all parties become familiar with the binder approval requirements.

Aggregates

Different aggregate types, shapes, sources, sizes, or combinations may be necessary to meet Superpave requirements. These materials may have different properties that could change the construction characteristics of the mix.

In order to meet all Superpave mix design requirements, several different types or sizes of aggregates may have to be blended. Depending on current capacity, this may call for having additional stockpiles and more cold-feed bins. The type of crusher used to process the aggregate can affect particle shape, which can ultimately influence the VMA. Cubical-shaped particles are preferred in Superpave mixes. The particle shape may be improved by utilizing a different type of crusher.

The blend chosen as the design aggregate structure may also handle differently through the plant. Minor modifications in drying time, screening rate, hot bin balance, mixing time and temperature, etc., should be recognized. Since the aggregates used in Superpave mixes are required to be "clean", any differences observed may be a positive improvement. The drying time may be reduced and the screening rate for batch plants may be improved. The hot bin balance will depend on how closely the cold-feed aggregates match the design aggregate structure. If there are sizable discrepancies between the anticipated grading of the individual aggregates and the selected final blend, the hot bins will be unbalanced, and some wasting of unneeded aggregate fractions will be necessary.



Mixtures

In general, experience has shown that Superpave mixes are produced like commonly used mixes. Differences in how the Superpave mixture handle through the plant will obviously depend on how much changed in the mixture specifications. Changes in production rate and the effect on motors, baghouse, potential for segregation, etc., may need to be considered.

For example, because Superpave mixes use substantially greater amounts of coarse aggregate (4.75mm to 19mm), slightly larger screens may be needed on the screen deck to maintain production rates. Higher concentrations of coarse aggregate can cause less veiling of aggregate in the drum, possibly resulting in increased stack temperatures. Mixes having these characteristics can be successfully produced as demonstrated by the routine production of open-graded mixes.

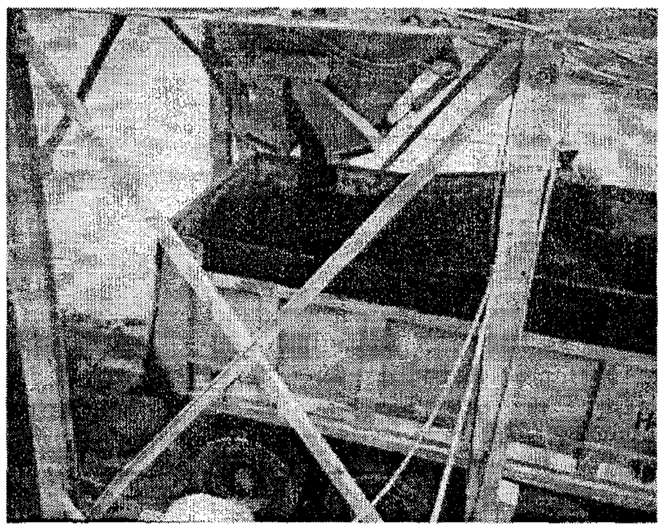
Differences encountered with Superpave mixes can be positive. If clean, low-absorptive aggregate is used, the loads on motors and the dust collection system may actually be reduced and the production rate increased.

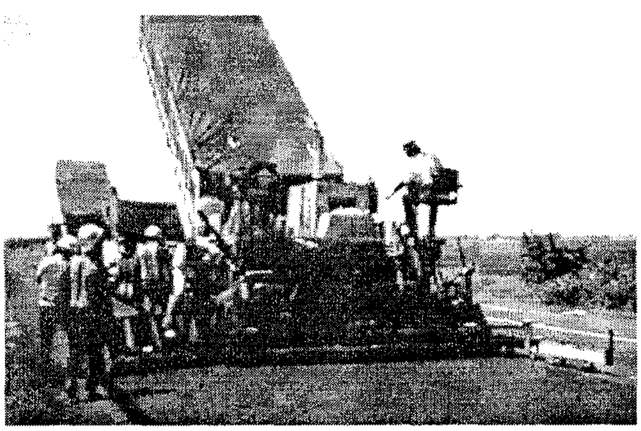
Segregation at the Plant

Typically, Superpave mixes will have a higher concentration of coarse aggregate than some conventional mixes. As a result, these mixes may be more prone to segregation than finer-graded mixes. However, segregation may not be as noticeable if the mix is uniformly coarse. Precautionary steps to minimize segregation throughout the plant-related operations should be implemented if they are not part of the current process.

The aggregate stockpiles should be constructed in the proper manner in uniform layers and to a maximum layer depth of four to five feet. The aggregate must be removed from the stockpile and placed (not dropped) into the cold-feed bins in such a way that the material is not segregated. The coated mix should be handled carefully. Conveyors should be aligned so that they do not toss and segregate the particles. If a surge-bin is used, the batcher (or other means of charging the bin) should be timed and operated to drop the material as a single, large mass within the silo.

The loading of the mix into the truck must be done properly. The mix should not be trickled into the truck; it should, again, be dumped in a mass. With surge storage, enough mix should be in the bin or silo to load the truck before starting the loading process. The drops within the truck bed should be positioned to limit the opportunities for larger particles to roll away from the mass. The initial drops should be positioned against the front and back of the truck bed, and then, subsequent drops should be made against the earlier drops.





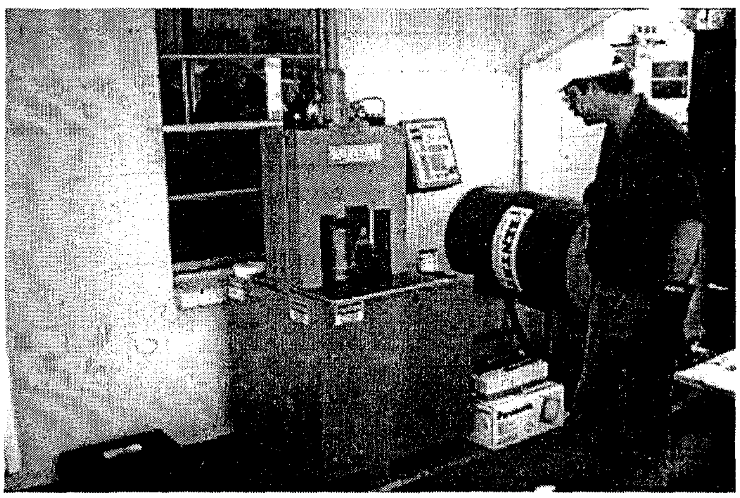
Modified binders will be stiffer than straight run asphalt, so retention of heat to facilitate workability and compaction is important. Covering the truck beds with tarps or insulating the trucks will help minimize the loss of heat.

Quality Control Operations

In selecting the Superpave gyratory compactor as the compaction device for use in the Superpave system, SHRP listed suitability for field quality control (QC) operations as a main concern. By the end of SHRP, many states had implemented, or had considered implementing, verification of the volumetric properties of the asphalt mixture. Therefore, it was necessary that the compaction device for the new mix design and analysis system be useful not only in mix design, but also in field quality control operations. The researchers believed that the Superpave gyratory compactor (SGC) would meet these needs.

Virtually every Superpave test section built since the first projects in 1992 had some form of field quality control testing involving the Superpave gyratory compactor. In 1993, a national research project, NCHRP 9-7, was authorized to study field procedures and equipment to implement the SHRP asphalt research. This research will provide recommendations for field quality control testing of Superpave mixtures. In addition, the Federal Highway Administration-sponsored asphalt trailers have provided Superpave field quality control testing assistance.

Although a definitive quality control program has not yet been developed, several key answers to the question of Superpave field quality control testing have been answered. Essentially, Superpave contractor quality control procedures should be very similar to current quality control testing programs. Determination of asphalt content and gradation will remain necessary components of a quality control testing program. Superpave has done nothing to dispel or lessen the necessity of these tests. Determination of asphalt mixture volumetric properties remains a key issue. The main difference in the Superpave QC program and conventional QC programs lies in determination of volumetric properties.



In the Superpave system, a sample will consist of a minimum of two specimens compacted using the Superpave gyratory compactor. Current Marshall QC testing plans generally require a minimum of three compacted specimens. The time required for compacting two SGC specimens is approximately the same as the time required for three Marshall specimens. Two SGC specimens are considered sufficient since studies have indicated that the bulk specific gravities of the SGC specimens have a smaller standard deviation than the Marshall specimens. No additional aging of the asphalt mixture is necessary in the Superpave system.

Once compacted, volumetric analysis is the same for SGC specimens as with Marshall specimens. A noted disadvantage of the SGC is that the specimens are much larger; approximately four times the mass of a Marshall specimen. The larger specimens will require longer to cool than the Marshall specimens, thereby slowing the ability to determine the bulk specific gravity of the compacted specimen. This in turn slows QC test results. Consequently, some researchers are attempting to devise a quicker turnaround time on test results.

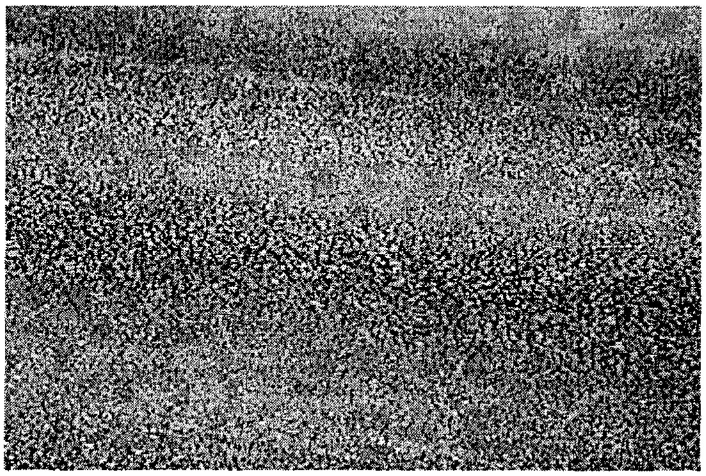
None of the SHRP research or subsequent Superpave implementation research addressed the frequency of sampling. It is assumed that sampling and testing frequency will remain the same using Superpave as with conventional mixtures.

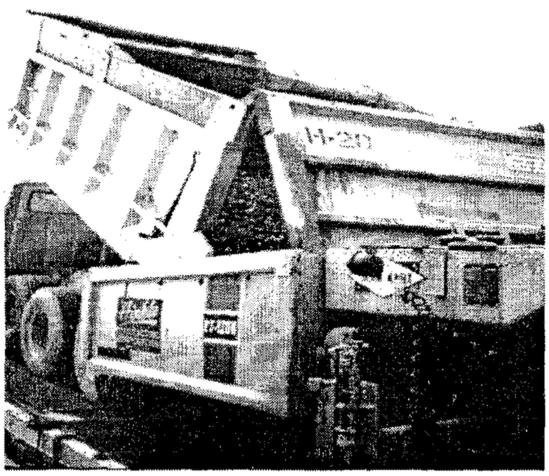
PAVING AND COMPACTION

Superpave mixes may also handle somewhat differently than current mixes during the paving and compaction operations. Some of the potential concerns with Superpave mixes, if coarser than the norm, include: minimizing segregation, lessening tender mix problems, limiting hand-working or raking, and achieving density.

Segregation at the Paver

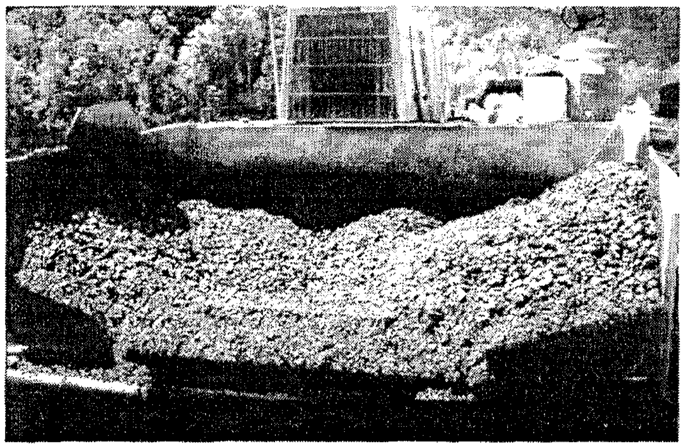
Superpave mixes are not inherently prone to segregation, but some of the design requirements lead to aggregate choices that can be susceptible to segregation. Typically, for projects with heavy traffic, the design aggregate structure of a Superpave mix will contain a relatively high concentration of coarse aggregate. Or, some specifiers may elect to use a mix having a larger maximum size aggregate than the mix that is routinely used. These situations can contribute to the potential for segregation to occur if proper construction practices are not followed.





In addition to the guidelines described previously for materials handling at the plant, standard precautions for handling the mix at the paving site apply. Minimizing segregation on-site begins with correctly unloading the trucks. The mix must be removed from the truck in mass rather than allowing the mix to trickle into the paver hopper from the truck. Truck beds should be lifted slowly to allow the mix to slide back against the tailgate <u>before</u> opening the gate to allow mix to drop into the paver hopper.

The commonly heard warning, "do not dump the hopper wings", is also appropriate for Superpave mixes. Similarly, the normal recommended practices of keeping an adequate depth of mix in the hopper, feeding sufficient material to the auger, etc., are also applicable. A materials transfer vehicle helps to minimize segregation by "reblending" multiple truckloads of mix as well as maintaining a constant supply of mix to the paver. The quality of the mat may be improved by including such a device in the paving process.

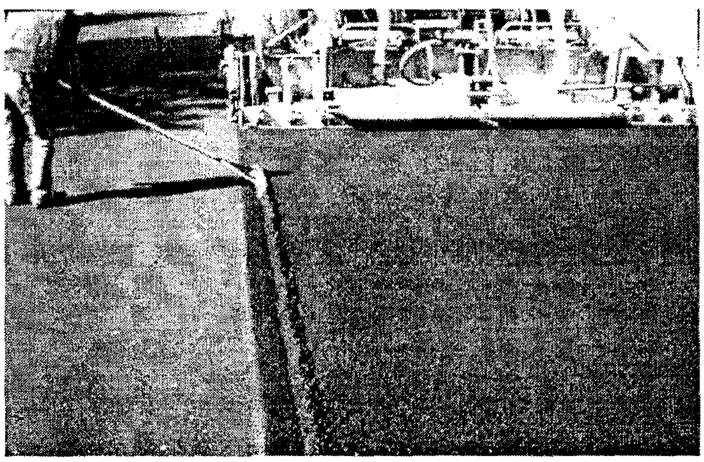


Paver Operations

Some adjustments to the normal operating settings of the paver may be necessary when constructing a Superpave pavement. In particular, the normal operation of the paver screed may be different with Superpave mixes. The same stiffer mix properties that improve rutting resistance are also likely to cause the Superpave mix to be more resistant to easy placement. Typical adjustments may include changing the vertical angle of the screed plate slightly or increasing the effort of the compacting mechanism (vibratory, tamping-bar, etc.) of the screed. If the Superpave mixtures are coarser than typically used, lift thickness may need increasing to ensure a smoothly constructed mat.

Handwork and Joint Construction

The specified properties of Superpave mixes (stiffer binders, higher concentrations of coarse aggregate, more angular coarse aggregate, less rounded sand, etc.) that contribute to the improved rutting resistance may also cause the mix to be more difficult to lute or otherwise work by hand. It is recommended that any handwork be kept to a minimum. Similarly, the relatively coarse aggregate structure selected for some Superpave mixtures may make the construction of a dense, low-permeability longitudinal joint more difficult than for finer graded mixes.



Compaction

Superpave mix design requirements emphasize the use of clean aggregate and proper volumetric properties. Early experience with some Superpave mixes has shown that some users are designing mixes that have a relatively high VMA. In order to meet the air voids requirements, high binder contents are used with these mixes. This results in a mix that is well lubricated and potentially tender despite meeting all Superpave criteria.

An extremely over-asphalted mixture could potentially be subject to asphalt draindown like that sometimes experienced with SMA mixtures. The designer should re-evaluate any mix that appears to have an unusually high VMA and determine if the grading can be revised to achieve a mix that is less susceptible to tenderness and potentially less expensive to produce. It is strongly recommended that asphalt content, VMA, and VFA be reviewed in terms of handling and construction as a final step of every mix design.

It is also possible that a Superpave mix will be tougher than current mixes. If the local mix generally contained excessive amounts of medium to fine-sized sands (such that the gradation plotted through or above the restricted zone), it may have been easier to place or even tender during rolling. These mixtures may have been compacted by allowing the mat to cool before the application of the rollers. Eliminating the excess sand to meet Superpave requirements may improve the compactability and resolve the tenderness issue.

Due to their relative coarseness, Superpave mixes may be somewhat more difficult to compact than fine-graded mixes. It is strongly recommended that a test strip be constructed to establish the optimal rolling pattern. The optimum type, size, and number of rollers and their operating patterns should be determined prior to mainline paving. In general, experience has shown that compaction of most Superpave mixes is best achieved by keeping the breakdown roller immediately behind the paver. The amount of time available to compact a Superpave mix before it stiffens and becomes extremely difficult to compact may be less than for commonly used mixes. To facilitate compaction, thicker courses of Superpave mixes may be necessary to retain heat and maintain the workability.

If the PG binder contains a modifier, exercise care with the use of pneumatic rollers. These binders are usually very sticky and tend to adhere to the rubber tires of the roller even when properly heated. This tackiness can cause pickup of particles from the freshly placed mat. For base courses, this problem may be tolerable, but pneumatic rollers are best avoided when compacting surface mixes containing polymer modifiers.

Conclusion

Some Superpave mixes may handle and respond somewhat differently from the present experience with some current mixes; however, with communication, planning, and attention to good construction practices, these mixes can deliver the superior performance they were designed to provide.

VII: Superpave Implementation Activities

Even before the SHRP research began, it was recognized that a "program designed without taking into account obstacles on implementation of research will fail," in TRB Report 202, which proposed the development of the Strategic Highway Research Program. Eventually, more money will be spent on the implementation of the SHRP products than on the research itself.

This section will discuss some of the activities surrounding the implementation of Superpave. Many of the Superpave implementation programs and activities are interrelated, and this text may reference some activities before they are fully explained. By the end of the section, the activities will be fully described, and the reader should know where any necessary help could be obtained.

FUNDING AND PLANS

Although the 5-year, $150 million SHRP program did not end until March 1993, planning for implementation had started well before that date. Funding and leadership for SHRP implementation was officially established on December 18, 1991, with the signing of the Intermodel Surface Transportation Efficiency Act (ISTEA). ISTEA allocated a total of $108 million to FHWA to implement the products of SHRP and to continue the Long Term Pavement Performance (LTPP) Program.

The strategic plan for implementation of all of the SHRP products is described in "Implementation Plan - SHRP Products", June 1993, FHWA-SA-93-054. The SHRP implementation plan describes the internal and external organizational structure, partners and partnerships, purpose, roles, implementation mechanisms, and support functions that are used to accomplish the FHWA Implementation Program. It also details the framework under which the various entities function in carrying out this mission.

The development and execution of national implementation plans for specific products or groups of products is accomplished through four Technical Working Groups (TWGs):

Asphalt
Concrete and Structures
Highway Operations
Long Term Pavement Performance

The initial meetings of these TWGs were held in the summer of 1993.

The review, finalizing, and acceptance of the tests and specifications within the Superpave asphalt mix design system are being handled within the committees of the Association of American State Highway and Transportation Officials (AASHTO). Provisional standards have been developed by AASHTO, and they are being reviewed and revised under the auspices of the AASHTO committees. The committees work with the many organizations involved with the development and implementation of the Superpave tests and specifications in establishing the final criteria.

FHWA IMPLEMENTATION ACTIVITIES

To provide a forum for the SHRP researchers to present the research and development results and to review the decision-making processes that took place, FHWA sponsored a SHRP Asphalt Technology Conference in Reno, Nevada. This technical forum, held October 24-28, 1994, served as a foundation for the many future implementation efforts.

FHWA has a series of initiatives underway that will provide assistance to State Highway Agencies and the asphalt industry in the implementation of the Superpave System. The FHWA implementation activities for asphalt actually began in 1992 and are projected to continue until the end of the decade. The program includes six major initiatives and numerous related projects:

Technical Assistance Program
Superpave Pooled-Fund Equipment Purchase
National Asphalt Training Center
Superpave Regional Centers
Mobile Laboratory Program
Research Activities

Superpave Technology Delivery Team

To serve as a focal point for all Superpave implementation activities, FHWA formed the Superpave Technology Delivery Team (TDT) using representatives from various offices. This team will provide leadership, coordination, and support for the many initiatives and staff involved in Superpave implementation. For more information, contact:

Gary Henderson, *Team Leader,* Highway Operations Division

Phone: 202 - 366 - 1549
FAX: 202 - 366 - 9981
e-mail : gary.henderson@fhwa.dot.gov

Technical Assistance Program

To implement a new technology, the industry must be familiar with it and comfortable with all of its many aspects. In 1993 and 1994, FHWA purchased five sets of the Superpave binder test equipment and loaned the equipment to the five newly-formed regional asphalt user-producer groups. This early trial period served to introduce the equipment to the asphalt industry and provide preliminary training for the tests. The user-producer groups typically placed this equipment in their associated Superpave Regional Center. During this period, equipment refinement continued, resulting in the final specifications of the Superpave binder equipment. There was significant redesign of all of the protocols, especially for the PAV, DTT, and SGC.

The asphalt user-producer groups consist of representatives from state, federal and local agencies (users) and material producers and suppliers (producers). By having a forum where each group can present their views on very complex issues, the points of view of all sides can be understood, and resolutions can be more easily reached by balancing the needs of all parties.

Under TE Project 39, *Superpave Asphalt Support Services*, engineers and technicians are available through several sources to assist the states and industry in setting up equipment and conducting preliminary training. This assistance includes workshops and mini-classes; equipment installation assistance, operation, and data collection; field tests (SPS-9); data analysis; and a variety of other activities.

Several states have had difficulty setting up and operating some of their binder equipment. The same FHWA support staff have worked with many of the DOTs and manufacturers to try to resolve these problems and ensure that each state has properly operating equipment.

Technicians were also involved in conducting the ruggedness testing of the binder equipment. Several engineers and technicians from private laboratories have also been trained at the FHWA Office of Technology Binder Lab at the Turner-Fairbank Highway Research Center (TFHRC).

Pooled-Fund Equipment

In February 1992, SHRP accepted the research recommendations for the accelerated performance tests (APT). In March 1992, FHWA, working with Draft Number 6 of the asphalt binder specifications, initiated the planning of the Highway Planning and Research (HPR) pooled-fund Superpave equipment purchase, after a joint meeting with the State Materials Engineers. The states had agreed to pool a portion of their Federal-Aid research money to purchase sets of testing equipment for the Superpave binder and mixture procedures. The original plan included these eight pieces of laboratory equipment:

Binder Equipment :

- Pressure Aging Vessel (PAV)
- Rotational Viscometer (RV)
- Bending Beam Rheometer (BBR)
- Dynamic Shear Rheometer (DSR)
- Direct Tension Tester (DTT)

Mix Equipment :

- Superpave Gyratory Compactor (SGC)
- Superpave Shear Tester (SST)
- Indirect Tensile Tester (IDT)

The original estimate for the pooled fund project was $335,000:

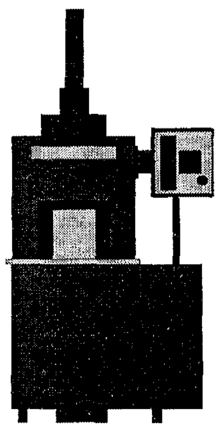
$98,000 for binder equipment
$227,000 for mix equipment
$10,000 for training

All of the pooled-fund asphalt binder equipment, except the DTT, has been delivered to the 52 participating highway agencies. Based on this purchase of equipment, the expected cost for a laboratory to buy a complete set of binder equipment is approximately $85,000:

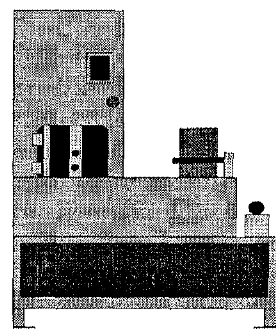
PAV	$10,000
RV	$5,000
DSR	$25,000
BBR	$20,000
DTT	$25,000

These costs are estimated based on individual purchases of each piece of equipment. The possibility exists for savings in a multiple purchase agreement. Any additional options desired to accompany the equipment would obviously increase the cost.

The pooled-fund buy for the DTT is on hold until new equipment, test procedures, and specifications are developed. A new prototype for the DTT was delivered in January 1996 and a preliminary evaluation has been completed. The new equipment can perform all the required functions and it has proven that repeatable results can be achieved. Five more units are being purchased prior to executing a general pooled-fund buy and these units will be used for ruggedness testing and resolving any questions about the accuracy of the test procedures. The general purchase is expected to occur in 1997.



The pooled-fund purchase of the SGC is complete; all 52 participating highway agencies have taken delivery of their first device. At the time, there were two manufacturers of the SGC. Both pieces of equipment cost about $25,000.

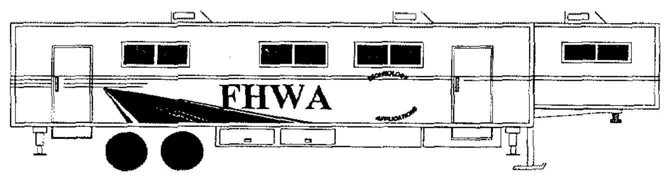


Long-Term Pavement Performance

Part of the SHRP research included the Long-Term Pavement Performance (LTPP) program. The LTPP program involved developing pavement monitoring and management tools for testing and evaluating the performance of in-place pavements. A major portion of the LTPP program was the selection and testing of hundreds of General Pavement Study (GPS) sites across North America. These GPS sites represented all types of pavements, climates and soil conditions, in an effort to analyze the hows and whys of long term pavement performance.

On-going work with the LTPP program will include the design and construction of Superpave test sections to validate the Superpave binder selection criteria, mix design requirements, and mix analysis predictions. The experimental project selection and development, data collection and analysis, and information sharing will continue into the next century.

Mobile Lab Program



Since 1987, the FHWA has had a mobile asphalt laboratory program, which has provided assistance in volumetric mix design and quality control of mixes at the plant site. This program was expanded in 1992, and two new mobile laboratories have been equipped to bring the principles of the Superpave volumetric mix design to the construction site. This effort was the first introduction of the concept of field management with volumetric mix design. At each project, there are two objectives:

1. Current mix is tested to Superpave standards.
2. A full independent Superpave mix design and analysis is performed.

At each site, the mobile lab personnel offer to hold a one- or two-day workshop. The workshop covers an introduction to the Superpave specifications and procedures for volumetric mix design. The two demonstration trailers are equipped with a Superpave Gyratory Compactor.

The first priority of the mobile labs are supporting the mix design activities involved with the SPS-9 studies. One of the trailers supported the construction of WesTrack, the test track for the Performance Related Asphalt Specification near Reno, Nevada. The trailer developed the Superpave mix designs and also assisted in the construction quality control testing for the track. These personnel have also been used to compare the results of different SGCs and operators with the same mixes.

A third trailer is equipped with a full set of the Superpave binder equipment and is available to provide states with technical support.

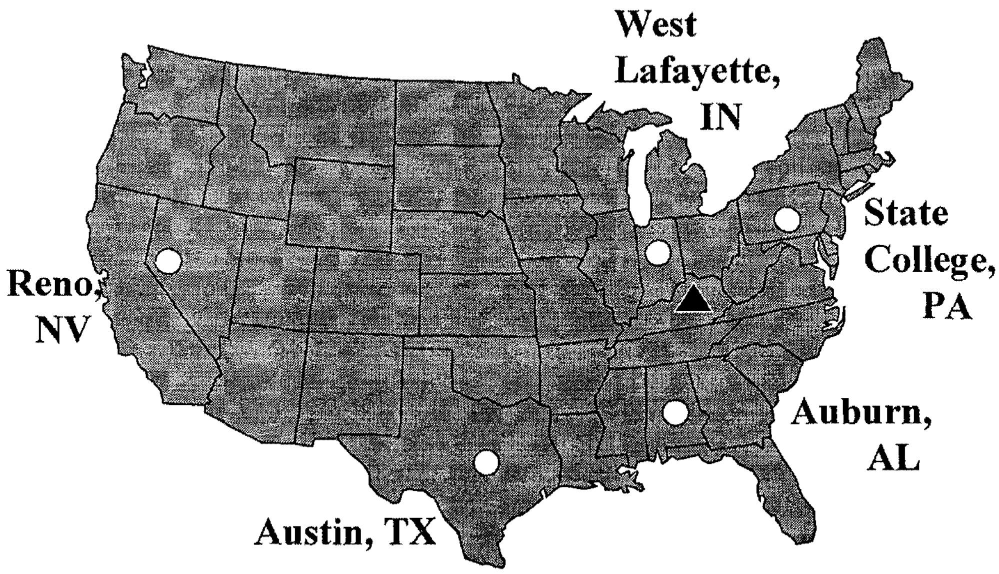
NATC

In September 1992, FHWA established the National Asphalt Training Center at the Asphalt Institute in Lexington, Kentucky. The primary activities of the NATC were to develop training materials for hands-on laboratory courses in Superpave asphalt binder testing and volumetric mix design. Over 550 participants representing State DOT, industry, university, and FHWA received training under this initial contract in 16, one-week, Superpave Binder courses and 14, one-week, Superpave Mix Design courses. As an additional part of the NATC activities, the Superpave Gyratory Compactor ruggedness experiment was conducted to establish sources of test procedure variability. These test data were later used to revise a few of the tolerances of the AASHTO provisional method, TP4, *Standard Method for Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the SHRP Gyratory Compactor.*

Superpave Regional Centers

Five Superpave Regional Centers were established in 1995 to provide technical leadership and assistance on a more-localized, regional basis for the implementation of Superpave. The Centers are evaluating Superpave equipment and procedures and helping the State highway agencies put the technology into practice. They provide another valuable source of hands-on training and experience for engineers and technicians in the area.

The Superpave Regional Centers were established in these locations:



Rocky Mountain Center Reno, Nevada Contact: Jon Epps	Phone: 702 - 784 - 6873 FAX: 702 - 784 - 1429 e-mail: epps@unr.edu
South Central Center Austin, Texas Contact: Bob McGennis	Phone: 512 - 475 - 7912 FAX: 512 - 475 - 7914 e-mail: bmcgennis@mail.utexas.edu
SouthEast Center Auburn, Alabama Contact: Doug Hanson	Phone: 334 - 844 - 6240 FAX: 334 - 844 - 6248 e-mail: hansodi@mail.auburn.edu
North Central Center West Lafayette, Indiana Contact: Rebecca McDaniel	Phone: 765 - 463 - 2317 FAX: 765 - 497 - 2402 e-mail: rsmcdani@ce.ecn.purdue.edu
NorthEast Center State College, Pennsylvania Contact: Anne Stonex	Phone: 814 - 863 - 5789 FAX: 814 - 865 - 3039 e-mail: asx60@psu.edu

Each host state has a working relationship with a local university and has established a detailed operations plan with them.

Beyond technical assistance, the initial plans for the Centers include performing the testing for the SST and IDT ruggedness experiments. After this testing is completed, this equipment will be used for testing SPS-9B field sections. Each Center is supporting the surrounding states with the evaluation of their mixes.

Superpave Models

Also during 1992, FHWA recognized the need to complete development of Superpave performance prediction models and revise the initial version of the Superpave software. On July 20, 1995, the Superpave Support and Performance Models Management contract was awarded to the University of Maryland. This contract is divided into two phases.

In Phase I, which ended on December 31, 1996, the existing Superpave performance models were critiqued and a revised version of the original software was beta-tested and evaluated for technical accuracy. A report documented necessary corrections, improvements and suggested enhancements to the models and software. After meeting with FHWA on their findings, in Phase II their main emphasis in the modeling area will be asphalt concrete materials characterization. In addition, the original Superpave software will be completely revised and updated based on the findings in Phase I. A Windows version 2.0 of the Superpave Mix Design software is expected to be complete in 1997.

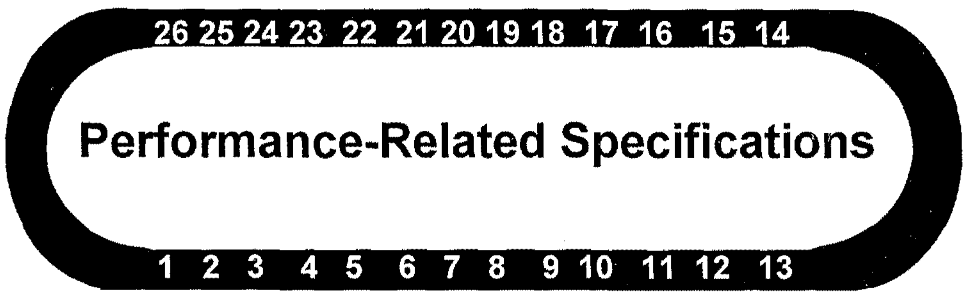
As an interim measure, until the entire modeling system can be completely revised, their staff will also work on a mixture strength test to supplement Superpave Volumetric Mix Design. This test is currently scheduled to be available in early 1999.

A long-term program is being planned to eventually develop the final performance models based on the revised system framework to be established under this contract. Using the data and materials that are currently being collected under LTPP, a field verification study of these new models will then be conducted to document their statistical accuracy.

WesTrack

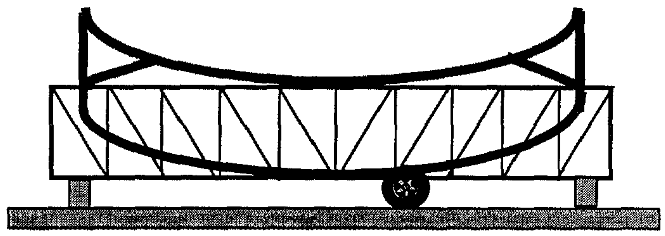
To accelerate the validation of the Superpave Mix Design method and to develop performance parameters for Performance Related Specifications (PRS) for asphalt pavements, FHWA awarded a contract to the Nevada Automotive Test Center in September 1994. An accelerated test track facility, "WesTrack," was completed in November 1995 about 100-km southeast of Reno, Nevada.

The 2.9-km track includes 26 test sections to evaluate the effect of variations in binder content, gradation, and density on the Superpave Mix Design System. Four sections were designed by strictly following all of the Superpave recommendations. Both a coarse gradation, made from crushed gravel and local natural sands, and a fine gradation, made from all crushed material, were used on the track. Another fine mix included three percent additional dust (minus 0.075 mm) material. Three levels of binder content (optimum, optimum -0.7 percent, optimum + 0.7 percent) and three levels of in-place air voids (4, 8, and 12 percent) were evaluated.



An automated vehicle guidance system was designed and installed in March 1996, and four heavily loaded triple-trailer driverless trucks began trafficking at 65 kph for 15 hours each day. As planned, the track will be subjected to ten million equivalent single axle loads (ESAL) in two years. Early findings indicate that the size of the aggregate is not as critical as the angularity, shape, and texture quality of the particles, when the pavement is being asked to endure heavy traffic.

Accelerated Loading Facility (ALF)



An Accelerated Loading Facility (ALF) is a mobile testing device that applies truck traffic loadings to pavement test sections. Much like the automated trafficking at WesTrack, an ALF can apply a concentrated number of loadings in a short period of time. The FHWA has an ALF at the Turner-Fairbank Highway Research Center.

The pavement sections for the ALF at TFHRC were reconstructed to isolate and evaluate Superpave binder effects on specific types of mixtures. A second ALF has also been delivered to further expedite the testing. Five different PG binders (52-34, 58-28, 64-28, 82-34, and 70-22) and two different gradations (19 mm and 38 mm maximum aggregate size) are included in this experiment.

The initial testing for the rutting and fatigue experiment is now completed. Future testing will be conducted on sections at differing temperatures to explore this effect.

NCHRP Studies

The National Cooperative Highway Research Program is a program administered by the National Academy of Sciences and funded by the individual states to investigate research needs identified by the state highway and transportation departments. NCHRP has several projects related to Superpave implementation and validation:

9-7 Field Procedures and Equipment to Implement SHRP Asphalt Specifications (This project is complete and the final report is being prepared.)

9-9 Refinement of the Superpave Gyratory Compaction Procedure

9-10 Superpave Protocols for Modified Asphalt Binders

9-12 Incorporation of Reclaimed Asphalt Pavement in the Superpave System

9-13 Evaluation of the AASHTO T283 Water Sensitivity Test

9-14 Investigation of the Restricted Zone in the Superpave Aggregate Gradation Specification

The latter five projects are in the initial stages of the study.

Expert Task Groups (ETG)

Expert Task Groups (ETGs) have been formed to provide technical guidance for the activities of the Technical Working Groups (TWGs). Three ETGs support the Asphalt TWG: the Binder ETG, the Mixtures ETG, and the Models/Software ETG.

The Binder ETG reviews issues related to the AASHTO provisional binder specifications and test methods. A number of issues are being considered, including:

Alternative binder fatigue criteria
New Direct Tension Tester and modification of criteria
Revisions necessary for testing modified binders
Revision of the low pavement temperature calculation
PG asphalt binder supplier certification system

The Mixtures ETG reviews issues related to the AASHTO provisional specifications, test methods, and practices related to mix and aggregate. A number of issues are being discussed, including:

Objectives of the gradation restricted zone
Refinements to gradation control points
Fine aggregate angularity test and level of criteria
Gyratory Compaction Levels (N_{design})
Short-term oven aging duration
Incorporation of Reclaimed Asphalt Pavement (RAP)

The Software/Models ETG reviews issues related to the AASHTO provisional test methods related to mix analysis; the various material, structural, and performance models; and the framework for the Superpave software. Ideas being discussed include:

Modeling of rutting
Modeling of fatigue
Characterization of material properties
Modeling the lower layers of the pavement structure
Need of a reflective cracking model
Form of traffic load input
Data necessary for field verification
Content of Superpave software

Lead States Pool of Expertise

With the goal of shortening the learning period for others, a "Lead State" initiative was advanced at a conference held in St. Louis, Missouri, in September 1996. The objective of AASHTO and the FHWA was to form teams of people from states that had lots of experience using a particular SHRP product or technology.

A team of lead states with Superpave experience is sharing what they have learned from implementing the Superpave technology. Engineers and technicians from these six Superpave lead states are available to provide technical support and assistance, by telephone, regarding binder testing, mix design and analysis, and construction:

Florida
Indiana
Maryland
New York
Texas
Utah

These personnel can help disseminate information concerning lessons learned or any proposed changes to the Superpave System. Some on-site technical assistance may be available and conference presentations are possible. The FHWA has dedicated some funding to support this activity.

Appendix A

Standard Specification for
Performance Graded Asphalt Binder

AASHTO Designation: MP1[1]

1. Scope - This specification covers asphalt binders graded by performance. Grading designations are related to average 7-day maximum pavement design temperatures and minimum pavement design temperatures.

Note 1 -- For asphalt cements graded by penetration at 25°C, see M20. For asphalt cements graded by viscosity at 60°C see M226.

Note 2 -- Guide PP5 provides information on the evaluation of modified asphalt binders.

Note 3 -- Guide PP6 provides information for determining the performance grade of an asphalt binder.

2. Reference Documents

2.1 AASHTO Documents:

PP5	Guide for the Laboratory Evaluation of Modified Asphalt Systems
PP6	Guide for Grading or Verifying the Performance Grade of an Asphalt Binder
PPX	Selection of Asphalt Binders (Being Developed)
M20	Specification for Penetration Graded Asphalt Cement
M226	Specification for Viscosity Graded Asphalt Cement
PP1	Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
T40	Practice for Sampling Bituminous Materials
T44	Solubility of Bituminous Materials in Organic Solvents
T48	Method for Flash and Fire Points by Cleveland Open Cup
T55	Method for Water in Petroleum Products and Bituminous Materials
T179	Test Method for Effect of Heat and Air on Asphalt Materials (Thin-Film Oven Test)
T201	Kinematic Viscosity of Asphalts
T202	Viscosity of Asphalts by Vacuum Capillary Viscometer
T240	Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin Film Oven Test)
TP1	Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)
TP3	Test Method for Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT)
TP5	Test Method for Determining Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)

[1] *This standard is based on SHRP Product 1001.*

2.2 ASTM Documents:

D8 Standard Definitions of Terms Relating to Materials for Roads and Pavements
D4402 Method for Viscosity Determinations of Unfilled Asphalt Using the Brookfield Thermosel Apparatus

2.3 SHRP Documents:

P00X Superpave Software (being developed)

3. Terminology

3.1 Definitions

3.1.1 Definitions for many terms common to asphalt cement are found in ASTM D8.

3.1.2 asphalt binder, n -- an asphalt-based cement that is produced from petroleum residue either with or without the addition of non-particulate organic modifiers.

4. Ordering Information - When ordering under this specification, include in the purchase order the performance grade of asphalt binder required from Table 1 (e.g. PG 52-16 or PG 64-34).

4.1 Asphalt binder grades may be selected by following the procedures described in provisional practice PPX, Selection of Asphalt Binders.

5. Materials and Manufacture

5.1 Asphalt cement shall be prepared by the refining of crude petroleum by suitable methods, with or without the addition of modifiers.

5.2 Modifiers may be any organic material of suitable manufacture, used in virgin or recycled condition, and that is dissolved, dispersed or reacted in asphalt cement to enhance its performance.

5.3 The base asphalt binder shall be homogeneous, free from water and deleterious materials, and shall not foam when heated to 175°C.

5.4 The base asphalt binder shall be at least 99.0% soluble in trichloroethylene as determined by AASHTO T44.
5.5 The bending beam rheometer test, TP1, the direct tension test, TP3, and the dynamic shear rheometer test, TP5, are not suitable for asphalt binders in which fibers or other discrete particles are larger than 250 μm in size.

5.6 The grades of asphalt binder shall conform to the requirements given in Table 1.

6. Sampling - The material shall be sampled in accordance with Method T 40.

7. Test Methods - The properties outlined in 5.3, 5.4 and 5.6 shall be determined in accordance with T44, T48, T55, T179, T240, PP1, TP1, TP3, TP5, and ASTM D4402.

8. Inspection and Certification - Inspection and certification of the material shall be agreed upon between the purchaser and the seller. Specific requirements shall be made part of the purchase contract.

9. Rejection and Rehearing - If the results of any test do not conform to the requirements of this specification, retesting to determine conformity is performed as indicated in the purchase order or as otherwise agreed upon between the purchaser and the seller.

10. Key Words - Asphalt binder, asphalt cement, modifier, performance specifications, rheology, direct tension, pressure aging, flash point.

Table 1. Performance Graded Asphalt Binder Specification

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C[a]	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

a Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program, may be provided by the specifying agency, or by following the procedures as outlined in PPX.

b This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

c For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of $G^*/\sin\delta$ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry (AASHTO T201 or T202).

d The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 64- and above, except in desert climates, where it is 110°C.

e Physical Hardening -- TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and *m*-value are reported for information purposes only.

f If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The *m*-value requirement must be satisfied in both cases.

Table 1. Performance Graded Asphalt Binder Specification
(Continued)

PERFORMANCE GRADE	PG 70-						PG 76-					PG 82-				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average 7-day Maximum Pavement Design Temp, °C[b]	<70						<76					<82				
Minimum Pavement Design Temperature, °C[b]	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																
Flash Point Temp, T48: Minimum °C	230															
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135															
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN (T179) RESIDUE																
Mass Loss, Maximum, percent	1.00															
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP1)																
PAV Aging Temperature, °C[d]	100(110)						100(110)					100(110)				
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening[e]	Report															
Creep Stiffness, TP1:[f] S, Maximum, 300.0 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24

Appendix B: Superpave Asphalt Mixture Gradation Requirements

37.5 MM NOMINAL SIZE

Sieve, mm	Control Points		Restricted Zone Boundary	
	Minimum	Maximum	Minimum	Maximum
50	100.0			
37.5	90.0	100.0		
25		90.0		
19				
12.5				
9.5				
4.75			34.7	34.7
2.36	15.0	41.0	23.3	27.3
1.18			15.5	21.5
0.600			11.7	15.7
0.300			10	10
0.150				
0.075	0.0	6.0		

25 MM NOMINAL SIZE

Sieve, mm	Control Points		Restricted Zone Boundary	
	Minimum	Maximum	Minimum	Maximum
37.5	100.0			
25	90.0	100.0		
19		90.0		
12.5				
9.5				
4.75			39.5	39.5
2.36	19.0	45.0	26.8	30.8
1.18			18.1	24.1
0.600			13.6	17.6
0.300			11.4	11.4
0.150				
0.075	1.0	7.0		

19 MM NOMINAL SIZE

	Control Points		Restricted Zone Boundary	
Sieve, mm	Minimum	Maximum	Minimum	Maximum
25	100.0			
19	90.0	100.0		
12.5		90.0		
9.5				
4.75				
2.36	23.0	49.0	34.6	34.6
1.18			22.3	28.3
0.600			16.7	20.7
0.300			13.7	13.7
0.150				
0.075	2.0	8.0		

12.5 MM NOMINAL SIZE

	Control Points		Restricted Zone Boundary	
Sieve, mm	Minimum	Maximum	Minimum	Maximum
19	100.0			
12.5	90.0	100.0		
9.5		90.0		
4.75				
2.36	28.0	58.0	39.1	39.1
1.18			25.6	31.6
0.600			19.1	23.1
0.300			15.5	15.5
0.150				
0.075	2.0	10.0		

9.5 MM NOMINAL SIZE

	Control Points		Restricted Zone Boundary	
Sieve, mm	Minimum	Maximum	Minimum	Maximum
12.5	100.0			
9.5	90.0	100.0		
4.75		90.0		
2.36	32.0	67.0	47.2	47.2
1.18			31.6	37.6
0.600			23.5	27.5
0.300			18.7	18.7
0.150				
0.075	2.0	10.0		

Appendix C: Outline of Steps in Superpave Mix Design

I. MATERIALS SELECTION

A. Selection of Asphalt Binder

1. Determine project weather conditions using weather database
2. Select reliability
3. Determine design temperatures
4. Verify asphalt binder grade
5. Temperature-viscosity relationship for lab mixing and compaction

B. Selection of Aggregates

1. Consensus properties
   a. Combined gradation
   b. Coarse aggregate angularity
   c. Fine aggregate angularity
   d. Flat and elongated particles
   e. Clay content
2. Agency and Other properties
   a. Specific gravity
   b. Toughness
   c. Soundness
   d. Deleterious materials
   e. Other

C. Selection of Modifiers

II. SELECTION OF DESIGN AGGREGATE STRUCTURE

A. Establish Trial Blends

1. Develop three blends
2. Evaluate combined aggregate properties

B. Compact Trial Blend Specimens

1. Establish trial asphalt binder content
   a. Superpave method
   b. Engineering judgment method
2. Establish trial blend specimen size
3. Determine N_{design} & $N_{initial}$ & $N_{maximum}$
4. Batch trial blend specimens
5. Compact specimens and generate densification tables
6. Determine mixture properties (G_{mm} & G_{mb})

C. Evaluate Trial Blends

1. Determine %G_{mm} @ $N_{initial}$ & N_{design} & $N_{maximum}$
2. Determine %Air Voids and %VMA
3. Estimate asphalt binder content to achieve 4% air voids
4. Estimate mix properties @ estimated asphalt binder content
5. Determine dust proportion
6. Compare mixture properties to criteria

D. Select Most Promising Design Aggregate Structure for Further Analysis

III. SELECTION OF DESIGN ASPHALT BINDER CONTENT

A. Compact Design Aggregate Structure Specimens at Multiple Binder Contents

1. Batch design aggregate structure specimens
2. Compact specimens and generate densification tables

B. Determine Mixture Properties versus Asphalt Binder Content

1. Determine $\%G_{mm}$ @ $N_{initial}$ & N_{design} & $N_{maximum}$
2. Determine volumetric properties
3. Determine dust proportion
4. Graph mixture properties versus asphalt binder content

C. Select Design Asphalt Binder Content

1. Determine asphalt binder content at 4% air voids
2. Determine mixture properties at selected asphalt binder content
3. Compare mixture properties to criteria

IV. EVALUATION OF MOISTURE SENSITIVITY OF DESIGN ASPHALT MIXTURE USING AASHTO T283

HHI-10/1-98(2M)QE