Preface

The *road network in Kurdistan Region has witnessed over the last decade an exceptional increase in road lengths by new construction, reconstruction and maintenance of many second carriage ways along old primary roads and construction of new rural roads, that connecting all cities, towns and villages of the Region within this network of roads that lengths over 5500 km.*

This *quantitative development of expanding lengths in road pavement construction associated with quantitative distresses in Hot Mix Asphalt (HMA) pavements especially the three primary common distresses;* ***permanent deformation*** *or* ***rutting****;* ***fatigue cracking****, which leads to alligator cracking; and* ***low temperature cracking*** *which are related to many reasons :*

- *Increase in axle loads because of increase of traffic volume and not controlling the travelled heavy axle loads.*
- *Undue selection of materials of aggregate and asphalt binder.*
- *Inadequate mix design procedures by conventional Marshall method*
- *Poor construction practices during the construction .*

I prepared this thesis about FUNDAMENTALS OF SUPERPAVE MIX DESIGN because Superpave method is a distinct and unique method which addresses (treats) the above mentioned three dominant distresses in HMA pavements. To ensure a high quality and soundness HMA pavement I see the resolution is by using Superpave method, because :

- ***The*** *mixes designed by Superpave method taking in account the traffic loading expected as well as the historical climatic conditions of the location for pavement.*
- ***Asphalt cement*** *in Superpave called Performance Graded (PG) binders, which is selected on the basis of climate and traffic loads at the location where it will be used.*
- ***The Superpave mixes*** *designed to match the expected traffic loads and the high and low expected pavement temperatures of the pavements.*
- ***Superpave mixes*** *have a strong aggregate structure which, in general, results in a coarser aggregate blend (which has a much greater stone-on-stone contact than standard mixes) and lower asphalt contents than standard mixes (Marshall Mixes).*
- ***The*** *Superpave laboratory Gyratory Compacter (SGC) which is the heart of the new mix design in Superpave method simulates the compactive effort of the pavement rollers.*

Engineer: Kamal Mohammed Mahdi

Kurdistan Engineering Union

Membership No. : 7822 in 8/2/2007

21/5/2016

List of Contents

INTRODUCTION

The name of **Superpave** comes from **SU**perior **PER**forming **PAVE**ments. Superpave is a new, Comprehensive asphalt mix design and analysis system, was developed through research performed during the **Strategic Highway Research Program (SHRP)**. United States 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. 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.

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.

The Superpave mix design method was designed to replace the Hveem and Marshall methods. The volumetric analysis common to the Hveem and Marshall methods provides the basis for the Superpave mix design method. The Superpave system ties asphalt binder and aggregate selection into the mix design process, and considers traffic and climate as well. The compaction devices from the Hveem and Marshall procedures have been replaced by a ***gyratory compactor*** and the compaction effort in mix design is tied to expected traffic.

Hveem Mix Design:

Francis Hveem of the California Department of Transportation developed the Hveem mix design procedure in 1930. Hveem and others refined the procedure, which is detailed in ASTM D 1560, *Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus*, (AASHTO T246) and ASTM D 1561, *Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor* (AASHTO T247). The Hveem method is not commonly used for HMA outside the western United States. The Hveem method also entails a **density/voids** and **stability analysis**. **The mixture's resistance to swell in the presence of water is also determined**.

Advantages of the Hveem method

1- The kneading method of laboratory compaction is thought to better simulate the densification characteristics of HMA in a real pavement.

2- Hveem stability is a direct measurement of the internal friction component of shear strength. It measures the ability of a test specimen to resist lateral displacement from application of a vertical load.

Disadvantage of the Hveem Method:

1- The testing equipment is somewhat expensive and not very portable.

2- Some important mixture volumetric properties that are related to mix durability are not routinely determined as part of the Hveem procedure.

3- Some engineers believe that the method of selecting asphalt content in the Hveem method is too subjective and may result in non-durable HMA with too little asphalt.

Marshall Mix design:

Most agencies currently use the Marshall mix design method. It is by far the most common procedure used in the world to design HMA. Developed by Bruce Marshall of the Mississippi State Highway Department the U.S. Army Corps of Engineers refined and added certain features to Marshall's approach in the 1940s for designing asphalt mixtures for airfield pavements, and it was formalized as ASTM D 1559, *Resistance to Plastic Flow of Bituminous Mixtures Using the Marshall Apparatus* (AASHTO T 245). The Marshall method entails a laboratory experiment aimed at developing a suitable asphalt mixture using **stability/flow** and **density/voids analyses**.

Advantages of Marshall Method:

1- One of the strengths of the Marshall method is its attention to density and voids properties of asphalt materials. This analysis ensures the proper volumetric proportions of mixture materials for achieving a durable HMA.

2- The required equipment is relatively inexpensive and portable.

Disadvantage of Marshall Method:

1- Many engineers believe that the impact compaction used with the Marshall method does not simulate mixture densification as it occurs in a real pavement.
2- Marshall stability does not adequately estimate the shear strength of HMA.
These two situations make it difficult to assure the rutting resistance of the designed mixture. Consequently, asphalt technologists agree that the Marshall method has outlived its usefulness for modern asphalt mixture design.

Superpave

The highways in the United States are continuously subjected to increasing traffic volumes, loads, and tire pressure. In 1982, the legal load limit was increased from 326 to 356 kN (73,280 to 80,000 lbs), resulting in approximately 40 to 50 percent higher pavement stresses for a given pavement structure. The use of radial tires further increased the stress levels in the pavement structure. These changes resulted in an accelerated rate of pavement deterioration as pavement distresses such as rutting and fatigue developed more quickly. It became clear that older methods of pavement design and materials selection needed updating in order to extend the service life of HMA pavements.

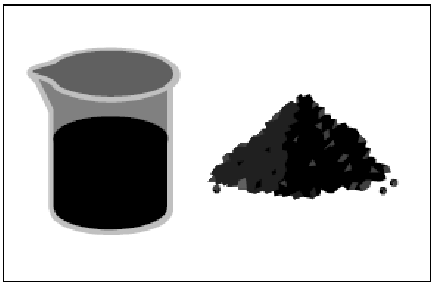
Superpave includes a new mixture design and analysis system based on performance characteristics of the pavement. It is a multi-faceted system with a tiered approach to designing asphalt mixtures based on desired performance. Superpave includes some old rules of thumb and some new and mechanistic based features. The Superpave mix design system is quickly becoming the standard system used in the United States (US), because of looking for a new system to overcome pavement problems such as rutting and low temperature cracking that had become common with the use of design systems such as Marshall and Hveem. The Superpave system offers solutions to these problems through a rational approach.

The Superpave system builds from the simple to the complex. The extent to which the designer utilizes the system is based on the traffic and climate for the pavement to be built. 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.
For low volume roads in moderate climates a simple system using materials selection and volumetric mix design is used. As the traffic level for the road to be designed increases the design requirements increase to improve reliability. At the higher traffic levels a complex system of extensive performance testing is recommended to assure the highest reliability.

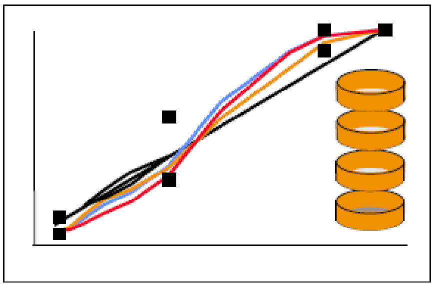
A unique feature of the Superpave system is that its tests are performed at temperatures and aging conditions that more realistically represent those encountered by in-service pavements
However, even with proper adherence to these procedures and the development of mix design criteria, asphalt technologists 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***.

Consequently, SHRP researchers set out to develop a chemically based asphalt binder specification and investigate improved methods of mix design. 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.

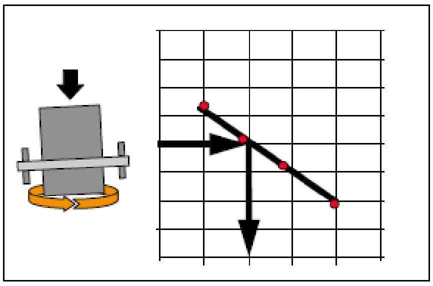
If the pavement distresses addressed by Superpave (rutting, fatigue cracking, and low temperature cracking) that occur in the pavement, that occur at relatively typical stages in a pavement's life and under relatively common temperature conditions. The Superpave performance graded (PG) binder specification 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 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** as shown in the following figures.



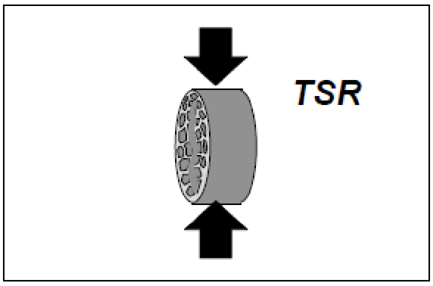
1. Materials Selection



2. Design Aggregate Structure



3. Design Binder Content



4. Moisture Sensitivity

4 Steps of Superpave Mix Design

SUPERPAVE ASPHALT MIX DESIGN

Key features in the Superpave system are laboratory compaction and testing for mechanical properties. Laboratory compaction is accomplished by means of a **Superpave Gyratory Compactor (SGC)**. It is a completely new device with new operational characteristics. Its main utility is to fabricate test specimens. The SGC can help avoid mixtures that exhibit tender mix behavior or densify to dangerously low air void contents under the long-term action of traffic.
The SHRP asphalt research program developed a number of HMA performance prediction tests. Output from these tests will eventually be used to make detailed predictions of pavement performance.

In other words, test procedures and the final performance prediction models will allow an engineer to estimate the performance life of HMA in terms of equivalent axle loads (ESALs) or time to achieve a certain level of rutting, fatigue cracking, and low temperature cracking. This integrated mixture and structural analysis system will allow the designer to evaluate and compare the costs associated with using various materials and applications

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 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.

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. The objectives will be to describe the material properties of HMA, both of the individual components of HMA (asphalt and aggregate) and the HMA mixture itself. Superpave system uses the tests and specifications to improve the three primary distresses in HMA pavements: **permanent deformation**, **fatigue cracking** and **low temperature cracking**.

For a comprehensive understanding of Superpave Asphalt Mixture Design we have to study the following topics in the following Chapters:

CHAPTER ONE SUPERPAVE BINDERS

CHAPTER TWO SUPERPAVE MINERAL AGGREGATES

CHAPTER THREE SUPERPAVE ASPHALT MIXTURE

CHAPTER FOUR SUPERPAVE MIX DESIGN

CHAPTER ONE
SUPERPAVE BINDERS

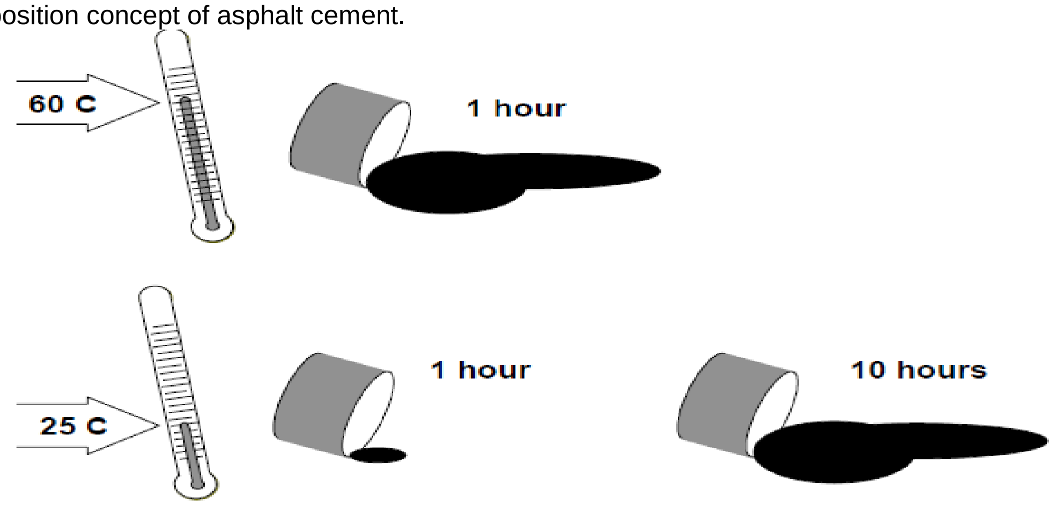
To understand Superpave binders in Superpave mixture design we have to know:

I. SUPERPAVE BINDER PROPERTY MEASUREMENTS
II. SUPERPAVE BINDER TESTS
III. SUPERPAVE BINDER SPECIFICATION
IV. SUPERPAVE BINDER SELECTION

I. SUPERPAVE BINDER PROPERTY MEASUREMENTS

I.1 HOW ASPHALT BEHAVES

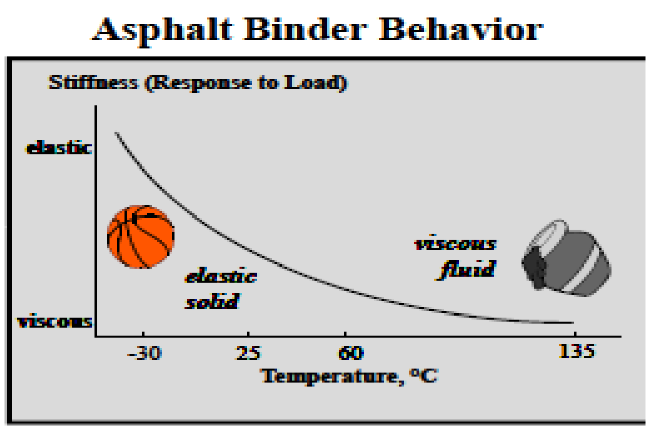
Asphalt is a *viscoelastic* material. This term means that asphalt has the properties of both a viscous material, such as motor oil, or more realistically, water, and an elastic material, such as a rubber. However, the property that asphalt exhibits, whether viscous, elastic, or most often, a combination of both, depends on *temperature* and *time of loading*. The flow 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.

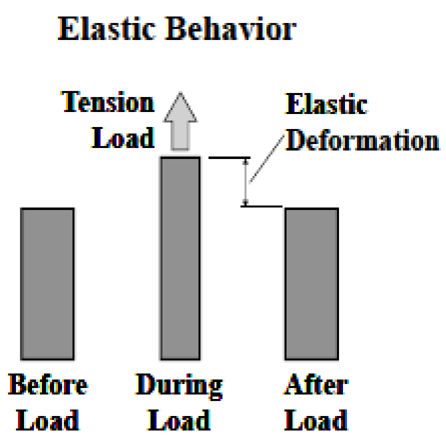


I.1.1- High Temperature Behavior

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.





I.1.2- 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).

I.1.3- 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. Most of the response is elastic or viscoelastic, (recoverable with time), while some of the response is plastic and non-recoverable.

I.1.4- 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 batching plant), 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.

II- SUPERPAVE BINDER TESTS

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

- describe the Superpave binder test equipment
- discuss where the tests fit into the range of material conditions (temperature and aging conditions)
- 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

II.1- Binder Aging Methods

An important thing 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:

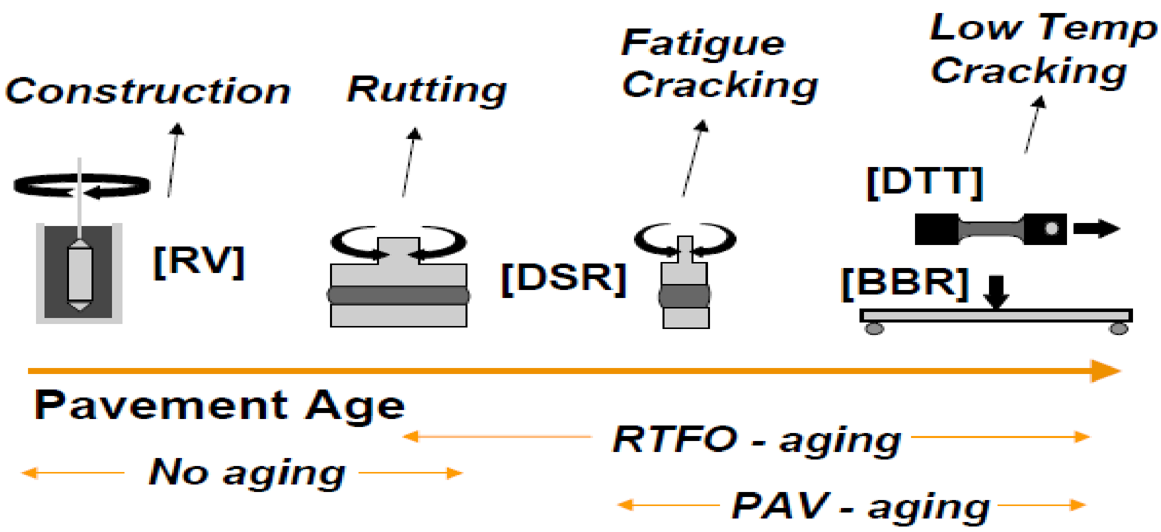
- during transport, storage, and handling,
- during mix production and construction, and
- after long periods in a pavement

Tests performed on **unaged** asphalt such as penetration and viscosity 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.

Superpave Binder Test	Purpose
Dynamic Shear Rheometer (DSR)	Measure properties at high and intermediate temperatures
Rotational Viscometer (RV)	Measure properties at high temperatures
Bending Beam Rheometer (BBR) Direct Tension Tester (DTT)	Measure properties at low temperatures
Rolling Thin Film Oven (RTFO) Pressure Aging Vessel (PAV)	Simulate hardening (durability) characteristics

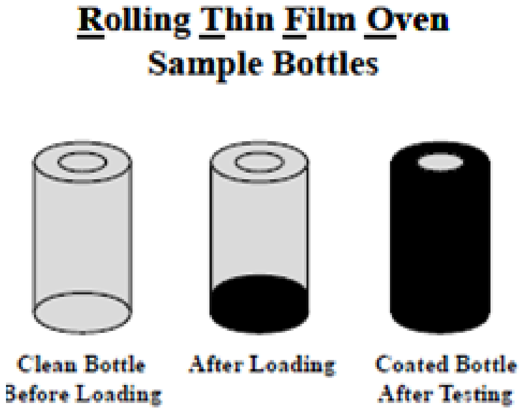


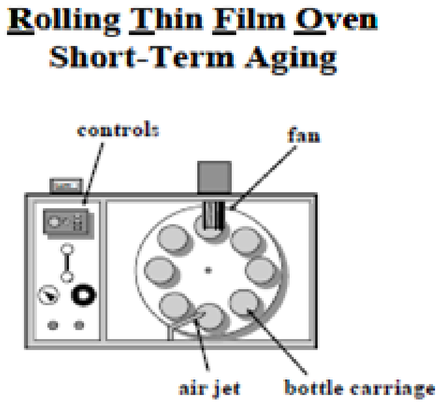
II.2- Required Superpave Binder Tests

II.2.1- Rolling Thin Film Oven (RTFO)

Specific equipment details can be found in AASHTO T 240, “*Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin Film Oven Test*).”

- RTFO bottles are loaded with 35 + 0.5 g of binder. The RTFO test temperature must be, 163 ± 0.5 C
- The device should be started and rotated at a rate of 15 + 0.2 rev/min.
  The air flow should be set at a rate of 4000 + 200 ml/min
- Time of Test: The samples are maintained under these conditions for 85 minutes

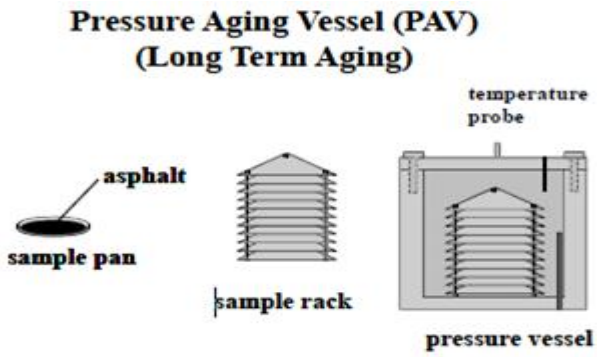




II.2.2- Pressure Aging Vessel (PAV)

Specific equipment details can be found in AASHTO PP1, *”Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)”*.

- Pressure test 2070 Pa
- Temperature test (90°, 100°, or 110° C)
- The Vessel must accommodate at least 10 sample pans.
- Each PAV sample should weigh 50 g, taken from residue of approximately two RTFO bottles
- Test time 20 hours.
- After the test, the pans are removed from the sample holder and placed in an oven at 163° C for 15 minutes.



II.2.3- Rotational Viscometer

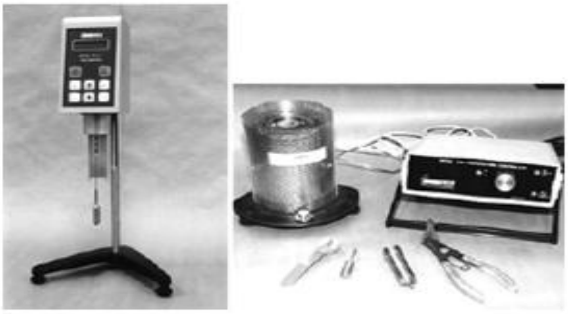
This method of measuring viscosity is detailed in AASHTO TP48, *"Viscosity Determination of Asphalt Binders Using Rotational Viscometer."*

Rotational viscosity is used to evaluate high temperature workability of binders. Rotational viscosity is measured on unaged or "tank" asphalt. Rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle while submerged in a sample at a constant temperature.

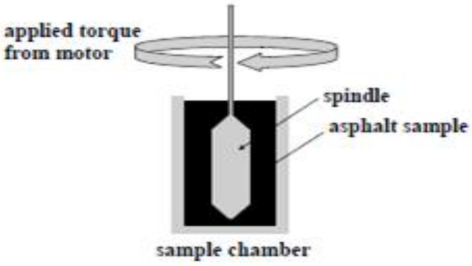
- Typically, less than 11 grams are used.
- Test temperature 150° C
- For most rotational viscometers and specification testing, the motor should be set at 20 rpm
- Many binders can be tested with only two spindles: Nos. 21 and 27. The spindle No. 27 is used most.

- Three viscosity readings should be recorded at 1-minute intervals. The viscosity at 135° C is reported as the average of three readings. Viscosity in units of centipoise (cP) while the Superpave binder specification uses Pa-s. 1000 cP = 1 Pa-s
- In some cases, it may be desirable to determine binder viscosity at temperatures other than 135° C, may be to the desired temperature, such as 165° C, to be equiviscous temperatures for mixing and compaction during mix design.

Rotational Viscometer



Rotational Viscometer

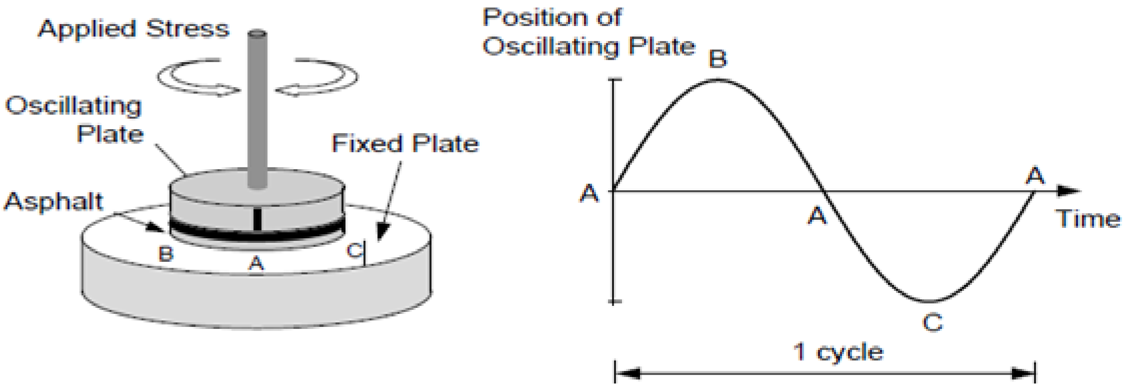


II.2.4- Dynamic Shear Rheometer (DSR)

Operational details of the DSR can be found in AASHTO TP5 *"Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer."*

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. (see the Figure below).

The thickness of gap used depends on the test temperature and the aged condition of the asphalt. Unaged and RTFO aged asphalt, tested at high temperatures of 46°C or greater, require a small gap of 1000 microns (1 mm). PAV aged asphalts, tested at intermediate test temperatures, in the range of 4° to 40°C, require a larger gap of 2000 microns (2 mm). High temperature tests require a large spindle (25 mm), and intermediate test temperatures require a small spindle (8 mm). The Superpave specifications require the oscillation speed to be 10 radians/second, which is approximately 1.59 Hz.

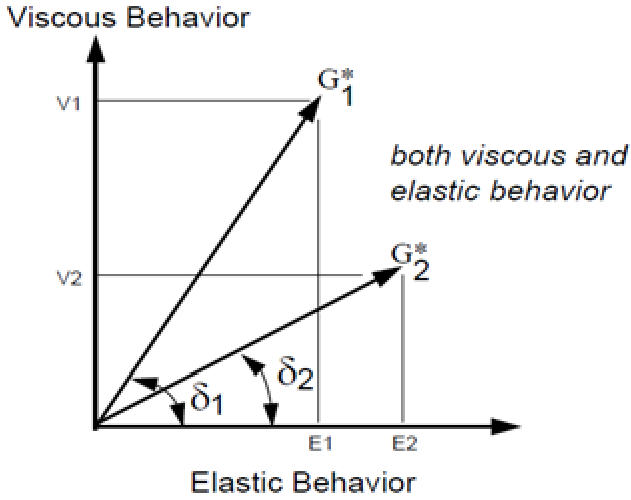


Shear strain values vary from one to12 percent and depend on the stiffness of the binder being tested. Relatively soft materials tested at high temperatures, (e.g., unaged binders and RTFO aged binders) are tested at strain values of approximately ten to twelve percent. Hard materials (e.g., PAV residues tested at intermediate temperatures) are tested at strain values of about one percent.

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 bindersAs 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 test 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.



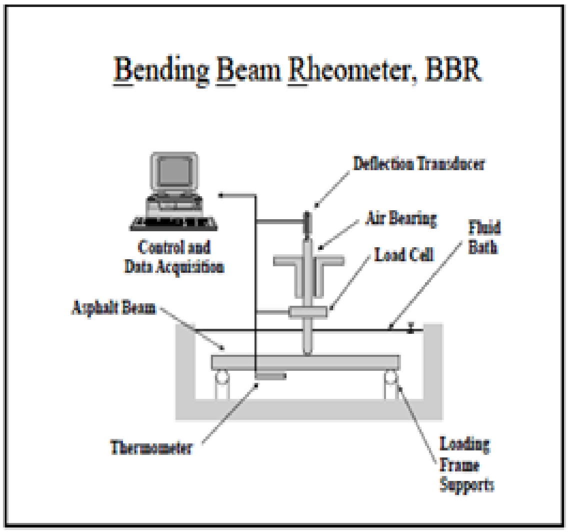
G* is the ratio of maximum shear stress (τ max) to maximum shear strain (γ max), and it is a measure of the total resistance of a material to deforming when repeatedly sheared. It consists of two parts: a part that is elastic (temporary deformation) as shown by the horizontal arrow, and a part that is viscous (permanent deformation) as indicated by the vertical arrow. δ , the angle made with the horizontal axis, indicates the relative amounts of temporary and permanent deformation. In this example, even though both asphalts are viscoelastic, asphalt 2 is more elastic than asphalt 1 because of its smaller δ . By determining both **G*** and δ , the DSR provides a more complete picture of the behavior of asphalt at pavement service temperatures.

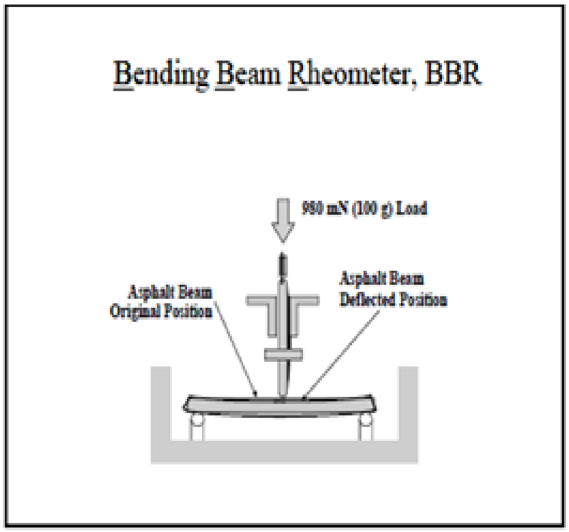
For asphalt, the values of **G*** and δ are highly dependent on the temperature and frequency of loading. Therefore, it is important to know the climate of the project where the pavement is being constructed, as well as the relative speed of the traffic to be using the facility.

II.2.5- Bending Beam Rheometer (BBR)

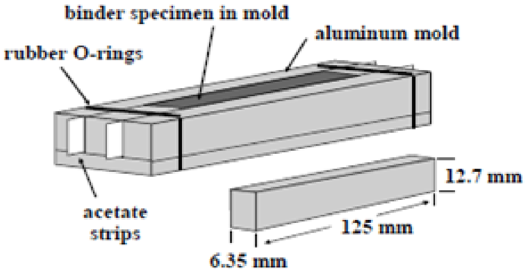
Details of the BBR test procedure can be found in AASHTO TP1 *"Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)."*

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.

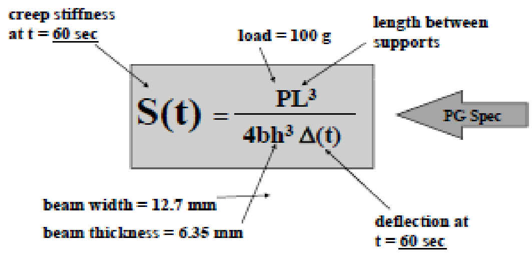




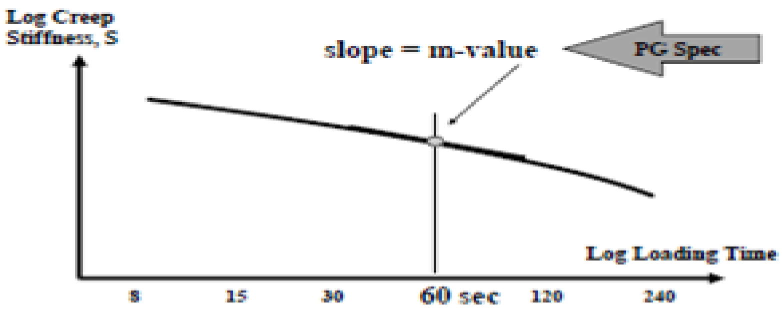
Bending Beam Rheometer, BBR



BBR Data - Stiffness



BBR Data - Relaxation

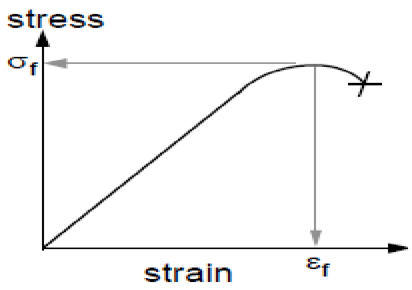


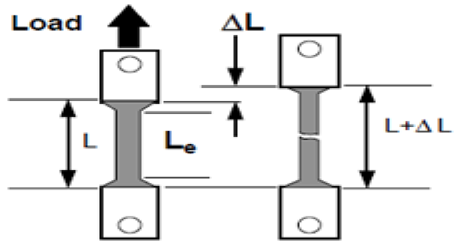
- The temperature bath contains a fluid consisting of ethylene glycol, methanol, and water.
- After molding asphalt beams they must be cooled for period about 45 to 60 minutes.
- To remove the molds of the specimens, cool the assembly in a freezer or ice bath at –5º C for five to ten minutes.
- Then the asphalt beams are placed in the test bath for 60 ± 5 minutes. At the end of this period the beams may be tested.
- At the end of the 60-minute thermal conditioning period, the asphalt beam is placed on the supports by gently grasping it with forceps.
- A 30 ± 5 mN preload is manually applied by the operator to ensure that the beam is firmly in contact with the supports. A 100-gram (980 mN) seating load is automatically applied for one second by the rheometer software. After this seating step, the load is automatically reduced to the preload for a 20-second recovery period. At the end of the recovery period, apply a test load ranging from 980 ± 50 mN, and maintain the load constant to ± 50 mN for the first five seconds and ± 10 mN for the remainder of the test. The deflection of the beam is recorded during this period.
- At the end of 240 seconds, the test load is automatically removed and the rheometer software calculates creep stiffness and m-value The m-value is the slope of the log stiffness versus log time curve at any time,t

II.2.6- Direct Tension Tester (DTT)

The test equipment and procedure are detailed in AASHTO TP3 *“Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT).”*

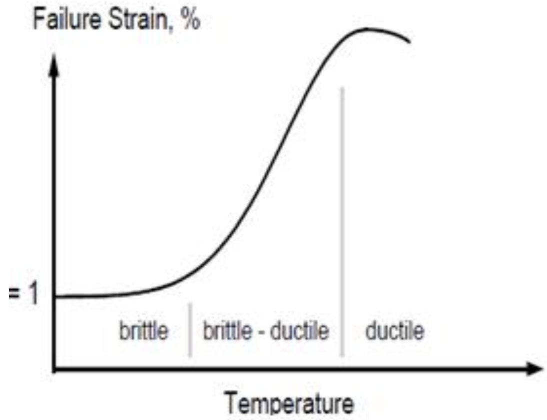
The direct tension test measures the low temperature ultimate tensile strain of an asphalt binder. The test is performed at relatively low temperatures ranging from +6° to -36° C, the temperature range within which asphalt exhibits brittle behavior. Furthermore, the test is performed on binders that 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.

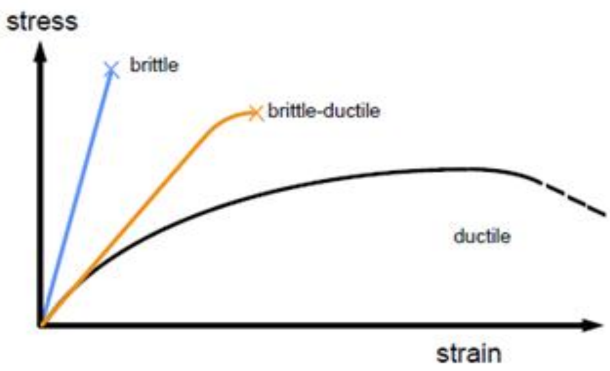




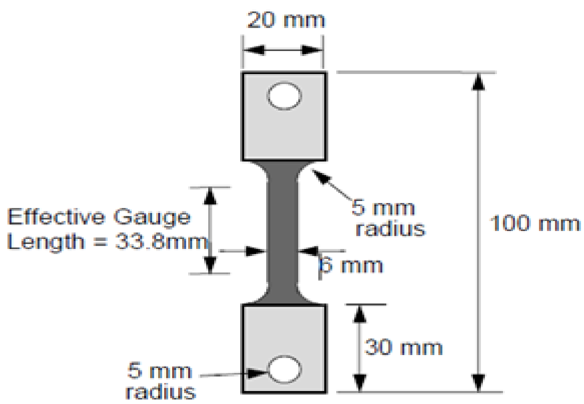
$$\text{failure strain } (\varepsilon_f) = \frac{\text{change in length } (\Delta L)}{\text{effective length } (L_e)}$$

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 (**Δ L**) divided by the effective gauge length (L).
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**.





This Figure shows the characteristic stress-strain relationships of three different lines that represent the same asphalt tested at multiple temperatures or different asphalts tested at the same temperature. Brittle behavior means that the asphalt very quickly picks up load and elongates only a small amount before it cracks. An asphalt that is ductile may not even crack in the direct tension test but rather "string-out" until its elongation exceeds the stroke of the loading frame.
That is why the point at which the specimen stops picking up load, which is the strain at peak stress, defines tensile failure strain.



III- SUPERPAVE BINDER SPECIFICATION

The Superpave asphalt binder specification is intended to improve performance by limiting *permanent deformation, low temperature cracking* and *fatigue cracking* in asphalt pavements. Specification provides for this improvement by designating various physical properties that are measured with the equipment described previously.

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

III.1- Permanent Deformation (Rutting)

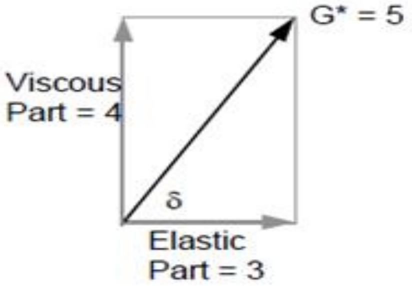
As discussed earlier in the section describing the DSR, the total response of asphalt binders to load consists of two components: elastic (recoverable) and viscous (non-recoverable). Pavement rutting or permanent deformation is the accumulation of the non-recoverable component of the responses to load repetitions at high service temperatures. So Superpave solves rutting using unaged binder and binder aged in the RTFO.

Viscosity, ASTM D 4402:[b] **Maximum, 3 Pa-s (3000 cP)** **Test Temp. C**	
Dynamic Shear, AASHTO TP5 **G*/sin δ, Minimum, 1.00 kPa** **Test Temp. @ 10 rad/s, C**	**Specification Requirements to Address Rutting**
Rolling Thin Film Oven AASHTO 240	
Mass Loss, Maximum %	
Dynamic Shear, AASHTO TP5 **G*/sin δ, Minimum, 2.20 kPa** **Test Temp. @ 10 rad/s, C**	**Specification Requirements to Address Rutting**

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.

Higher values of **G*** and lower values of δ are considered desirable from the standpoint of rutting resistance. For the two materials A and B shown below 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.

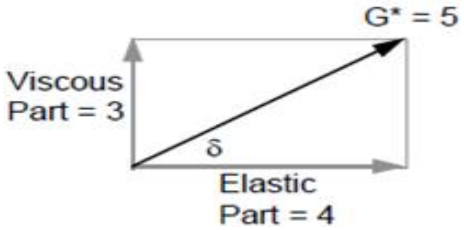
Material A



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

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

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

III.2- 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.

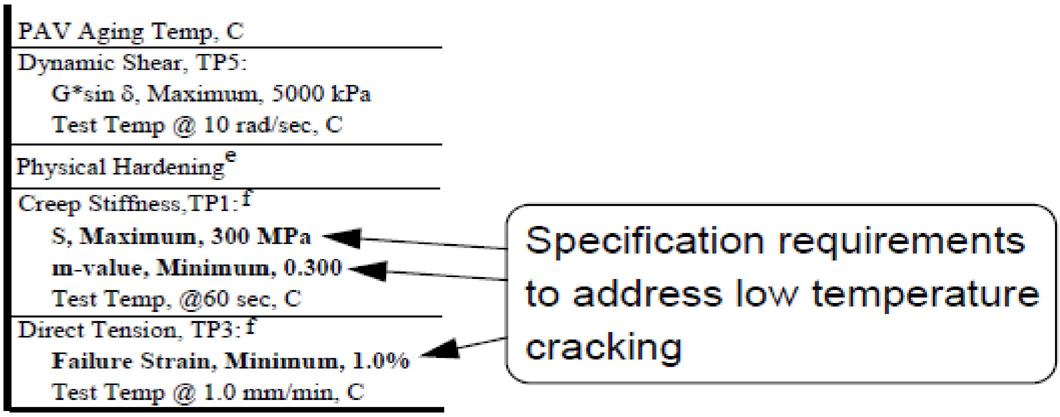
The DSR is again used to obtain **G*** and sin δ. However, instead of dividing the two parameters, the two are multiplied to produce a factor related to fatigue. The fatigue cracking factor is **G*sin δ**, which is called **"G star sine delta,"** or the **intermediate temperature stiffness**. It is the 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

III.3- 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. 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.



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.

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.

IV- SUPEPAVE BINDER SELECTION

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 must have a database of weather information from weather stations that allow 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 will not be used.

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:

SHRP High-Temperature Models

$$T\,(surf) = T\,(air) - 0.00618\,Lat^2 + 0.2289\,Lat + 24.4$$

Where: T (surf) = High pavement temperature at the surface, °C
T (air) = Air temperature, °C
Lat = Latitude of the section, degrees

$$T(d) = T(surf)\,(1 - 0.063\,d + 0.07\,d^2 - 0.0004\,d^3)$$

where: T(d) = High pavement temperature at a depth, d, in mm, °C

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

where ; **T20mm** = high pavement design temperature at a depth of 20 mm
T air = seven-day average high air temperature
L at = the geographical latitude of the project in degrees.

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

SHRP Low-Temperature Model

$$T\,(d) = T\,(air) + 0.051\,d - 0.000063\,d^2 \quad (4)$$

where: T(d) = Low pavement temperature at a depth, d, in mm, °C
T (air) = Air temperature, °C
d = Depth in pavement in mm

Federal Highway Administration (FHWA) recommended the adoption of the Long-Term Pavement Performance (LTPP) Program's new algorithms based upon the following rationale:
The current SHRP low-pavement-temperature algorithm does not correctly determine the low pavement temperature from the air temperature. The FHWA LTPP program has developed a new low-pavement-temperature algorithm from their weather stations at over 30 sites all over North America. The Binder Expert Task Group feels the LTPP algorithm is far more accurate and should be used in all AASHTO documents. Data supporting the LTPP algorithm is presented in *LTPP Seasonal Asphalt Concrete Pavement Temperature Models*, FHWA-RD-97- 103, September, 1998.
The LTPP proposed algorithms are as follows:

LTPP High-Temperature Model with Reliability

$$T\,(pav) = 54.32 + 0.78\,T(air) - 0.0025\,Lat^2 - 15.14\,\log_{10}(H + 25) + z\,(9 + 0.61\,\sigma_{air}^2)^{1/2} \quad (5)$$

where: T (pav) = High pavement temperature below the surface, °C
T (air) = High air temperature, °C
Lat = Latitude of the section, degrees
H = Depth from surface, mm
σ_{air} = Standard deviation of the high 7-day mean air temperature, °C
z = From the standard normal distribution table, z=2.055 for 98% reliability

LTPP Low-Temperature Model with Reliability

$$T(pav) = -1.56 + 0.72\,T(air) - 0.004\,Lat^2 + 6.26\,\log_{10}(H + 25) - z\,(4.4 + 0.52\,\sigma_{air}^2)^{1/2} \quad (6)$$

where: T (pav) = Low pavement temperature below the surface, °C

T (air) = Low air temperature, °C
Lat = Latitude of the section, degrees
H = Depth from surface, mm
σ_{air} = Standard deviation of the high 7-day mean air temperature, °C
z = From the standard normal distribution table, z=2.055 for 98% reliability

The average 7-day maximum pavement temperature (T_{max}) and the minimum pavement temperature (T_{min}) define the binder laboratory test temperatures. A factor of safety can be incorporated into the performance grading system based on temperature reliability. The 50 % reliability temperatures represent the straight average of the weather data. The 98 % reliability temperatures are determined based on the standard deviations of the low ($\sigma_{Low\ Temp}$) and high ($\sigma_{High\ Temp}$) temperature data. From statistics, 98 % reliability is two standard deviations from the average value, such that:

$$T_{max\ at\ 98\%} = T_{max\ at\ 50\%} + 2 * \sigma_{High\ Temp}$$

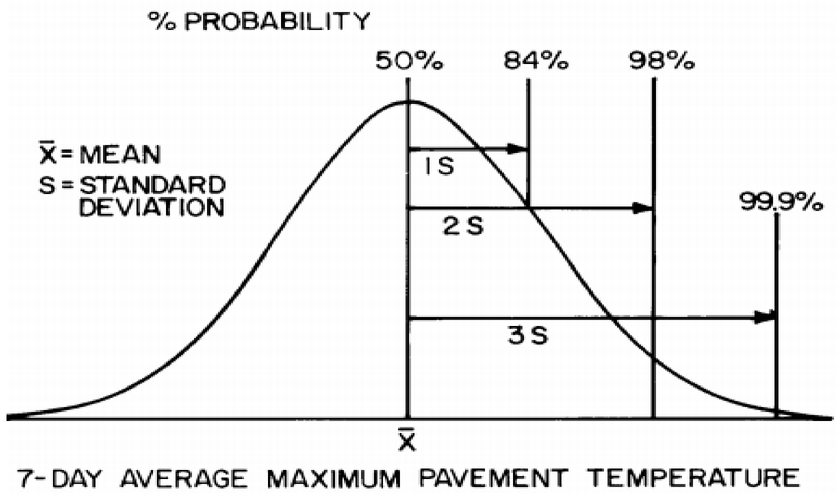
$$T_{min\ at\ 98\%} = T_{min\ at\ 50\%} - 2 * \sigma_{Low\ Tem}$$

The correct performance grade of asphalt binder is determined in the Superpave paving mix design process through consideration of the climate and the type of traffic loading at the site of the paving project. The Superpave software guides the mix designer through this process.

The Superpave software calculates the distribution of design pavement temperatures from the air temperature data, and guides selection of the minimum required performance grade of asphalt binder that will satisfy the conditions. These distributions may be viewed along with the degrees of probable risk associated with the selection of any particular design temperature. Thus, a binder performance grade may be selected for the project that either minimizes the probable design risk for high or low temperature pavement performance, or accepts some higher degree of probable risk when required by agency policy for the class of highway, the cost, and other relevant factors.

The Figure below illustrates the relationship between the mean and the standard deviation of the pavement temperature distributions, and the probability that in a given year the actual temperature will not deviate beyond a certain value. Specifically, there is an 50 percent probability in any given year that the actual temperature will not deviate beyond the mean, an 84 percent probability that it will not deviate beyond the mean plus one standard deviation (1S), a 98 percent probability that it will not deviate beyond the mean plus two standard deviations (2S), and a 99.9 percent probability that it will not deviate beyond the mean plus three standard deviations (3S)

Typical Probability Distribution of the 7-Day Average Maximum Pavement Temperature

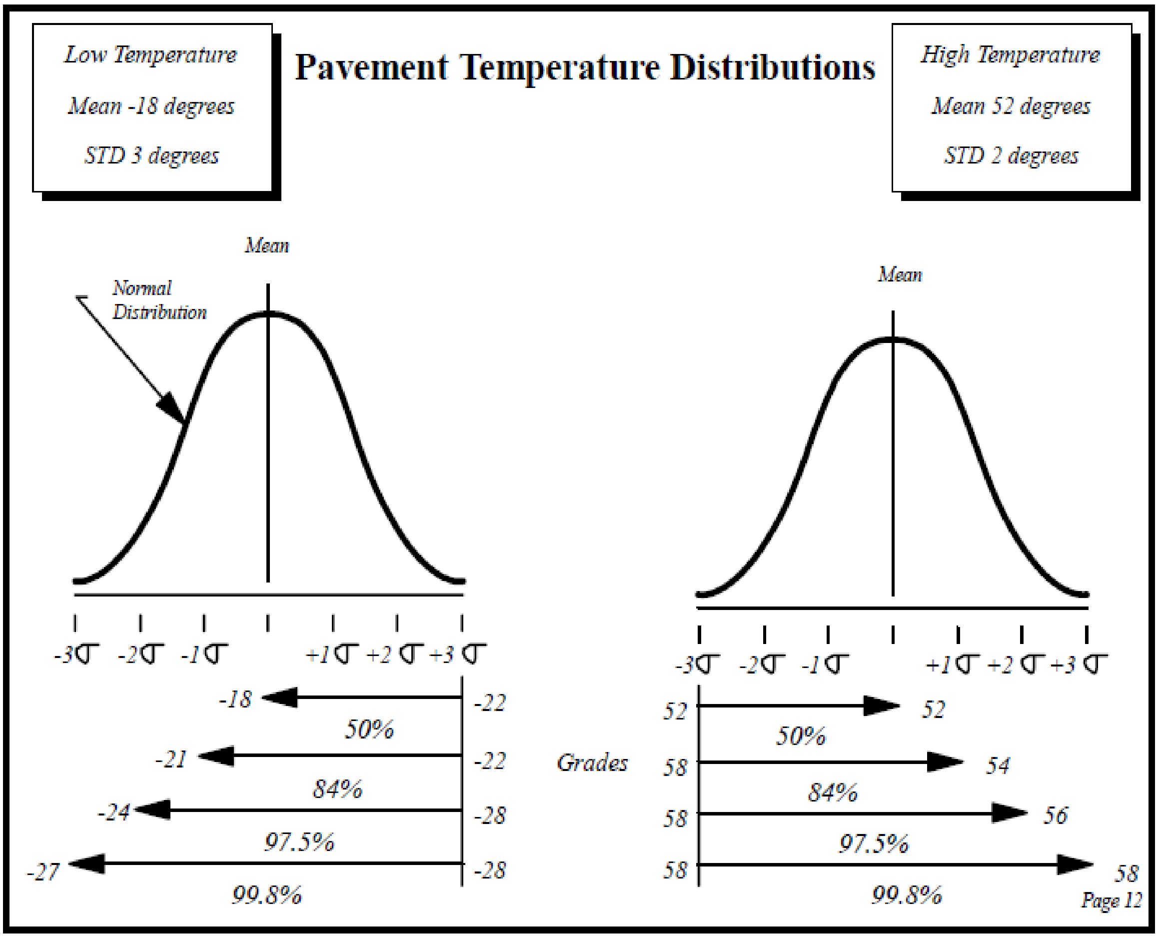


This procedure for performance grade selection assumes that the pavement will experience an average mix of car and truck traffic moving at moderate to high speeds (the fast transient condition). The following table permits an upward adjustment of the maximum design temperature-based performance grade to compensate for:

1- a larger than average proportion of slow-moving, heavy trucks or a frequent incidence of heavy standing loads;

2- expected traffic volumes in excess of 10^7 ESALs.

The Following Figure illustrates the Distributions of Pavement Temperature by FWHA



Selection of Asphalt Binder Performance Grades on the basis of Climate, Traffic Speed, and Traffic Volume

RECOMMENDATION FOR SELECTING BINDER PERFORMANCE GRADES

HIGH PAVEMENT DESIGN TEMPERATURE ° C

LOADS

STANDING →		28 TO 34→	34 TO 40→	40 TO 46→	46 TO 52→	52 TO 58→	58 TO 64→	64 TO 70→
(50 K/H) SLOW TRANSIENT		34 TO 40	40 TO 46	46 TO 52	52 TO 58	58 TO 64	64 TO 70	70 TO 76
(100 K/H) FAST TRANSIENT		34 TO 46	46 TO 52	52 TO 58	58 TO 64	64 TO 70	70 TO 76	76 TO 82
LOW PAVEMENT DESIGN TEMPERATURE °C	> -10	PG 46-10	PG 52-10	PG 58-10	PG 64-10	↓ PG 70-10	PG 76-10	PG 82-10
	-10 TO -16	PG 46-16	PG 52-16	PG 58-16	PG 64-16	↓ PG 70-16	PG 76-16	PG 82-16
	-16 TO -22	PG 46-22	PG 52-22	PG 58-22	PG 64-22	↓ PG 70-22	PG 76-22	PG 82-22
	-22 TO -28	PG 46-28	PG 52-28	PG 58-28	PG 64-28	PG 70-28	PG 76-28	PG 82-28
	-28 TO -34	PG 46-34	PG 52-34	PG 58-34	PG 64-34	PG 70-34	PG 76-34	PG 82-34
	-34 TO -40	PG 46-40	PG 52-40	PG 58-40	PG 64-40	PG 70-40		
	-40 TO -46	PG 46-46	PG 52-46	PG 58-46	PG 64-46			

1- Select the Type of Loading.

2- Move Horizontally to the High Pavement Design Temperature.

3- Move down the Low Pavement Design Temperature.

4- Identify the Binder Grade.

5- ESALS > 10^7 consider increase of one High Temperature Grade.

ESALS > 3* 10^7 increase one High Temperature Grade.

EXAMPLE

Standing Load, High Design Temperature = 57 ° C

Low Design Temperature = - 25 ° C

Grade = PG 70-28

AASHTO MP-2, Table Binder Selection on the Basis of Traffic Speed and Traffic Level

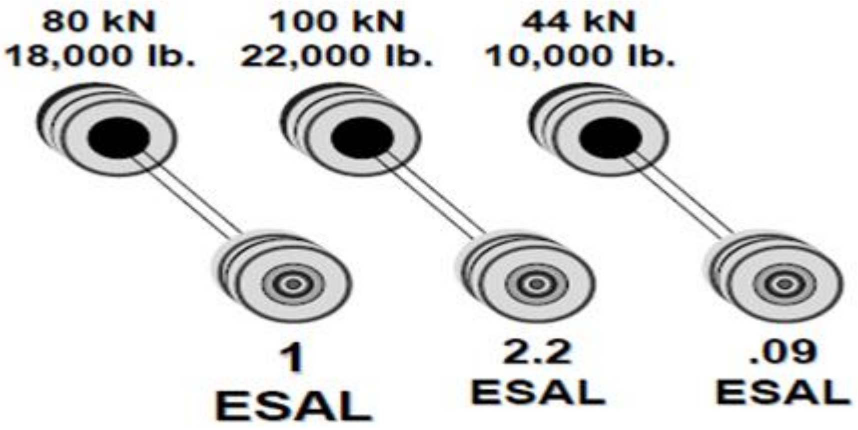
Design ESALs[b] (million)	Adjustment to Binder PG Grade[a]		
	Traffic Load Rate		
	Standing[c]	Slow[d]	Standard[e]
< 0.3	—[f]	-	-
0.3 to < 3	2	1	-
3 to < 10	2	1	-
10 to < 30	2	1	—[f]
≥ 30	2	1	1

a Increase the high temperature grade by the number of grade equivalents indicated (one grade is equivalent to 6ºC).
b The anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years.
c Standing Traffic - where the average traffic speed is less than 20 km/h.
d Slow Traffic - where the average traffic speed ranges from 20 to 70 km/h.
e Standard Traffic - where the average traffic speed is greater than 70 km/h.
f Consideration should be given to increasing the high temperature grade by one grade equivalent

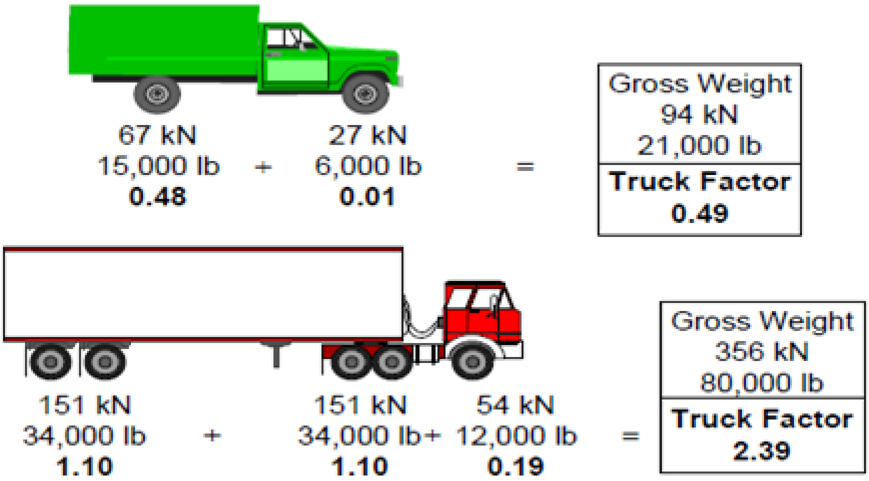
In summary, selecting a design asphalt binder grade requires the following steps carried out with the aid of the Superpave software:
1- Select weather stations in the vicinity of the paving project. Weather data from as many as three stations may be evaluated to estimate the climate at a paving site remote from established stations.
2- Select a degree of design reliability for high and low temperature performance. The reliability for a particular project is established by agency policy or assigned on the basis of the engineer's judgment of direct and indirect costs for maintenance and rehabilitation.
3- Estimate the design pavement temperatures corresponding to the assigned reliability at the location of the paving project.
4- Determine the minimum required performance grade of asphalt binder that will satisfy the selected maximum and minimum design pavement temperatures (and the associated risks).
5- For paving projects in locations that experience, slow or heavy truck traffic, frequent braking or acceleration of heavy vehicles, frequent, heavy standing loads, and (or) traffic volumes above 10^7 ESALs, adjust the performance grade determined in step 4 using the above mentioned Table

Traffic Analysis

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.

The difference between the high and low temperature grades can provide some indication whether the binder may be modified. A rule of thumb in the industry says if the difference is greater than 92, the binder may be modified, and the 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 (92) of this “rule”, such as the viscosity of the binder and the crude oil source.

CHAPTER TWO
SUPERPAVE MINERAL AGGREGATE

To understand Superpave Mineral Aggregate in Superpave mixture design we have to know:

I- MINERAL AGGREGATE BEHAVIOR
II- SUPERPAVE MINERAL AGGREGATE PROPERTY MEASUREMENTS
III- SUPERPAVE AGGREGATE GRADATION
IV- SUPERPAVE AGGREGATE TESTS AND SPECIFICATION

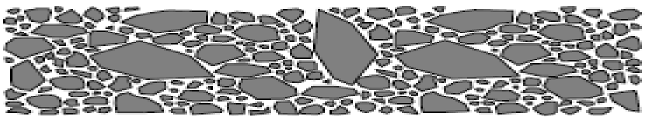
I- MINERAL AGGREGATE BEHAVIOR

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, quarry, or glacial deposits and are used without further processing to manufacture HMA. These are often called “bank-run” or “pit-run” materials.

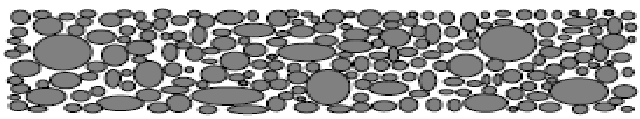
***Processed* aggregate** can include natural aggregate that has been separated into distinct size fractions, washed, crushed, or otherwise treated to enhance certain performance characteristics of the finished HMA. In most cases, the main processing consists of crushing and sizing.

***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. Regardless of the source, processing method, or mineralogy, the aggregate provision must have 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.

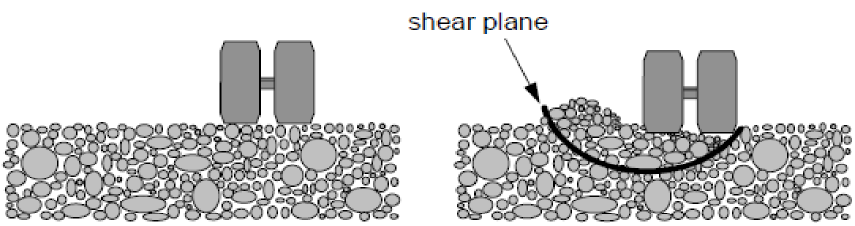


Cubical Aggregate



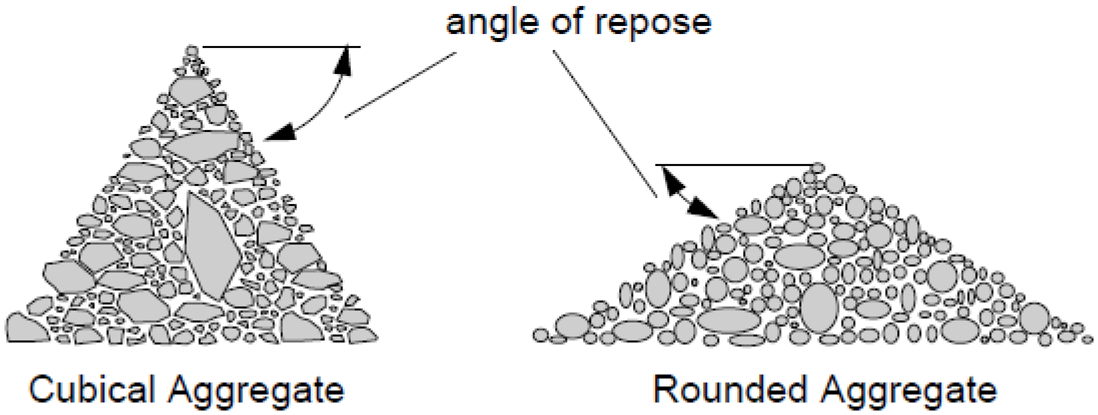
Rounded Aggregate

.



Before Load After Load

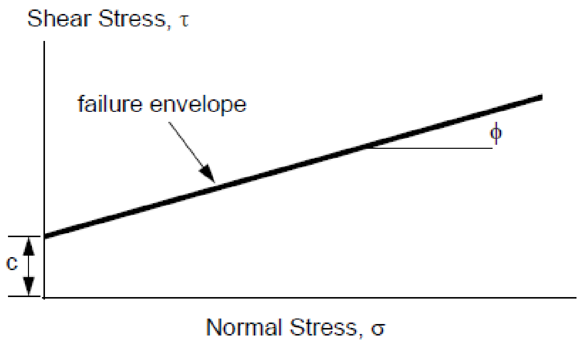
Aggregate shear strength behavior can easily be observed in aggregate stockpiles since crushed (i.e., mostly cubical) aggregates form steeper, more stable piles than rounded aggregates. The slope on stockpiles is the angle of repose. The angle of repose of a crushed aggregate stockpile is greater than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



The Engineers explain the shearing behavior of aggregates and other materials using Mohr-Coulomb theory. 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.



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.

II- SUPERPAVE MINERAL AGGREGATE PROPERTY MEASUREMENTS

During the SHRP research, pavement experts ascertained that 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***. 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.

II.1- Consensus Properties

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.

II.2- Source Properties

In addition to the consensus aggregate properties, pavement experts believed that certain other aggregate characteristics were critical, called source properties. A set of “source properties” was recommended. 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

III- SUPERPAVE AGGREGATE 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.

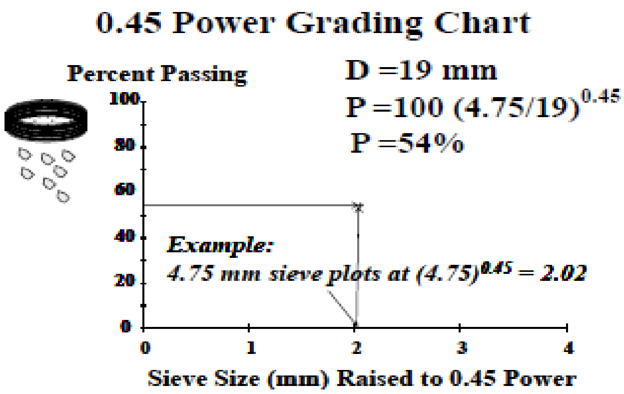
Starting Point for a Design Gradation
0.45 Power Gradation

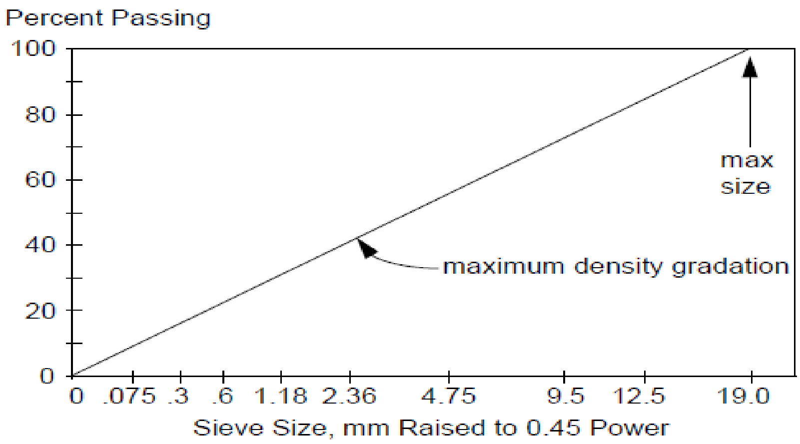
$$P = 100\left(\frac{d}{D}\right)^{0.45}$$

P: % Passing
d: size of aggregate
D: Max. aggregate size

Provides Maximum Density

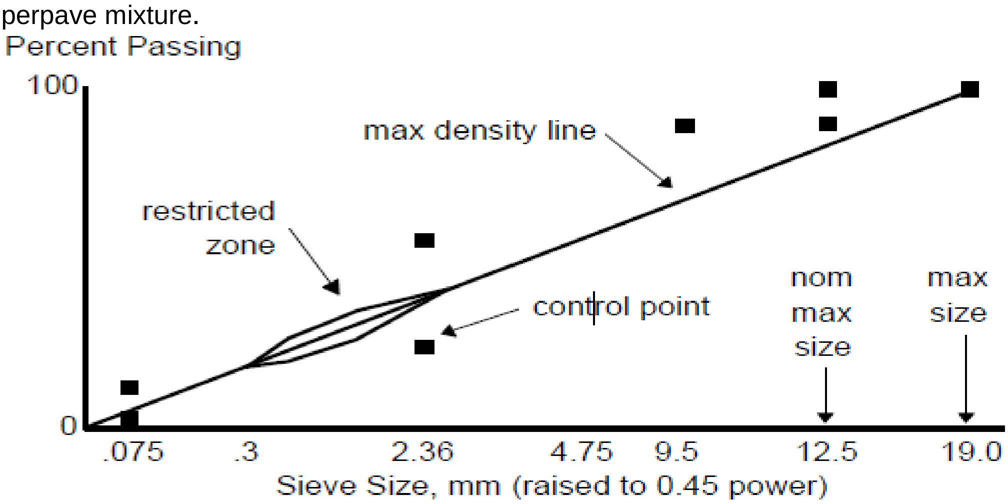
Pros	***Cons***
• High Strength	• Frost Damage
• Low Permeability	• Minimal space for Cement





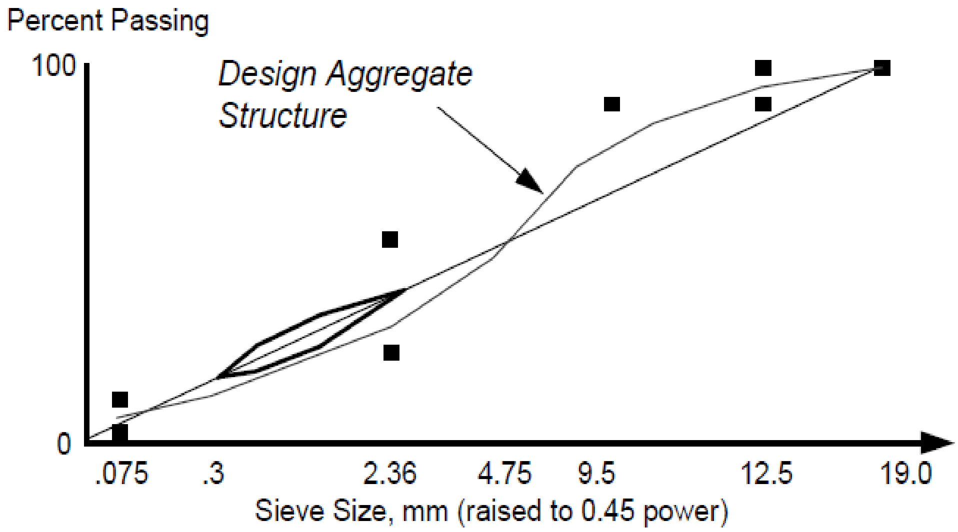
The maximum density gradation represents a gradation in which the aggregate particles fit together in their densest possible arrangement. Clearly this is a gradation to avoid because there would be very little aggregate space within which to develop sufficiently thick asphalt films for a durable mixture. The above Figure shows 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). The below illustration shows the control points and restricted zone for a 12.5 mm Superpave mixture.



The **restricted zone** is an area surrounding the maximum density line from (either 4.75 or 2.36 mm) sieve to the 0.3 mm sieve. Gradation should avoid passing through the restricted zone. 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 may 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



IV- SUPERPAVE AGGREGATE TESTS AND SPECIFICATIONS

IV.1- 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. 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/-	-/-
0.3 to < 3	75/-	50/-
3 to < 10	85/80	60/-
10 to < 30	95/90	80/75
$\geq$ 30	100/100	100/100

Note: "85/80" means that 85 % of the coarse aggregate has one fractured face and 80 % has two fractured faces.

Coarse Aggregate Angularity, CAA

0% Crushed

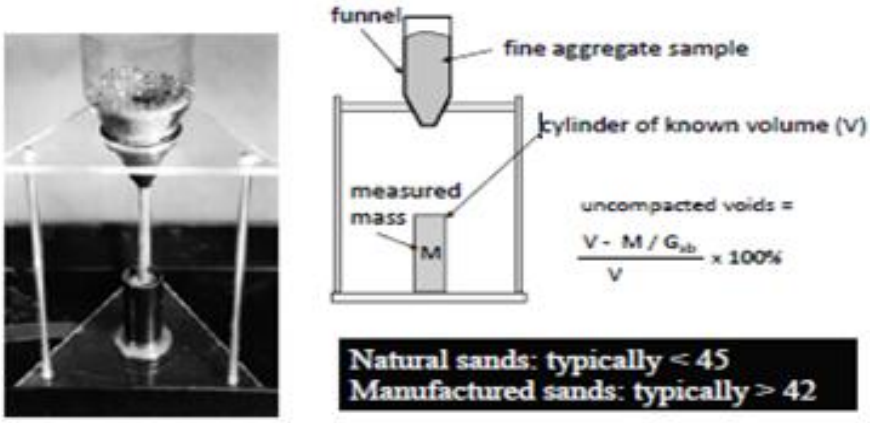
100% with 2 or More Crushed Faces

IV.2- 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.

Fine Aggregate Angularity, FAA

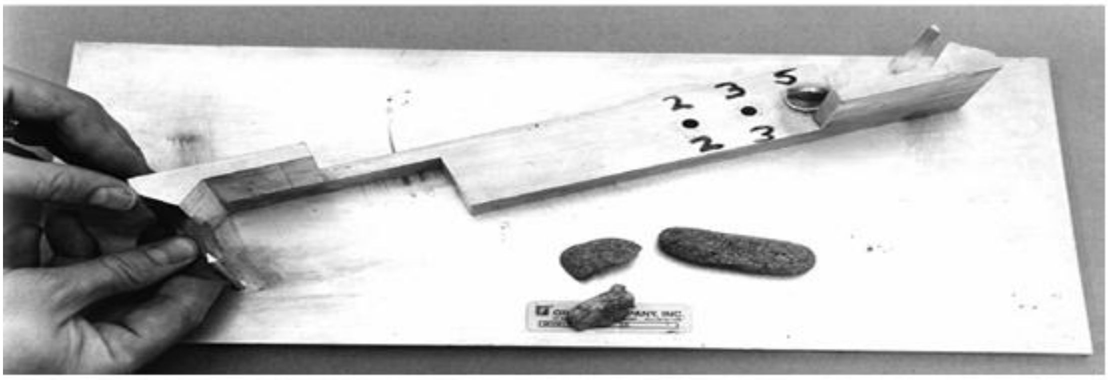


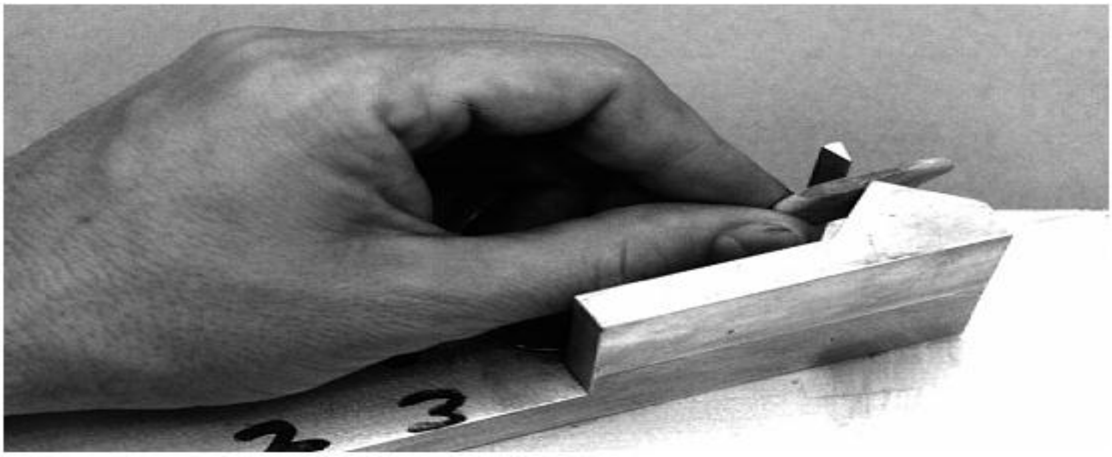
Superpave Fine Aggregate Angularity Requirements		
Traffic, million ESALs	Percent, Minimum	
	Depth from Surface	
	< 100 mm	> 100 mm
< 0.3	-	-
0.3 to < 3	40	40
3 to < 10	45	40
10 to < 30	45	40
≥ 30	45	45
Note: Criteria are presented as percent air voids in loosely compacted fine aggregate.		

IV.3- FLAT, ELONGATED PARTICLES

This characteristic is the percentage by weight of coarse aggregates that have a 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.

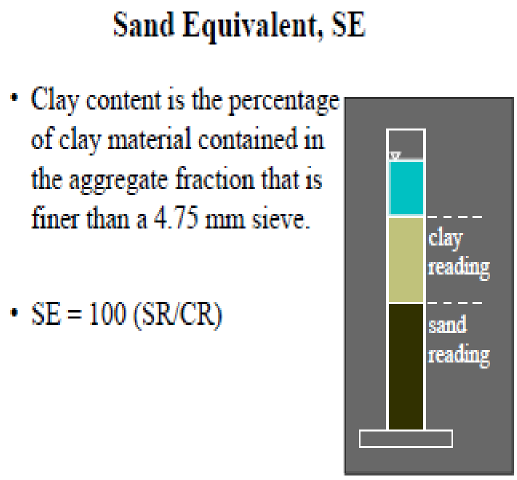
Superpave Flat, Elongated Particle Requirements	
Traffic, million ESALs	Percent, maximum
< 0.3	-
0.3 to < 3	10
3 to < 10	10
10 to < 30	10
≥ 30	10
Note: Criteria are presented as maximum percent by weight of flat and elongated particles.	





IV.4- 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*. 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
0.3 to < 3	40
3 to < 10	45
10 to < 30	45
$\geq$ 30	50

IV.5- 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.

IV.6- 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.

IV.7- DELETERIOUS MATERIALS

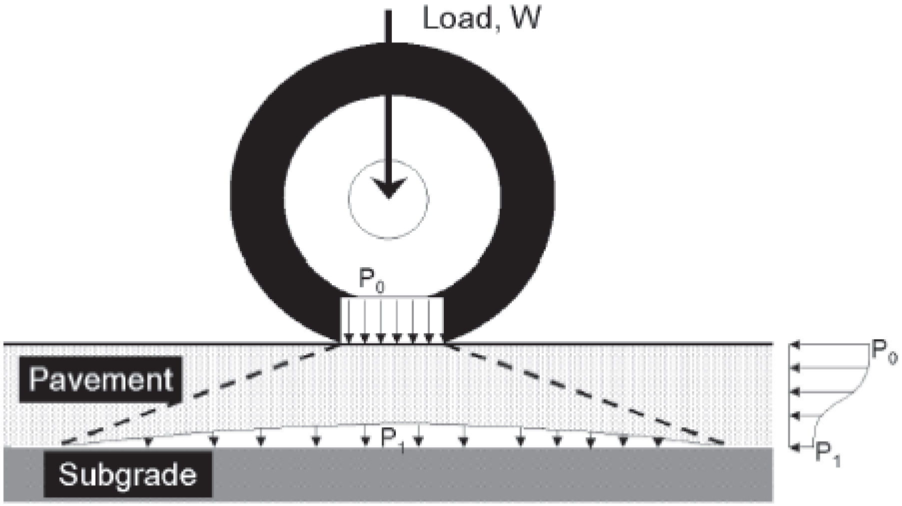
Deleterious materials are defined as the weight percentage of contaminants such as shale, wood, mica, and coal in the blended aggregate. This property is measured by AASHTO T 112, *"Clay Lumps and Friable Particles in Aggregates."* It can be performed on both coarse and fine aggregate. The test is performed 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.

CHAPTER THREE

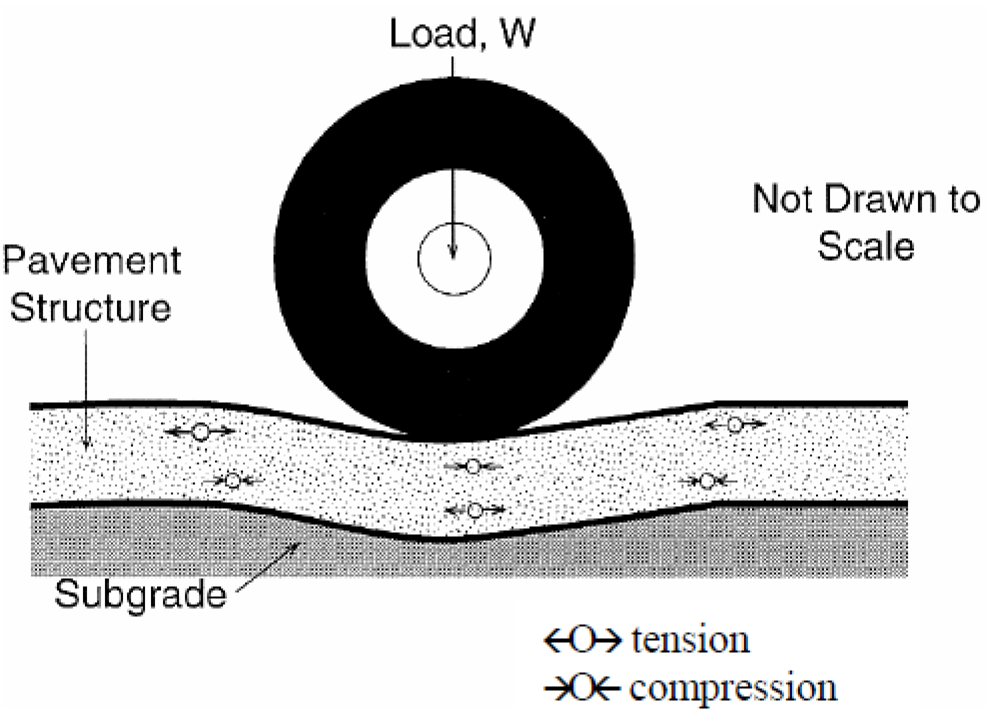
SUPERPAVE ASPHALT MIXTURE

I- ASPHALT MIXTURE BEHAVIOR

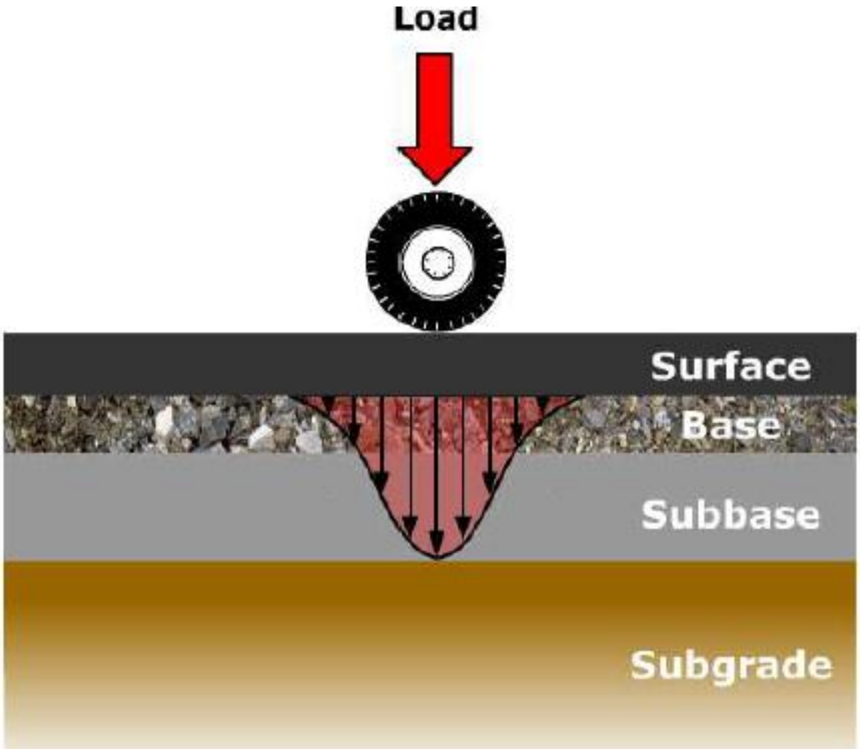
When a wheel load is applied to a pavement, two stresses are transmitted to the HMA: vertical compressive stress within the asphalt layer, and horizontal tensile stress at the bottom of the asphalt layer. The HMA must be internally strong and resilient to resist the compressive stresses and prevent permanent 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.



Spread of Wheel load pressure through the pavement structure



Pavement deflection under load

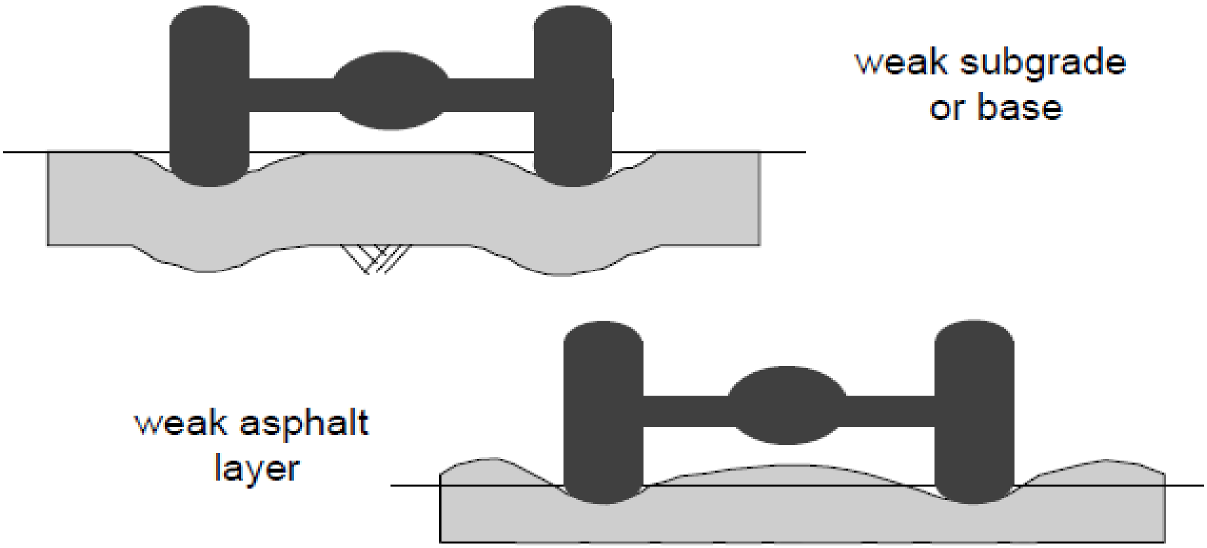


The only way to understand asphalt mixture behavior is to consider the primary asphalt pavement distress types that engineers try to avoid: **permanent deformation**, **fatigue cracking**, and **low temperature cracking**. These are the distresses analyzed in Superpave.

II- 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.

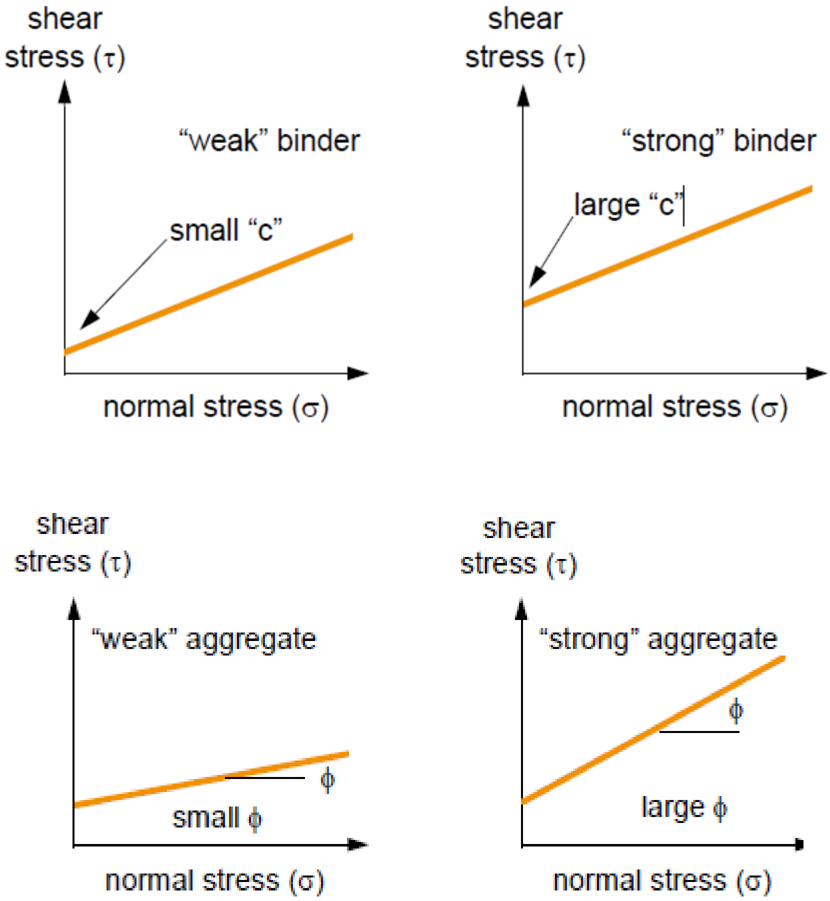
- Weak subgrade or base
- Weak asphalt layer



In one case, the rutting is caused by too much repeated stress being applied to the subgrade (or sub- base or base) below the asphalt layer. 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 rutting results from an asphalt mixture without enough shear strength to resist the repeated heavy loads.
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 solve rutting by considering the mineral aggregate and asphalt cement. In fact Mohr-Coulomb equation **(** $\tau = c + \sigma \times \tan \phi$ **)** can be used to illustrate how both materials can affect rutting.

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

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.

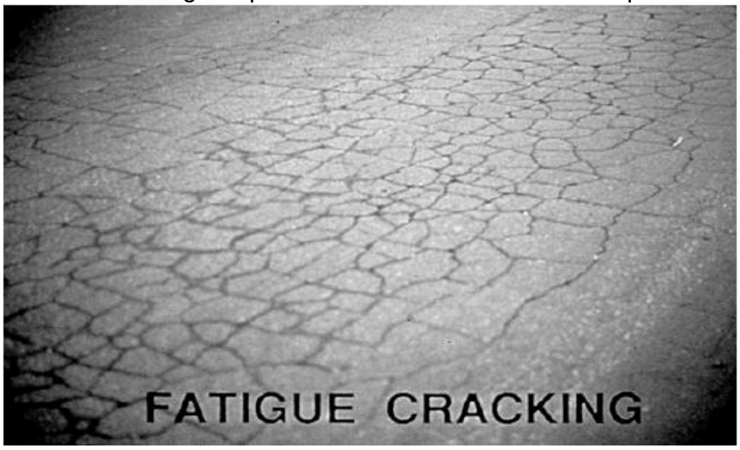


Another way to increase the shear strength of an asphalt mixture is by selecting an aggregate that has a high degree of internal friction **(ϕ)**. This is accomplished by selecting an aggregate that is cubical, has a rough surface texture, and graded in a manner to develop particle-to-particle contact. When a load is applied to the aggregate in the mixture, the aggregate particles lock tightly together and function not merely as a mass of individual particles, but more as **a *large, single, elastic stone***. As with the asphalt cement, the aggregate will act like a rubber band and spring back to its original shape when unloaded. In this case, no permanent deformation accumulates.

III- FATIGUE CRACKING

Fatigue cracking occurs when the applied loads overstress the asphalt materials, causing cracks to form. An early sign of fatigue cracking consists of intermittent longitudinal cracks in the traffic wheel path. Fatigue cracking progresses because at some point the initial cracks will join, causing even more cracks to form. An advanced stage of fatigue cracking is called alligator cracking, characterized by transverse cracks joining the longitudinal cracks. In extreme cases, a pothole forms when pavement pieces become dislodged by traffic.

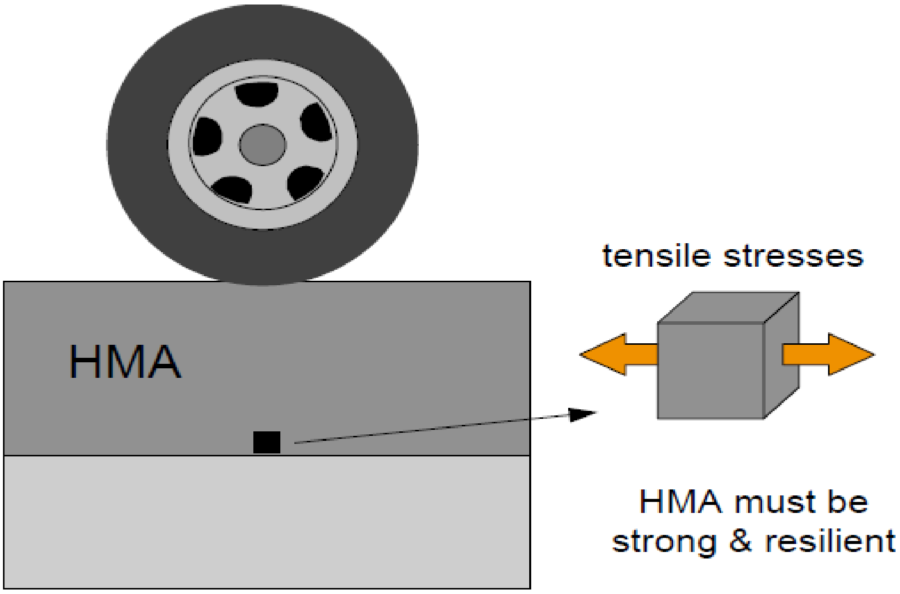
Fatigue cracking is usually caused by a number of factors occurring simultaneously. Obviously, repeated heavy loads must be present. Thin pavements or those with weak underlying layers are prone to high deflections under heavy wheel loads. High deflections increase the horizontal tensile stresses at the bottom of the asphalt layer, leading to fatigue cracking. Poor drainage, poor construction, and/or an under-designed pavement can contribute to this problem.



If the observed cracking occurs much sooner than the design period, it may be a sign that traffic loads were underestimated.

Consequently, the best ways to overcome fatigue cracking are:

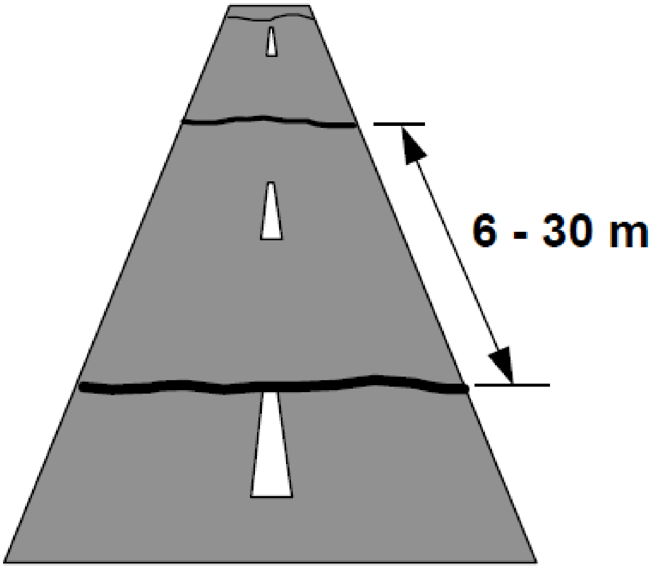
- adequately account for the expected number of heavy loads during design,
- keep the subgrade dry using whatever means available,
- use thicker pavements,
- use paving materials that are not excessively weakened in the presence of moisture, and
- use paving materials that are resilient enough to withstand normal deflections.

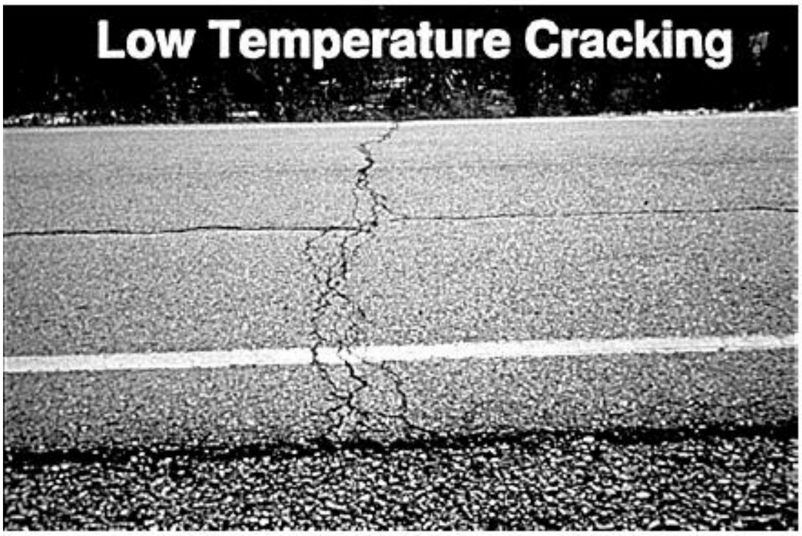


A good selection of resilient materials, can be solved by using **materials selection and design**. Thus, HMA must be designed to behave like a soft elastic material when loaded in tension to overcome fatigue cracking. This is accomplished by placing an upper limit on the asphalt cement's stiffness properties, since the tensile behavior of HMA is strongly influenced by the asphalt cement. In effect, soft asphalts have better fatigue properties than hard asphalts.

IV- LOW TEMOERATURE 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 a mixture has too many air voids, are also 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 the air void content and pavement density so that the binder does not become excessively oxidized.

CHAPTER FOUR
SUPERPAVE MIX DESIGN

This chapter presents a full Superpave volumetric mix design example.
Volumetric mix design plays a central role in Superpave mixture design. The best way of illustrating its steps is by means of an example for an assumed project . The information presented follows the logical progression of testing and data analysis involved in a Superpave mixture design and consists the concepts outlined in previous sections.
This chapter includes four major steps in the testing and analysis process for Superpave Mix Design:

I. SELECTION OF MATERIALS (aggregates, binders, modifiers, etc.)
II. SELECTION OF A DESIGN AGGREGATE STRUCTURE
III SELECTION OF A DESIGN ASPHALT BINDER CONTENT
IV. EVALUATIONOF MOISTURE SENSITIVITY OF THE DESIGN MIXTURE

I- SELECTION OF MATERIALS

For the assumed project, design ESALs are determined to be 18 million in the design lane. This places the design in the traffic category from 10 to 30 million ESALs. Traffic level is used to determine design requirements such as number of design gyrations for compaction, aggregate physical property requirements, and mixture volumetric requirements.
The mixture in this example is an intermediate course mixture. It will have a nominal maximum particle size of 19.0 mm. It will be placed at a depth less than 100 mm from the surface of the pavement.

I.1 BINDER SELECTION

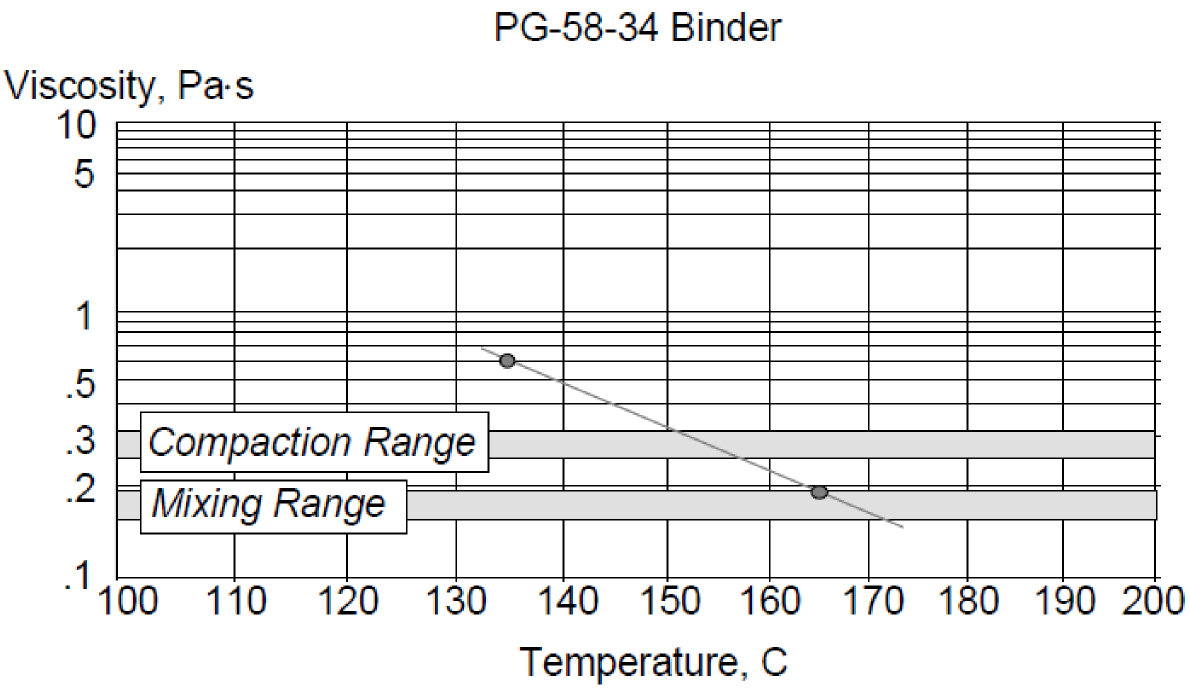
Environmental conditions are determined from weather station data stored in the Superpave weather database. The data can be retrieved from the report *Weather Database for the Superpave Mix Design System*, software released by the Long-Term Pavement Performance (LTPP) Division of the FHWA. Assuming there are two weather stations near the assumed project:

Project Environmental Conditions and Binder Grade				
Weather Station	**Min. Pavement Temp. (ºC)**	**Max. Pavement Temp. (ºC)**	**Binder Grade**	**Design Air Temp. (ºC)**
Low Reliability (50%)				
Station No. 1	- 26	51	PG 52 - 28	32
Station No. 2	- 25	51	PG 52 - 28	31
Paving Location (Assumed)	- 26	51	PG 52 - 28	32
High Reliability (98%)				
Station No. 1	- 32	55	PG 58 - 34	36
Station No. 2	- 33	54	PG 58 - 34	34
Paving Location (Assumed)	- 33	55	PG 58 - 34	35

Reliability is the percent probability that the actual temperature will not exceed the design pavement temperatures listed in the binder grade. In this example, the designer chooses high reliability for all conditions. Thus, a PG 58-34 binder is needed. The average Design High Air Temperature is 35°C. The selected binder should be tested for specification compliance. Binder test results are:

Test	Property	Test Result	Criteria (Specification)
Original Binder			
Flash Point	n/a	304 ºC	230 ºC minimum
Rotational Viscosity (RV)	135 ºC	0.575 Pa.s	3 Pa.s maximum
Rotational Viscosity (RV)	165 ºC	0.142 Pa.s	n/a
Dynamic Shear Rheometer (DSR)	G*/sin δ @ 58 ºC	1.42 kPa	1.00 kPa minimum
RTFO – aged Binder			
Mass Loss	n/a	0.14%	1.00% maximum
Dynamic Shear Rheometer (DSR)	G*/sin δ @ 58 ºC	2.42 kPa	2.20 kPa minimum
PAV –aged Binder			
Dynamic Shear Rheometer (DSR)	G*sin δ @ 16 ºC	1543 kPa	5000 kPa maximum
Bending Beam Rheometer (BBR)	Stiffness @- 24 ºC	172.0 MPa	300.0 MPa maximum
Bending Beam Rheometer (BBR)	m-value @- 24 ºC	0.321	0.300 minimum

Comparing the test results to specifications, the designer verifies that the asphalt binder meets the requirements of a PG 58-34 grade. Specification testing requires only that rotational viscosity be performed at 135° C. Additional testing was performed at 165° C to establish laboratory mixing and compaction temperatures. The illustration of the temperature-viscosity relationship for this binder shows that the mixing temperature range is selected between 165° C and 172° C. The compaction temperature range is selected between 151° C and 157° C.



I.2- AGGREGATE SELECTION

Next, the designer selects the aggregates to use in the mixture. For this example, there are 5 stockpiles of materials consisting of three coarse materials and two fine materials. It is assumed that the mixing facility will have at least 5 cold feed bins. If fewer cold feed bins are available, fewer stockpiles will be used. The materials are split into representative samples, and a washed sieve analysis is performed for each aggregate. These test results are shown in the section on selecting design aggregate structure.

The bulk and apparent specific gravities are determined for each aggregate. These specific gravities are used in VMA calculations and may be used if trial binder contents are calculated.

Aggregate Specific Gravities		
Aggregate	Bulk Sp. Gravity	Apparent Sp. Gravity
#1 Stone	2.703	2.785
12.5 mm Chip	2.689	2.776
9.5 mm Chip	2.723	2.797
Manuf. Sand	2.694	2.744
Screen Sand	2.679	2.731

In addition to sieve analysis and specific gravity determination, Superpave requires that consensus aggregate tests be performed to assure that the aggregates selected for the mix design are acceptable.

The four tests required are: coarse aggregate angularity, fine aggregate angularity, thin and elongated particles, and clay content. In addition, other aggregate tests can be selected that deemed important. These tests can include items such as soundness, toughness, and deleterious materials. For this example, the aggregate properties are measured for each stockpile as well as for the aggregate trial blends.

I.2.1- Coarse Aggregate Angularity

This test is performed on the coarse aggregate particles of the aggregate stockpiles. The coarse aggregate particles are defined as particles larger than 4.75 mm.

Coarse Aggregate Angularity Test Results				
Aggregate	1+ Fractured Faces	Criterion	2+ Fractured Faces	Criterion
#1 Stone	92%	95% min	88%	90% min
12.5 mm Chip	97%		94%	
9.5 mm Chip	99%		95%	

I.2.2- Fine Aggregate Angularity

This test is performed on the fine aggregate particles of the aggregate stockpiles. The fine aggregate particles are defined as particles smaller than 2.36 mm.

Fine Aggregate Angularity		
Aggregate	% Air Voids (Loose)	Criterion
Manufactured Sand	52%	45% min
Screen Sand	40%	

I.2.3- Flat, Elongated Particles

This test is performed on the coarse aggregate particles of the aggregate stockpiles. The coarse aggregate particles are defined as particles larger than 4.75 mm.

Flat, Elongated Particles		
Aggregate	% Flat/Elongated	Criterion
#1 Stone	0%	10% max
12.5 mm Chip	0%	
9.5 mm Chip	0%	

I.2.4- Clay Content (Sand Equivalent)

This test is performed on the fine aggregate particles of the aggregate stockpiles. The fine aggregate particles are defined as particles smaller than 4.75 mm.

Clay Content (Sand Equivalent)		
Aggregate	Sand Equivalent	Criterion
Manufactured Sand	47	45 min
Screen Sand	70	

II SELECTION OF A DESIGN AGGREGATE STRUCTURE

II-1 DETERMINATION OF A DESIGN AGGREGATE GRADATION

To select the design aggregate structure, the designer establishes trial blends by mathematically combining the gradations of the individual materials into a single gradation. The blend gradation is then compared to the specification requirements for the appropriate sieves. Gradation control is based on four control sieves: the maximum sieve, the nominal maximum sieve, the 2.36 mm sieve, and the 75 micron sieve.

The nominal maximum sieve is one sieve size larger than the first sieve to retain more than ten percent of combined aggregate. The maximum sieve size is one sieve size greater than the nominal maximum sieve. The restricted zone is an area on either side of the maximum density line. For a 19.0 mm nominal mixture, it starts at the 2.36 mm sieve and extends to the 300 micron sieve. Any proposed trial blend gradation has to pass between the control points established on the four sieves. In addition, it has to be outside of the area bounded by the limits set for the restricted zone. Some specifying agencies may allow gradations to pass through the Restricted Zone – if there is a history of successful performance or supporting test results.

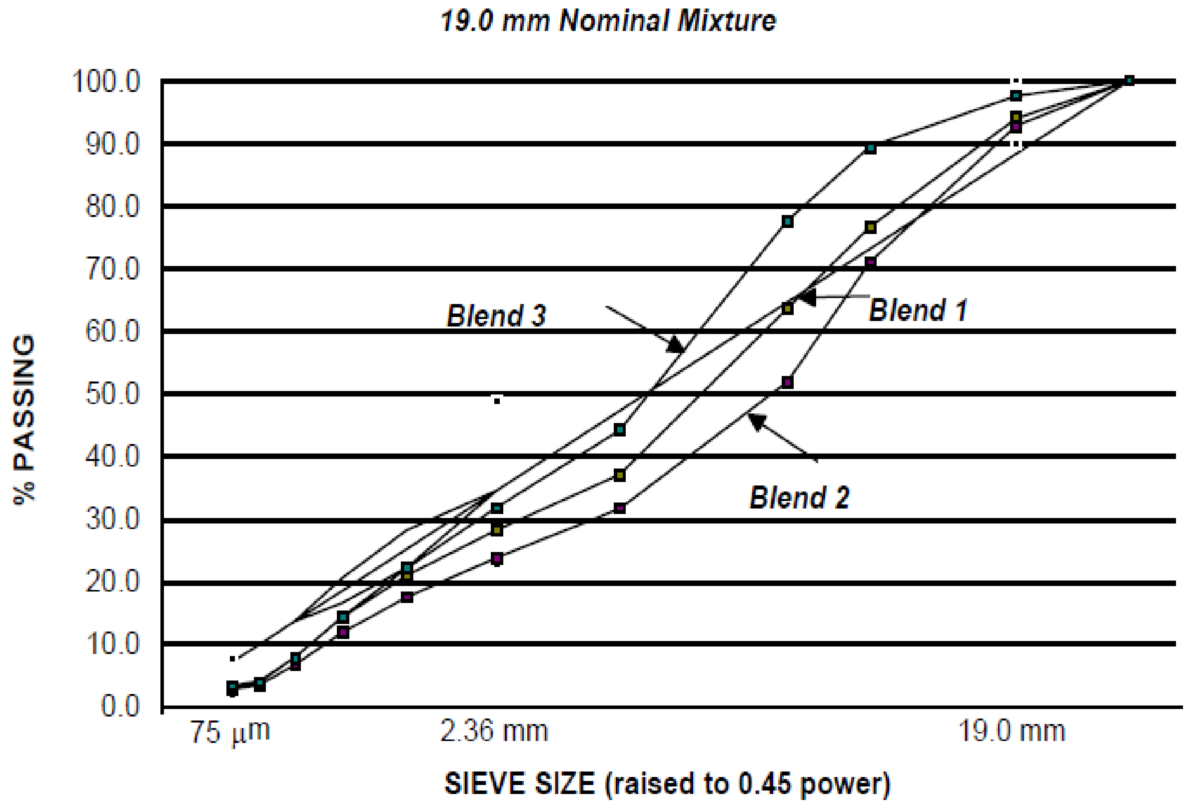
Gradation Criteria for 19.0 mm Nominal Mixture			
Gradation Control Item	Sieve Size, mm	Minimum, %	Maximum, %
Control Points	25.0	100.0	
	19.0	90.0	100.0
	12.5		90.0
	2.36	23.0	49.0
	0.075	2.0	8.0
Restricted Zone	2.36	34.6	34.6
	1.18	22.3	28.3
	0.600	16.7	20.7
	0.300	13.7	13.7

Any number of trial blends can be attempted, but three is the standard number of blends. Trial blending consists of varying stockpile percentages of each aggregate to obtain blend gradations meeting the gradation requirements for that particular mixture. For this example, three trial blends are used: an intermediate blend (Blend 1), a coarse blend (Blend 2), and a fine blend (Blend 3). The intermediate blend is combined to produce a gradation that is not close to either the gradation limits for the control sieves, or the restricted zone. The coarse blend is combined to produce a gradation that is close to the minimum criteria for the nominal maximum sieve, the 2.36 mm sieve, and the 75 micron sieve. The fine blend is combined to produce a gradation that is close to the maximum criteria for the nominal maximum sieve, and the restricted zone.

	#1 Stone	***12.5 mm chip***	***9.5 mm chip***	***Mfg sand***	***Scr. sand***
Blend 1	25.0%	15.0%	22.0%	18.0%	20.0%
Blend 2	30.0%	25.0%	13.0%	17.0%	15.0%
Blend 3	10.0%	15.0%	30.0%	31.0%	14.0%

Sieve						**Blend 1 Gradation**	**Blend 2 Gradation**	**Blend 3 Gradation**
25.0 mm	100.0	100.0	100.0	100.0	100.0	***100.0***	***100.0***	***100.0***
19.0 mm	76.1	100.0	100.0	100.0	100.0	***94.0***	***92.8***	***97.6***
12.5 mm	14.3	87.1	100.0	100.0	100.0	**76.6**	**71.1**	**89.5**
9.5 mm	3.8	26.0	94.9	100.0	99.8	**63.7**	**51.9**	**77.7**
4.75 mm	2.1	3.1	4.8	95.5	89.5	**37.1**	**31.7**	**44.3**
2.36 mm	1.9	2.6	3.0	63.5	76.7	***28.3***	***23.9***	***31.9***
1.18 mm	1.9	2.4	2.8	38.6	63.5	**21.1**	**17.6**	**22.2**
600 µm	1.8	2.3	2.6	21.9	45.6	**14.4**	**12.0**	**14.5**
300 µm	1.8	2.2	2.5	11.0	23.1	**7.9**	**6.8**	**7.9**
150 µm	1.7	2.1	2.4	5.7	8.4	**4.0**	**3.6**	**4.1**
75 µm	1.6	1.9	2.2	5.7	4.7	***3.1***	***2.9***	***3.5***

All three of the trial blends are shown graphically. Note that all three trial blends pass below the restricted zone. This is not a requirement. Superpave allows but does not recommend blends that plot above the restricted zone.



Once the trial blends are selected, a preliminary determination of the blended aggregate properties is necessary. This can be estimated mathematically from the 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

Values for coarse aggregate angularity are shown as percentage of one or more fractured faces followed by percentage of two or more fractured faces. Based on the estimates, all three trial blends are acceptable. When the design aggregate structure is selected, the blend aggregate properties will need to be verified by testing.

II.2- SELECTION OF TRIAL ASPHALT BINDER CONTENT

The next step is to evaluate the trial blends by compacting specimens and determining the volumetric properties of each trial blend. For each blend, a minimum of two specimens will be compacted using the Superpave Gyratory Compactor (SGC). The trial asphalt binder content can be estimated based on experience with similar materials. If there is no experience, the trial binder content can be determined for each trial blend by estimating the effective specific gravity of the blend and using the calculations shown below. The effective specific gravity (G_{se}) of the blend is estimated by:

$$G_{se} = G_{sb} + 0.8 \times (G_{sa} - G_{sb})$$

Where: G_{se} = Effective Specific Gravity
G_{sa} = Apparent Specific Gravity
G_{sb} = Bulk Specific Gravity

The factor, 0.8, can be adjusted at the discretion of the designer. Absorptive aggregates may require values closer to 0.6 or 0.5. The blend calculations are shown below:

Blend 1: $G_{se} = 2.699 + 0.8 \times (2.768 - 2.699) = 2.754$
Blend 2: $G_{se} = 2.697 + 0.8 \times (2.769 - 2.697) = 2.755$
Blend 3: $G_{se} = 2.701 + 0.8 \times (2.767 - 2.701) = 2.754$

The volume of asphalt binder (V_{ba}) absorbed into the aggregate is estimated using this equation:

$$V_{ba} = \frac{P_s \times (1 - V_a)}{\left(\frac{P_b}{G_b} + \frac{P_s}{G_{se}}\right)} \times \left(\frac{1}{G_{sb}} - \frac{1}{G_{se}}\right)$$

Where V_{ba} = volume of absorbed binder, cm³/ cm³ of mix
P_b = percent of binder (assumed 0.05),
P_s = percent of aggregate (assumed 0.95),
G_b = specific gravity of binder (assumed 1.02),
V_a = volume of air voids (assumed 0.04 cm³/ cm³ of mix)

Blend 1: $$V_{ba} = \frac{0.95 \times (1 - 0.04)}{\left(\frac{0.05}{1.02} + \frac{0.95}{2.754}\right)} \times \left(\frac{1}{2.699} - \frac{1}{2.754}\right) = 0.0171 \text{ cm}^3/\text{cm}^3 \text{ of mix}$$

Blend 2: $$V_{ba} = \frac{0.95 \times (1 - 0.04)}{\left(\frac{0.05}{1.02} + \frac{0.95}{2.755}\right)} \times \left(\frac{1}{2.697} - \frac{1}{2.755}\right) = 0.0181 \text{ cm}^3/\text{cm}^3 \text{ of mix}$$

Blend 3: $$V_{ba} = \frac{0.95 \times (1 - 0.04)}{\left(\frac{0.05}{1.02} + \frac{0.95}{2.754}\right)} \times \left(\frac{1}{2.701} - \frac{1}{2.754}\right) = 0.0165 \text{ cm}^3/\text{cm}^3 \text{ of mix}$$

The volume of the effective binder (V_{be}) can be determined from this equation:

$$V_{be} = 0.081 - 0.02931 \times [\ln(S_n)]$$

Where S_n = the nominal maximum sieve size of the aggregate blend (in inches)

Blend 1-3: $V_{be} = 0.081 - 0.02931 \times [\ln(0.75)] = 0.089$ cm³/ cm³ of mix

Finally, the initial trial asphalt binder (P_{bi}) content is calculated from this equation:

$$P_{bi} = \frac{G_b \times (V_{be} + V_{ba})}{(G_b \times (V_{be} + V_{ba})) + W_s} \times 100$$

Where P_{bi} = percent (by weight of mix) of binder
W_s = weight of aggregate, grams

$$W_s = \frac{P_s \times (1 - V_a)}{(\frac{P_b}{G_b} + \frac{P_s}{G_{se}})}$$

Blend 1:
$$W_s = \frac{0.95 \times (1 - 0.04)}{(\frac{0.05}{1.02} + \frac{0.95}{2.754})} = 2.315$$

$$P_{bi} = \frac{1.02 \times (0.089 + 0.0171)}{(1.02 \times (0.089 + 0.0171)) + 2.315} \times 100 = 4.46\% \quad \text{(by mass of mix)}$$

Blend 2:
$$W_s = \frac{0.95 \times (1 - 0.04)}{(\frac{0.05}{1.02} + \frac{0.95}{2.755})} = 2.316$$

$$P_{bi} = \frac{1.02 \times (0.089 + 0.0181)}{(1.02 \times (0.089 + 0.0171)) + 2.316} \times 100 = 4.46\% \quad \text{(by mass of mix)}$$

Blend 3:
$$W_s = \frac{0.95 \times (1 - 0.04)}{(\frac{0.05}{1.02} + \frac{0.95}{2.754})} = 2.315$$

$$P_{bi} = \frac{1.02 \times (0.089 + 0.0165)}{(1.02 \times (0.089 + 0.0171)) + 2.315} \times 100 = 4.46\% \quad \text{(by mass of mix)}$$

Next, a minimum of two specimens for each trial blend is compacted using the SGC. Two specimens are also prepared for determination of the mixture's maximum theoretical specific gravity (G_{mm}). An aggregate weight of 4500 grams is usually sufficient for the compacted specimens. An aggregate weight of 2000 grams is usually sufficient for the specimens used to determine maximum theoretical specific gravity (G_{mm}). AASHTO T 209 should be consulted to determine the minimum sample size required for various mixtures.

Specimens are mixed at the appropriate mixing temperature, which is 165° C to 172° C for the selected PG 58-34 binder. The specimens are then short-term aged by placing the loose mix in a flat pan in a forced draft oven at the compaction temperature, 151° C to 157°C, for 2 hours. Finally, the specimens are then removed and either compacted or allowed to cool loose (for G_{mm} determination).

The number of gyrations used for compaction is determined based on the traffic level.

Superpave Design Gyratory Compactive Effort			
Design ESALs (millions)	Compaction Parameters		
	$N_{initial}$	N_{design}	$N_{maximum}$
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 10	8	100	160
≥ 30	9	125	205

The number of gyrations for initial compaction, design compaction, and maximum compaction are:

N_{ini} = 8 gyrations
N_{des} = 100gyrations
N_{max} = 160 gyrations

Each specimen will be compacted to the design number of gyrations, with specimen height data collected during the compaction process. This is tabulated for each Trial Blend. SGC compaction data reduction is accomplished as follows.

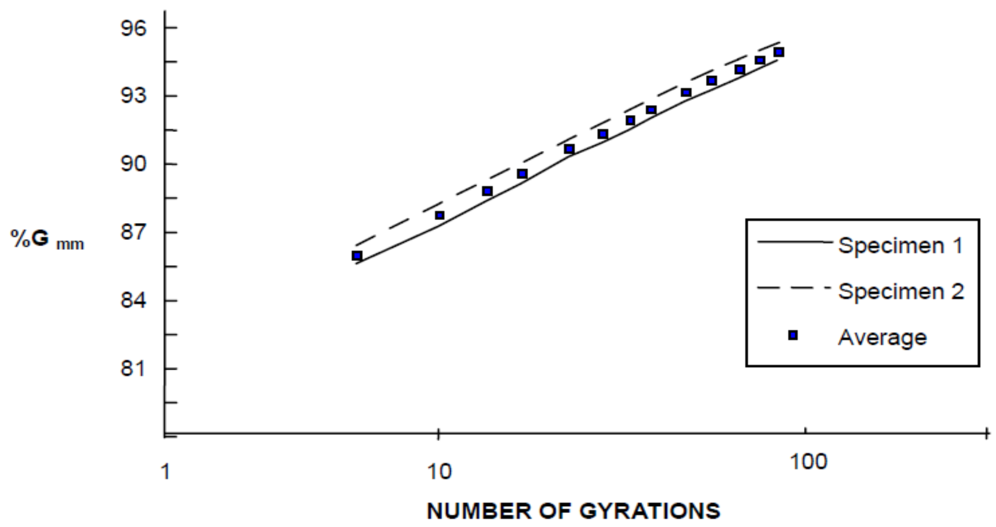
During compaction, the height of the specimen is continuously monitored. After compaction is complete, the specimen is extruded from the mold and allowed to cool. Next, the bulk specific gravity (G_{mb}) of the specimen is determined using AASHTO T166. The G_{mm} of each blend is determined using AASHTO T209. G_{mb} is then divided by G_{mm} to determine the % G_{mm} @ N_{des}. The % G_{mm} at any number of gyrations (N_x) is then calculated by multiplying % G_{mm} @ N_{des} by the ratio of the heights at N_{des} and N_x.

The SGC data reduction for the three trial blends is shown in the accompanying tables. The most important points of comparison are %G_{mm} at N_{ini}, N_{des}, and N_{max}, which are highlighted in these tables.

Figures illustrate the compaction plots for data generated in these tables. The figures show %G_{mm} versus the logarithm of the number of gyrations

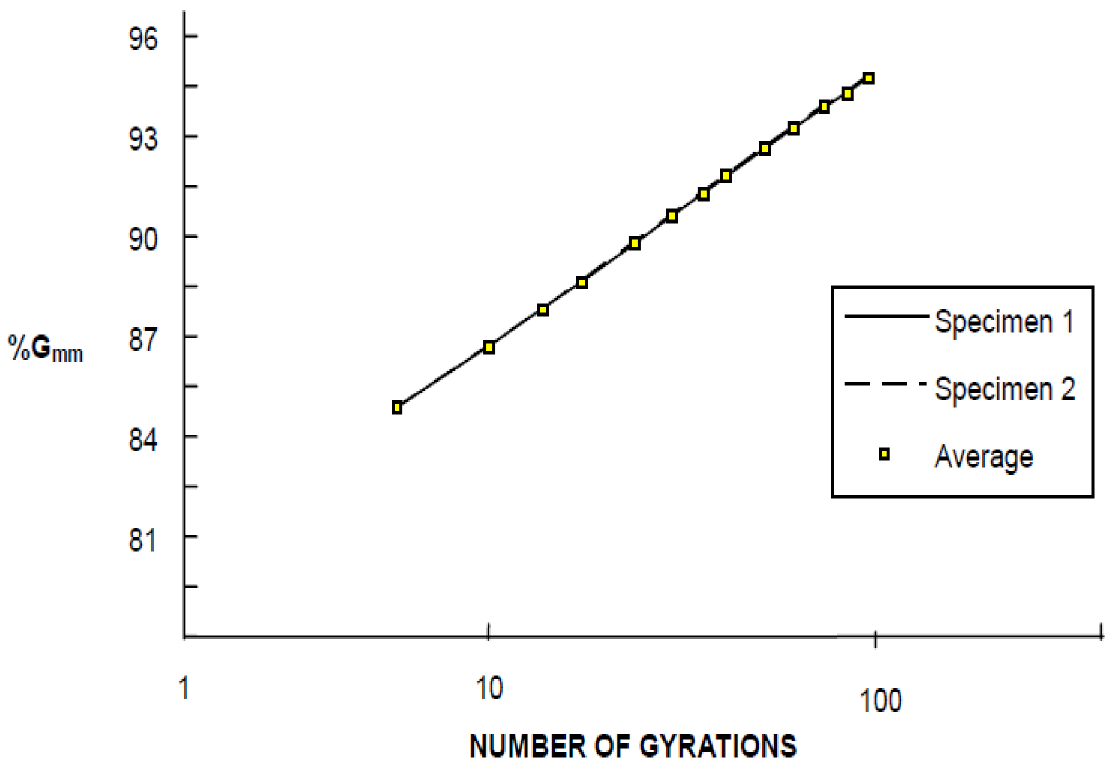
Densification Data for Trial Blend 1					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	%G_{mm}	Ht, mm	%G_{mm}	%G_{mm}
5	129.0	85.2	130.3	86.2	85.7
8	127.0	86.5	128.1	87.6	**87.1**
10	125.7	87.3	126.7	88.6	88.0
15	123.5	88.9	124.7	90.1	89.5
20	122.2	89.9	123.4	91.0	90.4
30	120.1	91.4	121.5	92.4	91.9
40	119.0	92.3	120.2	93.4	92.8
50	118.0	93.0	119.3	94.2	93.6
60	117.2	93.7	118.5	94.8	94.3
80	116.0	94.7	117.3	95.8	95.2
100	115.2	95.4	116.4	96.5	**95.9**
G_{mb}	2.445		2.473		
G_{mm}	2.563		2.563		

19.0 mm Nominal, 4.4% AC, Trial Blend 1.



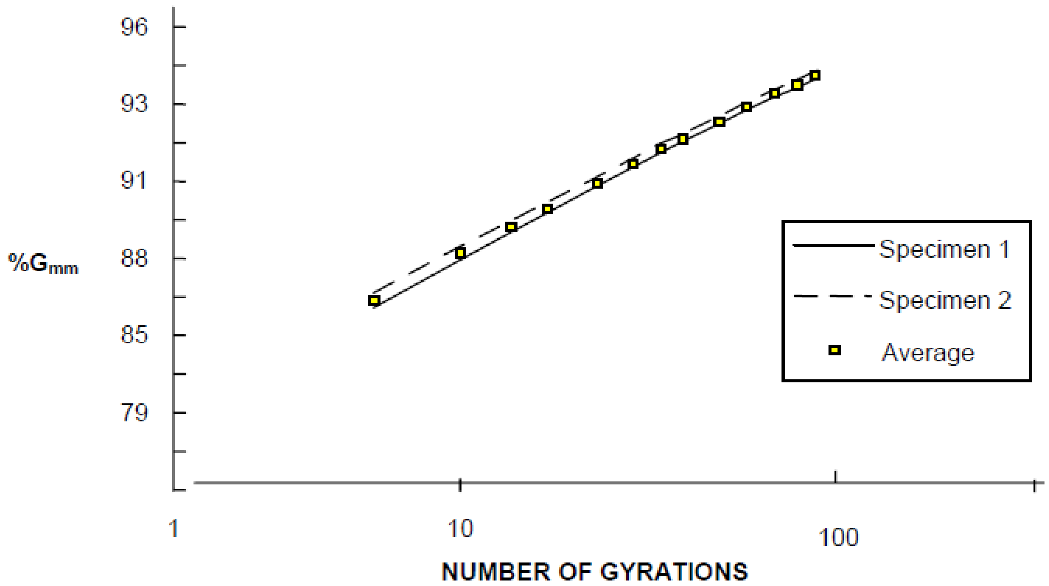
Densification Data for Trial Blend 2					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	$\%G_{mm}$	Ht, mm	$\%G_{mm}$	$\%G_{mm}$
5	131.7	84.2	132.3	84.2	84.2
8	129.5	85.6	130.1	85.6	**85.6**
10	128.0	86.6	128.7	86.6	86.6
15	125.8	88.1	126.5	88.1	88.1
20	124.3	89.2	124.9	89.2	89.2
30	122.2	90.7	122.7	90.8	90.7
40	120.7	91.8	121.2	91.9	91.9
50	119.6	92.7	120.1	92.8	92.7
60	118.7	93.4	119.2	93.5	93.4
80	117.3	94.5	117.8	94.6	94.5
100	116.3	95.3	116.8	95.4	**95.4**
G_{mb}	2.444		2.447		
G_{mm}	2.565		2.565		

19.0 mm Nominal, 4.4% AC, Trial Blend 2



Densification Data for Trial Blend 3					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	$\%G_{mm}$	Ht, mm	$\%G_{mm}$	$\%G_{mm}$
5	130.9	84.4	129.5	85.2	84.8
8	127.2	85.9	127.3	86.6	**86.3**
10	127.2	86.9	125.9	87.6	87.3
15	125.1	88.3	124.1	89.0	88.7
20	123.7	89.3	122.8	89.9	89.6
30	121.8	90.7	121.0	91.2	91.0
40	120.5	91.7	119.7	92.2	91.9
50	119.6	92.5	118.7	93.0	92.7
60	118.8	93.1	118.1	93.5	93.3
80	117.6	94.0	116.9	94.4	94.2
100	116.7	94.7	116.1	95.1	**94.9**
G_{mb}	2.432		2.442		
G_{mm}	2.568		2.568		

19.0 mm Nominal, 4.4% AC, Trial Blend 3



II.3- EVALUATEION OF TRIAL BLENDS

The average $\%G_{mm}$ is determined for N_{ini}, (8 gyrations) and N_{des} (100 gyrations) for each trial blend. This data is taken directly from the compaction data tables. The summary of these values for Trial Blends 1, 2, and 3 is:

Determination of $\%G_{mm}$ at N_{ini} and N_{des} for Trial Blends		
Trial Blend	$\% G_{mm}$ @ N_{ini}	$\%G_{mm}$ @ N_{des}
1	87.1	95.9
2	85.6	95.4
3	86.3	94.9

The $\%G_{mm}$ for N_{max} must also be evaluated. Two additional specimens can be compacted to N_{max} for each of the trial blends or just the selected trial blend can be checked.

The percent of air voids and voids in the mineral aggregate (VMA) are determined at N_{des}. The percent air voids is calculated using this equation:

%Air Voids = 100 - %G_{mm} @ N_{des}

Blend 1: %Air Voids = 100 - 95.9 = 4.1%
Blend 2: %Air Voids = 100 - 95.4 = 4.6%
Blend 3: %Air Voids = 100 - 94.9 = 5.1%

The percent voids in the mineral aggregate is calculated using this equation:

$$\%VMA = 100 - \left(\frac{\%G_{mm}@N_{des} \times G_{mm} \times P_s}{G_{sb}}\right)$$

Blend 1: $$\%VMA = 100 - \left(\frac{95.9\% \times 2.563 \times 0.956}{2.699}\right) = 12.9\%$$

Blend 2: $$\%VMA = 100 - \left(\frac{95.4\% \times 2.565 \times 0.956}{2.697}\right) = 13.3\%$$

Blend 3: $$\%VMA = 100 - \left(\frac{94.9\% \times 2.568 \times 0.956}{2.701}\right) = 13.7\%$$

Compaction Summary of Trial Blends					
Blend	AC%	%Gmm @ N=8	%Gmm @ N=100	% Air Voids (Va)	% VMA
1	4.4	87.1	95.9	4.1	12.9
2	4.4	85.6	95.4	4.6	13.3
3	4.4	86.3	94.9	5.1	13.7

The table above shows the compaction summary of the trial blends. The central premise in Superpave volumetric mix design is that the correct amount of asphalt binder is used in each trial blend so that each blend achieves exactly 96% of G_{mm} or 4% air void content at N_{des}. Clearly, this did not happen for any of the three trial blends. Because the trial blends exhibit different air void contents at N_{des}, the other volumetric and compaction properties cannot be properly compared. For example, Trial Blend 1 contained slightly too little asphalt to achieve 4 % air voids at N_{des}. Instead, it had 4.1% air voids. The VMA of Trial Blend 1 is too low. The designer must ask the question, “If I had used the asphalt content in Trial Blend 1 to achieve 4% air voids at N_{des}, would the VMA and other required properties improve to acceptable levels?”

Providing an answer to this question is an important step in volumetric mix design. To answer this question, an estimated asphalt binder content to achieve 4% air voids (96% G_{mm} at N_{des}) is determined for each trial blend using this formula:

$$P_{b,estimated} = P_{bi} - (0.4 \times (4 - V_a))$$

Where $P_{b,estimated}$ = estimated percent binder
P_{bi} = initial (trial) percent binder
V_a = percent air voids at N_{des}

Blend 1: $P_{b,estimated}$ = 4.4 - (0.4 × (4 – 4.1)) = 4.4%
Blend 2: $P_{b,estimated}$ = 4.4 - (0.4 × (4 - 4.6)) = 4.6%
Blend 3: $P_{b,estimated}$ = 4.4 - (0.4 × (4 – 5.1)) = 4.8%

The volumetric (VMA and VFA) and mixture compaction properties are then estimated at this asphalt binder content using the equations below. These steps are solely aimed at answering the question,
“What would have been the trial blend properties if I had used the right amount of asphalt to achieve 4% air voids at N_{des}?” It is by these steps that a proper comparison among trial blends can be accomplished.

For VMA:

$$\%VMA_{estimated} = \%VMA_{initial} + C \times (4 - V_a)$$

Where: $\%VMA_{initial}$ = %VMA from trial asphalt binder content
C = constant (either 0.1 or 0.2)
Note: C = 0.1 if V_a is less than 4.0%
C = 0.2 if V_a is greater than 4.0%

Blend 1: $\%VMA_{estimated} = 12.9 + (0.2 \times (4.0 - 4.1)) = 12.9\%$
Blend 2: $\%VMA_{estimated} = 13.3 + (0.2 \times (4.0 - 4.6)) = 13.2\%$
Blend 3: $\%VMA_{estimated} = 13.7 + (0.2 \times (4.0 - 5.1)) = 13.5\%$

For VFA:

$$\%VFA_{estimated} = 100\% \times \frac{(\%VMA_{estimated} - 4.0)}{\%VMA_{estimated}}$$

Blend 1: $$\%VFA_{estimated} = 100\% \times \frac{(12.9 - 4.0)}{12.9} = 69.0\%$$

Blend 2: $$\%VFA_{estimated} = 100\% \times \frac{(13.2 - 4.0)}{13.2} = 69.7\%$$

Blend 3: $$\%VFA_{estimated} = 100\% \times \frac{(13.5 - 4.0)}{13.5} = 70.4\%$$

For %Gmm at Nini:

$$\%G_{mm\ estimated} @ N_{ini} = \%G_{mm\ trial} @ N_{ini} - (4.0 - V_a)$$

Blend 1: $\%G_{mm\ estimated} @ N_{ini} = 87.1 - (4.0 - 4.1) =$ 87.2%
Blend 2: $\%G_{mm\ estimated} @ N_{ini} = 85.6 - (4.0 - 4.6) =$ 86.2%
Blend 3: $\%G_{mm\ estimated} @ N_{ini} = 86.3 - (4.0 - 5.1) =$ 87.4%

Finally, there is a required range on the **dust proportion**. This criteria is constant for all levels of traffic. It is calculated as the percent by mass of the material passing the 0.075 mm sieve (by wet sieve analysis) divided by the effective asphalt binder content (expressed as percent by mass of mix). The effective asphalt binder content is calculated using:

$$P_{be,\ estimated} = -(P_s \times G_b) \times \left(\frac{G_{se} - G_{sb}}{G_{se} \times G_{sb}}\right) + P_{b,\ estimated}$$

Blend 1: $$P_{be,\ estimated} = -(95.6 \times 1.02) \times \left(\frac{2.754 - 2.699}{2.754 \times 2.699}\right) + 4.4 = 3.7\%$$

Blend 2: $$P_{be,\ estimated} = -(95.4 \times 1.02) \times \left(\frac{2.755 - 2.697}{2.755 \times 2.697}\right) + 4.6 = 3.8\%$$

Blend 3: $$P_{be,\ estimated} = -(95.2 \times 1.02) \times \left(\frac{2.754 - 2.701}{2.754 \times 2.701}\right) + 4.8 = 4.1\%$$

Dust Proportion is calculated using:

$$DP = \frac{P_{.075}}{P_{be,\ estimated}}$$

Blend 1: $$DP = \frac{3.1}{3.7} = 0.84$$

Blend 2: $$DP = \frac{2.9}{3.8} = 0.76$$

Blend 3: $$DP = \frac{3.5}{4.1} = 0.85$$

The dust proportion must typically be between 0.6 and 1.2.

Dust Proportion of Trial Blends		
Blend	Dust Proportion	Criterion
Trial Blend 1	0.84	0.6 - 1.2
Trial Blend 2	0.76	
Trial Blend 3	0.85	

These tables show the estimated volumetric and mixture compaction properties for the trial blends at the asphalt binder content that should result in 4.0% air voids at N_{des}:

Estimated Mixture Volumetric Properties						
Blend	Trial % AC	Estimated % AC	% Air Voids	% VMA	% VFA	Dust Proportion
1	4.4	4.4	4.0	12.9	69.0	0.84
2	4.4	4.6	4.0	13.2	69.7	0.76
3	4.4	4.8	4.0	13.5	70.4	0.85

Estimated Mixture Compaction Properties			
Blend	Trial % AC	Estimated % AC	$\%G_{mm}$ @ N=8
1	4.4	4.4	87.2
2	4.4	4.6	86.2
3	4.4	4.8	87.4

Estimated properties are compared against the mixture criteria. For the design traffic and nominal maximum particle size, the volumetric and densification criteria are:

% Air Voids 4.0%
% VMA 13.0% (19.0 mm nominal mixture)
% VFA 65% - 75% (10-30 $\times$ 10^6 ESALs)
% G_{mm} @ N_{ini} less than 89%
Dust Proportion 0.6 - 1.2

Blend 1 is unacceptable based on a failure to meet the minimum VMA criteria. Both Blends 2 and 3 are acceptable. The **VMA**, **VFA**, **D. P.**, and **N_{ini}** criteria are met. For this example, Trial Blend 3 is selected as the design aggregate structure.

If none of the blends were acceptable additional combinations of the current aggregates could be tested, or additional materials from different sources could be obtained and included in the trial blend analysis.

III SELECTION OF A DESIGN ASPHALT BINDER CONTENT

III.1 DETERMINATION OF DESIGN ASPHALT BINDER CONTENT

Once the design aggregate structure is selected, Trial Blend 3 in this case, specimens are compacted at varying asphalt binder contents. The mixture properties are then evaluated to determine a design asphalt binder content.

A minimum of two specimens are compacted at each of the following asphalt contents:

- estimated binder content
- estimated binder content ± 0.5%, and
- estimated binder content + 1.0%.

For Trial Blend 3, the binder contents for the mix design are 4.3%, 4.8%, 5.3%, and 5.8%. Four asphalt binder contents are a minimum in Superpave mix design.

A minimum of two specimens is also prepared for determination of maximum theoretical specific gravity at the estimated binder content. Specimens are prepared and tested in the same manner as the specimens from the "Select Design Aggregate Structure" section.

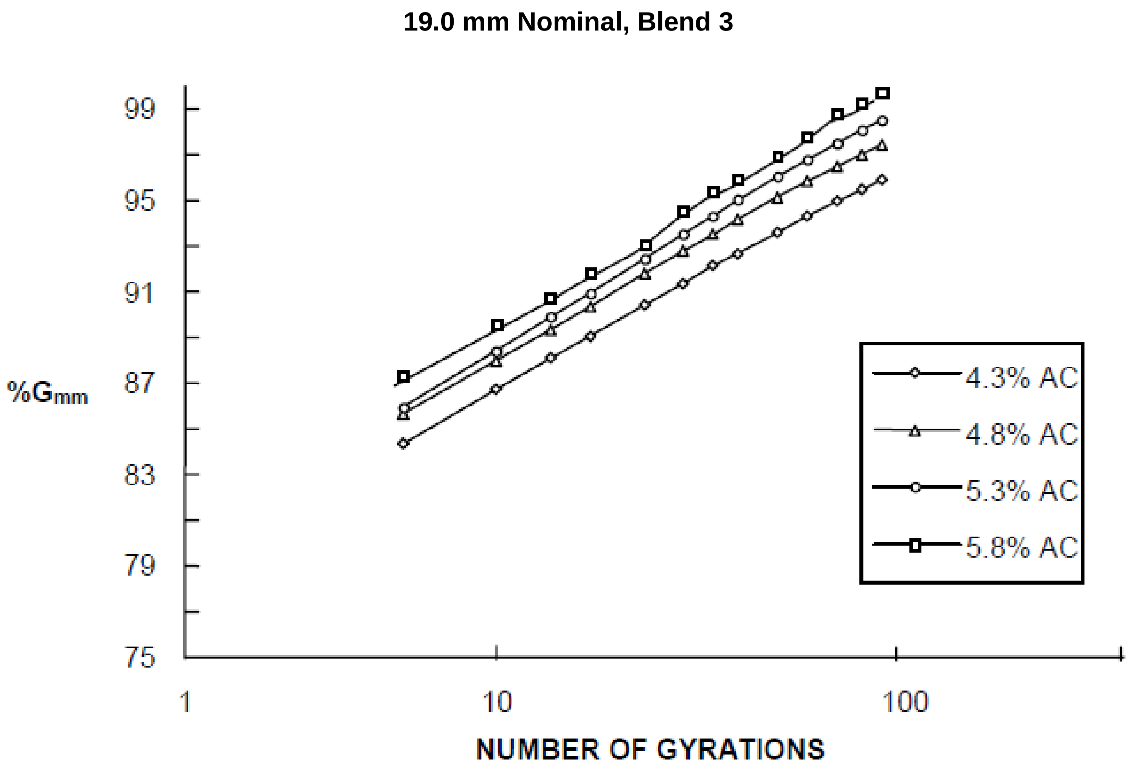
The following tables indicate the test results for each trial asphalt binder content. The average densification curves for each trial asphalt binder content are graphed for comparative illustration.

Densification Data for Blend 3, 4.3% Asphalt Binder					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	%G_{mm}	Ht, mm	%G_{mm}	%G_{mm}
5	131.3	83.9	131.0	84.7	84.3
8	129.0	85.4	128.8	86.1	**85.7**
10	127.5	86.4	127.4	87.1	86.7
15	125.4	87.8	125.5	88.4	88.1
20	124.0	88.8	124.2	89.3	89.1
30	122.1	90.2	122.4	90.6	90.4
40	120.9	91.1	121.1	91.6	91.4
50	119.9	91.9	120.1	92.4	92.1
60	119.1	92.5	119.4	92.9	92.7
80	117.9	93.4	118.3	93.8	93.6
100	117.0	94.1	117.4	94.5	**94.3**
G_{mb}	2.430		2.440		
G_{mm}	2.582		2.582		

Densification Data for Blend 3, 4.8% Asphalt Binder					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	%G_{mm}	Ht, mm	%G_{mm}	%G_{mm}
5	130.4	85.8	130.8	85.5	85.7
8	128.2	87.2	128.8	86.9	**87.1**
10	126.8	88.2	127.4	87.8	88.0
15	124.8	89.6	125.5	89.1	89.4
20	123.5	90.6	124.1	90.1	90.3
30	121.5	92.1	122.1	91.5	91.8
40	120.3	93.0	120.8	92.6	92.8
50	119.3	93.7	119.9	93.3	93.5
60	118.5	94.4	119.0	94.0	94.2
80	117.2	95.4	117.9	94.9	95.1
100	116.4	96.1	117.0	95.6	**95.8**
G_{mb}	2.462		2.449		
G_{mm}	2.562		2.562		

Densification Data for Blend 3, 5.3% Asphalt Binder					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	%G_{mm}	Ht, mm	%G_{mm}	%G_{mm}
5	132.0	86.0	132.6	85.8	85.9
8	129.8	87.5	130.4	87.4	**87.4**
10	128.3	88.5	128.9	88.4	88.4
15	126.2	90.0	126.7	89.8	89.9
20	124.8	91.0	125.2	90.9	91.0
30	122.8	92.5	123.2	92.4	92.4
40	121.4	93.5	121.7	93.5	93.5
50	120.3	94.4	120.7	94.3	94.3
60	119.5	95.1	119.9	95.0	95.0
80	118.2	96.1	118.6	96.0	96.0
100	117.3	96.8	117.7	96.7	**96.8**
G_{mb}	2.461		2.458		
G_{mm}	2.542		2.542		

Densification Data for Blend 3, 5.8% Asphalt Binder					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	$\%G_{mm}$	Ht, mm	$\%G_{mm}$	$\%G_{mm}$
5	130.4	87.4	131.5	87.2	87.3
8	128.6	88.7	129.4	88.6	**88.6**
10	127.4	89.5	128.0	89.6	89.5
15	125.4	90.8	126.2	90.8	90.8
20	124.0	91.9	124.9	91.8	91.8
30	122.4	93.1	123.1	93.1	93.1
40	120.5	94.6	121.3	94.5	94.5
50	119.4	95.5	120.2	95.4	95.4
60	118.9	95.9	119.5	96.0	95.9
80	117.6	96.9	118.2	97.0	96.9
100	116.7	97.7	117.2	97.8	**97.8**
G_{mb}	2.464		2.467		
G_{mm}	2.523		2.523		



Average Densification Curves for Blend 3, Varying Asphalt Binder Content

Mixture properties are evaluated for the selected blend at the different asphalt binder contents, by using the densification data at Nini (8 gyrations) and Ndes (100 gyrations). These tables show the response of the mixture's compaction and volumetric properties with varying asphalt binder contents:

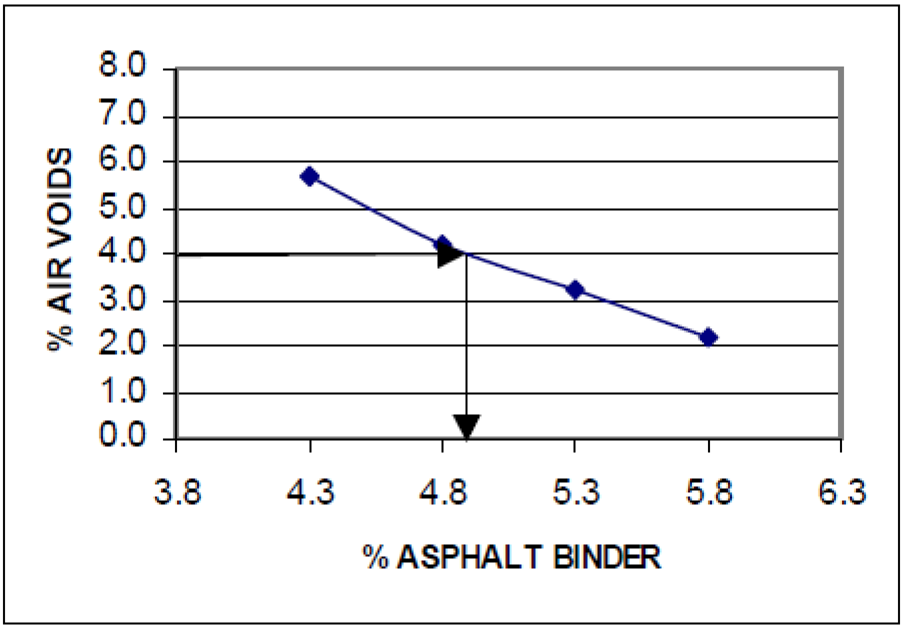
%AC	$\%G_{mm}$ @ N=8	$\%G_{mm}$ @ N=100
4.3%	85.8%	94.3%
4.8%	87.1%	95.8%
5.3%	87.4%	96.8%
5.8%	88.6%	97.8%

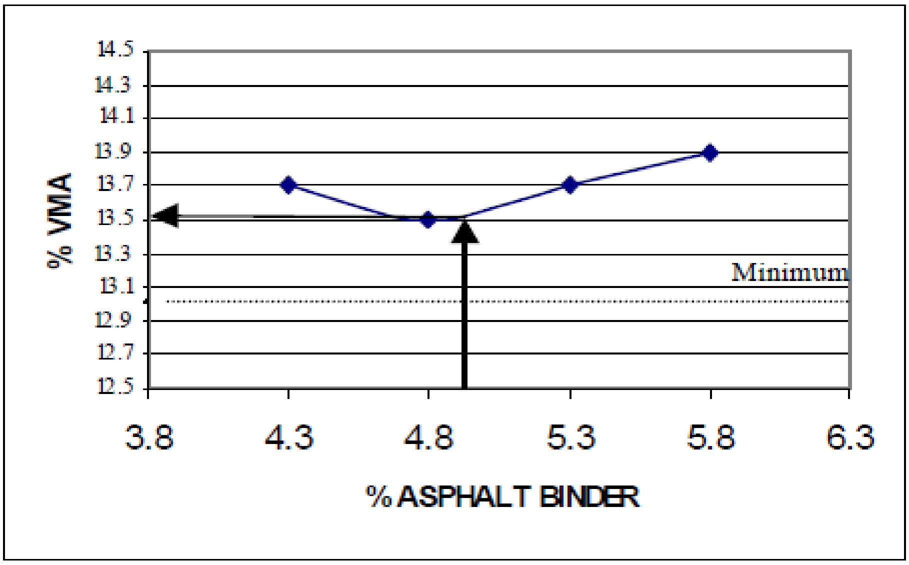
Summary of Blend 3 - Mix Volumetric Properties at N_{des}				
%AC	%Air Voids	%VMA	%VFA	Dust Proportion
4.3%	5.7%	13.7%	58.4%	1.21
4.8%	4.2%	13.5%	68.9%	1.05
5.3%	3.2%	13.7%	76.6%	0.91
5.8%	2.2%	13.9%	84.2%	0.82

The volumetric properties are calculated at the design number of gyrations (Ndes) for each trial asphalt binder content. From these data points, the designer can generate graphs of air voids, VMA, and VFA versus asphalt binder content. The design asphalt binder content is established at 4.0% air voids.

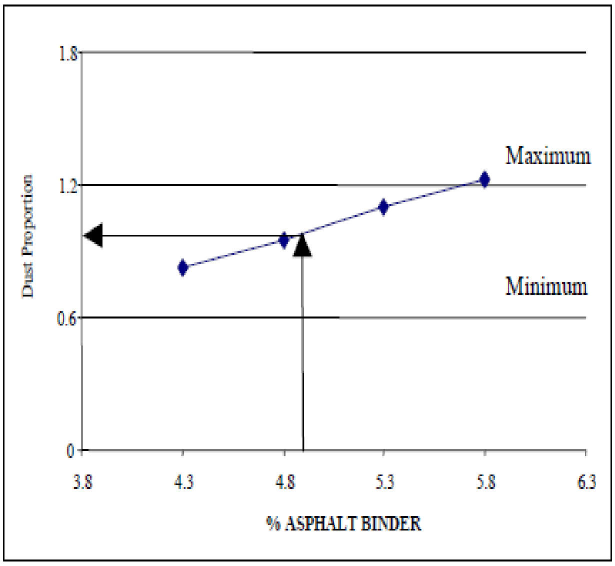
In this example, the design asphalt binder content is 4.9% - the value that corresponds to 4.0% air voids at Ndes = 100 gyrations. All other mixture properties are checked at the design asphalt binder content to verify that they meet criteria.

Design Mixture Properties at 4.9% Binder Content		
Mix Property	Result	Criteria
% Air Voids	4.0%	4.0%
%VMA	13.5%	13.0% min.
%VFA	71.0%	65% - 75%
Dust Proportion	1.00	0.6 - 1.2
$\%G_{mm}$ @ N_{ini} = 8	87.2%	less than 89%

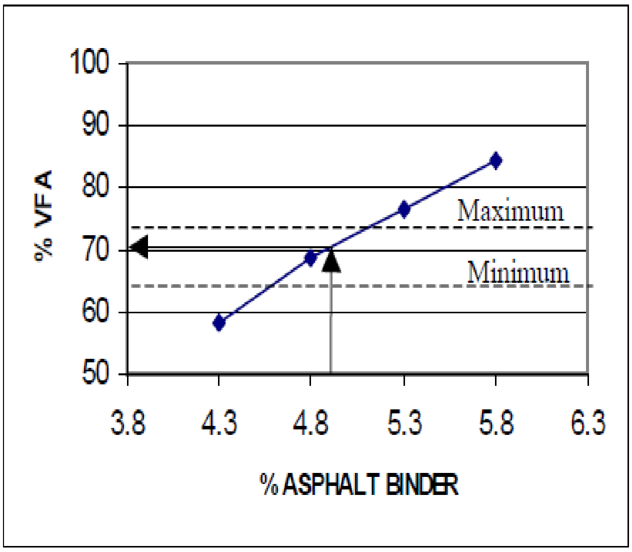




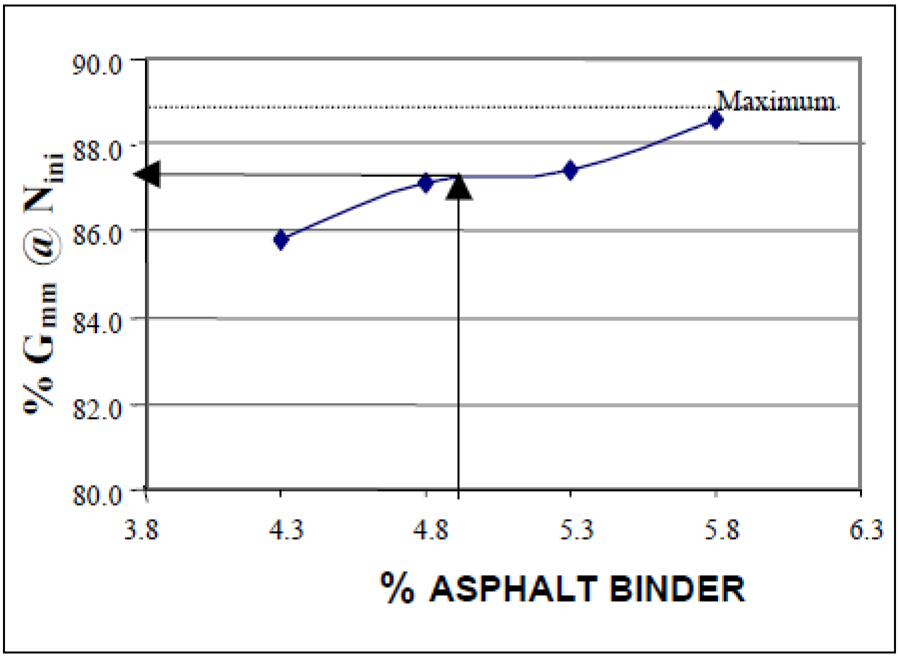
% VMA
Minimum
% ASPHALT BINDER



Dust Proportion
Maximum
Minimum
% ASPHALT BINDER



% VFA
Maximum
Minimum
% ASPHALT BINDER



% Gmm @ Nini
Maximum
% ASPHALT BINDER

III.2 VERIFICATION OF N_{MAX}

Superpave specifies a maximum density of 98% at N_{max}. Specifying a maximum density at N_{max} prevents design of a mixture that will compact excessively under 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 will not occur. After selecting the trial blend (#3) and selecting the design asphalt binder content (5.0%), two additional specimens are compacted to N_{max} (160 gyrations).

The table shows the compaction data.

N_{max} Densification Data for Blend 3, 4.9% Asphalt Binder					
	Specimen 1		Specimen 2		AVG
Gyrations	Ht, mm	%G_{mm}	Ht, mm	%G_{mm}	%G_{mm}
5	130.4	85.8	130.8	85.5	85.7
8	128.2	87.2	128.8	86.9	87.1
10	126.8	88.2	127.4	87.8	88.0
15	124.8	89.6	125.5	89.1	89.4
20	123.5	90.6	124.1	90.1	90.3
30	121.5	92.1	122.1	91.5	91.8
40	120.3	93.0	120.8	92.6	92.8
50	119.3	93.7	119.9	93.3	93.5
60	118.5	94.4	119.0	94.0	94.2
80	117.2	95.4	117.2	95.4	95.1
100	116.4	96.1	117.0	95.6	95.8
125	115.6	96.8	116.2	96.2	96.5
150	115.0	97.3	115.5	96.8	97.0
160	114.5	97.7	115.0	97.2	97.5
G_{mb}	2.495		2.490		
G_{mm}	2.554		2.554		

Blend 3, with %G_{mm} @ N_{max} equal to 97.5, satisfies the Superpave criteria.

IV EVALUATATION OF MOISTURE SENSITIVITY

The final step in the Superpave mix design process is to evaluate the moisture sensitivity of the design mixture. This step is accomplished by performing AASHTO T 283 testing on the design aggregate blend at the design asphalt binder content. Six specimens are compacted to approximately 7% air voids.

One subset of three specimens is considered control specimens. The other subset of three specimens is the conditioned subset. The conditioned subset is subjected to vacuum saturation followed by an optional freeze cycle, followed by a 24 hour thaw cycle at 60° C. All specimens are tested to determine their indirect tensile strengths. The moisture sensitivity is determined as a ratio of the tensile strengths of the conditioned subset divided by the tensile strengths of the control subset.

The table shows the moisture sensitivity data for the mixture at the design asphalt binder content.

Moisture Sensitivity Data for Blend 3 at 4.9% Design Asphalt Binder Content							
SAMPLE		1	2	3	4	5	6
Diameter, mm	D	150.0	150.0	150.0	150.0	150.0	150.0
Thickness, mm	t	99.2	99.4	99.4	99.3	99.2	99.3
Dry mass, g	A	3986.2	3981.3	3984.6	3990.6	3987.8	3984.4
SSD mass, g	B	4009.4	4000.6	4008.3	4017.7	4013.9	4008.6
Mass in Water, g	C	2329.3	2321.2	2329.0	2336.0	2331.5	2329.0
Volume, cc (B-C)	E	1680.1	1679.4	1679.3	1681.7	1682.4	1679.6
Bulk Sp Gravity (A/E)	F	2.373	2.371	2.373	2.373	2.370	2.372
Max Sp Gravity	G	2.558	2.558	2.558	2.558	2.558	2.558
% Air Voids(100(G-F)/G)	H	7.2	7.3	7.2	7.2	7.3	7.3
Vol Air Voids (HE/100)	I	121.8	123.0	121.6	121.7	123.4	122.0
Load, N	P				20803	20065	20354
Saturated							
SSD mass, g	B'	4060.9	4058.7	4059.1			
Mass in water, g	C'	2369.4	2373.9	2372.8			
Volume, cc (B'-C')	E'	1691.5	1684.8	1686.3			
Vol Abs Water, cc (B'-A)	J'	74.7	77.4	74.5			
% Saturation (100J'/I)		61.3	62.9	61.3			
% Swell (100(E'-E)/E)		0.7	0.3	0.4			
Conditioned							
Thickness, mm	t"	99.5	99.4	99.4			
SSD mass, g	B"	4070.8	4074.9	4074.8			
Mass in water, g	C"	2373.7	2380.3	2379.0			
Volume, cc (B"-C")	E"	1697.1	1694.6	1695.8			
Vol Abs Water, cc (B"-A)	J"	84.6	93.6	90.2			
% Saturation (100J"/I)		69.5	76.1	74.2			
% Swell (100(E"-E)/E)		1.0	0.9	1.0			
Load, N	P"	16720	16484	17441			
Dry Str. (2000P/(tDp))	S_{td}				889	858	870
Wet Str. (2000P"/(t"Dp))	S_{tm}	713	704	745			

Average Dry Strength (kPa)	872
Average Wet Strength (kPa)	721
%TSR	82.6%

The minimum criteria for tensile strength ratio 80%. The design blend (82.6%) exceeded the criteria.

The Superpave volumetric mix design is now complete for the intermediate mixture.

References

1- Superpave Volumetric Mix Design Workshop 2011 NECEPT(North East Center for Excellence Pavement Technology)

2- Superpave Fundamentals Reference Manual - NHI (National Highway Institute)

3- The Superpave Mix Design Maual for New Construction and Overlays (SHRP-A-407) Ronald J. Cominsky – University of Texas at Austin

4- Superpave Mix Design –NCAT (National Center for Asphalt Technology) - (Rown University)

5- Marshall and Superpave Mix Design Procedures by Pedro Romero, Ph.D.,P.E. The University of Utah

6- Superpave Mix Design Tests Methods and Requirements by John A. D'Angelo U.S. Federal Highway Administration

7- Asphalt Institute. (2001). Superpave Mix Design. Superpave Series No. 2 (SP-02). Asphalt Institute. Lexington, KY

8- Bituminous Materials and Mixtures by Mang Tia, Professor of Civil and Coastal Engineering, University of Florida

9- Basic of a Good Road – 2014 Cornell Local Roads Program – Revised by Paul Male, P.E., L.S

10- Asphalt Paving Principles by Christopher Blades & Edward Kearney- Cornell Local Roads Program

11- Comparison of Three Methods of Pavement Design by David L. Allen- Kentucky Transportation Center (KTC)- College of Engineering – University of Kentucky

12- Procedures for Aggregate Inspection – MODT (Michigan Department of Transportation)

13- American Association of State Highway and Transportation Officials (AASHTO) 1997

14- American Society for Testing and Materials (ASTM)