

Bituminous Materials and Mixtures

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45.3 Bituminous Mixtures

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45.1 Introduction

The term *bituminous materials* is generally used to denote substances in which bitumen is present or from which it can be derived [Goetz and Wood, 1960]. *Bitumen* is defined as an amorphous, black or dark-colored, (solid, semi-solid, or viscous) cementitious substance, composed principally of high molecular weight hydrocarbons, and soluble in carbon disulfide. For civil engineering applications, bituminous materials include primarily *asphalts* and *tars*. Asphalts may occur in nature (natural asphalts) or may be obtained from petroleum processing (petroleum asphalts). Tars do not occur in nature and are obtained as condensates in the processing of coal, petroleum, oil-shale, wood or other organic materials. *Pitch* is formed when a tar is partially distilled so that the volatile constituents have evaporated off from it. *Bituminous mixtures* are generally used to denote the combinations of bituminous materials (as binders), aggregates and additives.

This chapter presents the basic principles and practices of the usage of bituminous materials and mixtures in pavement construction. In recent years, the use of tars in highway construction has been very limited due to the concern with the possible emission of hazardous flumes when tars are heated. Thus, this chapter deals primarily with asphalts and asphalt mixtures.

45.2 Bituminous Materials

Types of Bituminous Materials Used in Pavement Construction

Asphalt cement is an asphalt that has been specially refined as to quality and consistency for direct use in the construction of asphalt pavements. An asphalt cement has to be heated to an appropriate high temperature in order to be fluid enough to be mixed and placed.

Cutback asphalt is a liquid asphalt that is a blend of asphalt and petroleum solvents (such as gasoline and kerosine). A cutback asphalt can be mixed and placed with little or no application of heat. After a cutback asphalt is applied and exposed to the atmosphere, the solvent will gradually evaporate, leaving the asphalt cement to perform its function as a binder.

Emulsified asphalt (or *asphalt emulsion*) is an emulsion of asphalt cement and water that contains a small amount of emulsifying agent. In a normal emulsified asphalt, the asphalt cement is in the form of minute globules in suspension in water. An emulsified asphalt can be mixed and applied without any application of heat. After an asphalt emulsion is applied, sufficient time is required for the emulsion to break and the water to evaporate to leave the asphalt cement to perform its function as a binder. In an *inverted emulsified asphalt*, minute globules of water are in suspension in a liquid asphalt, which is usually a cutback asphalt. Inverted asphalt emulsions are seldom used in pavement applications.

Conventional Tests on Asphalt Cements and Their Significance

In this section, the purpose and significance of the commonly used tests on asphalt cements are described. Readers may refer to the appropriate standard test methods for detailed description of the test procedures.

Penetration Test

The penetration test is one of the oldest and most commonly used tests on asphalt cements or residues from distillation of asphalt cutbacks or emulsions. The standardized procedure for this test can be found in ASTM D5 [ASTM, 2001]. It is an empirical test that measures the consistency (hardness) of an asphalt at a specified test condition. In the standard test condition, a standard needle of a total load of 100 g is applied to the surface of an asphalt sample at a temperature of 25 °C for 5 seconds. The amount of penetration of the needle at the end of 5 seconds is measured in units of 0.1 mm (or penetration unit). A softer asphalt will have a higher penetration, while a harder asphalt will have a lower penetration. Other test conditions that have been used include (1) 0°C, 200 g, 60 sec., and (2) 46°C, 50 g, 5 sec.

The penetration test can be used to designate grades of asphalt cement, and to measure changes in hardness due to age hardening or changes in temperature.

Flash Point Test

The flash point test determines the temperature to which an asphalt can be safely heated in the presence of an open flame. The test is performed by heating an asphalt sample in an open cup at a specified rate and determining the temperature at which a small flame passing over the surface of the cup will cause the vapors from the asphalt sample temporarily to ignite or flash. The commonly used flash point test methods include (1) the Cleveland Open Cup (ASTM D92) and (2) Tag Open Cup (ASTM D1310). The Cleveland Open-Cup method is used on asphalt cements or asphalts with relatively higher flash points, while the Tag Open-Cup method is used on cutback asphalts or asphalts with flash points of less than 79°C.

Minimum flash point requirements are included in the specifications for asphalt cements for safety reasons. Flash point tests can also be used to detect contaminating materials such as gasoline or kerosine in an asphalt cement. Contamination of an asphalt cement by such materials can be indicated by a substantial drop in flash point. When the flash point test is used to detect contaminating materials, the Pensky-Martens Closed Tester method (ASTM D93), which tends to give more indicative results, is normally used. In recent years, the flash point test results have been related to the hardening potential of asphalt. An asphalt with a high flash point is more likely to have a lower hardening potential in the field.

Solubility Test

Asphalt consists primarily of bitumens, which are high-molecular-weight hydrocarbons soluble in carbon disulfide. The bitumen content of a bituminous material is measured by means of its solubility in carbon disulfide. In the standard test for bitumen content (ASTM D4), a small sample of about 2 g of the asphalt is dissolved in 100 ml of carbon disulfide and the solution is filtered through a filtering mat in a filtering crucible. The material retained on the filter is then dried and weighed, and used to calculate the bitumen content as a percentage of the weight of the original asphalt.

Due to the extreme flammability of carbon disulfide, solubility in trichloroethylene, rather than solubility in carbon disulfide, is usually used in asphalt cement specifications. The standard solubility test using trichloroethylene is designated as ASTM D 2042.

The solubility test is used to detect contamination in asphalt cement. Specifications for asphalt cements normally require a minimum solubility in trichloroethylene of 99.0 percent.

Unfortunately, trichloroethylene has been identified as a carcinogen and contributing to the depletion of the earth's ozone layer. The use of trichloroethylene will most likely be banned in the near future. There is a need to use a less hazardous and non-chlorinated solvent for this purpose. Results of several investigations have indicated that the solvent n-Propyl Bromide appears to be a feasible alternative to trichloroethylene for use in this application [Collins-Garcia et al, 2000].

Ductility Test

The ductility test (ASTM D113) measures the distance a standard asphalt sample will stretch without breaking under a standard testing condition (5 cm/min at 25°C). It is generally considered that an asphalt with a very low ductility will have poor adhesive properties and thus poor performance in service. Specifications for asphalt cements normally contain requirements for minimum ductility.

Viscosity Tests

The viscosity test measures the viscosity of an asphalt. Both the viscosity test and the penetration test measure the consistency of an asphalt at some specified temperatures and are used to designate grades of asphalts. The advantage of using the viscosity test as compared with the penetration test is that the viscosity test measures a fundamental physical property rather than an empirical value.

Viscosity is defined as the ratio between the applied shear stress and induced shear rate of a fluid. The relationship between shear stress, shear rate and viscosity can be expressed as:

$$\text{Shear Rate} = \text{Shear Stress} / \text{Viscosity} \quad (45.1)$$

When shear rate is expressed in units of 1/sec. and shear stress in units of Pascal, viscosity will be in units of Pascal-seconds. One Pascal-second is equal to 10 Poises. The lower the viscosity of an asphalt, the faster the asphalt will flow under the same stress.

For a Newtonian fluid, the relationship between shear stress and shear rate is linear, and thus the viscosity is constant at different shear rates or shear stress. However, for a non-Newtonian fluid, the relationship between shear stress and shear rate is not linear, and thus the apparent viscosity will change as the shear rate or shear stress changes. Asphalts tend to behave as slightly non-Newtonian fluids, especially at lower temperatures. When different methods are used to measure the viscosity of an asphalt, the test results might be significantly different, since the different methods might be measuring the viscosity at different shear rates. It is thus very important to indicate the test method used when viscosity results are presented.

The most commonly used viscosity test on asphalt cements is the Absolute Viscosity Test by Vacuum Capillary Viscometer (ASTM D2171). The standard test temperature is 60°C. The absolute viscosity test measures the viscosity in units of Poise. The viscosity at 60°C represents the viscosity of the asphalt at the maximum temperature a pavement is likely to experience in most parts of the U.S.

When the viscosity of an asphalt at a higher temperature (such as 135°C) is to be determined, the most commonly-used test is the Kinematic Viscosity Test (ASTM D2170), which measures the kinematic viscosity in units of Stokes or centi-Stokes. Kinematic viscosity is defined as:

$$\text{Kinematic Viscosity} = \text{Viscosity} / \text{Density} \quad (45.2)$$

When viscosity is in units of Poise and density in units of g/cm³, the kinematic viscosity will be in units of Stokes. To convert from kinematic viscosity (in units of Stokes) to absolute viscosity (in units of Poises), one simply multiplies the number of Stokes by the density in units of g/cm³. However, due

to the fact that an asphalt might be non-Newtonian and that the kinematic viscosity test and the absolute viscosity test are run at different shear rates, conversion by this method will not produce accurate results and can only serve as a rough estimation. The standard temperature for the kinematic test on asphalt cement is 135°C. The viscosity at 135°C approximately represents the viscosity of the asphalt during mixing and placement of a hot mix.

Thin Film Oven and Rolling Thin Film Oven Tests

When an asphalt cement is used in the production of asphalt concrete, it has to be heated to an elevated temperature and mixed with a heated aggregate. The hot asphalt mixture is then hauled to the job site, placed and compacted. By the time the compacted asphalt concrete cools down to the normal pavement temperature, significant hardening of the asphalt binder has already taken place. The properties of the asphalt in service are significantly different from those of the original asphalt.

Since the performance of the asphalt concrete in service depends on the properties of the hardened asphalt binder in service rather than the properties of the original asphalt, the properties of the hardened asphalt in service need to be determined and controlled.

The Thin Film Oven Test (TFOT) procedure (ASTM D1754) was developed to simulate the effects of heating in a hot-mix plant operation on an asphalt cement. In the standard TFOT procedure, the asphalt cement sample is poured into a flat-bottomed pan to a depth of about 1/8 in. (3.2 mm). The pan with the asphalt sample in it is then placed on a rotating shelf in an oven and kept at a temperature of 163°C for five hours. The properties of the asphalt before and after the TFOT procedure are measured to determine the change in properties that might be expected after a hot-mix plant operation.

The Rolling Thin Film Oven Test (RTFOT) procedure (ASTM D2872) was developed for the same purpose as the TFOT and designed to produce essentially the same effect as the TFOT procedure on asphalt cement. The advantages of the RTFOT over the TFOT are that (1) a larger number of samples can be tested at the same time, and (2) less time is required to perform the test. In the standard RTFOT procedure, the asphalt cement sample is placed in a specially designed bottle, which is then placed on its side on a rotating shelf, in an oven kept at 163°C, and rolled continuously for 85 minutes. Once during each rotation, the opening of the bottle passes an air jet, which provides fresh air to the asphalt in the bottle for increased oxidation rate.

While the RTFOT and TFOT have generally worked well on pure asphalts, problems were encountered when modified asphalts were used. Asphalts modified with crumb rubber and SBR tended to spill out from the RTFOT bottles during the RTFOT process. When TFOT was used on these modified binders, a thin skin tended to form on the surface of the modified asphalt, which reduced the homogeneity and the aging of the samples.

A feasible alternative to the RTFOT and TFOT for use on modified asphalts appears to be the modified rotavapor aging procedure [Sirin et al, 1998]. The rotavapor apparatus, which was originally used for recovery of asphalt from solution (ASTM D5404), was modified to work as an aging device for asphalt. The binder to be aged is placed in a rotating flask, which is immersed in a temperature-controlled oil bath. An air pump is used to provide a controlled air flow to the flask. Different aging effects can be produced by using different combinations of process temperature, process duration and sample size. Using a process temperature of 163°C, process duration of 165 minutes and a sample size of 200 g has been found to produce aging severity similar to that of the RTFOT.

Ring & Ball Softening Point Test

The ring and ball softening point test (ASTM D36) measures the temperature at which an asphalt reaches a certain softness. When an asphalt is at its softening point temperature, it has approximately a penetration of 800 or an absolute viscosity of 13,000 poises. This conversion is only approximate and can vary from one asphalt to another, due to the non-Newtonian nature of asphalts and the different shear rates used by these different methods.

The softening point temperature can be used along with the penetration to determine the temperature susceptibility of an asphalt. Temperature susceptibility of an asphalt is often expressed as:

$$M = [\log(p_2) - \log(p_1)] / (t_2 - t_1) \quad (45.3)$$

where M = temperature susceptibility

t_1, t_2 = temperatures in °C

p_1 = penetration at t_1

p_2 = penetration at t_2

Since an asphalt has approximately a penetration of 800 at the softening point temperature, the softening point temperature can be used along with the penetration at 25°C to determine the temperature susceptibility as:

$$M = [\log(\text{pen at } 25^\circ\text{C}) - \log(800)] / (25 - \text{S.P. Temp.}) \quad (45.4)$$

The M computed in this manner can then be used to compute a Penetration Index (PI) as follows:

$$PI = (20 - 500 M) / (1 + 50 M) \quad (45.5)$$

The Penetration Index is an indicator of the temperature susceptibility of the asphalt. A high PI indicates low temperature susceptibility. Normal asphalt cements have a PI between -2 and +2. Asphalt cements with a PI of more than +2 are of low temperature susceptibility, while those with a PI of less than -2 are of excessively high temperature susceptibility.

Conventional Methods of Grading and Specifications of Asphalt Cements

There are three conventional methods of grading asphalt cements. These three methods are (1) grading by penetration at 25°C, (2) grading by absolute viscosity at 60°C, and (3) grading by absolute viscosity of aged asphalt residue after the rolling thin film oven test (RTFOT) procedure. These three methods of grading and the associated ASTM specifications of asphalt cements are presented and discussed in this section.

The method of grading of asphalt cements by standard penetration at 25°C is the first systematic method developed and is still used by a few highway agencies in the world. The standard grades by this method include 40/50, 60/70, 85/100, 120/150 and 200/300 asphalts, which have penetrations of 40 to 50, 60 to 70, 85 to 100, 120 to 150, and 200 to 300, respectively. The Asphalt Institute recommends the use of a 120/150 or 85/100 pen. asphalt in the asphalt concrete for cold climatic condition with a mean annual temperature of 7°C or lower. For warm climatic condition with a mean air temperature between 7 and 24°C, a 85/100 or 60/70 pen. asphalt is recommended. For hot climatic condition with a mean annual air temperature of 24°C or greater, the use of a 40/50 or 60/70 pen. asphalt is recommended [Asphalt Institute, 1991].

ASTM D946 [ASTM, 2001] provides a specification for penetration-graded asphalt cements. [Table 45.1](#) shows the specification for 60/70 and 85/100 pen asphalts as examples. According to this specification, the only requirement on the consistency of the asphalt cements is the penetration at 25°C. There is no requirement on the consistency at either a higher or lower temperature, and thus no requirement on the temperature susceptibility of the asphalt cements. Two asphalts may be of the same penetration grade and yet have substantially different viscosities at 60°C. This problem is illustrated in [Fig. 45.1](#). Thus, it is clear that specifying the penetration grade alone will not ensure that the asphalt used will have the appropriate viscosities at the expected service temperatures. Other requirements in the specification are (1) minimum flash point temperature, (2) minimum ductility at 25°C, (3) minimum solubility in trichloroethylene, and (4) penetration and ductility at 25°C of the asphalt after aging by the TFOT procedure.

Since penetration is an empirical test, grading by penetration was thought to be unscientific. Considerable efforts were made in the 1960s to grade asphalts using fundamental units. Early attempts were

TABLE 45.1 Requirements for 60/70 and 85/100 Penetration Asphalt Cements

Test	Penetration Grade			
	60/70		85/100	
	Min	Max	Min	Max
Penetration at 25 °C, 0.1mm	60	70	85	100
Flash point (Cleveland open cup), °C	232	—	232	—
Ductility at 25 °C, cm	100	—	100	—
Solubility in trichloroethylene, %	99	—	99	—
Retained penetration after TFOT, %	52	—	47	—
Ductility at 25 °C after TFOT, cm	50	—	75	—

Source: ASTM 1994. ASTM D 946 Standard Specification for Penetration-Graded Asphalt Cement for Use in Pavement Construction, *Annual Book of ASTM Standards*, Volume 04.03, 1994, pp.91–92, Philadelphia, PA.

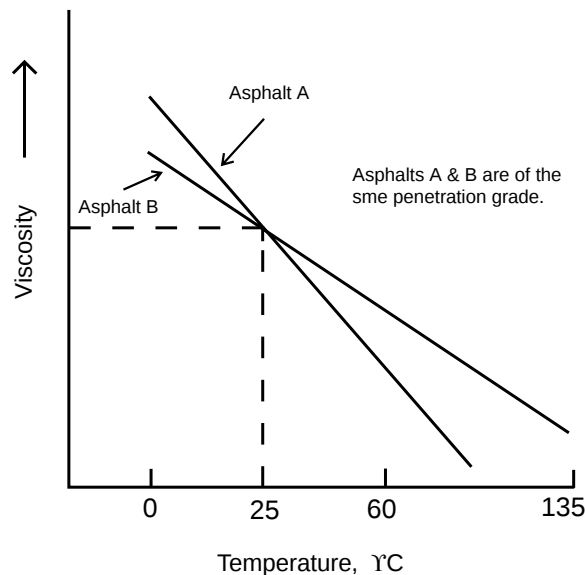


FIGURE 45.1 Variation in viscosity of two penetration-graded asphalts at different temperatures.

made to grade asphalts by viscosity at 25°C and 20°C. However, problems were encountered in measuring viscosity at such low temperatures. With some reluctance, the temperature for grading asphalt by viscosity was moved to 60°C, which represents approximately the highest temperature pavements may experience in most parts of the United States. When an asphalt is graded by this system, it is designated as AC followed by a number which represents its absolute viscosity at 60°C in units of 100 poises. For example, an AC-20 would have an absolute viscosity of around 2,000 poises at 60°C. An AC-20 roughly corresponds to a 60/70 pen. asphalt. However, due to the possible effects of different temperature susceptibility and non-Newtonian behavior, the conversion from a viscosity grade to a penetration grade may be different for different asphalts. Figure 45.2 shows the effects of different temperature susceptibility on the viscosity variation of two asphalts that have the same viscosity grade. In an effort to control this variation, the requirements for a minimum penetration at 25°C and a minimum viscosity at 135°C were added to the specification.

ASTM D3381 [ASTM, 2001] provides two different specifications for asphalt cements graded by absolute viscosity of the original asphalt at 60°C. Table 45.2 shows the requirements for AC-10 and AC-20 grade asphalts in the two specifications as examples. The main difference between these two specifications is that one of them requires a lower temperature susceptibility than the other. Limits on temperature

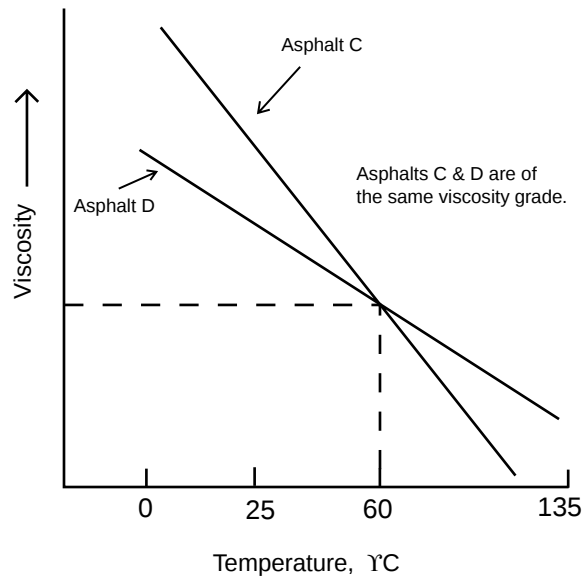


FIGURE 45.2 Variation in viscosity of two viscosity-graded asphalts at different temperatures.

TABLE 45.2 Requirements for AC-10 and AC-20 Asphalt Cements

Test on Original Asphalt	Viscosity Grade			
	AC-10		AC-20	
	Spec. 1	Spec. 2	Spec. 1	Spec. 2
Absolute Viscosity at 60 °C, poises	1000 ± 200	1000 ± 200	2000 ± 400	2000 ± 400
Kinematic Viscosity at 135 °C, cSt	150	250	210	300
Penetration at 25 °C, 0.1mm	70	80	40	60
Flash point (Cleveland open cup), °C	219	219	232	232
Solubility in trichloroethylene, min.,%	99.0	99.0	99.0	99.0
Tests on Residue				
Viscosity at 60 °C, max., poises	5000	5000	10,000	10,000
from TFOT				
Ductility at 25 °C, min., cm	50	75	20	50

Source: ASTM 1994. ASTM D 3381 Standard Specification for Viscosity-Graded Asphalt Cement for Use in Pavement Construction, *Annual Book of ASTM Standards*, Volume 04.03, 1994, pp.297–298, Philadelphia, PA.

susceptibility are specified through a minimum required penetration at 25°C and a minimum required kinematic viscosity at 135°C. The other requirements are similar to those in the specification of penetration-graded asphalts. The other requirements are (1) minimum flash point temperature, (2) minimum ductility at 25°C, (3) minimum solubility in trichloroethylene, and (4) required properties of the asphalt after aging by the TFOT procedure (by means of maximum viscosity at 60 °C and ductility at 25°C).

The third asphalt grading system is to grade asphalts according to their viscosity when placed on the road (after aging due to the heating and mixing process). This grading system has been adopted by several western states in the U.S. Grading is to be based on the absolute viscosity at 60°C of the asphalt residue after the Rolling Thin Film Oven Test (RTFOT) procedure, which simulates the effects of the hot-mix plant operation. An asphalt graded by this system is designated as AR followed by a number which represents the viscosity of the aged residue at 60°C in units of poises. For example, an AR-6000 would have an aged residue with an absolute viscosity of around 6000 poises. An AR-6000 would roughly correspond to an AC-20 or a 60/70 pen. asphalt. However, it should be recognized that the conversion from an AR grade to an AC grade depends on the hardening characteristics of the asphalt.

ASTM D3381 [ASTM, 2001] provides a specification for asphalt cements graded by viscosity of aged residue after the RTFOT process. Table 45.3 shows the specification for AR-4000 and AR-8000 grade

TABLE 45.3 Requirements for AR-4000 and AR-8000 Asphalt Cements

Test on Residue from RTFOT		Viscosity Grade	
		AR-4000	AR-8000
Absolute Viscosity at 60 °C, poises		4000 ± 1000	8000 ± 2000
Kinematic Viscosity at 135 °C, min., cSt		275	400
Penetration at 25 °C, min., 0.1 mm		25	20
% of original penetration, min.		45	50
Ductility at 25 °C, min., cm		75	75
Test on Original	Flash point (Cleveland open cup), min., °C	227	232
Asphalt	Solubility in trichloroethylene, min., %	99.0	99.0

Source: ASTM 1994. ASTM D 3381 Standard Specification for Viscosity-Graded Asphalt Cement for Use in Pavement Construction, *Annual Book of ASTM Standards*, Volume 04.03, 1994, pp.297–298, Philadelphia.

asphalts as examples. According to this specification, temperature susceptibility is specified through requiring a minimum penetration at 25°C and a minimum kinematic viscosity at 135°C of the residue after the RTFOT. Similar to the requirements in the specifications for the other two grading systems, there are requirements on (1) ductility at 25°C of the aged residue, (2) minimum flash point of the original asphalt, and (3) minimum solubility in trichloroethylene of the original asphalt. Another requirement in this specification is a minimum percent of retained penetration after the RTFOT, which can serve as a check on the composition and aging characteristics of the asphalt.

Superpave Binder Tests

The Strategic Highway Research Program (SHRP) conducted a \$50 million research effort from October 1987 through March 1993 to develop performance-based test methods and specifications for asphalts and asphalt mixtures. The resulting product is a new system called Superpave (Superior PERforming asphalt PAVements), which includes a binder specification and an asphalt mixture design method. The Superpave binder tests and specifications have been standardized by the American Association of State Highway and Transportation Officials (AASHTO). The significance of the Superpave binder tests are described in this section. The detailed procedures can be found in the AASHTO publications for these tests [AASHTO, 1999].

Pressure Aging Vessel

The Superpave Pressure Aging Vessel (PAV) procedure is used for simulation of long-term aging of asphalt binders in service. According to the method (AASHTO Designation PP1–98), the asphalt samples are first aged in the standard RTFOT. Pans containing 50 grams of RTFOT residue are then placed in the PAV, which is pressurized with air at 2.1 ± 0.1 MPa, and aged for 20 hours. As many as 10 pans can be placed in the PAV. The proposed range of PAV temperature to be used is between 90 and 110°C. The PAV temperature to be used will depend on the climatic condition of the region where the binders will be used. A higher PAV temperature could be used for a warmer climatic condition, while a lower temperature could be used for a colder climatic condition.

Dynamic Shear Rheometer Test

The dynamic shear rheometer test measures the viscoelastic properties of an asphalt binder by testing it in an oscillatory mode. The general method had been used by researchers long before the SHRP researchers adopted and standardized the method for the purpose of asphalt specification. Typically, in a dynamic shear rheometer test, a sample of asphalt binder is placed between two parallel steel plates. The top plate is oscillated by a precision motor with a controlled angular velocity, w , while the bottom plate remains fixed. From the measured torque and angle of rotation, the shear stress and shear strain can be calculated. The oscillatory strain, γ , can be expressed as:

$$\gamma = \gamma_o \sin wt \quad (45.6)$$

where γ_o = peak shear strain
 w = angular velocity in radian/second

The shear stress, τ , can be expressed as:

$$\tau = \tau_o \sin(wt + \delta) \quad (45.7)$$

where τ_o = peak shear stress
 δ = phase shift angle

The following parameters are usually computed from the test data:

$$(1) \text{ Complex Shear Modulus, } G^* = \tau_o / \gamma_o \quad (45.8)$$

$$(2) \text{ Dynamic Viscosity, } \eta^* = G^* / w \quad (45.9)$$

$$(3) \text{ Storage Modulus, } G' = G^* \cos \delta \quad (45.10)$$

$$(4) \text{ Loss Modulus, } G'' = G^* \sin \delta \quad (45.11)$$

$$(5) \text{ Loss Tangent, } \tan \delta = G'' / G' \quad (45.12)$$

How are the results of a dynamic rheometer test related to the basic rheologic properties of the tested binder? This question can be answered by analyzing how a viscoelastic material would behave in such a test. For simplicity, the test binder is modeled by a Maxwell model with a shear modulus of G and a viscosity of η . When the test binder is modeled in this manner, it can be shown analytically that the complex shear modulus, G^* is equal to:

$$G^* = \tau_o / \gamma_o = w\eta / \left(1 + \eta^2 w^2 / G^2\right)^{1/2} \quad (45.13)$$

It can be noted that, from the above equation, at very high w , the dynamic modulus G^* will approach the true shear modulus G .

The dynamic viscosity, η^* , can be derived to be:

$$\eta^* = G^* / w = \eta / \left(1 + \eta^2 w^2 / G^2\right)^{1/2} \quad (45.14)$$

It can be noted that, at very low w , the dynamic viscosity η^* will approach the true viscosity η . The dynamic viscosity determined at very low w has been referred to as “zero shear viscosity”.

The loss tangent, $\tan \delta$, can be derived to be:

$$\tan \delta = G / w\eta \quad (45.15)$$

SHRP standardized the dynamic shear rheometer test for use in measuring the asphalt properties at high and intermediate service temperatures for specification purposes. In the standardized test method (AASHTO Designation TP5–98), the oscillation speed is specified to be 10 radians/second. The amplitude of shear strain to be used depends on the stiffness of the binder, and varies from 1% for hard materials tested at low temperatures to 13% for relatively softer materials tested at high temperatures. There are two standard sample sizes. For relatively softer materials, a sample thickness (gap) of 1 mm and a sample diameter (spindle diameter) of 25 mm are to be used. For harder materials, a sample thickness of 2 mm

and a sample diameter of 8 mm are to be used. The two values to be measured from each test are the complex shear modulus, G^* , and the phase angle, δ . These two test values are then used to compute $G^*/\sin \delta$ and $G^*\sin \delta$. In the Superpave asphalt specification, permanent deformation is controlled by requiring the $G^*/\sin \delta$ of the binder at the highest anticipated pavement temperature to be greater than 1.0 kPa before aging and 2.2 kPa after the RTFOT process. Fatigue cracking is controlled by requiring that the binder after PAV aging should have a $G^*\sin \delta$ value of less than 5000 kPa at a specified intermediate pavement temperature.

Bending Beam Rheometer Test

The bending beam rheometer test (AASHTO Designation TP1–98) was used to measure the stiffness of asphalts at low service temperatures. The standard asphalt test specimen is a rectangular prism with a width of 12.5 mm, a height of 6.25 mm and a length of 125 mm. The test specimen is to be submerged in a temperature-controlled fluid bath and to be simply supported with a distance between supports of 102 mm. For specification testing, the test samples are to be fabricated from PAV-aged asphalt binders, which simulate the field-aged binders. In the standard testing procedure, after the beam sample has been properly pre-conditioned, a vertical load of 100 gram-force is applied to the middle of the beam for a total of 240 seconds. The deflection of the beam at the point of load is recorded during this period, and used to compute for the creep stiffness of the asphalt by the following equation:

$$S(t) = PL^3/4bh^3\delta(t) \quad (45.16)$$

where: $S(t)$ = creep stiffness at time t

P = applied load, 100 g

L = distance between beam supports, 102 mm

b = beam width, 12.5 mm

h = beam height, 6.25 mm

$\delta(t)$ = deflection at time t

The above equation is similar to the equation that relates the deflection at the center of the beam to the elastic modulus of an elastic beam according to the classical beam theory. The instantaneous deflection in the original equation is replaced by the time dependent deflection $\delta(t)$, while the elastic modulus in the original equation is replaced by the time dependent creep stiffness $S(t)$.

For Superpave binder specification purpose, the bending beam rheometer test is to be run at 10°C above the expected minimum pavement temperature, $T_{\min}$. SHRP researchers [Anderson & Kennedy, 1993] claimed that the stiffness of an asphalt after 60 seconds at $T_{\min} + 10^\circ\text{C}$ is approximately equal to its stiffness at $T_{\min}$ after 2 hours loading time, which is related to low-temperature cracking potential. The Superpave binder specification as stated in AASHTO Designation MP1–98 [AASHTO, 1999] requires the stiffness at the test temperature after 60 seconds to be less than 300 MPa to control low-temperature cracking.

The second parameter obtained from the bending beam rheometer test result is the m -value. The m -value is the slope of the log stiffness versus log time curve at a specified time. A higher m -value would mean that the asphalt would creep at a faster rate to reduce the thermal stress and would be more desirable to reduce low-temperature cracking. The Superpave binder specification as stated in AASHTO MP1–98 requires the m -value at 60 seconds to be greater than or equal to 0.30.

Direct Tension Test

The Superpave direct tension test (AASHTO Designation TP3–98) measures the stress-strain characteristics of an asphalt binder in direct tension at low temperature. In this test, a small “dog bone” shaped asphalt specimen is pulled at a constant rate of 1 mm/min until it breaks. The amount of elongation at failure is used to compute the failure strain. The maximum tensile load taken by the specimen is used to compute the failure stress. The test specimen is 30 mm long and has a cross section of 6 mm by 6 mm at the middle portion. For Superpave binder specification purpose, the direct tension test is to be run

on PAV-aged binders at the same test temperature as for the bending beam rheometer test, which is run at 10°C above the minimum expected pavement temperature. According to the Superpave binder specification as stated in AASHTO Designation MP1–98, the failure strain at this condition should not be less than 1% in order to control low temperature cracking.

Brookfield Rotational Viscometer Test

The Superpave binder specification uses the Brookfield rotational viscometer test as specified by ASTM D4402 for use in measuring the viscosity of binders at elevated temperatures to ensure that the binders are sufficiently fluid when being pumped and mixed at the hot mix plants. In the Brookfield rotational viscometer test, the test binder sample is held in a temperature-controlled cylindrical sample chamber, and a cylindrical spindle, which is submerged in the sample, is rotated at a specified constant speed. The torque that is required to maintain the constant rotational speed is measured and used to calculate the shear stress according to the dimensions of the sample chamber and spindle. Similarly, the rotational speed is used to calculate the shear rate of the test. Viscosity is then calculated by dividing the computed shear stress by the computed shear rate.

As compared with the capillary tube viscometers, the rotational viscometer provides larger clearances between the components. Therefore, it can be used to test modified asphalts containing larger particles, which could plug up a capillary viscometer tube. Another advantage of the rotational viscometer is that the shear stress versus shear rate characteristics of a test binder can be characterized over a wide range of stress or strain levels.

For Superpave binder specification purpose, the rotational viscosity test is to be run on the original binder at 135°C. The maximum allowable viscosity at this condition is 3 Pa-s.

Superpave Binder Specification

The Superpave performance graded asphalt specification (AASHTO Designation MP1–98) uses grading designations which correspond to the maximum and minimum pavement temperatures of the specified region. The designation starts with “PG,” and is followed by the maximum and the minimum anticipated service temperature in °C. For example, A “PG-64–22” grade asphalt is intended for use in a region where the maximum pavement temperature (based on average 7-day maximum) is 64°C and the minimum pavement temperature is –22°C. A “PG-52–46” grade asphalt is for use where the maximum pavement temperature is 52°C and the minimum pavement temperature is –46°C.

Table 45.4 shows the Superpave specification for three different performance grades of asphalts (PG-52–16, PG-52–46 & PG-64–22) as examples. The specified properties are constant for all grades, but the temperatures at which these properties must be achieved vary according to the climate in which the binder is to be used. It is possible that an asphalt can meet the requirements for several different grades.

All grades are required to have a flash point temperature of at least 230°C for safety purpose, and to have a viscosity of no greater than 3 Pa-s at 135°C to ensure proper workability during mixing and placement.

Dynamic shear rheometer tests are to be run on the original and RTFOT-aged binders at the maximum pavement design temperature. The minimum required values of $G^*/\sin\delta$ at this temperature are 1.0 kPa and 2.2 kPa for the original and RTFOT-aged binders, respectively. These requirements are intended to control pavement rutting.

Dynamic shear rheometer tests are also to be run on PAV-aged binders at an intermediate temperature, which is equal to 4°C plus the mean of the maximum and minimum pavement design temperatures. For example, for a PG-52–46 grade, the intermediate temperature is 7°C. The maximum allowable value of $G^*\sin\delta$ at this condition is 5000 kPa. This requirement is intended to control pavement fatigue cracking.

Bending beam rheometer tests and direct tension tests are to be run on PAV-aged binders at a temperature which is 10°C above the minimum pavement design temperature. For example, for a PG-52–46, the test temperature is –36°C. At a loading time of 60 seconds, the stiffness is required to be no greater than 300 MPa, and the m-value is required to be no less than 0.3. The failure strain from the

TABLE 45.4 Examples of Superpave Performance Graded Binder Specification

Performance Grade	PG-52-16	PG-52-40	PG-64-22
Average 7-Day Maximum Pavement Design Temperature, °C	52	52	64
Minimum Pavement Design Temperature, °C	-16	-40	-22
Original Binder			
Flash Point Temperature, Minimum, °C	230		
Viscosity: Maximum, 3 Pa·s Test Temperature, °C	135		
Dynamic Shear @ 10 rad/s : G*/sinδ, Minimum, 1.00 kPa Test Temperature, °C	52	52	64
Rolling Thin Film Oven Residue			
Mass Loss, Maximum, %	1.00		
Dynamic Shear @ 10 rad/s : G*/sinδ, Minimum, 2.20 kPa Test Temperature, °C	52	52	64
Pressure Aging Vessel Residue			
PAV Aging Temperature, °C	90	100	100
Dynamic Shear @ 10 rad/s : G* sinδ, Maximum, 5000 kPa Test Temperature, °C	22	7	25
Creep Stiffness @ 60 s : S, Maximum, 300 MPa m-value, Minimum, 0.30 Test Temperature, °C	-6	-36	-12
Direct Tension @ 1.0 mm/min : Failure Strain, Minimum, 1.0% Test Temperature, °C	-6	-36	-12

Source: AASHTO 1999. AASHTO Designation MP1 Standard Specification for Performance Graded Asphalt Binder, *AASHTO Provisional Standard*, Washington, D.C.

direct tension test is required to be at least 1%. However, the direct tension test criterion is applicable only if an asphalt does not meet the bending beam rheometer stiffness requirement and has a stiffness between 300 MPa and 600 MPa.

It is to be pointed out that AASHTO is in the process of revising the low-temperature criteria based on the bending beam rheometer and direct tension test results. It is expected that the revised criteria will be incorporated in an AASHTO Designation MP1(a), which is to be published in 2002.

Effects of Properties of Asphalt Binders on the Performance of Asphalt Pavements

Effects of Viscoelastic Properties of Asphalt

When an asphalt concrete surface is cooled in winter time, stresses are induced in the asphalt concrete. These stresses can be relieved by the flowing of the asphalt binder within the asphalt mixture. However, if the viscosity of the asphalt binder is too high at this low temperature, the flow of the asphalt binder may not be fast enough to relieve the high induced stresses. Consequently, low-temperature cracking may occur. The viscosity of asphalt at which low-temperature cracking would occur is dependent on the cooling rate of the pavement as well as the characteristics of the asphalt concrete. However, as a rough prediction of low-temperature cracking, a limiting viscosity of 2×10^{10} poises could be used [Davis, 1987]. If the viscosity of the asphalt binder at the lowest anticipated temperature is kept lower than this limiting value, low-temperature cracking would be unlikely to occur.

The effects of the elastic property of asphalt on low-temperature cracking can be understood by analyzing how a viscoelastic material as modeled by a Maxwell model with a shear modulus of G and a viscosity of η would release its stress after it is subjected to a forced strain γ_0 (which could be caused by a sudden drop in pavement temperature). If the material is subjected to a forced strain of γ_0 at $t = 0$, the instantaneous induced stress would be equal to $\gamma_0 G$, but the stress will decrease with time according to the following expression:

$$\tau = \gamma_0 G e^{-Gt/\eta} \quad (45.17)$$

It can be seen that the rate of stress release is proportional to G/η . The reciprocal of this parameter, η/G , is commonly known as the relaxation time. To maximize the rate of relaxation, it is desirable to have a low relaxation time, η/G , or a higher G/η . As presented in Section 45.15, the parameter $\tan \delta$ as obtained from the dynamic shear rheometer test is directly proportional to G/η . Thus, a high $\tan \delta$ value would be desirable to reduce the potential for low-temperature pavement cracking. Experimental data show that $\tan \delta$ of an asphalt always decrease with decreasing temperature. Goodrich [1991] stated that when testing is done at an angular velocity, w , of 0.1 radian/second, the temperature at which $\tan \delta$ of the binder is equal to 0.4 corresponds approximately to the temperature at which the asphalt mixture would reach its limiting stiffness.

Another critical condition of an asphalt concrete is at the highest pavement temperature, at which the asphalt mixture is the weakest and most susceptible to plastic flow when stressed. When the other factors are kept constant, an increase in the viscosity of the asphalt binder will increase the shear strength and subsequently the resistance to plastic flow of the asphalt concrete. With respect to resistance to plastic flow of the asphalt concrete, it is preferable to have a high asphalt viscosity at the highest anticipated pavement temperature. Results by Goodrich [1988] indicate that a low $\tan \delta$ value of the binder (as obtained from the dynamic rheometer test) tends to correlate with a low creep compliance of the asphalt mixture, which indicates high rutting resistance. Thus, a low $\tan \delta$ value of the binder is desirable to reduce rutting potential.

The effectiveness of the mixing of asphalt cement and aggregate, and the effectiveness of the placement and compaction of the hot asphalt mix are affected greatly by the viscosity of the asphalt. The Asphalt Institute recommends that the mixing of asphalt cement and aggregate should be done at a temperature where the viscosity of the asphalt is 1.7 ± 0.2 poises. Compaction should be performed at a temperature where the viscosity of the asphalt cement is 2.8 ± 0.3 poises [Epps et al, 1983]. These viscosity ranges are only offered as guidelines. The actual optimum mixing and compaction temperatures will depend on the characteristics of the mixture as well as the construction environment.

In the selection of a suitable asphalt cement to be used in a certain asphalt paving project, the main concerns are (1) whether the viscosity of the asphalt at the lowest anticipated service temperature would not be low enough to avoid low-temperature cracking of the asphalt concrete, (2) whether the viscosity of the asphalt at the highest anticipated temperature would be high enough to resist rutting, and (3) whether the required temperatures for proper mixing and placement would not be too high.

Effects of Newtonian and Non-Newtonian Flow Properties of Asphalt

The flow behavior of asphalt cements can be classified into four main categories, namely (1) Newtonian, (2) pseudoplastic, (3) Bingham-plastic, and (4) dilatant. Asphalt cements usually exhibit Newtonian or near-Newtonian flow behavior, especially at temperatures in excess of 25°C. A Newtonian flow behavior is characterized by a linear shear stress-shear rate relationship, as shown in Figure 45.3. The shear susceptibility, C , is defined as the slope of the plot of $\log(\text{shear stress})$ vs. $\log(\text{shear rate})$. For a Newtonian flow behavior, C is equal to 1.00.

The type of flow behavior where a reduction in viscosity is experienced with increased stress is termed “pseudoplastic.” The shear stress-shear rate relationship for a pseudoplastic fluid is shown in Fig. 45.4. It can be seen that the shear rate increases more rapidly at higher stresses. The shear susceptibility, C , is less than 1.0 in this case.

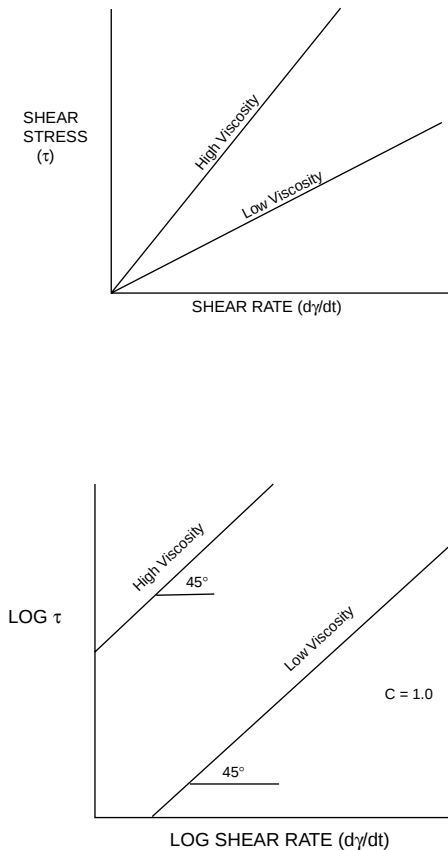


FIGURE 45.3 Newtonian flow characteristics.

The shear stress-shear rate relationship for a “Bingham plastic” material is illustrated in Figure 45.5. When the stress is below a certain stress level, there is no flow. When the stress is above the yield point, the flow characteristic is likely to be highly pseudoplastic with a C of less than 0.5. Highly air-blown asphalts usually exhibit Bingham plastic behavior at low temperatures.

The type of flow behavior where the apparent viscosity increases with increased stress is referred to as “dilatant.” The shear stress-shear rate characteristics of dilatant behavior are shown in Fig. 45.6. For this type of flow behavior, C is greater than 1.

What are the effects of the flow behavior of the asphalt cement on the performance of the asphalt pavement? The answer to these questions is still not definitive at this point. However, some research results have indicated that asphalts with high shear susceptibility (c) have been related to tender mixes [Epps, Button and Gallaway 1983], and to high temperature susceptibility and high aging indices [Kandhal, Sandvig and Wenger 1973].

The effect of non-Newtonian flow behavior on the measured viscosity is clear. When an asphalt exhibits a non-Newtonian flow behavior, the measured viscosity will change as the shear stress or shear rate used for the test changes. This effect must be properly accounted for. When the viscosity of the asphalt is used to predict the behavior of the asphalt concrete in service, the viscosity at a stress level close to the anticipated stress level in service should be used.

Effects of Hardening Characteristics of Asphalt

An important factor that affects the durability of an asphalt concrete is the rate of hardening of the asphalt binder. The causes of hardening of asphalt have been attributed to oxidation, loss of volatile oils,

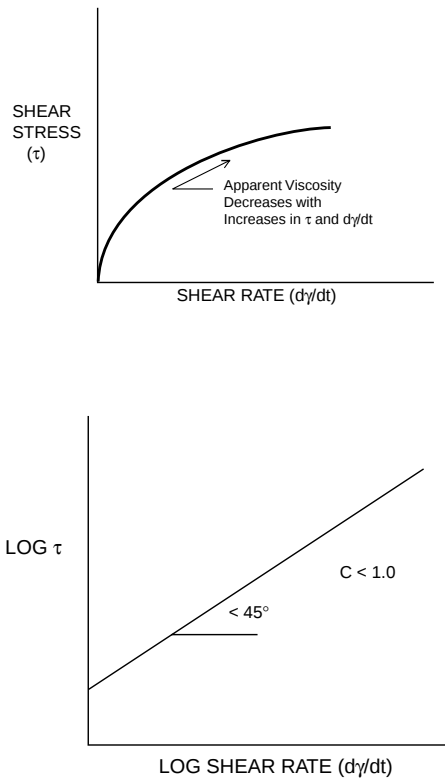


FIGURE 45.4 Pseudoplastic flow characteristics.

and polymerization (changes in structure). Among all these possible factors, oxidation is generally considered to be the prime cause of asphalt hardening.

The most severe hardening of asphalt occurs during the mixing process. The viscosity of the asphalt binder immediately after the asphalt concrete is placed on the road is usually 2 to 4 times the viscosity of the original asphalt cement. The asphalt binder continues to harden through service; its viscosity could reach as high as 10 to 20 times the viscosity of the original asphalt cement. The rate of asphalt hardening is dependent on asphalt composition, mixing temperature, air voids content, and climatic conditions. It usually increases with increased mixing temperature, increased air voids content in the asphalt mix, and increased service (air) temperature.

Excessive hardening of the asphalt binder will cause the asphalt concrete to be too brittle and low-temperature cracking to occur. It may also cause the asphalt binder to partially lose its adhesion and cohesion, and subsequently it may cause raveling (progressive disintegration of pavement material and separation of aggregates from it) in the asphalt concrete.

A certain amount of hardening of the asphalt binder during the mixing process is usually expected and designed for. If an asphalt binder has not hardened sufficiently during the mixing process (due to low mixing temperature or the peculiar nature of the asphalt), the asphalt binder may be too soft at placement. This may cause the asphalt mix to be difficult to compact (tender mix) and to have a low resistance to rutting in service. If the tenderness of an asphalt concrete disappears within a few weeks after construction, the problem is most likely caused by slow setting asphalt. This type of asphalt requires an excessive amount of time to “set up” after they are heated up and returned to normal ambient temperature. Asphalts containing less than 10 percent asphaltenes appear to have a greater probability of producing slow-setting asphalt mixtures. Asphaltene is the high molecular weight fraction of asphalt which can be separated from the other asphalt fractions by dissolving an asphalt in a specified solvent (such as n-heptane as used in the ASTM D4124 Methods for Separation of Asphalt into Four Fractions)

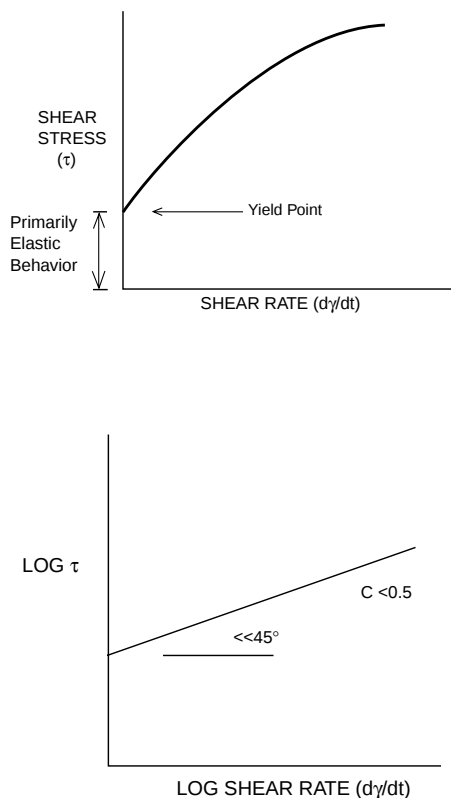


FIGURE 45.5 Bingham-plastic flow characteristics.

[ASTM, 2001]. The asphaltenes, which are insoluble in the solvent, would be precipitated out in this method.

Types and Grades of Cutback Asphalts

Cutback asphalts are classified into three main types on the basis of the relative speed of evaporation of the solvents in them. A Rapid-Curing (RC) cutback asphalt is composed of an asphalt cement and a solvent of a volatility similar to that of naphtha or gasoline, which evaporates at a fast speed. A Medium-Curing (MC) cutback asphalt contains a solvent of a volatility similar to that of kerosine, which evaporates at a medium speed. A Slow-Curing (SC) cutback asphalt contains an oil of relatively low volatility.

Within each type, cutback asphalts are graded by kinematic viscosity at 60°C. It is designated by the type followed by the lower limit of the kinematic viscosity at 60°C in units of centi-stokes (cSt). The upper limit for the viscosity is twice its lower limit. For example, an “RC-70” designates a rapid-curing cutback asphalt with a kinematic viscosity at 60°C ranging between 70 and 140 cSt, while an “SC-800” designates a slow-curing cutback asphalt with a viscosity ranging between 800 and 1600 cSt. The standard specifications for SC, MC and RC cutback asphalts can be found in ASTM Designation D2026, D2027 and D2028, respectively [ASTM, 2001].

The standard practice for selection of cutback asphalts for pavement construction and maintenance can be found in ASTM Designation D2399 [ASTM, 2001].

Types and Grades of Emulsified Asphalts

Emulsified asphalts (or asphalt emulsions) are divided into three major kinds, namely anionic, cationic and nonionic, on the basis of the electrical charges of the asphalt particles in the emulsion. An anionic

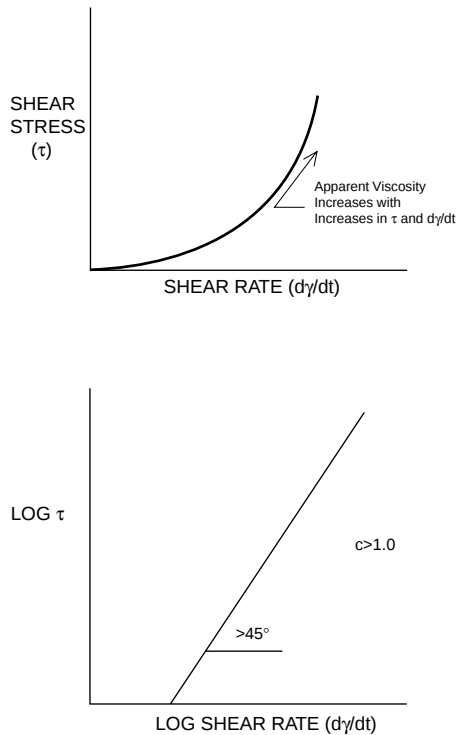


FIGURE 45.6 Dilatant flow characteristics.

asphalt emulsion has negatively-charged asphalt particles, and is usually more suitable for use with calcareous aggregates, which tend to have positive surface charges. A cationic asphalt emulsion has positively charged asphalt particles, and is usually more suitable for use with siliceous aggregates, which tend to have negative surface charges. A nonionic asphalt emulsion contains asphalt particles that are electrically neutral. Nonionic asphalt emulsions are not used in pavement applications.

Asphalt emulsions are further classified into three main types on the basis of how quickly the suspended asphalt particles revert to asphalt cement. The three types are Rapid-setting (RS), Medium-Setting (MS) and Slow-Setting (SS). An RS emulsion is designed to demulsify (to break away from the emulsion form such that asphalt particles are no longer in suspension) upon contact with an aggregate, and thus has little or no ability to mix with an aggregate. It is best used in spraying applications where mixing is not required but fast setting is desirable. An MS emulsion is designed to have good mixing characteristics with coarse aggregates and to demulsify after proper mixing. It is suitable for applications where mixing with coarse aggregate is required. An SS emulsion is designed to be very stable in the emulsion form, and is suitable for use where good flowing characteristics are desired or where mixing with fine aggregates is required. The three types of cationic asphalt emulsions are denoted as CRS, CMS and CSS. The absence of the letter “C” in front of the emulsion type denotes an anionic type.

Two other standard types of anionic asphalt emulsions available are High-Float Rapid Setting (HFRS) and High-Float Medium Setting (HFMS). This type of asphalt emulsion contains an asphalt cement which has a Bingham plastic characteristic (resistant to flow at low stress level). This flow property of the asphalt permits a thicker film coating on an aggregate without danger of runoff.

Within each type, asphalt emulsions are graded by the viscosity of the emulsion and the hardness of the asphalt cement. The lower viscosity grade is designated by a number “1” and the higher viscosity grade is designated by a number “2,” which is placed after the emulsion type. A letter “h” that follows the number “1” or “2” designates that a harder asphalt cement is used. For example, an “RS-1” designates a rapid-setting anionic type with a relatively low viscosity. An “HFMS-2h” designates a high-float medium

setting anionic type having a relatively higher viscosity and containing a hard base asphalt. A “CSS-1h” designates a slow-setting cationic type having a relatively lower viscosity and containing a hard base asphalt. Standard specifications for anionic and cationic emulsified asphalts can be found in ASTM Designations D977, and D2397, respectively.

The standard practice for selection and use of emulsified asphalts in pavement construction and maintenance can be found in ASTM Designation D3628 [ASTM, 2001].

45.3 Bituminous Mixtures

Types of Bituminous Mixtures used in Pavement Construction

A *bituminous mixture* is a combination of bituminous materials (as binders), properly graded aggregates and additives. Since tar is rarely used in bituminous mixtures in recent years and asphalt is the predominant binder material used, the term “*asphalt mixture*” is now more commonly used to denote a combination of asphalt materials, aggregates and additives. Asphalt mixtures used in pavement applications are usually classified by (1) their methods of production, or (2) their composition and characteristics.

Classification by Method of Production

Hot-mix asphalt (HMA) is produced in a hot asphalt mixing plant (or hot-mix plant) by mixing a properly controlled amount of aggregate with a properly controlled amount of asphalt at an elevated temperature. The mixing temperature has to be sufficiently high such that the asphalt is fluidic enough for proper mixing with and coating the aggregate, but not too high as to avoid excessive aging of the asphalt. A HMA mixture must be laid and compacted when the mixture is still sufficiently hot so as to have proper workability. HMA mixtures are the most commonly used paving material in surface and binder courses in asphalt pavements.

Cold-laid plant mix is produced in an asphalt mixing plant by mixing a controlled amount of aggregate with a controlled amount of liquid asphalt without the application of heat. It is laid and compacted at ambient temperature.

Mixed-in-place or road mix is produced by mixing the aggregates with the asphalt binders in proper proportions on the road surface by means of special road mixing equipment. A medium setting (MS) asphalt emulsion is usually used for open-graded mixtures while a slow setting (SS) asphalt emulsion is usually used for dense-graded mixtures.

Penetration macadam is produced by a construction procedure in which layers of coarse and uniform-size aggregate are spread on the road and rolled, and sprayed with appropriate amounts of asphalt to penetrate the aggregate. The asphalt material used may be hot asphalt cement or a rapid setting (RS) asphalt emulsion.

Classification by Composition and Characteristics

Dense-graded HMA mixtures, which use a dense-graded aggregate and have a relatively low air voids after placement and compaction, are commonly used as surface and binder courses in asphalt pavements. The term *Asphalt Concrete* is commonly used to refer to a high-quality, dense-graded HMA mixture.

A dense graded HMA mixture with maximum aggregate size of greater than 25 mm (1 in.) is called a *large stone dense-grade HMA* mix. A dense-grade HMA mix with 100% of the aggregate particles passing the 9.5 mm (3/8 in.) sieve is called a *sand mix*.

Open-graded asphalt mixtures, which use an open-graded aggregate and have a relatively high air void after placement and compaction, are used where high water permeability is desirable. Two primary types of open-graded mixes are (1) *open-graded base mix* and (2) *open-graded friction course (OGFC)*.

Open-graded base mixes are used to provide a strong base for an asphalt pavement as well as rapid drainage for subsurface water. Open-graded base mixes usually use a relatively larger size aggregate that contains very little or no fines. Due to the lower aggregate surface area, these mixes have relatively lower

asphalt content than that of a dense-graded HMA mix. Open-graded base mixes can be produced either hot or cold in an asphalt plant.

Open-graded friction courses (OGFC) are placed on top of surface courses to improve skid resistance and to reduce hydroplaning of the pavement surface. OGFC mixtures use aggregates with a small proportion of fines to produce high air voids and good drainage characteristics. Even though the voids content is higher, the asphalt film thickness is usually greater than that for a dense-graded HMA, and thus a typical OGFC mixture has about the same or higher asphalt content than that of a dense-graded HMA. A typical OGFC uses an aggregate of $\frac{1}{2}$ in. (12.5mm) maximum size, and is placed at a thickness of $\frac{3}{4}$ in. (19 mm). An OGFC mixture is produced in a hot-mix plant in the same way as a dense-graded HMA mixture. Crumb rubber modified asphalt has been used in OGFC mixtures in recent years to improve their performance and durability. Due to the higher viscosity of the crumb rubber modified binder, thicker film thickness can be used. This results in a higher binder content and thus better durability for the crumb rubber modified OGFC mixtures.

Stone Matrix Asphalt (SMA), which was originally developed in Europe, was a special asphalt mixture of improved rutting resistance and increased durability. SMA mixtures are designed to have a high coarse aggregate content (typically 70–80%), a high binder content (typically over 6%) and high filler content (typically about 10%). Asphalts modified with polymers and/or fibers are typically used. The improved rutting resistance of the SMA mixture is attributed to the fact that it carries the load through the coarse aggregate matrix (or the stone matrix), as compared with a dense-graded HMA, which carries the load through the fine aggregate. The use of polymer and/or fiber modified asphalts, which have increased viscosity, and the use of high filler content, which increases the stiffness of the binder, allow the SMA mixtures to have a higher binder film thickness and higher binder content without the problem of draindown of asphalt during construction. The increased durability of the SMA mixtures can be attributed to the higher binder film thickness and the higher binder content. SMA mixtures require the use of strong and durable aggregates with a relatively lower L.A. Abrasion Loss. SMA mixtures can be produced in a hot-mix plant in a similar way as a dense-grade HMA mixture. The main disadvantage of using a SMA as compared with a dense-grade HMA is its relatively higher cost due to the requirement for the use of higher quality aggregates, polymer, fibers and fillers.

Effects of Aggregate Characteristics on Performance of Asphalt Pavements

Aggregate makes up 90 to 95% by weight and 75 to 85% by volume of most asphalt paving mixtures. Aggregate provides most of the load-bearing capacity of the asphalt mixture. Thus, the performance of an asphalt mixture is greatly influenced by the properties of the aggregate used. The effects of aggregate characteristics on the performance of asphalt pavements, and the commonly used methods to determine these aggregate characteristics are presented in this section.

Aggregate Gradation

One of the most important characteristics of an aggregate, which affect the performance of an asphalt mixture, is its gradation. The properties of an asphalt mixture could be changed substantially when the aggregate gradation is altered.

What is the ideal gradation of an aggregate to be used in asphalt mixture? From the standpoint of achieving maximum strength and bearing capacity of the asphalt mixture, since an asphalt mixture derives its strength mainly from the aggregate, it would be preferable to have a well-graded aggregate to achieve maximum volume of aggregate in the mix. However, if the aggregate is too well graded, the voids in mineral aggregate (VMA) of the mix may be too low to accommodate the proper amount of asphalt, which is needed to produce a certain minimum asphalt film thickness on the aggregate surface. If the VMA is too low and the required amount of asphalt is added to the mix, the phenomenon of bleeding may occur as there would not be enough voids in the mix to accommodate the asphalt. However, if a lower asphalt content is used, the asphalt film thickness on the aggregate may be too low. The asphalt concrete produced would not be durable, and the problem of raveling may occur. Thus, the ideal aggregate

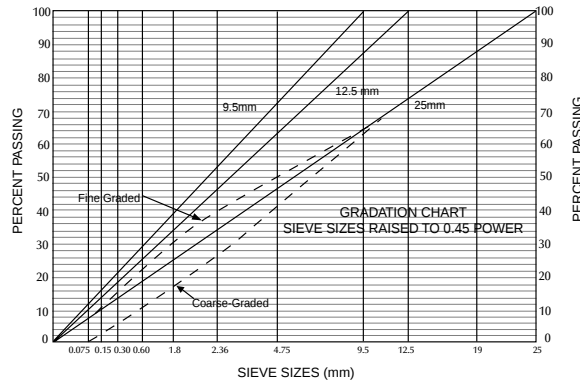


FIGURE 45.7 Maximum density gradations plotted on a 0.45 power gradation chart.

should be fairly well graded to produce a high volume of aggregate in the asphalt mix, but it should not be too well graded such that the VMA becomes too low.

Figure 45.7 shows a 0.45 power gradation chart, which was developed by the Federal Highway Administration (FHWA) of the U.S. in 1962 to plot aggregate gradation. The 0.45 power gradation chart was chosen so that a gradation that plots as a straight line on the chart would define the maximum density gradation. Three different maximum-density aggregate gradations are shown in Fig. 45.7, each with a different top size coarse aggregate. The equation for the FHWA's maximum density gradation is:

$$P = 100 \left(\frac{d}{D} \right)^{0.45} \quad (45.18)$$

where P = total percent passing the specific sieve
 d = the specific sieve size in question
 D = maximum size of the aggregate

Asphalt mixes that have aggregate gradations that plot above the maximum density line are called fine-graded mixtures. Conversely, mixes that have gradations that fall below the maximum density line are called coarse-graded mixes. When the other factors are constant, the coarser-graded aggregate will require less asphalt binder in the mix to achieve adequate coating and mix properties. It is also more tolerant to an increase in asphalt content than the finer-graded mixtures. In general, the coarser mixes, when properly designed, are more resistant to permanent deformation [Ruth et al, 1989].

Recommended aggregate gradation specifications for dense-grade asphalt mixtures of various nominal maximum aggregate sizes can be found in ASTM Standard Specification D3515 [ASTM, 2001]. For the most part, the FHWA maximum density curves fall inside the limits of these gradation specifications for the corresponding maximum aggregate sizes.

The aggregate gradation specification in the Superpave mix design method is presented in the subsection "Superpave Mix Design Method" in this section.

Maximum Aggregate Size

Maximum aggregate size is the smallest sieve through which 100% of the aggregate particle pass. Generally, using a larger maximum aggregate size in the asphalt mixture will increase the bearing capacity and rutting resistance of the asphalt pavement. Using a larger maximum aggregate size also reduces the design asphalt content and cost of the mix. However, mixtures using a larger stone size are harder to place and to compact to the desired smoothness. The lift thickness also limits the maximum aggregate size to be used. The maximum aggregate size is limited to 0.5 times the lift thickness.

Asphalt mixture designations typically use the *nominal maximum aggregate size* rather than the maximum aggregate size. However, the definition of nominal maximum size may vary from one agency to another. ASTM C125 Standard [ASTM, 2001] defines nominal maximum size as the smallest sieve

opening through which the entire amount of the aggregate is permitted to pass, but up to 10 percent of the aggregate may be retained on the nominal maximum size. In the Superpave mix design system, nominal maximum size is defined as one sieve size larger than the first sieve to retain more than 10 percent, while maximum size is one sieve larger than the nominal maximum size.

Mineral Filler

Mineral filler is the aggregate finer than the No. 200 mesh size. Proper amount of mineral filler added to an asphalt mixture could improve the performance of the mix. Adding mineral filler to an asphalt mixture has the effect of increasing the apparent viscosity of the asphalt binder. It could be used to decrease mixture tenderness during placement. Increased filler content reduces the VMA in the asphalt mixture. A mineral filler content of 2 to 6% is usually used in dense-grade HMA mixtures. General requirements for mineral filler can be found in ASTM D242 Standard Specification for Mineral Filler For Bituminous Paving Mixtures [ASTM, 2001].

Affinity for Asphalt

The affinity for asphalt of an aggregate is its tendency to accept and retain an asphalt coating. An asphalt concrete using an aggregate with high affinity for asphalt will be less susceptible to stripping of asphalt when exposed to water and thus more durable. Aggregates that are basic, such as limestone and dolomite, are usually less susceptible to stripping. Aggregates that are more acidic, such as sand and gravel, are usually more susceptible to stripping. However, there are exceptions to this generalization. Some limestones have been known to have stripping problems, while some gravels have been known to have no stripping problem at all.

Stripping resistance of an aggregate is typically evaluated by testing the asphalt aggregate mixture. A commonly used test for this purpose is AASHTO Designation T 283 Standard Test for Resistance of Compacted Bituminous Mixture to Moisture Induced Damage [AASHTO 1997]. In this test, a set of replicate specimens of the asphalt mixtures to be evaluated are compacted to $7 \pm 1\%$ air voids. The specimens are divided into two subsets. One subset is tested in the dry condition for indirect tensile strength. The other subset is subjected to vacuum saturation followed by a freeze and warm-water soaking cycle and then tested for indirect tensile strength. The tensile strength ratio, which is calculated by dividing the average tensile strength of the conditioned subset by the average tensile strength of the dry subset, is used as a indicator of stripping resistance.

Other tests for stripping resistance include ASTM D1075 Standard Test Method for Effect of Water on Compressive Strength of Compacted Bituminous Mixtures [ASTM, 2001], and AASHTO T182 Standard Specification for Coating and Stripping of Bitumen-Aggregate Mixtures [AASHTO 1997].

It is to be pointed out that none of the existing stripping resistance tests have been found to be completely reliable in predicting the performance of the asphalt-aggregate mixtures in actual service.

Aggregate Shape and Texture

The shape of an aggregate used in an asphalt mixture has a great effect on the tendency of the mix to deform. Rounded aggregates have no interlocking ability and can easily “slide by” each other when subjected to shear stresses. Increasing the amount of crushed coarse and fine aggregates in an asphalt mixture can significantly increase the resistance of the mix to plastic deformation. Thus, in order to increase the rutting resistance of the asphalt mixtures, many asphalt mixture specifications have required a large percentage of the coarse aggregate to have at least one or two crushed faces, and have limited the percentage of natural sand to be used.

Flat or elongated particles are typically defined as particles having a ratio of maximum to minimum dimension greater than five. Flat or elongated particles are undesirable. These particles can be easily broken by traffic compaction and can reduce the strength of the asphalt mixture. These particles also reduce the workability of the asphalt mixture and can impede the compaction of the mixture during construction. Flat or elongated particles can be determined using ASTM D4791 Standard Test Method for Flat or Elongated Particles in Coarse Aggregate [ASTM, 2001].

Surface texture of aggregate particles also has a great effect on the ability of the mix to resist plastic deformation. Some researchers consider this factor to be more important than particle shape. A rough aggregate surface texture can provide good skid resistant characteristics of the pavement surface. Asphalt can bond better to rough surfaces than to smooth ones. Aggregates that have smooth surfaces, such as gravels, have a higher tendency to rut than do crushed limestone aggregate, which have rougher surfaces.

The overall measure of particle shape and texture characteristics of an aggregate can be quantified by a *particle index value* by means of ASTM D3398 Standard Test for Index of Aggregate Particle Shape and Texture [ASTM, 2001]. In this test, the aggregate to be evaluated is sieved into different specified size fractions. The aggregate from each of the different size fractions is placed in 3 layers in a special cylindrical steel mold and each layer is compacted with 10 tamps by a special tamping rod. This procedure is repeated using a compaction of 50 tamps per layer. The percent of voids in the compacted aggregate in each condition is determined from the weight of the compacted aggregate and the bulk specific gravity of the aggregate. The particle index for the aggregate in each size fraction is calculated from the following equation:

$$I_a = 1.25 V_{10} - 0.25 V_{50} - 32.0 \quad (45.19)$$

where I_a = particle index value

V_{10} = percent voids in the aggregate compacted with 10 tamps per layer

V_{50} = percent voids in the aggregate compacted with 50 tamps per layer

The particle index of the aggregate is computed as the weighted average of the particle index values from the different size fractions based on the percentages of the fractions in the original aggregate. An aggregate containing round particles with smooth surface texture may have a low particle index of 5 to 8, while an aggregate containing highly angular particles with rough texture may have a particle index of 15 to 20.

A test that has been used to measure the angularity and texture characteristics of fine aggregates is ASTM C1252 or AASHTO TP56–99 Method for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading) [AASHTO, 1999]. In this test, the fine aggregate to be evaluated is dropped through the orifice of a funnel into a calibrated 100-cm³ cylinder. Excess material is struck off and the cylinder with aggregate is weighed. Uncompacted void content of the sample is computed using this weight and the bulk specific gravity of the aggregate. There are three different variations of this method. Method A uses a sample of specified gradation. Method B uses three different size fractions. Method C uses the actual gradation of the aggregate to be evaluated. Superpave mix design system uses Method A.

A higher uncompacted void content is generally associated with higher angularity and rougher texture of the fine aggregate. However, since the results of the uncompacted void content test are influenced by the gradation of the aggregate, comparisons between different aggregates can only be made when they are tested in the same grading.

Strength and Toughness

Since the aggregates provide most of the load carrying capacity of the asphalt mixtures, aggregates must be sufficiently strong and tough to resist the applied loads. Insufficiently strong and tough aggregates in the asphalt mixtures can be excessively broken and degraded by the applied loads during construction and by traffic during service.

The Los Angeles (L.A.) abrasion test is commonly used to control the desired strength and toughness of the aggregate. ASTM C131 Standard Test Method for Resistance to Degradation of Small-Size Aggregate by Abrasion and Impact in the Los Angeles Machine is used for coarse aggregate smaller than 37.5 mm (1½ in.). ASTM C535 Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine is used for coarse aggregate larger than 19 mm (¾ in.) and up to 76 mm (3 in.) maximum aggregate size.

The L.A. abrasion test reports the results in terms of percent L.A. abrasion loss. A higher percent L.A. abrasion loss generally indicates a less abrasion-resistant aggregate. Typical test results range from 10% for extremely hard rocks to more than 60% for soft aggregates. Specifications for aggregate for use in HMA mixtures typically limit the maximum allowable L.A. abrasion loss to a certain level, which may vary from 40% by some agencies to 60% by others.

It is to be pointed out that the L.A. abrasion loss is mainly a measure of the resistance to abrasion. Many aggregates have given satisfactory performance even though their L.A. abrasion loss is high.

Durability

In order to ensure a durable aggregate, specifications for coarse aggregate for use in asphalt mixtures typically include a soundness test using sodium or magnesium sulfate (ASTM C88). This test involves submerging the different size fractions of the aggregate in a solution of sodium or magnesium sulfate for 18 hours followed by oven drying. The process is repeated for a specified number of cycles. The loss in weight for each size fraction is determined, and the weighted average percent loss for the entire sample is computed and reported as percent soundness loss. A higher percent soundness loss indicates a less durable aggregate. ASTM D692 Standard Specification for Coarse Aggregate for Bituminous Paving Mixtures specifies a maximum of 12% loss after 5 cycles when using sodium sulfate and 18% loss when using magnesium sulfate.

The sodium and magnesium sulfate soundness test was originally developed to simulate the damaging effects of freezing and thawing on aggregates. However, this test is now used to screen aggregates regardless of whether or not the aggregate is to be used in a freezing and thawing environment.

Cleanliness

Clean aggregates that are free of deleterious materials are desirable for use in asphalt mixtures. Deleterious materials that are to be avoided include clay, dust, friable particles and organic impurities.

The sand equivalent test (ASTM D2419) is used to determine the proportions of clay and sands in a fine aggregate. In this test, a sample of the fine aggregate to be tested is placed in a specified transparent cylinder filled with water and a flocculating agent. The mixture is agitated, and allowed to settle for 20 min. The sand will separate from the flocculated clay, and the heights of clay and sand in the cylinder are measured. The sand equivalent is the ratio of the height of sand to the height of clay times 100. A higher sand equivalent value indicates a cleaner aggregate. Specifications for aggregates in asphalt mixtures typically specify a minimum sand equivalent of 25 to 35.

Clay and friable particles in aggregate can be determined in accordance with ASTM C142 Standard Test Method for Clay Lumps and Friable Particles in Aggregates. The amount of clay lumps and friable particles are typically limited to a maximum of 1%.

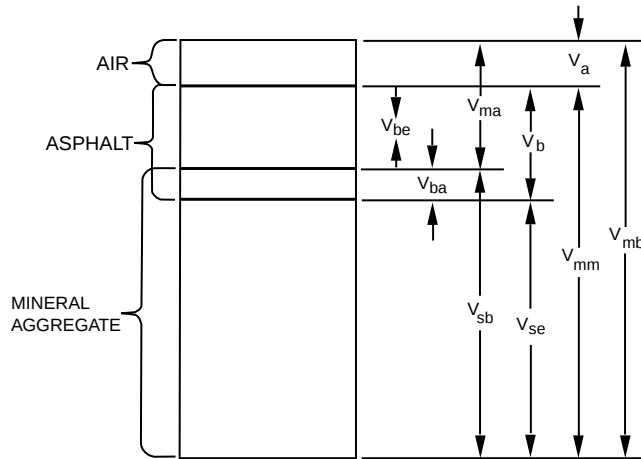
The amount of plastic fines in a fine aggregate can be indicated by the plasticity index (PI) (ASTM D4318). ASTM D1073 Standard Specification for Fine Aggregate for Bituminous Paving Mixtures limits the PI of the fraction passing the No. 40 (425 μm) to 4.0.

Volumetric Properties of Asphalt Mixtures

A compacted asphalt mixture consists primarily of aggregate, asphalt and air. The composition of an asphalt mixture can be characterized by the proportioning of the volumes of these three components. *Volumetric properties* of asphalt mixtures are properties that are directly related to the proportioning of the volumes of these three components. Volumetric properties have been widely used in the design and control of production of asphalt mixtures. The commonly used volumetric properties of asphalt mixtures are presented in this section.

Different Volumes in a Compacted Asphalt Mixture

Although there are only three components in a compacted asphalt mixture, numerous different volumes can be computed when different combinations of the three components are combined. This is further



- V_{ma} = Volume of voids in mineral aggregate
- V_{mb} = Bulk volume of compacted mix
- V_{mm} = Voidless volume of paving mix
- V_a = Volume of air voids
- V_b = Volume of asphalt
- V_{ba} = Volume of absorbed asphalt
- V_{be} = Volume of effective asphalt
- V_{sb} = Volume of mineral aggregate (by bulk specific gravity)
- V_{se} = Volume of mineral aggregate (by effective specific gravity)

FIGURE 45.8 Representation of volumes in a compacted asphalt mixture.

complicated by the fact that some asphalt can be absorbed into the aggregate and occupy part of the bulk volume of the aggregate. The representation of the different volumes in a compacted mixture is shown in Fig. 45.8.

These volumes and their corresponding notations will be used to define the volumetric properties in the subsequent subsections.

Percent Air Voids

The percent air voids (P_a) of a compacted mixture is the ratio of the volume of air voids to the total volume of the mixture. It can be expressed by the following equation:

$$P_a = \left(V_a / V_{mb} \right) \times 100\% \quad (45.20)$$

Percent Voids in Mineral Aggregate

Percent voids in mineral aggregate (VMA) is the ratio of the volume of voids in mineral aggregate to the total volume of the mixture. It can be expressed by the following equation:

$$VMA = V_{ma} / V_{mb} \times 100\% = \left(V_a + V_{be} \right) / V_{mb} \times 100\% \quad (45.21)$$

Percent Voids Filled with Asphalt

Voids filled with asphalt (VFA) is the ratio of the volume of effective asphalt to the volume of the voids in mineral aggregate. It can be expressed by the following equation:

$$VFA = V_{be} / V_{ma} \times 100\% = V_{be} / \left(V_{be} + V_a \right) \times 100\% \quad (45.22)$$

Computation of Volumetric Properties

The maximum specific gravity (G_{mm}) of the asphalt mixture is needed in order to calculate the percent air voids. The maximum specific gravity is the specific gravity when there are no air voids in the mixture. The maximum specific gravity of the mixture can be determined by running the ASTM D2041 Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixture on the loose mixture. This test is also known as the Rice test.

The percent air voids (P_a) can be computed from the maximum specific gravity (G_{mm}) and the bulk specific gravity of the mixture (G_{mb}) as follows:

$$P_a = (G_{mm} - G_{mb}) / G_{mm} \times 100 \quad (45.23)$$

The percent voids in mineral aggregate (VMA) can be computed as follows:

$$VMA = 100 - (G_{mb} P_s) / G_{sb} \quad (45.24)$$

where P = aggregate percent by total weight of the mixture
 G = bulk specific gravity of aggregate

The percent voids filled with asphalt (VFA) can be computed as follows:

$$VFA = (VMA - P_a) / VMA \times 100 \quad (45.25)$$

Design of HMA Mixtures

The design of an asphalt paving mixture usually involves selecting the aggregates, asphalt and additives to be used, testing the asphalt mixtures at various different proportions of the ingredients, and selecting the optimum mix design which would give the best anticipated performance in service. Ideally, the mixtures to be tested should be prepared and compacted to as close to the field condition as possible, so that they can be representative of the mixtures to be produced and put in service. The properties of the mixtures to be determined should be good indicators of performance of the mixtures in service, so that these properties can be used to determine the acceptability of the mixtures and to select the optimum mix design to be used.

A design procedure for asphalt mixtures generally involves (1) preparing and compacting the asphalt mixtures in the laboratory to simulate the field condition, (2) characterizing the laboratory compacted specimens, and (3) determining the optimum mix design based on the properties of the tested specimens and the set criteria for these properties. Different design methods generally differ from one another by (1) the equipment and method used to prepare and compact the asphalt mixtures, (2) the properties of the compacted specimens to be measured, and (3) the criteria used for selecting acceptable and optimum mix designs.

This section presents the general methodologies of four different mix design methods for dense-grade HMA mixtures, which include the Marshall, Hveem, Superpave and GTM methods. Emphasis is placed on the three main elements as described above, so that these different design methods could be compared with one another in a more meaningful manner.

Marshall Mix Design Method

The concept of the Marshall method of mix design was originally conceived by Mr. Bruce Marshall, formerly a bituminous engineer with the Mississippi State Highway Department. The Marshall method was later further improved by the U.S. Corps of Engineers who added certain features and developed the mix design criteria. The Marshall mix design method and criteria were originally developed for airfield pavements, but were later also adopted for use in highway pavements. Due to its simplicity, the Marshall method of mix design was the most commonly used mix design method in the U.S. before the introduction

of the Superpave design system, and it is still the most commonly-used mix design methods in the rest of the world.

The Marshall mix design procedure as recommended by the Asphalt Institute is described in detail in the Manual “Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types” by the Asphalt Institute [1997]. The Marshall mix design procedure consists of the following main elements:

1. Selection of aggregates — The aggregates must meet all the requirements as specified by the local highway agency. These requirements typically include limits on L.A. abrasion loss, soundness loss, sand equivalent, percent of deleterious substance, percent of natural sand, percent of particles with crushed faces, and percent of flat or elongated particles. (See Section 45.3 for a description of aggregate properties.) The gradation of the aggregate blend to be used must meet the gradation requirements for dense-grade HMA mixture as set by the local highway agency.
2. Selection of asphalt binder — The asphalt must meet the specification requirements as set by the local highway agency.
3. Preparation of asphalt mixture samples — Samples of asphalt mixtures at five different asphalt contents, with three replicates per asphalt content are prepared. The asphalt contents are selected at 0.5% increments with at least two asphalt contents above the estimated optimum and at least two below it. The aggregate and asphalt are mixed at a temperature at which the asphalt kinematic viscosity is 170 ± 20 centistokes.
4. Compaction of the asphalt mixtures — The asphalt mixture is compacted in a 101.6-mm (4-in.) diameter cylindrical mold by a Marshall compaction hammer, which is 6.5 kg (10 lb) in weight and dropped from a height of 457 mm (18 in.) for a specified number of blows per side of the specimen. The number of blows to be applied per side is 35, 50 or 75 for light, medium or heavy designed traffic, respectively. Light traffic is defined as having less than 10^4 ESALs. Medium traffic has between 10^4 and 10^6 ESALs, while heavy traffic has more than 10^6 ESALs. Compaction of the mixtures is done at a temperature at which the asphalt kinematic viscosity is 280 ± 20 centistokes. The compacted specimen is 101.6 mm (4 inches) in diameter and approximately 63.5 mm (2.5 in.) in height.
5. Testing of the compacted Marshall specimens — The tests to be run on the Marshall specimens include (1) determination of bulk specific gravity in accordance with AASHTO T166 [AASHTO, 1997] or ASTM D2726 [ASTM, 2001] and (2) Marshall stability test, which measures the Marshall stability and Marshall flow, in accordance with ASTM D1559 [ASTM, 2001].

The Marshall stability is the maximum load the specimen can withstand before failure when tested in the Marshall stability test. The configuration of the Marshall stability test is close to that of the indirect tensile strength test, except for the confinement of the Marshall specimen imposed by the Marshall testing head. Thus, the Marshall stability is related to the tensile strength of the asphalt mixture.

The Marshall flow is the total vertical deformation of the specimen, in units of 0.01 in., when it is loaded to the maximum load in the Marshall stability test. The Marshall flow can provide some indication of the resistance of an asphalt mixture to plastic deformation. Mixtures with low flow numbers are stiff and may be difficult to compact. However, these mixtures are more resistant to rutting than those with high flow numbers. Mixtures with flow numbers above the normal range may be “tender mixes,” which are susceptible to permanent deformation.

1. Computation of volumetric properties of the specimens — Using the bulk specific gravity of the specimen, the maximum specific gravity of the mixture and the bulk specific gravity of the aggregate, the percent air voids and VMA of the specimen are determined. Percent air voids of the specimen can be computed from the bulk specific gravity of the specimen and the maximum specific gravity of the mixture according to Eq. (45.23). VMA can be computed from the bulk specific gravity of the mixture, the bulk specific gravity of the aggregate and the aggregate percent by weight of the mix according to Eq. (45.24).
2. Marshall mix design criteria — The Marshall mix design method as recommended by the Asphalt Institute uses five mix design criteria. They are (1) a minimum Marshall stability, (2) a range of

TABLE 45.5 Marshall Mix Design Requirements on Stability, Flow, Air Voids and VFA

Traffic Category Compaction, No. of blows/side	Light 35		Medium 50		Heavy 75	
	Min.	Max.	Min.	Max.	Min.	Max.
Stability, lb (N)	750 (3333)	—	1200 (5333)	—	1800 (8000)	—
Flow, 001 in. (0.25 mm)	8	18	8	16	8	14
Air Voids, %	3	5	3	5	3	5
VFA, %	65	75	65	78	70	80

Source: Asphalt Institute 1997. *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types*, Manual Series No. 2, Sixth Edition, The Asphalt Institute, Lexington, KY.

TABLE 45.6 Marshall Mix Design Criteria on VMA

Nominal Maximum Aggregate Size	Minimum Required VMA, %		
	Design Air Voids, %		
	3.0	4.0	5.0
#8 (2.36 mm)	19.0	20.0	21.0
#4 (4.75 mm)	16.0	17.0	18.0
3/8 in. (9.5 mm)	14.0	15.0	16.0
1/2 in. (12.5 mm)	13.0	14.0	15.0
3/4 in. (19.0 mm)	12.0	13.0	14.0
1 in. (25.0 mm)	11.0	12.0	13.0
1.5 in. (37.5 mm)	10.0	11.0	12.0
2 in. (50 mm)	9.5	10.5	11.5
2.5 in. (63 mm)	9.0	10.0	11.0

Source: Asphalt Institute 1997. *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types*, Manual Series No. 2, Sixth Edition, The Asphalt Institute, Lexington, KY.

acceptable Marshall flow, (3) a range of acceptable air voids, (4) percent voids filled with asphalt (VFA), and (5) a minimum amount of VMA. Table 45.5 shows the requirements for stability, flow, air voids and VFA, while Table 45.6 shows the requirements for VMA. A mix design to be adopted must satisfy all these five criteria.

3. Determination of design asphalt content — To facilitate the selection of optimum asphalt content, the following six plots are made:
 - a. Average unit weight versus asphalt content
 - b. Average air voids versus asphalt content
 - c. Average Marshall stability versus asphalt content
 - d. Average Marshall flow versus asphalt content
 - e. Average VMA versus asphalt content
 - f. Average VFA versus asphalt content

From the plot of air voids versus asphalt content, determine the asphalt content at an air voids content of 4%. Using plots (3) through (6), determine the Marshall stability, Marshall flow, VMA and VFA at this asphalt content, and compare them with the Marshall mix design criteria as given in Tables 45.5 and 45.6. If all the mix criteria are met, this asphalt content is the preliminary design asphalt content. The preliminary design asphalt content can then be adjusted within the range where all the mix criteria are met according to the special need of the project to arrive at the final design asphalt content.

If one or more of the mix criteria cannot be met, adjustments in aggregate type, aggregate gradation and/or asphalt type will need to be made and the mix design procedure will need to be re-conducted.

Hveem Mix Design Method

The Hveem mix design method was developed in the 1930s by Francis N. Hveem and his co-workers at the California Division of Highways. The Hveem mix design method has been adopted by several western states in the U.S., and is still being used by the California Department of Transportation Department today. The Hveem mix design procedure as recommended by the Asphalt Institute is described in detail in the Manual “Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types” by the Asphalt Institute [1997]. The Hveem mix design procedure consists of the following main elements:

1. Selection of aggregate and asphalt — The selection of aggregate and asphalt to be used is similar to that in the Marshall mix design method as described in the subsection on Marshall mix design method.
2. Estimation of optimum asphalt content — The Centrifuge Kerosene Equivalent test is run on the fine aggregate to determine the CKE (percentage of kerosene retained by the fine aggregate). The surface capacity test is run on the coarse aggregate to determine the Percent Oil Retained. The CKE and Percent Oil Retained, along with the calculated surface area of the aggregate and apparent specific gravities of the fine and coarse aggregates, are then used to estimate the optimum asphalt content through a series of five charts.
3. Preparation of asphalt mixtures — Samples of asphalt mixtures at seven different asphalt contents are prepared. The asphalt contents are selected at 0.5% increments with four asphalt contents above the estimated optimum, one at the estimated optimum and two below it. Mixing temperature is a function of the grade of the asphalt. For an AC-10, the specified mixing temperature is 135 to 149°C, while for an AC-40, it is 149 to 163°C.
4. Compaction of specimens — After mixing, the mixture is placed in oven for a 15-hour curing period at 60°C. After curing is complete, the mixture is reheated to 110°C and compacted in the California kneading compactor. The detailed description of the compaction equipment and procedure are given in ASTM D1561. Basically, the specimen is compacted in a 101.6-mm (4-in.) diameter cylindrical mold by means of the kneading action of a compactor ram without impact, followed by a static load to smoothen out the surface of the compacted specimen. The compacted specimen is 101.6 mm (4 in.) in diameter and approximately 63.5 mm (2.5 in.) in height.
5. Testing of the compacted mixture — The compacted Hveem specimens are tested at 60°C in the Hveem stabilometer to determine the Hveem Stabilometer (S) value. Since the Hveem stabilometer test is a non-destructive test, the bulk specific gravity of the specimen is determined after the completion of the Hveem stabilometer test.

The Hveem stabilometer apparatus and test procedure are described in details in ASTM D1560. Hveem S-value is expressed as a number that may vary from 0 to 100. A higher Hveem S-value would indicate a mix of higher stability. In the development of the Hveem method, it was found that a Hveem S-value of 28 to 30 could be used to distinguish between pavements that would be susceptible to rutting and those that would not. The minimum required Hveem S-value was later increased to 37 (for heavy traffic condition) to account for the increased traffic loads. The Hveem S-value is generally considered to be a measure of the angle of internal friction in the Coulomb shear strength equation. Thus the Hveem S-value is related to the shear strength and thus to the rutting resistance of the asphalt mixture.

A swell test is also run on the compacted specimens to determine the mixture's resistance to water. Basically, a swell test measures the swelling of a compacted specimen after it is submerged in water for 24 hours. A detailed description of the swell test can be found in the Manual “Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types” by the Asphalt Institute [1997].

6. Computation of air voids of the specimens — The air voids of the specimens are computed from the bulk specific gravity of the specimen and the maximum specific gravity of the mixture by according to Equation 45.23.
7. Hveem mix design criteria — The three mix design criteria used by the Hveem method are (1) a minimum required Hveem S-value, (2) a minimum air voids of 4 percent, and (3) a maximum

allowable swell as measured by the swell test. The minimum required Hveem S-value as recommended by the Asphalt Institute is 30, 35 and 37 for light, medium and heavy designed traffic.

8. Determination of optimum asphalt content — The optimum asphalt content is the highest possible asphalt content such that the three Hveem mix design criteria are met.

Superpave Volumetric Mix Design Method

The Superpave mix design method is a new mix design method, which was introduced as a result of the Strategic Highway Research Program (SHRP) conducted from 1988 through 1993. When the Superpave mix design method was first developed by the SHRP researchers, it was intended to have three levels of sophistication and design effort. Level 1 design would involve only materials selection and volumetric proportioning, and was intended for use on low-traffic roads with less than 1 million ESALs. Level 2 design would include Level 1 design effort plus conductance of performance prediction tests, and was intended for use on pavements with between 1 and 10 million ESALs. Level 3 design would include Level 1 design effort plus conductance of enhanced performance prediction tests, and was intended for use on high-traffic pavements with over 10 million ESALs. However, at present, only Level 1 design has been implemented, and Level 1 design has been used for all levels of traffic. Superpave Level 1 mix design is now also referred to as Superpave volumetric mix design.

The detailed specification for Superpave Volumetric Design can be found in AASHTO MP2–99 [AASHTO, 1999]. The description of the Superpave volumetric mix design procedure can be found in AASHTO PP28–99 [AASHTO, 1999]. However, it is to be pointed out that since the Superpave design method is not well established yet and changes are still being made to it periodically, the most updated versions of these documents should be consulted when performing a Superpave volumetric mix design.

The Superpave volumetric mix design procedure consists of the following main elements:

1. Selection of asphalt — The asphalt binder should be a PG grade asphalt meeting the requirements of AASHTO MP1, which is appropriate for the climate and traffic condition at the project site.
2. Selection of aggregate — The combined aggregate must meet the following requirements:
 - a. Nominal maximum size — Nominal maximum aggregate size should be 9.5 to 19.0 mm for surface course HMA and 19.0 to 37.5 mm for base course HMA.
 - b. Gradation control points — The gradation must pass through the control points as specified in [Table 45.7](#).
 - c. Gradation restricted zone — It is recommended that the gradation does not pass through the restricted zones as specified in [Table 45.8](#). However, results of recent research studies have indicated that mixtures which had aggregate gradations that violated the restricted zone could perform similarly or better than those that had not violated it [NCAT, 2001].

TABLE 45.7 Superpave Mix Design Criteria on Aggregate Gradation Control Points

Sieve Size	Nominal Maximum Aggregate Size									
	37.5 mm		25.0 mm		19.0 mm		12.5 mm		9.5 mm	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
50 mm	100	—	—	—	—	—	—	—	—	—
37.5 mm	90	100	100	—	—	—	—	—	—	—
25.0 mm	—	90	90	100	100	—	—	—	—	—
19.0 mm	—	—	—	90	90	100	100	—	—	—
12.5 mm	—	—	—	—	—	90	90	100	100	—
9.5 mm	—	—	—	—	—	—	—	90	90	100
4.75 mm	—	—	—	—	—	—	—	—	—	90
2.36 mm	15	41	19	45	23	49	28	58	32	67
0.075 mm	0	6	1	7	2	8	2	10	2	10

Source: AASHTO 1999. AASHTO MP2–99 Standard Specification for Superpave Volumetric Mix Design, *AASHTO Provisional Standards*, Interim Edition, AASHTO, Washington, D.C.

TABLE 45.8 Boundaries of Aggregate Restricted Zone as recommended in Superpave Mix Design Method

Sieve Size	Nominal Maximum Aggregate Size									
	37.5 mm		25.0 mm		19.0 mm		12.5 mm		9.5 mm	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
0.30 mm	10.0	10.0	11.4	11.4	13.7	13.7	15.5	15.5	18.7	18.7
0.60 mm	11.7	15.7	13.6	17.6	16.7	20.7	19.1	23.1	23.5	27.5
1.18 mm	15.5	21.5	18.1	24.1	22.3	28.3	25.6	31.6	31.6	37.6
2.36 mm	23.3	27.3	26.8	30.8	34.6	34.6	39.1	39.1	47.2	47.2
4.75 mm	34.7	34.7	39.5	39.5	—	—	—	—	—	—

Source: AASHTO 1999. AASHTO MP2–99 Standard Specification for Superpave Volumetric Mix Design, *AASHTO Provisional Standards*, Interim Edition, AASHTO, Washington, D.C.

TABLE 45.9 Superpave Mix Design Criteria on Consensus Aggregate Properties

Design Traffic (million ESALs)	Coarse Aggregate Angularity, Minimum (% with one fractured face/% with two fractured faces)		Uncompacted Void Content of Fine Aggregate, Minimum (%)		Sand Equivalent, Minimum (%)	Flat and Elongated, Maximum (%)
	Thickness ≤100 mm	Thickness >100 mm	Thickness ≤100 mm	Thickness >100 mm		
	≤100 mm	>100 mm	≤100 mm	>100 mm	(%)	(%)
<0.3	55/—	—/—	—	—	40	—
0.3 to <3	75/—	50/—	40	40	40	10
3 to <10	85/80	60/—	45	40	45	
10 to <30	95/90	80/75	45	40	45	
≥30	100/100	100/100	45	45	50	

Source: AASHTO 1999. AASHTO MP2–99 Standard Specification for Superpave Volumetric Mix Design, *AASHTO Provisional Standards*, Interim Edition, AASHTO, Washington, D.C.

- d. Consensus aggregate property requirements — There are four consensus aggregate property requirements. The coarse aggregate must meet the angularity requirements in terms of the minimum percentages of particles with crushed faces as measured by ASTM D5821. The fine aggregate must meet the fine aggregate angularity requirements in terms of the minimum uncompacted void contents as measured by AASHTO T304 Method A. The aggregate must meet the sand equivalent requirement in terms the minimum sand contents as measured by AASHTO T176. The aggregate must meet the requirement on the maximum allowable percentage of flat and elongated particles as measured by ASTM D4791. The Superpave mix design criteria for these four consensus properties are shown in [Table 45.9](#).
- e. Aggregate source property requirements — The aggregate must meet all the source property requirements, such as L.A. abrasion loss, soundness and deleterious materials as specified by the local highway agency.
3. Preparation of asphalt mixtures — Aggregate and asphalt are mixed at the temperature at which the kinematic viscosity of the asphalt is 170 ± 20 cSt. The loose asphalt mixture is then cured in a forced-draft oven at 135°C for 4 hours before compaction. The detailed description of the curing procedure can be found in AASHTO PP2–99 [AASHTO, 1999].
4. Compaction of asphalt mixtures — Compaction of the asphalt mixture is done in the Superpave gyratory compactor, as described in AASHTO TP4–99. The Superpave gyratory compactor differs from the Corps of Engineers GTM in that the angle of gyration in the Superpave gyratory compactor is fixed, while the gyratory angle in the GTM can vary according to the stability of the tested mixture. The Superpave gyratory compactor configurations are as follows:

TABLE 45.10 Numbers of Gyration
for Superpave Gyratory Compaction

Design Traffic (million ESALs)	N_{initial}	N_{design}	N_{max}
<0.3	6	50	75
0.3 to <3	7	75	115
3 to 30	8	100	160
≥ 30	9	125	205

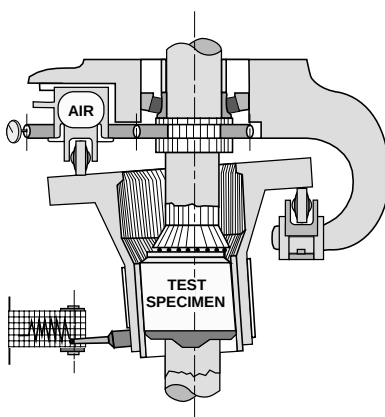
Source: AASHTO 1999, AASHTO PP28–99
Standard Practice for Superpave Volumetric
Mix Design for Hot Mix Asphalt, *AASHTO
Provisional Standards*, Interim Edition,
AASHTO, Washington, D.C.

- a. Vertical ram pressure: 600 kPa (87 psi)
 - b. Gyratory angle: 1.25° fixed angle
 - c. Speed: 30 gyrations per minute
 - d. Specimen diameter: 150 mm (5.9 in.)
 - e. Specimen height: 115 mm (4.5 in.)
4. Compaction is done at a temperature at which the kinematic viscosity of the asphalt is 280 ± 30 cSt. The number of gyrations to be applied is a function of the designed traffic level, as given in [Table 45.10](#). For each level of designed traffic, there are three levels of compaction, namely N_{ini} , N_{des} and N_{max} gyrations. The specimen is compacted to N_{des} gyrations, while the specimen height is recorded continuously. After compaction, the specimen is removed from the mold and its bulk specific gravity and $\%G_{\text{mm}}$ is determined. $\%G_{\text{mm}}$ is equal to 100% minus % air voids. The actual measured bulk density is compared with the calculated density based on the specimen height, and a correction factor is calculated. This correction factor and the specimen height at N_{ini} are then used to calculate the density and $\%G_{\text{mm}}$ of the specimen at N_{ini} . After the determination of the design asphalt content, duplicate samples at the design asphalt content are also compacted to N_{max} gyrations to determine the $\%G_{\text{mm}}$ of the mixture at N_{max} gyrations.
 5. Determination of design asphalt content — The design asphalt content is the asphalt content at which the asphalt mixture has an air voids content of 4% (or a $\%G_{\text{mm}}$ of 96%) when compacted to N_{des} gyrations, while all the mix design requirements are met. These mix design requirements are presented in the next section..
 6. Superpave mix design requirements — The asphalt mixture design must meet all the following requirements:
 - a. The asphalt and the aggregate must meet all the requirements as presented in the preceding sections.
 - b. The asphalt mixture must have a target air voids of 4% when compacted to N_{des} gyrations.
 - c. The VMA of the compacted mixture at N_{des} gyrations must meet the minimum VMA requirements as shown in [Table 45.11](#).
 - d. The VFA (Voids Filled with Asphalt) of the compacted mixture at N_{des} gyrations must fall within the range as shown in [Table 45.11](#).
 - e. The dust-to-binder ratio, which is the ratio of the weight of the mineral filler to the weight of the binder, must be between 0.6 and 1.2.
 - f. The $\%G_{\text{mm}}$ of the asphalt mixture compacted to N_{ini} must not exceed the limits as shown in [Table 46.11](#). The $\%G_{\text{mm}}$ of the mixture compacted to N_{max} must not exceed 98%.
 - g. The asphalt mixture, when compacted by the Superpave gyratory compactor to 7% air voids and tested in the AASHTO Designation T 283 Standard Test for Resistance of Compacted Bituminous Mixture to Moisture Induced Damage, must have a retained tensile-strength ratio of at least 80%.

TABLE 45.11 Superpave Mix Design Criteria on %G_{mm}, VMA, VFA and Dust-to-Binder Ratio

Design Traffic (million ESALs)	VFA (%)	Required %G _{mm}			Required minimum VMA (%)					Dust-to-Binder Ratio
					Nominal Max.Agg. Size, mm					
		N _{initial}	N _{design}	N _{max}	37.5	25.0	19.0	12.5	9.5	
<0.3	70–80	≤91.5	96.0	≤98.0	11.0	12.0	13.0	14.0	15.0	0.6 – 1.2
0.3 to <3	65–78	≤90.5								
3 to <10	65–75	≤89.0								
10 to <30										
≥30										

Source: AASHTO 1999. AASHTO MP2–99 Standard Specification for Superpave Volumetric Mix Design, AASHTO Provisional Standards, Interim Edition, AASHTO, Washington, D.C.

**FIGURE 45.9** Schematic of the Gyratory Testing Machine (GTM).

GTM Mix Design Method

The Gyratory Testing Machine (GTM) was developed by John McRae while working for the U.S. Corps of Engineers Waterway Experimental Station in Mississippi. The GTM is both a compaction device and a testing machine for asphalt mixture. A description of the GTM and testing procedure can be found in ASTM D 3387. The schematic of the GTM is shown in Fig. 45.9. The compaction variables in the GTM include the following:

1. Ram pressure — The vertical ram pressure simulates the tire contact pressure on the pavement as the pavement is compacted by roller during construction and by traffic during service. A ram pressure of 120 psi is typically used to simulate the highest anticipated tire contact pressure on highway pavements.
2. Gyratory angle — The gyratory angle used is empirically related to the applied strain on the pavement. A higher gyratory angle will produce relatively higher compactive effort. Typically an initial gyratory angle of 1 to 3 degrees is used.
3. Type of roller — Three different types of rollers can be used. They are fixed, oil and air rollers. Fixed rollers are the easiest to use. However, the use of the oil rollers or air rollers enables the loads applied by the rollers to be measured, and the gyratory shear strength of the specimen to be determined.
4. Number of gyrations — Compactive effort increases with higher number of gyrations. Typically 60 to 300 gyrations are used to produce ultimate compaction condition.

The following properties are measured during GTM compaction and testing:

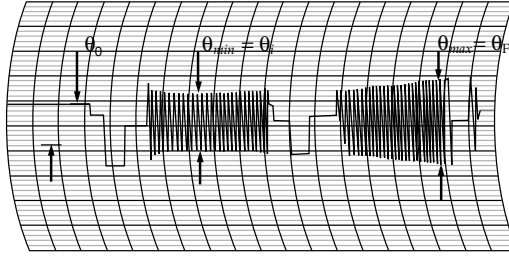


FIGURE 45.10 Gyrograph from a GTM test.

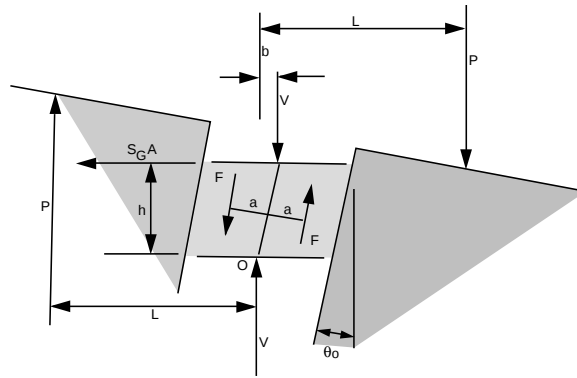


FIGURE 45.11 Forces acting on a GTM test sample.

1. Gyrograph — gyrograph is a recording of the shear strain experienced by the sample during compaction. An example of gyrograph is shown in Fig. 45.10. From the gyrograph, the following property can be calculated:

$$\text{Gyratory Stability Index (GSI)} = \theta_F / \theta_I \quad (45.26)$$

where θ_F = maximum gyratory angle
 θ_I = intermediate gyratory angle

A stable mix will have a GSI of 1.0. A GSI of greater than 1 indicates instability of the mix.

2. Gyratory shear strength — when the oil or air roller is used, the shear strength of the sample during compaction can be measured. Figure 45.11 shows all the forces acting on the sample during the GTM compaction. By balancing the moments acting around point O as shown in the figure, the following equation can be written:

$$2 P L = S_G A h + 2 F a - V b \quad (45.27)$$

The gyratory shear strength, S_G can be determined as:

$$S_G = [2(P L - F a) + V b] / A h \quad (45.28)$$

Neglecting wall friction (F) and moment due to eccentricity ($V b$), the gyratory shear strength can be determined as:

$$S_G = 2 P L / A h \quad (45.29)$$

3. Specimen height — specimen heights can be measured during compaction, and can be used to determine the density of the sample at different stages of compaction.

The GTM mix design method uses an approach which is completely different from that of the Superpave volumetric design method. The GTM design method does not use volumetric properties as mix design criteria. Instead, it uses properties that are directly related to shear strength and rutting resistance as indicators of performance. ASTM D3387–83 describes a mix design procedure using the GTM. This standard is currently being updated. The latest GTM mix design method, which will be described in the updated ASTM D3387 standard, uses the following GTM test configurations:

1. Ram pressure: maximum anticipated tire contact pressure.
2. Roller type: Air or oil
3. Initial gyratory angle: 2°
4. Initial roller pressure: A roller pressure that would give a computed gyratory shear strength equal to the maximum anticipated shear stress in the pavement.
5. Compaction temperature: Anticipated plant temperature

The mix design procedure consists of the following main steps:

1. Prepare at least two replicate specimens for each combination of aggregate and binder to be tested. Use at least three asphalt contents - one at estimated optimum, one at 0.5% below, and one at 0.5% above.
2. Mix the aggregate and binder at the anticipated plant temperature.
3. Compact the mixture in the GTM until equilibrium condition is reached. Equilibrium condition is considered to be reached when density changes by less than 0.5 lb/ft^3 (0.008 g/cm^3) per 50 gyrations.
4. Determine GSI (from gyrograph) and S_G (from roller pressure) at equilibrium condition.
5. Optimum asphalt content is the maximum asphalt content such that GSI is equal to 1, and S_G is equal to or greater than the maximum anticipated pavement shear stress. A conservative estimate of the maximum pavement shear stress can be taken to be equal to $(\text{max. tire contact pressure})/\pi$.

Defining Terms

Bingham-plastic material — A material which behaves as a solid (which would not flow) when the shear stress is below its yield strength, but behaves as a fluid (which would flow under stress) when the shear stress is above this yield point.

Dilatant fluid — A non-Newtonian fluid whose viscosity increases as the shear rate or shear stress increases.

Newtonian fluid — A fluid whose viscosity remains constant with changes in shear rate or shear stress. The relationship between shear stress and shear rate is linear for this type of fluid.

Pseudoplastic fluid — A non-Newtonian fluid whose viscosity decreases as the shear rate or shear stress increases.

Rutting — Permanent vertical depression of pavement surface along the wheel paths.

Shear susceptibility — The slope of the plot of $\log(\text{shear stress})$ versus $\log(\text{shear rate})$. It is usually denoted as “C.” For a Newtonian fluid, C is equal to 1.

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Manuals and Handbook

A whole host of manuals dealing with all aspects of asphalt pavement design, construction and testing are published and updated regularly by the Asphalt Institute (Research Park Drive, P.O. Box 14052, Lexington, KY 40512) and the National Asphalt Pavement Association (6811 Kenilworth Avenue, Riverdale, MD 20737).

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Proceedings of Canadian Technical Asphalt Association, published annually by CTAA, Victoria, British Columbia, Canada.

Proceedings of International Conference on Asphalt Pavements, published every five years by the International Society for Asphalt Pavement, c/o Texas Research and Development Foundation, Austin, TX.

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