9

ASPHALT BINDERS AND ASPHALT MIXTURES

Asphalt is one of the oldest materials used in construction. Asphalt binders were used in 3000 B.C., preceding the use of the wheel by 1000 years. Before the mid-1850s, asphalt came from natural pools found in various locations throughout the world, such as the Trinidad Lake asphalt, which is still mined. However, with the discovery and refining of petroleum in Pennsylvania, use of asphalt cement became widespread. By 1907, more asphalt cement came from refineries than came from natural deposits. Today, practically all asphalt cement is from refined petroleum.

Bituminous materials are classified as asphalts and tars, as shown in Figure 9.1. Several asphalt products are used; asphalt is used mostly in pavement construction, but is also used as sealing and waterproofing agents. Tars are produced by the destructive distillation of bituminous coal or by cracking petroleum vapors. In the United States, tar is used primarily for waterproofing membranes, such as roofs. Tar may also be used for pavement treatments, particularly where fuel spills may dissolve asphalt cement, such as on parking lots and airport aprons.

The fractional distillation process of crude petroleum is illustrated in Figure 9.2. Different products are separated at different temperatures. Figure 9.2 shows the main products, such as gasoline, kerosene, diesel oil, and asphalt residue (asphalt cement). Since asphalt is a lower-valued product than other components of crude oil, refineries are set up to produce the more valuable fuels at the expense of asphalt production. The quantity and quality of the asphalt depends on the crude petroleum source and the refining method. Some crude sources, such as the Nigerian oils, produce little asphalt, while others, such as many of the Middle Eastern oils, have a high asphalt content.

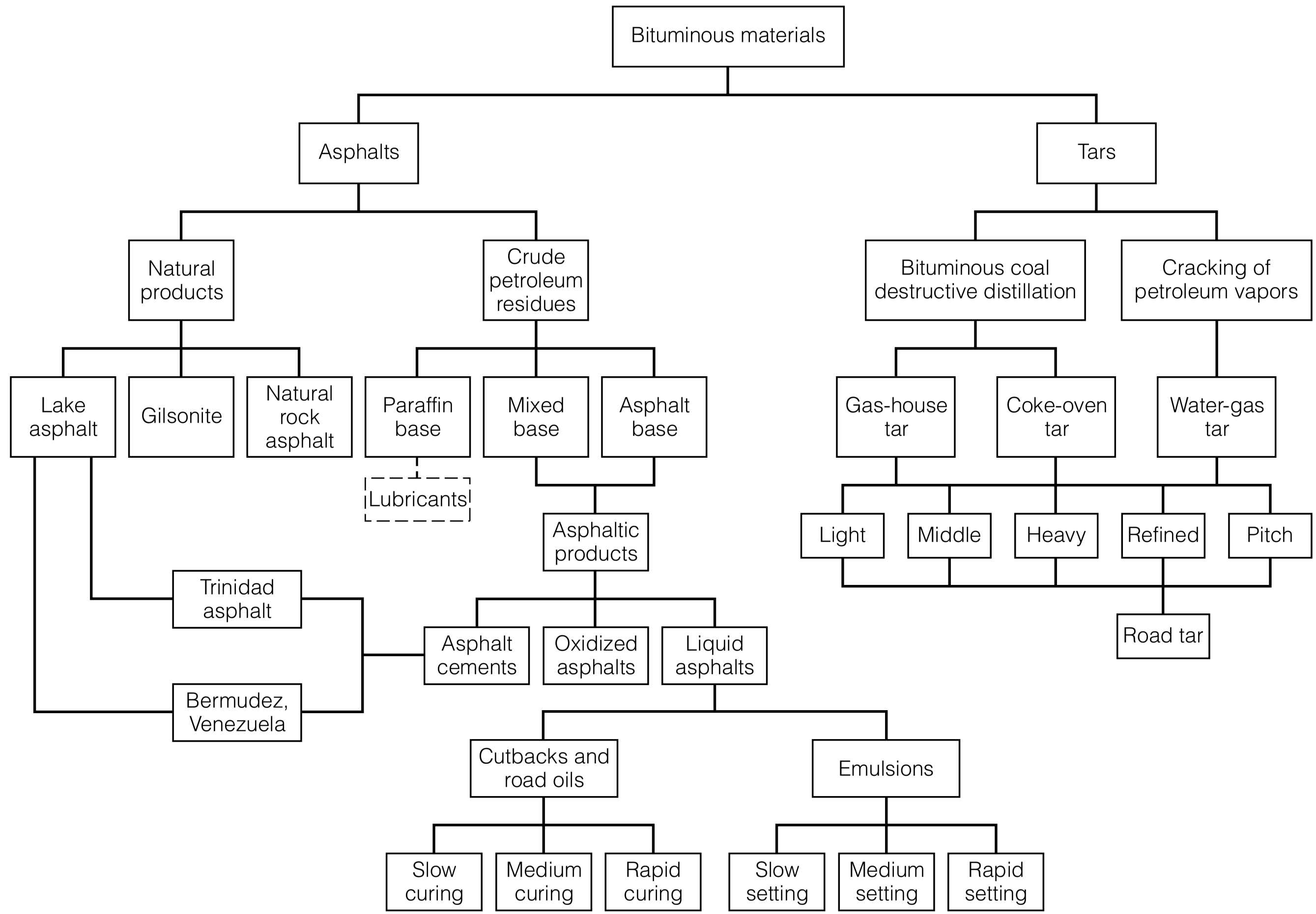

FIGURE 9.1 Classification of bituminous materials. (Goetz and Wood 1960).

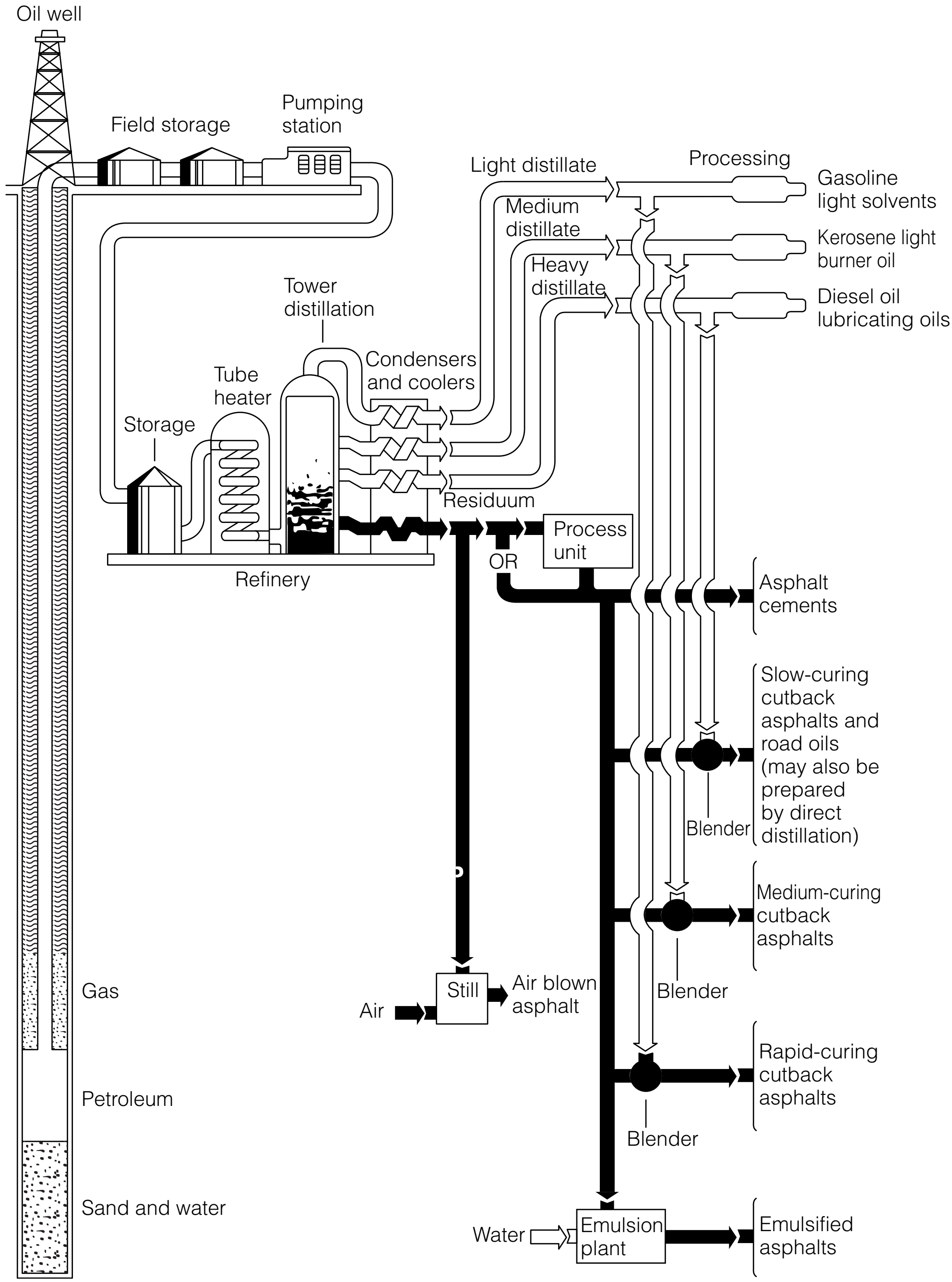

FIGURE 9.2 Distillation of crude petroleum. (The Asphalt Institute 1989).

This chapter reviews the types, uses, and chemical and physical properties of asphalt. The asphalt concrete used in road and airport pavements, which is a mixture of asphalt and aggregates, is also presented. The chapter discusses the recently developed Performance Grade asphalt binder specifications and Superpave mix design. Recycling of pavement materials and additives used to modify the asphalt properties are also included.

9.1 Types of Asphalt Products

Asphalt used in pavements is produced in three forms: *asphalt cement, asphalt cutback,* and *asphalt emulsion.* Asphalt cement is a blend of hydrocarbons of different molecular weights. The characteristics of the asphalt depend on the chemical composition and the distribution of the molecular weight hydrocarbons. As the distribution shifts toward heavier molecular weights, the asphalt becomes harder and more viscous. At room temperatures, asphalt cement is a semisolid material that cannot be applied readily as a binder without being heated. Liquid asphalt products, cutbacks and emulsions, have been developed and can be used without heating (The Asphalt Institute 1989).

Although the liquid asphalts are convenient, they cannot produce a quality of asphalt concrete comparable to what can be produced by heating neat asphalt cement and mixing it with carefully selected aggregates. Asphalt cement has excellent adhesive characteristics, which make it a superior binder for pavement applications. In fact, it is the most common binder material used in pavements.

A cutback is produced by dissolving asphalt cement in a lighter molecular weight hydrocarbon solvent. When the cutback is sprayed on a pavement or mixed with aggregates, the solvent evaporates, leaving the asphalt residue as the binder. In the past, cutbacks were widely used for highway construction. They were effective and could be applied easily in the field. However, three disadvantages have severely limited the use of cutbacks. First, as petroleum costs have escalated, the use of these expensive solvents as a carrying agent for the asphalt cement is no longer cost effective. Second, cutbacks are hazardous materials due to the volatility of the solvents. Finally, application of the cutback releases environmentally unacceptable hydrocarbons into the atmosphere. In fact, many regions with air pollution problems have outlawed the use of any cutback material.

An alternative to dissolving the asphalt in a solvent is dispersing the asphalt in water as emulsion. In this process the asphalt cement is physically broken down into micron-sized globules that are mixed into water containing an emulsifying agent. Emulsified asphalts typically consist of about 60% to 70% asphalt residue, 30% to 40% water, and a fraction of a percent of emulsifying agent. There are many types of emulsifying agents; basically they are a soap material. The emulsifying molecule has two distinct components, the head portion, which has an electrostatic charge, and the tail portion, which has a high affinity for asphalt. The charge can be either positive to produce a *cationic* emulsion or negative to produce an *anionic* emulsion. When asphalt is introduced into the water with the emulsifying agent, the tail portion of the emulsifier attaches itself to the asphalt, leaving the head exposed. The electric charge of the emulsifier causes a repulsive force between the asphalt globules, which maintains their separation in the water. Since the specific gravity of asphalt is very near that of water, the globules have a neutral buoyancy and, therefore, do not tend to float or sink. When the emulsion is mixed with aggregates or used on a pavement, the water evaporates, allowing the asphalt globs to come together, forming the binder. The phenomenon of separation between the asphalt residue and water is referred to as *breaking* or *setting*. The rate of emulsion setting can be controlled by varying the type and amount of the emulsifying agent.

Since most aggregates bear either positive surface charges (such as limestone) or negative surface charges (such as siliceous aggregates), they tend to be compatible with anionic or cationic emulsions, respectively. However, some emulsion manufacturers can produce emulsions that bond well to aggregate-specific types, regardless of the surface charges.

Although emulsions and cutbacks can be used for the same applications, the use of emulsions is increasing because they do not include hazardous and costly solvents.

9.2 Uses of Asphalt

The main use of asphalt is in pavement construction and maintenance. In addition, asphalt is used in sealing and waterproofing various structural components, such as roofs and underground foundations.

The selection of the type and grade of asphalt depends on the type of construction and the climate of the area. Asphalt cements, also called asphalt binders, are used typically to make hot-mix asphalt concrete for the surface layer of asphalt pavements (See Figures 9.3 and 9.4). Asphalt concrete is also used in patching and repairing both asphalt and portland cement concrete pavements. Liquid asphalts (emulsions and cutbacks) are used for pavement maintenance applications, such as fog seals, chip seals, slurry seals, and microsurfacing (See Figures 9.5 and 9.6) (The Asphalt Institute 1989, Mamlouk and Zaniewski 1998). Liquid asphalts may also be used to seal the cracks in pavements. Liquid asphalts are mixed with aggregates to produce cold mixes, as well. Cold mixtures are normally used for patching (when hot-mix asphalt concrete is not available), base and subbase stabilization, and surfacing of low-volume roads. Table 9.1 shows common paving applications for asphalts.

FIGURE 9.3 Placing hot mix asphalt (asphalt concrete) used for the surface layer of asphalt pavement.

FIGURE 9.4 Compaction of hot mix asphalt.

FIGURE 9.5 Applying fog seal for preserving existing pavement.

FIGURE 9.6 Applying microsurfacing for preserving existing pavement.

TABLE 9.1 Paving Applications of Asphalt

Term	Description	Application
Hot mix asphalt	Carefully designed mixture of asphalt and aggregates	Pavement surface, patching
Cold mix	Mixture of aggregates and liquid asphalt	Patching, low volume road surface, asphalt stabilized base
Fog seal	Spray of diluted asphalt emulsion on existing pavement surface	Seal existing pavement surface
Prime coat	Spray coat to bond aggregate base and asphalt concrete surface	Construction of flexible pavement
Tack coat	Spray coat between lifts of asphalt concrete	Construction of new pavements or between an existing pavement and an overlay
Chip seal	Spray coat of asphalt emulsion (or asphalt cement or cutback) followed with aggregate layer	Maintenance of existing pavement or low volume road surfaces
Slurry seal	Mixture of emulsion, well-graded fine aggregate and water	Resurface low volume roads
Microsurfacing	Mixture of polymer modified emulsion, well-graded crushed fine aggregate, mineral filler, water, and additives	Texturing, sealing, crack filling, rut filling, and minor leveling

9.3 Temperature Susceptibility of Asphalt

The consistency of asphalt is greatly affected by temperature. Asphalt gets hard and brittle at low temperatures and soft at high temperatures. Figure 9.7 shows a conceptual relation between temperature and logarithm of viscosity. The viscosity of the asphalt decreases when the temperature increases. Asphalt's temperature susceptibility can be represented by the slope of the line shown in Figure 9.7. The steeper the slope the higher the temperature susceptibility of the asphalt. However, additives can be used to reduce this susceptibility.

When asphalt is mixed with aggregates, the mixture will perform properly only if the asphalt viscosity is within an optimum range. If the viscosity of asphalt is higher than the optimum range, the mixture will be too

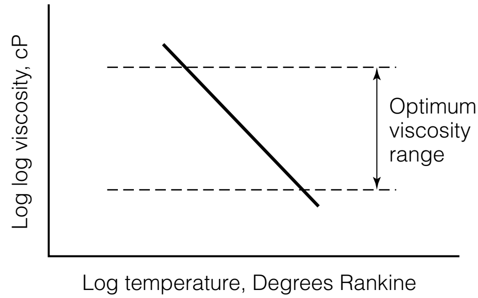

FIGURE 9.7 Typical relation between asphalt viscosity and temperature.

FIGURE 9.8 Thermal cracking resulting from the use of too stiff asphalt in a cold climate area.

brittle and susceptible to low-temperature cracking (Figure 9.8). On the other hand, if the viscosity is below the optimum range, the mixture will flow readily, resulting in permanent deformation (rutting) as shown in Figure 9.9.

Due to temperature susceptibility, the grade of the asphalt cement should be selected according to the climate of the area. The viscosity of the asphalt should be mostly within the optimum range for the area's annual temperature range; soft-grade asphalts are used for cold climates and hard-grade asphalts for hot climates (See Figure 9.10).

FIGURE 9.9 Rutting that could result from the use of too soft asphalt.

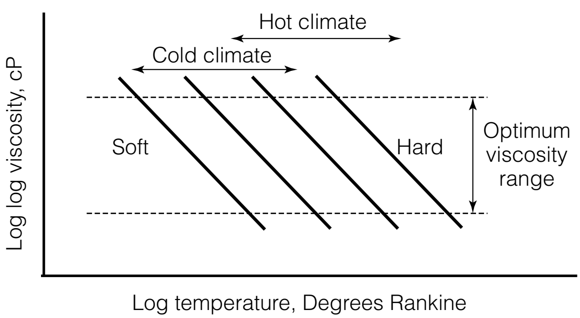

FIGURE 9.10 Selecting the proper grade of asphalt binder to match the climate.

9.4 Chemical Properties of Asphalt

Asphalt is a mixture of a wide variety of hydrocarbons primarily consisting of hydrogen and carbon atoms, with minor components such as sulfur, nitrogen, and oxygen (heteroatoms), and trace metals. The percentages of the chemical components, as well as the molecular structure of asphalt, vary depending on the crude oil source (Peterson 1984).

The molecular structure of asphalt affects the physical and aging properties of asphalt, as well as how the asphalt molecules interact with each other and with aggregate. Asphalt molecules have three arrangements, depending on the carbon atom links: (1) *aliphatic* or parraffinic, which form straight or branched chains, (2) *saturated rings,* which have the highest hydrogen to carbon ratio, and (3) *unsaturated rings* or aromatic. Heteroatoms attached to carbon alter the molecular configuration. Since the number of molecular structures of asphalt is extremely large, research on asphalt chemistry has focused on separating asphalt into major fractions that are less complex or more homogeneous. Each of these fractions is a complex chemical structure.

Asphalt cement consists of asphaltenes and maltenes (petrolenes). The maltenes consist of resins and oils. The asphaltenes are dark brown friable solids that are chemically complex, with the highest polarity among the components. The asphaltenes are responsible for the viscosity and the adhesive property of the asphalt. If the asphaltene content is less than 10%, the asphalt concrete will be difficult to compact to the proper construction density. Resins are dark and semisolid or solid, with a viscosity that is largely affected by temperature. The resins act as agents to disperse asphaltenes in the oils; the oils are clear or white liquids. When the resins are oxidized, they yield asphaltene-type molecules. Various components of asphalt interact with each other to form a balanced or compatible system. This balance of components makes the asphalt suitable as a binder.

Three fractionation schemes are used to separate asphalt components, as illustrated in Figure 9.11. The first scheme [Figure 9.11(a)] is partitioning with partial solvents in which *n*-butranol is added to separate (precipitate) the asphaltics. The butranol is then evaporated, and the remaining component is dissolved in acetone and chilled to $-23°C$ to precipitate the paraffinics and leave the cyclics in solution. The second scheme [Figure 9.11(b)] is selective adsorption–desorption, in which *n*-heptane is added to separate asphaltene. The remaining maltine fraction is introduced to a chromatographic column and desorbed using solvents with increasing polarity to separate other fractions. The third scheme [Figure 9.11(c)] is chemical precipitation in which *n*-pentane is added to separate the asphaltenes. A sulfuric acid (H_2SO_4) is added in increasing strengths to precipitate other fractions.

In addition, asphalt can be separated based on the molecular size with the use of high-pressure liquid chromatography (gel-permeation chromatography).

9.5 Superpave and Performance Grade Binders

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials and designing asphalt mixes.

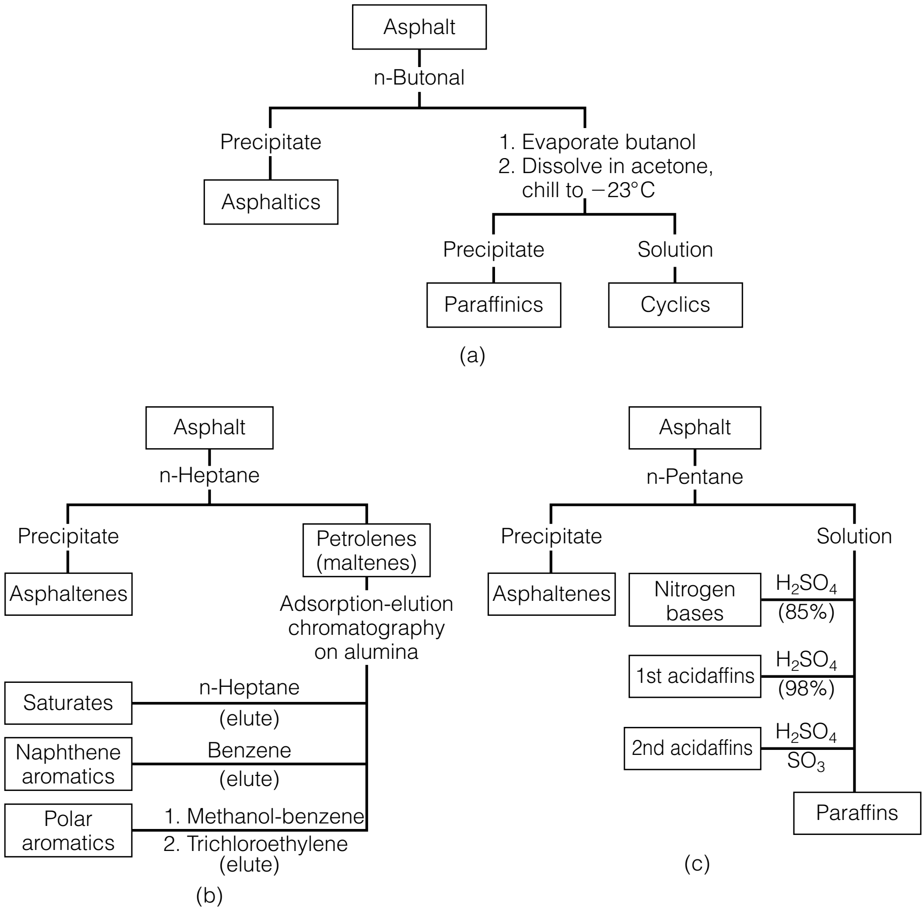

FIGURE 9.11 Schematic diagrams of three asphalt fractionation schemes: (a) partitioning with partial solvents, (b) selective-adsorbtion-descripition, and (c) chemical precipitation (Peterson 1984).

The SHRP research program produced the Superpave (*Su*perior *Per*forming Asphalt *Pave*ments) mix design method for asphalt concrete and the Performance Grading method for asphalt binder specification (McGennis 1994; 1995). The objectives of SHRP's asphalt research were to extend the life or reduce the life-cycle costs of asphalt pavements, to reduce maintenance costs, and to minimize premature failures. An important result of this research effort was the development of performance-based specifications for asphalt binders and mixtures to control three distress modes: rutting, fatigue cracking, and thermal cracking. Note that the Performance Grade specifications use the term *asphalt binder,* which refers to asphalt cement with or without the addition of modifiers.

9.6 Characterization of Asphalt

Many tests are available to characterize asphalt cement. Some tests are commonly used by highway agencies, while others are used for research. Since the properties of the asphalt are highly sensitive to temperature, all asphalt tests must be conducted at a specified temperature within very tight tolerances (The Asphalt Institute 1989).

Before the SHRP research, the asphalt cement specifications typically were based on measurements of viscosity, penetration, ductility, and softening point temperature. These measurements are not sufficient to properly describe the viscoelastic and failure properties of asphalt cement that are needed to relate asphalt binder properties to mixture properties and to pavement performance. The new Performance Grade binder specifications were designed to provide performance-related properties that can be related in a rational manner to pavement performance (McGennis 1994).

9.6.1 Performance Grade Characterization Approach

The Performance Grade tests used to characterize the asphalt binder are performed at pavement temperatures to represent the upper, middle, and lower range of service temperatures. The measurements are obtained at temperatures in keeping with the distress mechanisms. Therefore, unlike previous specifications that require performing the test at a fixed temperature and varying the requirements for different grades of asphalt, the Performance Grade specifications require performing the test at the critical pavement temperature and fixing the criteria for all asphalt grades. Thus, the Performance Grade philosophy ensures that the asphalt properties meet the specification criteria at the critical pavement temperature.

Three pavement design temperatures are required for the binder specifications: a maximum, an intermediate, and a minimum temperature. The maximum and minimum pavement temperatures for a given geographical location in the United States can be generated using algorithms contained within the SHRP software, based on weather information from 7500 weather stations. The maximum pavement design temperature is selected as the highest successive seven-day average maximum pavement temperature. The minimum pavement design temperature is the minimum pavement temperature expected over the life of the pavement. The intermediate pavement design temperature is the average of the maximum and minimum pavement design temperatures plus 4°C.

Laboratory tests that evaluate rutting potential use the maximum pavement design temperature, whereas tests that evaluate fatigue potential use the intermediate pavement design temperature. Thermal-cracking tests use the minimum pavement design temperature plus 10°C (18°F). The minimum pavement design temperature is increased by 10°C to reduce the testing time. These results are corrected to the minimum temperature using the time–temperature shift factor (McGennis 1994).

9.6.2 Performance Grade Binder Characterization

Several tests are used in the Performance Grade method to characterize the asphalt binder. Some of these tests have been used before for asphalt testing, while others are new. The discussion that follows summarizes the main steps and the significance of SHRP tests. With the exception of the rotational (Brookfield) viscometer, solubility, and flash point tests, the test temperatures are selected based on the temperature at the design location. The binder specification indicates the specific test temperatures used for various binders for each test (McGennis 1994). Four tests are performed on the neat or tank asphalt: flash point, solubility, rotational viscosity, and dynamic shear rheometer. To simulate the effect of aging on the properties of the binder, the rolling thin-film oven and pressure-aging vessel condition the binder for short-term and long-term effects. Samples are conditioned with both the rolling thin film oven and pressure-aging vessel before determining their characteristics with respect to fatigue and low-temperature cracking.

Rolling Thin-Film Oven The rolling thin-film oven (RTFO) procedure is used to simulate the short-term aging that occurs in the asphalt during production of asphalt concrete. In the RTFO method (ASTM D2872), the asphalt binder is poured into special bottles, as shown in Figure 9.12. The bottles are placed in a rack in a forced-draft oven at a temperature of 163°C (325°F) for 75 min. The rack rotates vertically, continuously exposing fresh asphalt. The binder in the rotating bottles is also subjected to an air jet to speed up the aging process. The Performance Grade specifications limit the amount of mass loss during RTFO conditioning. Rolling thin-film oven conditioning is used to prepare samples for evaluation for rutting potential with the dynamic shear

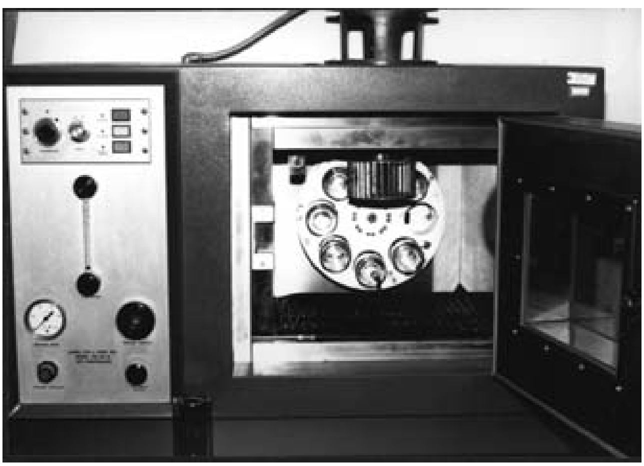

FIGURE 9.12 Rolling thin film oven test apparatus.

rheometer and prior to conditioning with the pressure-aging vessel. Under the penetration and viscosity grading methods, the aged binder is usually tested for penetration or viscosity and the results are compared with those of new asphalt.

Pressure-Aging Vessel The pressure-aging vessel (PAV) consists of a temperature-controlled chamber, and pressure- and temperature-controlling and measuring devices, as illustrated in Figure 9.13 (ASTM D6521). The asphalt binder is first aged, using the rolling thin-film oven (RTFO) (ASTM D2872). A specified thickness of residue from the RTFO is placed in the PAV pans. The asphalt is then aged at the specified aging temperature for 20 hours in a vessel under 2.10 MPa (305 psi) of air pressure. Aging temperature, which ranges between 90°C and 110°C, is selected according to the grade of the asphalt binder. Since the procedure forces oxygen into the sample, it is necessary to use a vacuum oven to remove any air bubbles from the sample prior to testing.

The PAV is designed to simulate the oxidative aging that occurs in asphalt binders during pavement service. Residue from this process may be used to estimate the physical or chemical properties of an asphalt binder after 5 to 10 years in the field.

Flash Point At high temperatures, asphalt can flash or ignite in the presence of open flame or spark. The flash point test is a safety test that measures the temperature at which the asphalt flashes; asphalt cement may be heated to

FIGURE 9.13 Pressure aging vessel apparatus.

a temperature below this without becoming a fire hazard. The Cleveland open cup method (ASTM D92) requires partially filling a standard brass cup with asphalt cement. The asphalt is then heated at a specified rate and a small flame is periodically passed over the surface of the cup, as shown in Figure 9.14. The flash point is the temperature of the asphalt when the volatile fumes coming off the sample will sustain a flame for a short period of time. The minimum temperature at which there are sufficient volatile fumes to sustain a flame for an extended period of time is the fire point.

Rotational Viscometer Test The rotational (Brookfield) viscometer test (ASTM D4402) consists of a rotational coaxial cylinder viscometer and a temperature control unit, as shown in Figure 9.15. The test is performed on unaged binders. The asphalt binder sample is placed in the sample chamber at 135°C (275°F); then both are placed in the thermocell. A spindle is placed in the asphalt sample and rotated at a specified speed. The viscosity is determined by the amount of torque required to rotate the spindle at the specified speed. The spindle size used is determined based on the viscosity being measured. The viscosity is computed in centipoises (cP) by the testing machine. The Performance Grade specification limit is stated in Pascal seconds (Pa·s), which is equal to cP divided by 1000. The viscosity is recorded as the average of three readings, at one-minute intervals, to the nearest 0.1 Pa·s. In addition to testing at the temperature required by the specification, additional tests are performed at higher temperatures to establish the temperature susceptibility relationship used to determine the compaction and mixing temperatures required for the mix design process.

Dynamic Shear Rheometer Test The dynamic shear rheometer test system, Figure 9.16, consists of two parallel metal plates, an environmental chamber, a loading device, and a control and data acquisition system (AASHTO T315). The dynamic shear rheometer is used to measure three specification requirements in the Performance Grading system. For testing the neat binder

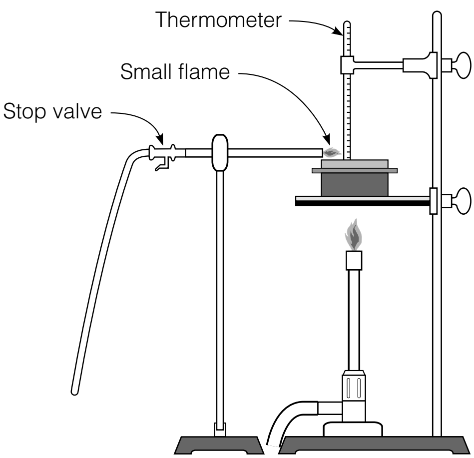

FIGURE 9.14 Cleveland open cup flash point test apparatus.

FIGURE 9.15 Rotational viscometer.

FIGURE 9.16 Dynamic shear rheometer apparatus.

and for the rutting potential test, the test temperature is equal to the upper temperature for the grade of the asphalt binder (e.g., a PG 64–22 is tested at 64°C). For these tests, the sample size is 25 mm in diameter and 1 mm thick. Prior to testing for rutting potential, the sample is conditioned in the rolling thin-film oven. For evaluating fatigue potential, the intermediate temperature is used; 25°C for PG 64-22. The sample size is 8 mm in diameter by 2 mm thick. Prior to testing, the sample is conditioned in the rolling thin-film oven, followed by the pressure-aging vessel.

During testing, one of the parallel plates is oscillated with respect to the other at preselected frequencies and rotational deformation amplitudes (or torque amplitudes). The required amplitude depends upon the value of the complex shear modulus of the asphalt binder being tested. Specification testing is performed at an angular frequency of 10 rads/s. The complex shear modulus (G^*) and phase angle (δ) are calculated automatically by the rheometer's computer software. The complex shear modulus and the phase angle define the asphalt binder's resistance to shear deformation in the linear viscoelastic region.

Bending Beam Rheometer Test The bending beam rheometer measures the midpoint deflection of a simply supported prismatic beam of asphalt binder subjected to a constant load applied to its midpoint (ASTM D6648). The bending beam rheometer test system consists of a loading frame, a controlled temperature bath, and a computer-controlled automated data acquisition unit, as shown in Figure 9.17. The test temperature is 10°C higher than the lower temperature rating of the binder (e.g., a PG 64–22 is tested at −12°C). The sample is conditioned in both the rolling thin-film oven and pressure-aging vessel prior to testing. An asphalt binder beam is placed in the bath and loaded with a constant force of 980 ± 50 mN for 240 seconds. As the beam creeps, the midpoint deflection is monitored after 8, 15, 30, 60, 120, and 240 seconds. The constant maximum stress in the beam is calculated from the load magnitude and the dimensions of the beam. The maximum strain is calculated from the deflection and the dimensions of the beam. The flexural creep stiffness of the beam is then calculated by dividing the maximum stress by the maximum strain for each of the specified loading times.

The low-temperature thermal-cracking performance of paving mixtures is related to the creep stiffness, defined as the slope of the logarithm of the creep stiffness versus the logarithm of the time curve of the asphalt binder contained in the mix.

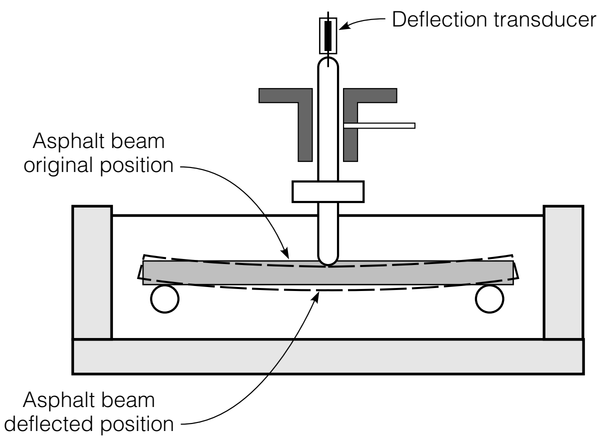

FIGURE 9.17 Schematic of the bending beam rheometer.

Direct Tension Test The direct tension test system consists of a displacement-controlled tensile loading machine with gripping system, a temperature controlled chamber, measuring devices, and a data acquisition system, as shown in Figure 9.18 (ASTM D6723). In this test, an asphalt binder specimen, conditioned in the rolling thin-film oven and pressure-aging vessel, is pulled at a constant rate of deformation of 1 mm/min. The test temperature equals the low temperature rating plus 10°C. A noncontact extensometer measures the elongation of the specimen. The maximum load developed during the test is monitored. The tensile strain and stress in the specimen when the load reaches a maximum is reported as the failure strain and failure stress, respectively.

The strain at failure is a measure of the amount of elongation that the asphalt binder can sustain without cracking. Strain at failure is used as a criterion for specifying the low-temperature properties of the binder.

FIGURE 9.18 Direct tension test apparatus.

9.6.3 Traditional Asphalt Characterization Tests

Traditional tests that have been used to characterize asphalt before the development of the Performance Grade system include the penetration and absolute and kinematic viscosities.

Penetration The penetration test (ASTM D5) measures asphalt cement consistency. An asphalt sample is prepared and brought to 25°C (77°F). A standard needle with a total mass of 100 g is placed on the asphalt surface. The needle is released and allowed to penetrate the asphalt for 5 seconds, as shown in Figure 9.19. The depth of penetration, in units of 0.1 mm, is recorded and reported as the penetration value. A large penetration value indicates soft asphalt.

Absolute and Kinematic Viscosity Tests Similar to the penetration test, the viscosity test is used to measure asphalt consistency. Two types of viscosity are commonly measured: absolute and kinematic. The absolute viscosity procedure (ASTM D2171) requires heating the asphalt cement and pouring it into a viscometer placed in a water or oil bath at a temperature of 60°C (140°F) (Figure 9.20). The viscometer is a U-tube, with a reservoir where the asphalt is introduced and a section with a calibrated diameter and timing marks. For absolute viscosity tests vacuum is applied at one end. The time during which the asphalt flows between two timing marks on the viscometer is measured using a stopwatch. The flow time, measured in seconds, is multiplied by the viscometer calibration factor to obtain the absolute viscosity in units of poises. Different-sized viscometers are used for different asphalt grades to meet minimum and maximum flow time requirements of the test procedure.

The kinematic viscosity test procedure (ASTM D2170) is similar to that of the absolute viscosity test, except that the test temperature is 135°C (275°F). Since the viscosity of the asphalt at 135°C is fairly low, vacuum is not used. The time it takes the asphalt to flow between the two timing marks is multiplied by the calibration factor to obtain the kinematic viscosity in units of cenistokes (cSt).

9.6.4 Characterization of Emulsion and Cutback

Common methods used to characterize emulsion include distillation and Saybolt–Furol viscosity tests. Cutback is characterized by distillation.

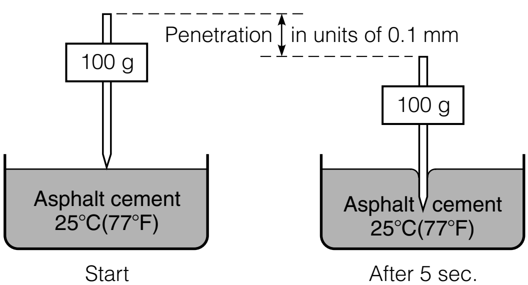

FIGURE 9.19 Penetration test.

FIGURE 9.20 Absolute viscosity test apparatus.

Distillation of Cutback and Emulsion The distillation test of cutback asphalt (ASTM D402) measures the amount and character of volatile constituents it contains. The procedure requires that the percentages, by volume, of the distillate fractions at specified temperatures be determined. The distillation test of emulsified asphalt (ASTM D244) determines the percent of residue and oil distillates by weight.

Saybolt Furol Viscosity of Emulsion Emulsion viscosity is an important factor in field applications. When applied in a spray, the emulsion must be thin enough to be uniformly applied through the spray bar of the distributor truck, yet viscous enough that it will not flow from the crown or grade of the road. Emulsion viscosity is measured using the Saybolt–Furol viscometer, as shown in Figure 9.21 (ASTM D244). In this test, the emulsion is bought to a temperature of either 25°C (77°F) or 50°C (122°F) and allowed to flow through a specific orifice. The Saybolt–Furol viscosity is the time (in seconds) required to fill a special flask.

FIGURE 9.21 Saybolt Furol viscosity.

9.7 Classification of Asphalt

Several methods are used to characterize asphalt binders, asphalt cutbacks, and asphalt emulsions.

9.7.1 Asphalt Binders

Asphalt binder is produced in several grades or classes. There are four methods for classifying asphalt binders:

1. performance grading
2. penetration grading
3. viscosity grading
4. viscosity of aged residue grading

Performance Grade Specifications and Selection Several grades of binder are available, based on their performance in the field. Names of grades start with PG (Performance Graded) followed by two numbers representing the maximum and minimum pavement design temperatures in Celsius. For example, an asphalt binder PG 52–28 would meet the specification for a design high pavement temperature up to 52°C (126°F) and a design low temperature warmer than −28°C (−18°F). These temperatures are calculated 20 mm (0.75 in.) below the pavement surface. The high and low pavement temperatures are related to the air temperature as well as other factors. Table 9.2 shows the binder grades in the Performance Grade specifications. PG 76 and 82 are intended to accommodate only slow transient or standing loads, such as those that occur near intersections or in truck climbing lanes.

The performance-graded asphalt binder specifications are shown in Table 9.3 (ASTM D6373). The table shows the design criteria of various test

TABLE 9.2 Binder Grades in the Performance Grade Specifications

High Temperature Grades (°C)	Low Temperature Grades (°C)
PG 46	−34, −40, −46
PG 52	−10, −16, −22, −28, −34, −40, −46
PG 58	−16, −22, −28, −34, −40
PG 64	−10, −16, −22, −28, −34, −40
PG 70	−10, −16, −22, −28, −34, −40
PG 76	−10, −16, −22, −28, −34
PG 82	−10, −16, −22, −28, −34

TABLE 9.3 Performance Graded Asphalt Binder Specifications

	PG 46-			PG 52-							PG 58-					PG 64-					
Performance Grade	**34**	**40**	**46**	**10**	**16**	**22**	**28**	**34**	**40**	**46**	**16**	**22**	**28**	**34**	**40**	**10**	**16**	**22**	**28**	**34**	**40**
Average seven-day Maximum Pavement Design Temperature, °C	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C	>−34	>−40	>−46	>−10	>−16	>−22	>−28	>−34	>−40	>−46	>−16	>−22	>−28	>−34	>−40	>−10	>−16	>−22	>−28	>−34	>−40
Original Binder																					
Flash Point Temp: Minimum °C	230																				
Viscosity, ASTM D4402: Maximum, 3Pa.s, Test Temp, °C	135																				
Dynamic Shear: G*/ sin δ, Minimum, 1.00kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
Rolling Thin-Film Oven Residue																					
Mass Loss, Maximum,Percent	1.00																				
Dynamic Shear: G*/ Sin δ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
Pressure-Aging Vessel (PAV) Residue																					
PAV Aging Temperature, °C	90			90							100					100					
Dynamic Shear: G* · sin δ, Maximum, 5000 kPa Test Temp @ 10rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Creep Stiffness:S, Maximum, 300 MPa, m-value, Minimum,0.300 Test Temp @ 60s, °C	−24	−30	−36	0	−6	−12	−18	−24	−30	−36	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	−30
Direct Tension: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	−24	−30	−36	0	−6	−12	−18	−24	−30	−36	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	−30

TABLE 9.3 (Continued)

Performance Grade	PG 70-						PG 76-					PG 82-				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average seven-day Maximum Pavement Design Temperature, °C	<70						<76					<82				
Minimum Pavement Design Temperature, °C	>−10	>−16	>−22	>−28	>−34	>−40	>−10	>−16	>−22	>−28	>−34	>−10	>−16	>−22	>−28	>−34
Original Binder																
Flash Point Temp: Minimum °C							230									
Viscosity, ASTM D4402: Maximum, 3 Pa.s, Test Temp, °C							135									
Dynamic Shear: $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
Rolling Thin-Film Oven Residue																
Mass Loss, Maximum, percent							1.00									
Dynamic Shear: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp@ 10 rad/s, °C	70						76					82				
Pressure-aging Vessel (PAV) Residue																
PAV Aging Temperature, °C	100 (110)						100 (110)					100 (110)				
Dynamic Shear, $G^* \cdot \sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Creep Stiffness: S, Maximum, 300 MPa, m-value, Minimum, 0.300 Test Temp @ 60s, °C	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	0	−6	−12	−18	−24
Direct Tension: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	0	−6	−12	−18	−24

parameters at the specified test temperatures. One important difference between the Performance Grade specifications and the traditional specifications is in the way the specifications work. As shown in Table 9.3, the physical properties (criteria) remain constant for all grades, but the temperatures at which these properties must be achieved vary, depending on the climate at which the binder is expected to be used. The temperature ranges shown in Table 9.3 encompass all pavement temperature regimes that exist in the United States and Canada.

The binder is selected to satisfy the maximum and minimum design pavement temperature requirements. The average seven-day maximum pavement temperature is used to determine the design maximum, whereas the design minimum pavement temperature is the lowest pavement temperature. Since the maximum and minimum pavement temperatures vary from one year to another, a reliability level is considered. As used in the Performance Grade, reliability is the percent probability in a single year that the actual pavement temperature will not exceed the design high pavement temperature or be lower than the design low pavement temperature.

It is assumed that the design high and design low pavement temperatures throughout the years follow normal distributions as illustrated in Figure 9.22(a). In this example, the average seven-day maximum pavement temperature is 56°C and the standard deviation is 2°C. Similarly, the average one-day minimum pavement temperature is −23°C and the standard deviation is 4°C. Since the area under the normal distribution curve represents the probability as illustrated in Figure 1.19, the range of temperature that satisfies the assumed probability can be calculated. For example, the range between −23°C and 56°C results in a 50% reliability for both high and low temperatures. By subtracting two standard deviations from the minimum pavement temperature and adding two standard deviations to the maximum pavement temperature, the range between −31°C and 60°C results in 98% reliability. In selecting the appropriate grade, the designer should select the standard PG grade that most closely satisfies the required reliability level. This "rounding" typically results in a higher reliability level than is intended, as shown in Figure 9.22(b). Note that the reliability levels at the high- and low-temperature grades do not need to be the same, depending on the specific pavement conditions.

Sample Problem 9.1

What standard PG asphalt binder grade should be selected under the following conditions:

The seven-day maximum pavement temperature has a mean of 57°C and a standard deviation of 2°C.

The minimum pavement temperature has a mean of −6°C and a standard deviation of 3°C.

Reliability is 98%.

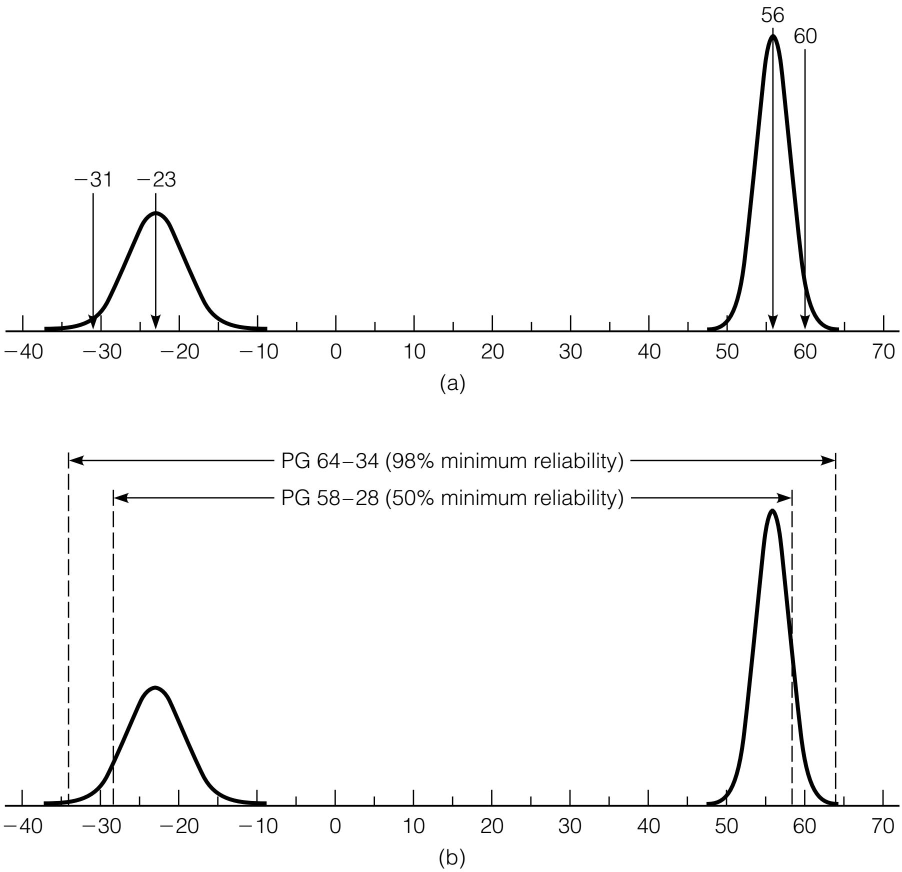

FIGURE 9.22 Example of the distribution of design pavement temperatures and selection of binder grades: (a) distribution of high and low design pavement temperatures, and (b) binder grade selection.

Solution

High-temperature grade $\geq 57 + (2 \times 2) \geq 61°\text{C}$

Low-temperature grade $\leq -6 - (2 \times 3) \leq -12°\text{C}$

The closest standard PG asphalt binder grade that satisfies the two temperature grades is PG 64–16.

Other Asphalt Binder Grading Methods Table 9.4 shows various asphalt cement grades based on penetration and on their properties (ASTM D946). The grades correspond to the allowable penetration range; that is, the penetration of a 40–50 grade must be in the range of 40 to 50. Various grades based on viscosity and their properties are shown in Table 9.5 (ASTM D3381). The AC

TABLE 9.4 Penetration Grading System of Asphalt Cement

Grade	Penetration min.	Penetration max.	Flash Point °C (°F)	Ductility (cm)
40–50	40	50	232 (450)	100
60–70	60	70	232 (450)	100
85–100	85	100	232 (450)	100
120–150	120	150	219 (425)	100
200–300	200	300	177 (350)	100

TABLE 9.5 Viscosity Grading System of Asphalt Cement

Grade	Viscosity Absolute (poises)	Viscosity Kinematic* (cSt)	Penetration*	Flash Point* °C (°F)
AC-2.5	250 ± 50	125	220	163 (325)
AC-5	500 ± 100	175	140	177 (350)
AC-10	1000 ± 200	250	80	219 (425)
AC-20	2000 ± 400	300	60	232 (450)
AC-30	3000 ± 600	350	50	232 (450)
AC-40	4000 ± 800	400	40	232 (450)

*Specification is for the minimum acceptable values

grade numbers are 1/100 of the middle of the allowable viscosity range; that is, an AC-5 has an absolute viscosity of 500 ± 100 poises. Therefore, high viscosity asphalt cements have a high designation number. Aged-residue grades are based on the absolute viscosity of the asphalt after it has been conditioned to simulate the effects of the aging that occur when the asphalt cement is heated to make asphalt concrete. The aged-residue grade numbers are at the middle of the allowable viscosity range after conditioning, as shown in Table 9.6 (ASTM D3381). For example, an AR-1000 has an absolute viscosity of 1000 ± 250 poises.

TABLE 9.6 Aged-Residue Grading System of Asphalt Cement

Grades	Viscosity: Absolute (poises)	Viscosity: Kinematic* (cSt)	Penetration*	Flash Point*@ °C (°F)
AR-1000	1000 ± 250	140	65	205 (400)
AR-2000	2000 ± 500	200	40	219 (425)
AR-4000	4000 ± 1000	275	25	227 (440)
AR-8000	8000 ± 2000	400	20	232 (450)
AR-16000	16000 ± 4000	550	20	238 (460)

*Specification is for the minimum acceptable values
@Flash point specification is for the asphalt cement before rolling thin-film oven conditioning. All other specifications are for samples that have been conditioned.

9.7.2 Asphalt Cutbacks

Three types of cutbacks are produced, depending on the hardness of the residue and the type of solvent used. *Rapid-curing cutbacks* are produced by dissolving hard residue in a highly volatile solvent, such as gasoline. *Medium-curing cutbacks* use medium hardness residue and a less volatile solvent, such as kerosene. *Slow-curing cutbacks* are produced by either diluting soft residue in nonvolatile or low-volatility fuel oil or by simply stopping the refining process before all of the fuel oil is removed from the stock.

Curing the cutback refers to the evaporation of the solvent from the asphalt residue. Rapid-curing (RC) cutbacks cure in about 5 to 10 minutes, while medium-curing (MC) cutbacks cure in a few days. Slow-curing (SC) cutbacks cure in a few months. In addition to the three types, cutbacks have several grades defined by the kinematic viscosity at 60°C (140°F). Grades of 30, 70, 250, 800, and 3000 are manufactured, with higher grades indicating higher viscosities. Thus, cutback asphalts are designated by letters (RC, MC, or SC), representing the type, followed by a number that represents the grade. For example, MC-800 is a medium-curing cutback with a grade of 800. The different grades of cutback are produced by varying the amounts and types of solvent and base asphalt. The specifications of cutbacks are standardized by ASTM D2026, D2027, and D2028.

9.7.3 Asphalt Emulsions

Asphalt emulsions are produced in a variety of combinations of the electric charge of the emulsifying agent, the rate the emulsion sets (brakes), the viscosity of the emulsion, and the hardness of the asphalt cement. Both anionic and cationic are produced These emulsions have negative and positive charges respectively. Depending on the emulsion concentration, the set or break time is varied from rapid to medium to slow set. Rapid-setting emulsion sets in about 5 to 10 minutes, medium-setting emulsion sets in several hours, and slow-setting emulsion sets in a few months. The viscosity of the emulsion is rated as normal flow or slow flow, based on the Saybolt–Furol viscosity (Figure 9.21). The consistency of the asphalt in the emulsion is evaluated with the penetration test. Asphalt residues with a penetration of 100–200 are typically used. However soft asphalts, with a penetration of greater than 200, or hard asphalts, with a penetration of 60–100, may be used.

Asphalt emulsion types are designated based on the setting rate as RS, MS or SS for rapid, medium, and slow set respectively. If a cationic emulsifying agent was used, the set designation is preceded by a C (e.g., a cationic rapid set emulsion is designated as CRS). If the first letter of the emulsion type is not a C, then it is an anionic emulsion. The flow rate of the emulsion is designated next with a 1 or 2 for a normal or slow flow respectively. Finally, the consistency of the asphalt is identified. Not all possible combinations of charge type, set rate, viscosity, and asphalt hardness are produced. Table 9.7 is a summary of the emulsion grades and types (Jansich and Gaillard, 1998).

Other emulsion types are also produced, such as the high float residue emulsion and the quick-set emulsion. The specifications of various asphalt emulsions are standardized by ASTM D977.

TABLE 9.7 Asphalt Emulsion Grades

Charge	Grade	Setting Speed	Viscosity of Emulsion, Saybolt Furol at 25°C	Penetration of Residue
Anionic	RS-1	Rapid	20–100	100–200
	RS-2	Rapid	75–400*	100–200
	MS-1	Medium	20–100	100–200
	MS-2	Medium	≥100	100–200
	MS-2h	Medium	≥100	60–100
	SS-1	Slow	20–100	100–200
	SS-1h	Slow	20–100	60–100
Cationic	CRS-1	Rapid	20–100	100–250
	CRS-2	Rapid	100–400	100–250
	CMS-2	Medium	50–450	100–250
	CMS-2h	Medium	50–450	60–100
	CSS-1	Slow	20–100	100–250
	CSS-1h	Slow	20–100	60–100

*Test at 50°C

9.8 Asphalt Concrete

Asphalt concrete, also known as hot-mix asphalt (HMA), consists of asphalt cement and aggregates mixed together at a high temperature and placed and compacted on the road while still hot. Asphalt (flexible) pavements cover approximately 93% of the 2 million miles of paved roads in the United States, while the remaining 7% of the roads are portland cement concrete (rigid) pavements. The performance of asphalt pavements is largely a function of the asphalt concrete surface material.

The objective of the asphalt concrete mix design process is to provide the following properties (Roberts et al. 1996):

1. stability or resistance to permanent deformation under the action of traffic loads, especially at high temperatures
2. fatigue resistance to prevent fatigue cracking under repeated loadings
3. resistance to thermal cracking that might occur due to contraction at low temperatures
4. resistance to hardening or aging during production in the mixing plant and in service
5. resistance to moisture-induced damage that might result in stripping of asphalt from aggregate particles
6. skid resistance, by providing enough texture at the pavement surface
7. workability, to reduce the effort needed during mixing, placing and compaction

Regardless of the set of criteria used to state the objectives of the mix design process, the design of asphalt concrete mixes requires compromises. For example, extremely high stability often is obtained at the expense of lower durability, and vice versa. Thus, in evaluating and adjusting a mix design for a particular use, the aggregate gradation and asphalt content must strike a favorable balance between the stability and durability requirements. Moreover, the produced mix must be practical and economical.

9.9 Asphalt Concrete Mix Design

The purpose of asphalt concrete mix design is to determine the design asphalt content using the available asphalt and aggregates. The design asphalt content varies for different material types, material properties, loading levels, and environmental conditions. To produce good-quality asphalt concrete, it is necessary to accurately control the design asphalt content. If the appropriate design asphalt content is not used, the pavement will lack durability or stability, resulting in premature pavement failure. For example, if not enough asphalt binder is used, not all the aggregate particles will be coated with asphalt, which will result in a less stable and less durable material. Also, if too much binder is used, aggregate particles may have too much "lubrication" and may move relative to each other upon application of the load, resulting in a less stable material. Typical design asphalt contents range from 4% to 7% by weight of total mix.

Before the Superpave mix design method was developed during the SHRP program, there were two common asphalt concrete design methods: Marshall (ASTM D1559), and Hveem (ASTM D1560). The Marshall method was more commonly used than the Hveem method, due to its relative simplicity and its ability to be used for field control. Both methods are empirical in nature; that is, they are based on previous observations. Both methods have been used satisfactorily for several decades and have produced long-lasting pavement sections. However, due to their empirical nature they are not readily adaptable to new conditions, such as modified binders, large-sized aggregates, and heavier traffic loads.

The Superpave design system is performance based and is more rational than the Marshall and Hveem methods. Many highway agencies are implementing the Superpave system.

9.9.1 Specimen Preparation in the Laboratory

Asphalt concrete specimens are prepared in the laboratory for mix-design and quality-control tests. To prepare specimens in the lab, aggregates are batched and heated, according to a specified gradation. Asphalt cement is also heated separately and added to the aggregate at a specified rate. Aggregates and asphalt are mixed with a mechanical mixer until the aggregate particles are completely coated with asphalt. Three compaction machines are commonly used:

1. Superpave gyratory compactor
2. Marshall hammer
3. California kneading compactor

Regardless of the compaction method, the procedure for preparing specimens basically follows the same four steps:

1. Heat and mix the aggregate and asphalt cement
2. Place the material into a mold
3. Apply compactive force
4. Allow the specimen to cool and extrude from the mold

The specific techniques for placing the material into the mold vary among the three compaction methods, and the standards for the test must be followed.

The greatest difference among the compaction procedures is the manner in which the compactive force is applied. For the gyratory compaction, the mixture in the mold is placed in the compaction machine at an angle to the applied force. As the force is applied the mold is gyrated, creating a shearing action in the mixture. Gyratory compaction devices have been available for a long time, but their use was limited due to the lack of a mix-design procedure based on this type of compaction. However, the Superpave mix design method (FHWA, 1995) uses a gyratory compactor; thus, this compaction method is now common. Figure 9.23 shows the Superpave gyratory compactor.

In the Marshall procedure (Figure 9.24), a slide hammer weighing 4.45 kg (10 lb) is dropped from a height of 0.46 m (18 in.) to create an impact compaction force (ASTM D1559). The head of the Marshall hammer has a diameter equal to the specimen size, and the hammer is held flush with the specimen at all times.

In the California kneading compactor method (Figure 9.25), the area of the compactor foot is smaller than the area of the mold. After each compaction stroke the mold is rotated, subjecting the asphalt mixture to a kneading action (ASTM D1561). After the kneading compaction is complete, the specimen is

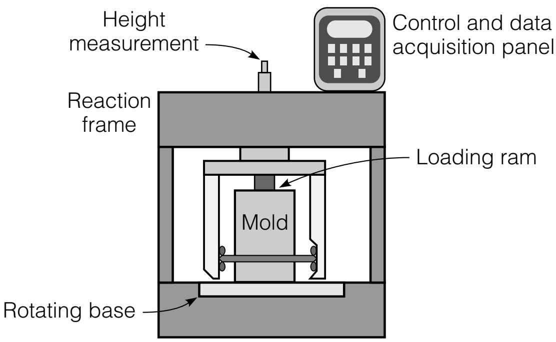

FIGURE 9.23 Superpave gyratory compactor.

FIGURE 9.24 Marshall compactor.

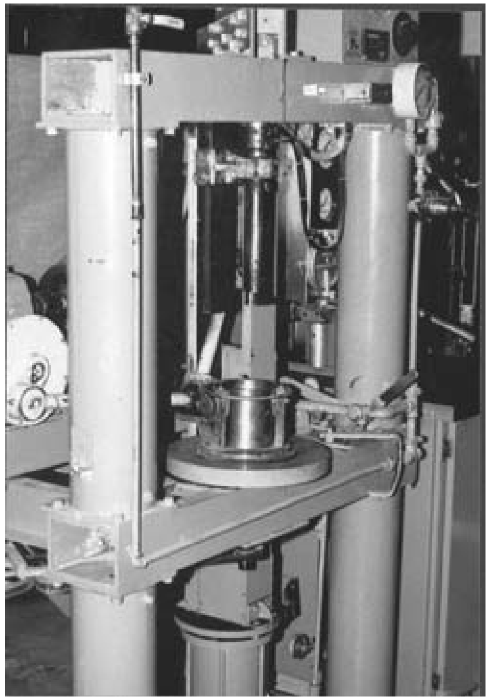

FIGURE 9.25 California kneading compactor.

reheated while still in the mold; then a compression machine is used to apply a static force to level the face of the specimen.

The Superpave gyratory compactor is used for the Superpave mix design, whereas the Marshall hammer and the California kneading compactor are used for the Marshall and Hveem methods of mix design, respectively. The Superpave gyratory compactor produces specimens 150 mm in diameter and 95 mm to 115 mm high, allowing the use of aggregates with a maximum size of more than 25 mm (1 in.). Specimens prepared with both Marshall and California kneading compactors, as well as some gyratory compactors, are typically 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high.

9.9.2 Density and Voids Analysis

It is important to understand the density and voids analysis of compacted asphalt mixtures for both mix design and construction control. Regardless of the method used, the mix design is a process to determine the volume of asphalt binder and aggregates required to produce a mixture with the desired properties. However, since volumes are difficult and not practical to measure, weights are used instead; the specific gravity is used to convert from weight to volume. Figure 9.26 shows that the asphalt mixture consists of aggregates, asphalt binder, and air voids. Note that a portion of the asphalt is absorbed by aggregate particles. Three important parameters commonly used

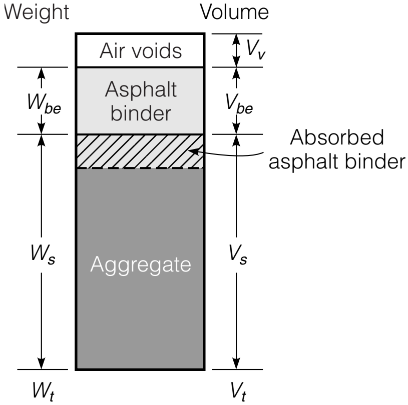

FIGURE 9.26 Components of compacted asphalt mixture.

are percent of air voids (voids in total mix) (VTM), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). These are defined as

$$\text{VTM} = \frac{V_v}{V_m}100 \tag{9.1}$$

$$\text{VMA} = \frac{V_v + V_{\text{be}}}{V_m}100 \tag{9.2}$$

$$\text{VFA} = \frac{V_{\text{be}}}{V_{\text{be}} + V_V}100 \tag{9.3}$$

where

V_v = volume of air voids
V_{be} = volume of effective asphalt binder
V_m = total volume of the mixture

The effective asphalt is the total asphalt minus the absorbed asphalt.

Sample Problem 9.2

A compacted asphalt concrete specimen contains 5% asphalt binder (Sp. Gr. 1.023) by weight of total mix, and aggregate with a specific gravity of 2.755. The bulk density of the specimen is 2.441 Mg/m^3. Ignoring absorption, compute VTM, VMA, and VFA.

Solution

Referring to Figure SP 9.2, assume $V_t = 1\text{ m}^3$
Determine mass of mix and components:

Total mass $= 1 \times 2.441 = 2.441$ Mg

Mass of asphalt binder $= 0.05 \times 2.441 = 0.122$ Mg

Mass of aggregate $= 0.95 \times 2.441 = 2.319$ Mg

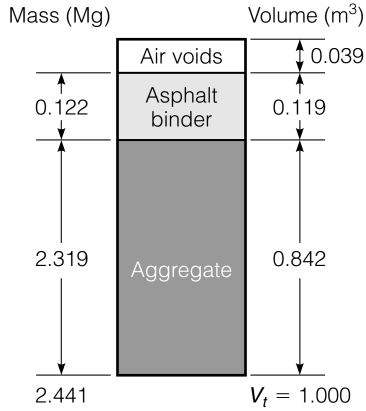

FIGURE SP9.2

Determine volume of components:

$$V_b = \frac{0.122}{1.023} = 0.119 \text{ m}^3$$

Since the problem statement specified no absorption, $V_{be} = V_b$

$$V_s = \frac{2.319}{2.755} = 0.842 \text{ m}^3$$

V_s = Volume of stone (aggregate)

Determine volume of voids:

$$V_v = V_m - V_b - V_s = 1 - 0.199 - 0.842 = 0.039 \text{ m}^3$$

Volumetric calculations:

$$\text{VTM} = \frac{V_v}{V_m}100 = \frac{0.093}{1.00}100 = 3.9\%$$

$$\text{VMA} = \frac{V_v + V_{be}}{V_m}100 = \frac{0.039 + 0.119}{1.00}100 = 15.8\%$$

$$\text{VFA} = \frac{V_{be}}{V_{be} + V_V}100 = \frac{0.119}{0.119 + 0.039}100 = 75\%$$

The density and void analysis requires using the effective specific gravity of the asphalt-coated aggregate, determined from the theoretical maximum specific gravity of the mix (ASTM D2041). The weight of the loose mixture specimen in air *A* is measured along with the weight of the measurement bowl filled with water *D* and the weight of the bowl containing the asphalt mix and filled with water *E*. When the loose mixture specimen is submerged in water, a vacuum is used to remove all air from the sample. The theoretical maximum specific gravity is

$$G_{mm} = \frac{A}{A + D - E} \tag{9.4}$$

It is necessary to determine only the theoretical maximum specific gravity of the sample at one asphalt content. However, the result should be based on the average of three samples (with a minimum of two). By definition, the theoretical maximum specific gravity of asphalt concrete is

$$G_{mm} = \frac{100}{\left(\frac{P_s}{G_{se}} + \frac{P_b}{G_b}\right)} \tag{9.5}$$

Solving this equation for G_{se} produces

$$G_{se} = \frac{P_s}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)} \tag{9.6}$$

where

G_{mm} = theoretical maximum specific gravity of the asphalt concrete
P_s = percent weight of the aggregate
P_b = percent weight of the asphalt cement
G_{se} = effective specific gravity of aggregate coated with asphalt
G_b = specific gravity of the asphalt binder

Although G_{se} is determined for only one asphalt content, it is constant for all asphalt contents. Thus, once G_{se} is determined based on the results of the theoretical maximum specific gravity test, it can be used in Equation 9.5 to calculate G_{mm} for the different asphalt contents.

The next step in the process is to determine the bulk specific gravity G_{mb} (ASTM D2726) of each of the compacted specimens. This requires weighing the sample in three conditions: dry, saturated-surface dry, and submerged. The bulk specific gravity is computed as

$$G_{mb} = \frac{\text{Weight in air}}{(\text{Weight SSD} - \text{Weight in water})} \tag{9.7}$$

The unit weight of each specimen is computed by multiplying the bulk specific gravity by the density of water, 1 Mg/m^3 (62.4 lb/ft^3). The average bulk specific gravity and unit weight for each asphalt content are computed and used to calculate VTM as follows:

$$\text{VTM} = 100\left(1 - \frac{G_{mb}}{G_{mm}}\right) \tag{9.8}$$

The percent voids in the mineral aggregate (VMA), is a measure of the space available in the aggregates for the addition of the asphalt cement. The percent VMA is the volume of the mix minus the volume of the aggregates, divided by the volume of the mix and converted to a percent. VMA is commonly computed from the bulk specific gravity of the aggregate G_{sb}, the bulk specific gravity of the mix G_{mb} and the percent weight of aggregate as

$$\mathrm{VMA} = \left(100 - G_{mb}\frac{P_s}{G_{sb}}\right) \tag{9.9}$$

The percent of the voids filled with asphalt, %VFA, is determined as

$$\mathrm{VFA} = 100\left(\frac{\mathrm{VMA} - \mathrm{VTM}}{\mathrm{VMA}}\right) \tag{9.10}$$

Sample Problem 9.3

An asphalt concrete specimen has the following properties:

asphalt content = 5.9% by total weight of mix
bulk specific gravity of the mix = 2.457
theoretical maximum specific gravity = 2.598
bulk specific gravity of aggregate = 2.692

Calculate the percents VTM, VMA, and VFA.

Solution

$$\mathrm{VTM} = 100\left(1 - \frac{G_{mb}}{G_{mm}}\right) = 100\left(1 - \frac{2.457}{2.598}\right) = 5.4\%$$

$$\mathrm{VMA} = \left(100 - G_{mb}\frac{P_s}{G_{mm}}\right) = \left(100 - 2.457\frac{100 - 5.9}{2.692}\right) = 14.1\%$$

$$\mathrm{VFA} = 100\left(\frac{\mathrm{VMA} - \mathrm{VTM}}{\mathrm{VMA}}\right) = 100\left(\frac{14.1 - 5.4}{14.1}\right) = 61.7\%$$

9.9.3 Superpave Mix Design

The Superpave mix-design process consists of

- Selection of aggregates
- Selection of binder
- Determination of the design aggregate structure
- Determination of the design binder content.
- Evaluation of moisture susceptibility

Aggregate Selection Aggregate properties under the Superpave mix-design system are described as either consensus or source properties. The following *consensus* aggregate properties are required:

- coarse aggregate angularity measured by the percentage of fractured faces
- fine aggregate angularity (AASHTO TP 33) (see apparatus in Figure 5.5)
- flat and elongated particles (ASTM D4791)
- clay content (ASTM D2419)

Specification limits for these properties depend on the traffic level and how deep under the pavement surface the materials will be used, as shown in Table 9.8. In addition to these properties, highway agencies may consider other factors that are critical to the specific local conditions. These are called *source* properties and may include Los Angeles abrasion (see apparatus in Figure 5.6), soundness, and deleterious materials. Source properties are defined at the local level; consensus properties are defined at the national level.

Aggregate used in asphalt concrete must be well graded. Superpave recommends using the 0.45 power chart discussed in Chapter 5. The gradation curve should go through control points specified by Superpave. Figure 9.27 shows the gradation requirements for the 12.5 mm (1/2 in.) nominal-sized mix.

To control segregation, aggregates are sorted into stockpiles based on size. The designer of an asphalt concrete mix must select a blend of stockpiles that meets the source, consensus, and gradation requirements.

Binder Selection The binder is selected based on the maximum and minimum pavement temperatures, as discussed earlier. In addition to the specification tests, the specific gravity and the rotational viscosity versus temperature relationship for the selected asphalt binder must be measured. The specific gravity is needed for the void analysis. The viscosity–temperature relationship is needed to determine the required mixing and compaction temperatures. The Superpave method requires mixing the asphalt and aggregates at a temperature at which the rotational viscosity of the asphalt binder is 0.170 ± 20 Pa · s and the compacting temperature corresponds to a viscosity of 0.280 ± 30 Pa · s.

TABLE 9.8 Superpave Consensus Aggregate Properties

Design Level	Course Aggregate Angularity (% min)	Fine Aggregate Angularity (% min)	Sand Equivalency (% min)	Flat and Elongated (% min)
Light Traffic	55/-	—	40	—
Medium Traffic	75/-	40	40	10
Heavy Traffic	85/80*	45	45	10

*85/80 denotes minimum percentages of one fractured face / two or more fractured faces

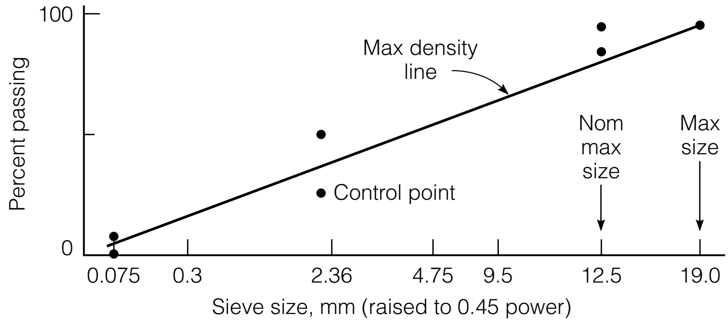

FIGURE 9.27 Superpave gradation limits for 12.5 mm nominal maximum size.

Design Aggregate Structure After selecting the appropriate aggregate, binder, and modifiers (if any), trial specimens are prepared with three different aggregate gradations and asphalt contents. There are equations for estimating the optimum asphalt content for use in the specimens prepared for determining the design aggregate structure. However, these equations are empirical and the designer is given latitude to estimate the asphalt content. Specimens are compacted using the Superpave gyratory compactor (Figure 9.23) with a gyration angle of 1.25 degrees and a constant vertical pressure of 600 kPa (87 psi). The number of gyrations used for compaction is determined based on the traffic level, as shown in Table 9.9.

As shown in Table 9.9, the Superpave method recognizes three critical stages of compaction, initial, design, and maximum. The design compaction level N_{des} corresponds to the compaction that is anticipated at the completion of the construction process. The maximum compaction N_{max} corresponds to the ultimate density level that will occur after the pavement has

TABLE 9.9 Number of Gyrations at Specific Design Traffic Levels

	Traffic Level (10^6 ESAL*)			
	<0.3	0.3 to 3	3 to 30	>30
N_{ini}	6	7	8	9
N_{des}	50	75	115	125
N_{max}	75	100	160	205

*ESAL is the 18,000-lb equivalent single axle load. It is a design factor used in the design of pavement that considers both traffic volume and loads (Huang 2004).

been subjected to traffic for a number of years. The initial compaction level N_{ini} was implemented to assist with identifying "tender" mixes. A tender mix lacks stability during construction, and hence will displace under the rollers rather than densifying. For the initial stage of determining the design aggregate structure, samples are compacted with N_{des} gyrations. The volumetric properties are determined by measuring the bulk specific gravity G_{mb} of the compacted mix and the maximum theoretical specific gravity G_{mm} of a loose mix with the same asphalt content and aggregate composition. The volumetric parameters, VTM, VMA, and VFA, are determined and checked against the criteria. Two additional parameters are evaluated in the Superpave method: percent of G_{mm} at N_{ini}, and the dust-to–effective binder content ratio. The percent of G_{mm} at N_{ini} is determined as

$$\text{Percent } G_{\text{mm,Nini}} = \text{Percent } G_{\text{mm,Ndes}} \frac{h_{\text{des}}}{h_{\text{ini}}} \tag{9.11}$$

where h_{ini} and h_{des} are the heights of the specimen at the initial and design number of gyrations, respectively. Note that the percent $G_{mm,Ndes}$ is equal to (100-VTM).

The dust-to–effective binder ratio is the percent of aggregate passing the 0.075 mm (#200) sieve divided by the effective asphalt content, computed as

$$P_{\text{ba}} = 100\left(\frac{G_{\text{se}} - G_{\text{sb}}}{G_{\text{sb}}G_{\text{se}}}\right)G_b \tag{9.12}$$

$$P_{\text{be}} = P_b - \left(\frac{P_{\text{ba}}}{100}\right)P_s \tag{9.13}$$

$$D/B = \frac{P_D}{P_{\text{be}}} \tag{9.14}$$

where

D/B = dust to binder ratio
P_{ba} = percent absorbed binder based on the mass of aggregates
G_{sb} = bulk specific gravity of aggregate
P_D = percent dust, or % of aggregate passing the 0.075mm sieve
P_{be} = percent effective binder content

The Superpave method requires determining the volumetric properties at 4% VTM. The samples prepared for the evaluation of the design aggregate content are not necessarily at the binder content required for achieving this level of air voids. Therefore, the results of the volumetric evaluation are "corrected" to four percent air voids, as

$$P_{\text{b,est}} = P_{\text{bt}} - 0.4(4 - \text{VTM}_{\text{t}}) \tag{9.15}$$

$$\text{VMA}_{\text{est}} = \text{VMA}_{\text{t}} + C(4 - \text{VTM}_{\text{t}}) \tag{9.16}$$

$$C = 0.1 \text{ for VTM}_{\text{t}} < 4.0\%$$

$$C = 0.2 \text{ for VTM}_{\text{t}} \geq 4.0\%$$

$$\text{VFA}_{\text{est}} = 100\frac{\text{VMA}_{\text{t}} - 4.0}{\text{VMA}_{\text{t}}} \tag{9.17}$$

TABLE 9.10 Superpave Mix Design Criteria

Design Air Voids	4%					
Dust to Effective Asphalt[1]	0.6–1.2					
Tensile strength ratio	80% min					
	Nominal Maximum Size (mm)					
	37.5	25	19	12.5	9.5	4.75
Minimum VMA (%)	11	12	13	14	15	16

G_{mm} and VFA Requirements

Design EASL in millions	Percent Maximum Theoretical Specific Gravity N_{init}	N_{des}	N_{max}	Percent Voids Filled with Asphalt[2,3,4]
<0.3	≤91.5	96	≤98.0	70–80
0.3–3	≤90.5	96	≤98.0	65–78
3–10	≤89.0	96	≤98.0	65–75
10–30	≤89.0	96	≤98.0	65–75
≥30	≤89.0	96	≤98.0	65–75

Notes

1. Dust-to-binder ratio range is 0.9 to 2.0 for 4.75 mm mixes.
2. For 9.5 mm nominal maximum aggregate size mixes and design VFA ≥ 3 million, VFA range is 73 to 76% and for 4.75 mm mixes the range is 75 to 78%.
3. For 25 mm nominal maximum aggregate size mixes, the lower limit of the VFA range shall be 67% for design traffic levels <0.3 million ESALs.
4. For 37.5 mm nominal maximum aggregate size mixes, the lower limit of the VFA range shall be 64% for all design traffic levels.

$$P_{be,est} = P_{b,est} - \frac{P_s G_b (G_{se} - G_{sb})}{G_{se} G_{sb}} \tag{9.18}$$

$$D/B_{est} = \frac{P_D}{P_{be,est}} \tag{9.19}$$

where

$P_{b,est}$ = adjusted estimated binder content
VMA_{est} = adjusted VMA
VMA_t = VMA determined from volumetric analysis
VFA_{est} = adjusted VFA

VTM_t = VTM determined from the volumetric analysis
$P_{be,est}$ = adjusted percent effective binder
D/B_{est} = adjusted dust-to–binder ratio
P_D = percent aggregate passing 0.075mm sieve

In Equation 9.17, the air voids are assumed to be 4.0%, the target air void content for a Superpave mix design.

The adjusted results of the design aggregate structure evaluation are compared to the Superpave mix-design criteria, as shown in Table 9.10.

The design aggregate blend is the one whose adjusted volumetric parameters meet all of the criteria. In the event that more than one of the blends meets all of the criteria, the designer can use discretion for the selection of the blend. A pair of samples is compacted at the adjusted binder content for the selected aggregate blend. The average percent of maximum theoretical specific gravity is determined and compared to the design criteria. If successful, the procedure continues with the determination of the design binder content. If unsuccessful, the design process is started over with the selection of another blend of aggregates.

Sample Problem 9.4

Select a 19-mm Superpave design aggregate structure based on the following data:

	Blend		
Data	**1**	**2**	**3**
G_{mb}	2.457	2.441	2.477
G_{mm}	2.598	2.558	2.664
G_b	1.025	1.025	1.025
P_b (%)	5.9	5.7	6.2
P_s (%)	94.1	94.3	93.8
P_d (%)	4.5	4.5	4.5
G_{sb}	2.692	2.688	2.665
h_{ini} (mm)	125	131	125
h_{des} (mm)	115	118	115

Solution

In this problem, the lab data were entered into an Excel spreadsheet and the appropriate equations were entered, producing the results shown in the accompanying table. The steps in the calculation are shown for the first blend only; the reader can verify the calculations for the other blends

Volumetric Analysis

Computed	Equation		Using Data for Blend 1	Blend 1	Blend 2	Blend 3	
G_{se}	9.6	$\dfrac{P_s}{\left(\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}\right)}$	$\dfrac{94.1}{\left(\dfrac{100}{2.598} - \dfrac{5.9}{1.025}\right)}$	2.875	2.812	2.979	
VTM (%)	9.8	$100\left(1 - \dfrac{G_{mb}}{G_{mm}}\right)$	$100\left(1 - \dfrac{2.457}{2.598}\right)$	5.4	4.6	7.0	
VMA (%)	9.9	$\left(100 - G_{mb}\dfrac{P_s}{G_{sb}}\right)$	$\left(100 - 2.457\dfrac{94.1}{2.692}\right)$	14.1	14.4	12.8	
VFA (%)	9.10	$100\left(\dfrac{\mathrm{VMA} - \mathrm{VTM}}{\mathrm{VMA}}\right)$	$100\left(\dfrac{14.1 - 5.4}{14.1}\right)$	61.7	68.1	45.3	
$\% G_{mm,Nini}$	9.11	Percent $G_{mm,Ndes}\dfrac{h_{des}}{h_{ini}}$	$94.6\dfrac{115}{125}$	87.0	86.0	85.5	
P_{ba} (%)	9.12	$100\left(\dfrac{G_{se} - G_{sb}}{G_{sb}G_{se}}\right)G_b$	$100\left(\dfrac{2.875 - 2.692}{2.692 * 2.875}\right)1.025$	2.42	1.68	4.05	
P_{be} (%)	9.13	$P_b - \left(\dfrac{P_{ba}}{100}\right)P_s$	$5.9 - \left(\dfrac{2.42}{100}\right)94.1$	3.62	4.12	2.4	
D/B	9.14	$\dfrac{P_D}{P_{be}}$	$\dfrac{4.5}{3.62}$	1.2	1.1	1.9	
Adjusted Values							**Criteria**
$P_{b,est}$ (%)	9.15	$P_{bt} - 0.4(4 - \mathrm{VTM}_t)$	$5.9 - 0.4(4 - 5.4)$	6.5	5.9	7.4	
VMA_{est} (%)	9.16	$\mathrm{VMA}_t + C(4 - \mathrm{VTM}_t)$	$14.1 + 0.2(4 - 5.4)$	13.8	14.3	12.2	≥ 13
VFA_{est} (%)	9.17	$100\dfrac{\mathrm{VMA}_t - 4.0}{\mathrm{VMA}_t}$	$100\dfrac{13.8 - 5.4}{13.8}$	71.0	72.0	67.2	65–75
$P_{be,est}$ (%)	9.18	$P_{b,est} - \dfrac{P_s G_b (G_{se} - G_{sb})}{G_{se}G_{sb}}$	$6.5 - \dfrac{94.1 * 1.025(2.875 - 2.692)}{2.875 * 2.692}$	4.2	4.3	3.6	
D/B_{est}	9.19	$\dfrac{P_D}{P_{be,est}}$	$\dfrac{4.5}{4.2}$	1.1	1.0	1.3	0.8–1.2

Although both mixes 1 and 2 meet the criteria, blend 2 is preferred, since it has a higher VMA value.

Design Binder Content The design binder content is obtained by preparing eight specimens—two replicates at each of four binder contents: estimated optimum binder content, 0.5% less than the optimum, 0.5% more than the optimum, and 1% more than the optimum. Specimens are compacted using N_{des} gyrations. The volumetric properties are computed and plots are prepared of each volumetric parameter versus the binder content. The binder content that corresponds to 4% VTM is determined. Then the plots are used to determine the volumetric parameters at the selected binder content. If the properties meet the criteria in Table 9.10, two specimens are prepared and compacted using N_{max} gyrations. If these specimens meet the criteria, then the moisture sensitivity of the mix is determined.

Sample Problem 9.5

Based on the previous problem, determine the recommended asphalt content according to Superpave mix design for an equivalent single axle load (ESAL) of 20 millions.

Solution
Samples were prepared at four asphalt contents. An Excel spreadsheet was prepared to present and analyze the data. The following results are obtained:

The results are plotted against binder content, as shown in Figure SP9.5.

Design Binder Content

	Asphalt Content Trial			
Data	**1**	**2**	**3**	**4**
P_b (%)	5.4	5.9	6.4	6.9
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.570	2.558	2.530	2.510
G_b	1.025	1.025	1.025	1.025
P_s (%)	94.6	94.1	93.6	93.1
P_d (%)	4.5	4.5	4.5	4.5
G_{sb}	2.688	2.688	2.688	2.688
h_{ini} (mm)	125	131	126	130
h_{des} (mm)	115	118	114	112

Design Binder Content

Volumetric Analysis

		Asphalt Content Trial			
Computed	**Equation**	**1**	**2**	**3**	**4**
G_{se}	9.6	2.812	2.812	2.812	2.812
VTM (%)	9.8	8.5	4.6	3.0	1.6
VMA (%)	9.9	17.3	14.5	14.5	14.5
VFA (%)	9.10	50.9	68.3	79.3	89.0
$\% G_{mm}, N_{ini}$	9.11	84.2	86.0	87.8	84.7
P_{ba} (%)	9.12	1.68	1.68	1.68	1.68
P_{be} (%)	9.13	3.81	4.32	4.83	5.34
D/B	9.14	1.2	1.0	0.9	0.8

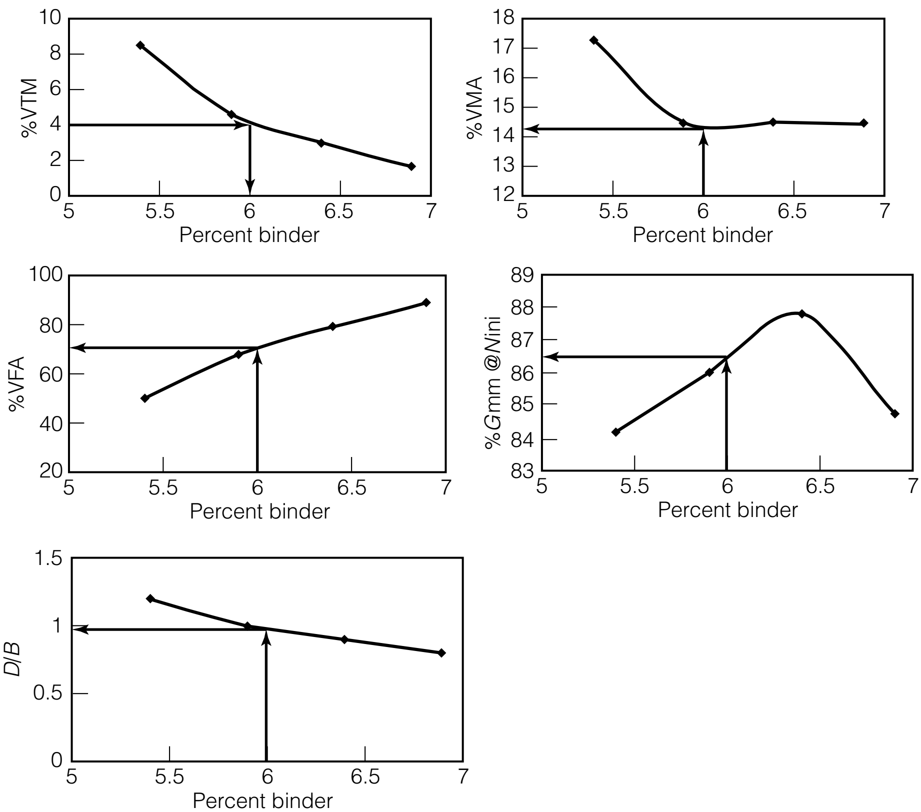

FIGURE SP9.5

The figure shows that, at 4% VTM, the binder content is 6%. The design values at 6% binder content are

- VMA = 14.5%
- VFA = 71%
- G_{mm} @ N_{ini} = 86.5%
- D/B ratio = 0.9

These values satisfy the design criteria shown in Table 9.10. Therefore, the design binder content is 6.0%.

Moisture Sensitivity Evaluation The moisture sensitivity of the design mixture is determined using the AASHTO T283 procedure on six specimens prepared at the design binder content and 7% air voids. Three specimens are conditioned by vacuum saturation, then freezing and thawing; three other specimens are not conditioned. The tensile strength of each sample is measured using a Marshall stability machine with a modified loading head. The tensile strength ratio is determined as the ratio of the average tensile strength of conditioned specimens to that of unconditioned specimens. The minimum Superpave criterion for tensile strength ratio is 80%.

9.9.4 Superpave Simple Performance Tests (SPT)

The volumetric procedure has been widely implemented with successful results. However, the method lacks a strength test to verify the suitability of the Superpave mixes. Research completed in NCHRP Project 9-19 has identified three candidate simple performance tests (SPT) for HMA (Witczak et al. 2002). Tests based on measurement of dynamic modulus (for both of permanent deformation and fatigue cracking), flow time (permanent deformation), and flow number (permanent deformation) were selected for further field validation. The three tests used to obtain these parameters are the dynamic modulus test, triaxial static creep test, and triaxial repeated load permanent deformation test. All these tests use cylindrical specimens 100 mm (4 in.) in diameter and 150 mm (6 in.) high. Specimens are cored in the lab from specimens compacted using the Superpave gyratory compactor (Figure 9.23). Figure 9.28 shows a photo of the SPT.

Dynamic Modulus Test The dynamic modulus test in triaxial compression has been around the pavement community for many years (ASTM D3497). The test consists of applying an axial sinusoidal compressive stress to an unconfined or confined HMA cylindrical test specimen, as shown in Figure 9.29.

Assuming that HMA is a linear viscoelastic material, a complex number called the complex modulus, E^*, can be obtained from the test to define the relationship between stress and strain. The absolute value of the complex modulus, $|E^*|$, is called the dynamic modulus. The dynamic modulus is mathematically defined as the peak dynamic stress σ_o divided by the peak recoverable axial strain ε_o:

$$|E^*| = \frac{\sigma_o}{\varepsilon_0} \tag{9.20}$$

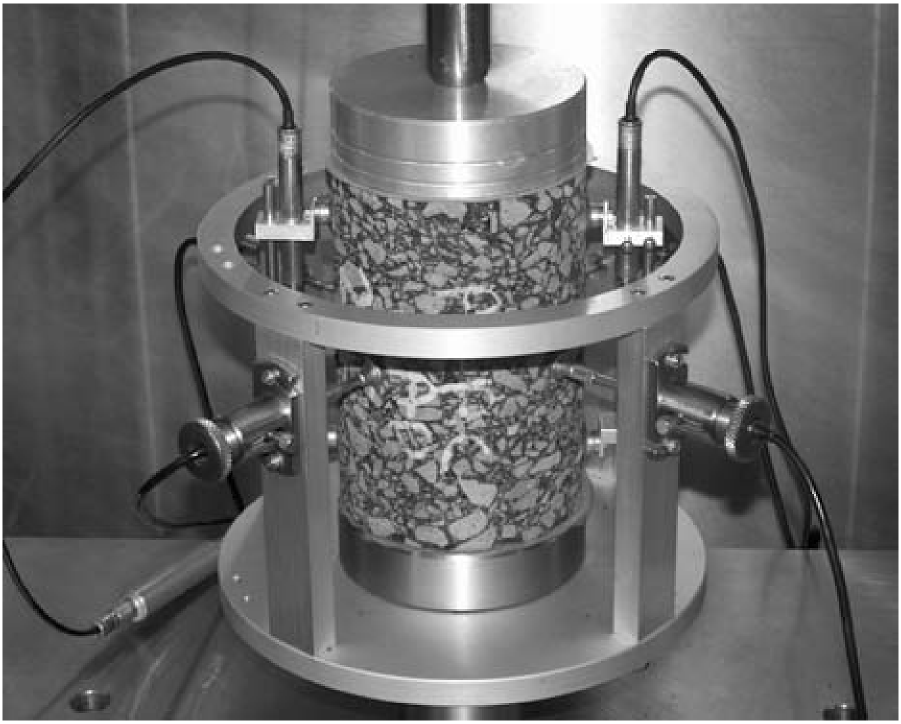

FIGURE 9.28 Superpave simple performance test assembly.

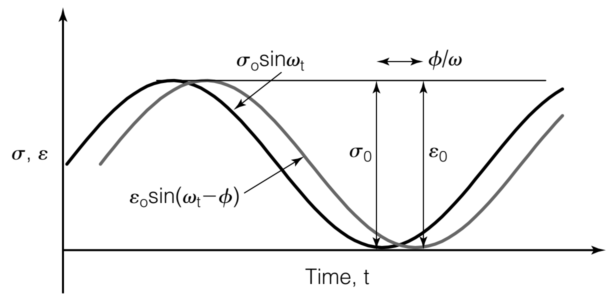

FIGURE 9.29 Stress and strain pulses for the dynamic modulus test.

The real and imaginary portions of the complex modulus E^* can be written as

$$E^* = E' + iE'' \tag{9.21}$$

E' is generally referred to as the storage or elastic modulus component of the complex modulus and E'' is referred to as the loss or viscous modulus. The phase angle ϕ is the angle by which ε_o lags behind σ_o. It is an indicator of the viscous properties of the material being evaluated. Mathematically, this is expressed as

$$E^* = |E^*| \cos \phi + i|E^*| \sin \phi \tag{9.22}$$

where

$$\phi = \frac{t_i}{t_p} \times 360$$

t_i = time lag between of stress and strain, s
t_p = time for a stress cycle, s
i = imaginary number

For a pure elastic material, $\phi = 0$, and the complex modulus E^* is equal to the absolute value, or the dynamic modulus. For a pure viscous material, $\phi = 90°$.

The dynamic modulus obtained from this test is indicative of the stiffness of the asphalt mixture at the selected temperature and load frequency. The dynamic modulus is correlated to both rutting and fatigue cracking of HMA.

Triaxial Static Creep Test In the static compressive creep test, a total strain–time relationship for a mixture is measured in the laboratory under unconfined or confined conditions. The static creep test, using either one load–unload cycle or incremental load–unload cycles, provides sufficient information to determine the instantaneous elastic (recoverable) and plastic (irrecoverable) components (time independent), and the viscoelastic and viscoplastic components (time dependent) of the material's response. In this test, the compliance, *D(t)* is determined by dividing the strain as a function of time $\varepsilon(t)$, by the applied constant stress σ_o:

$$D(t) = \frac{\varepsilon(t)}{\sigma_o} \tag{9.23}$$

Figure 9.30 shows typical test results between the calculated compliance and loading time. As shown, the compliance can be divided into three

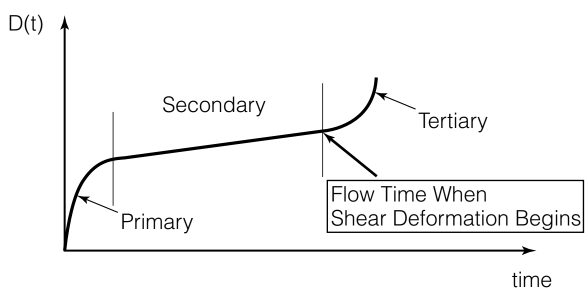

FIGURE 9.30 Compliance versus loading time for the static triaxial creep test.

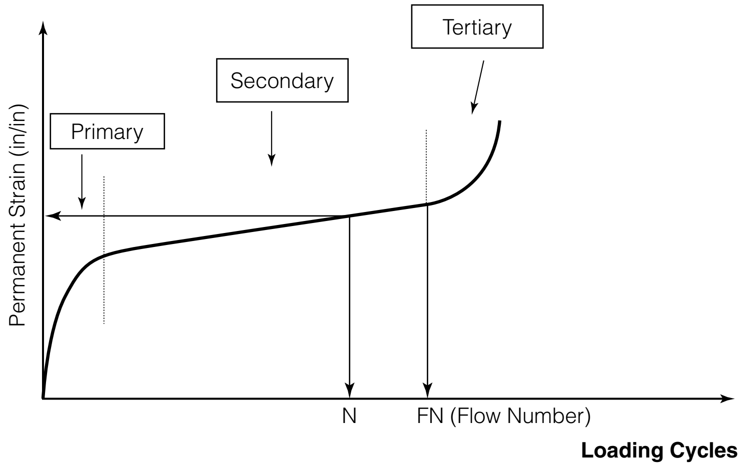

FIGURE 9.31 Typical relationship between total cumulative plastic strain and loading cycles.

major zones: primary zone, secondary zone, and tertiary flow zone. The time at which tertiary flow starts is referred to as the *Flow Time*. The flow time is a significant parameter in evaluating the rutting resistance of HMA.

Triaxial Repeated Load Permanent Deformation Test Another approach to measuring the permanent deformation characteristics of HMA is to use a repeated load test for several thousand repetitions and to record the cumulative permanent deformation as a function of the number of load repetitions. In this test, a haversine pulse load consisting of a 0.1 second and 0.9 second dwell (rest) time is applied for the test duration, typically about three hours or 10,000 loading cycles.

Results from the repeated load tests typically are presented in terms of the cumulative permanent strain versus the number of loading cycles. Figure 9.31 illustrates a typical relationship between the cumulative plastic strain and number of load cycles. In a manner similar to the triaxial static creep test, the cumulative permanent strain curve can be divided into three zones: primary, secondary, and tertiary. The cycle number at which tertiary flow starts is referred to as the *Flow Number*. In addition to the flow number, the test can provide the resilient strain and modulus, all of which are correlated to the rutting resistance of HMA.

9.9.5 Marshall Method of Mix Design

The basic steps required for performing Marshall mix design are as follows (The Asphalt Institute 1995):

1. aggregate evaluation
2. asphalt cement evaluation

3. specimen preparation
4. Marshall stability and flow measurement
5. density and voids analysis
6. design asphalt content determination

1. **Aggregate Evaluation** The aggregate characteristics that must be evaluated before it can be used for an asphalt concrete mix include the durability, soundness, presence of deleterious substances, polishing, shape, and texture. Agency specifications define the allowable ranges for aggregate gradation. The Marshall method is applicable to densely graded aggregates with a maximum size of not more than 25 mm (1 in.).
2. **Asphalt Cement Evaluation** The grade of asphalt cement is selected based on the expected temperature range and traffic conditions. Most highway agencies have specifications that prescribe the grade of asphalt for the design conditions.
3. **Specimen Preparation** The full Marshall mix-design procedure requires 18 specimens 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high. The stability and flow are measured for 15 specimens. In addition, 3 specimens are used to determine the theoretical maximum specific gravity G_{mm}. This value is needed for the void and density analysis. The specimens for the theoretical maximum specific gravity determination are prepared at the estimated design asphalt content. Samples are also required for each of five different asphalt contents; the expected design asphalt content, ±0.5% and ±1.0%. Engineers use experience and judgment to estimate the design asphalt content.

   Specimen preparation for the Marshall method uses the Marshall compactor discussed earlier (Figure 9.24). The Marshall method requires

FIGURE 9.32 Marshall stability machine.

mixing of the asphalt and aggregates at a temperature where the kinematic viscosity of the asphalt cement is 170 ± 20 cSt and compacting temperature corresponds to a viscosity of 280 ± 30 cSt.

The Asphalt Institute permits three different levels of energy to be used for the preparation of the specimens: 35, 50, and 75 blows on each side of the sample. Most mix designs for heavy-duty pavements use 75 blows, since this better simulates the required density for pavement construction.

4. **Marshall Stability and Flow Measurement** The Marshall stability of the asphalt concrete is the maximum load the material can carry when tested in the Marshall apparatus, Figure 9.32. The test is performed at a deformation rate of 51 mm/min. (2 in./min.) and a temperature of 60°C (140° F). The Marshall flow is the deformation of the specimen when the load starts to decrease. Stability is reported in newtons (pounds) and flow is reported in units of 0.25 mm (0.01 in.) of deformation. The stability of specimens that are not 63.5 mm thick is adjusted by multiplying by the factors shown in Table 9.11. All specimens are tested and the average stability and flow are determined for each asphalt content.
5. **Density and Voids Analysis** The values of VTM, VMA, and VFA are determined as using Equations 9.8, 9.9, and 9.10, respectively.
6. **Design Asphalt Content Determination** Traditionally, test results and calculations are tabulated and graphed to help determine the factors that must be used in choosing the optimum asphalt content. Table 9.12 presents examples of mix design measurements and calculations. Figure 9.33 shows plots of results obtained from Table 9.12, which include asphalt content versus air voids, VMA, VFA, unit weight, Marshall stability, and Marshall flow.

TABLE 9.11 Marshall Stability Adjustment Factors

Approximate Thickness of Specimen, mm (in.)	Adjustment Factor	Approximate Thickness of Specimen, mm (in.)	Adjustment Factor
50.8 (2)	1.47	65.1 (2 9/16)	0.96
52.4 (2 1/16)	1.39	66.7 (2 5/8)	0.93
54.0 (2 1/8)	1.32	68.3 (2 11/16)	0.89
55.6 (2 3/16)	1.25	69.8 (2 3/4)	0.86
57.2 (2 1/4)	1.19	71.4 (2 13/16)	0.83
58.7 (2 5/16)	1.14	73.0 (2 7/8)	0.81
60.3 (2 3/8)	1.09	74.6 (2 15/16)	0.78
61.9 (2 7/16)	1.04	76.2 (3)	0.76
63.5 (2 1/2)	1.00		

TABLE 9.12 Examples of Mix Design Measurements and Calculations by the Marshall Method (The Asphalt Institute, 1995)

% AC by Wt. of Mix, Spec. No.	Spec. Height, mm	Wt. in Air, g	Wt. in Water, g	SSD Wt., g	Bulk Vol., cm^3	Bulk Sp. Gr.	Max. Theo. Sp. Gr. (loose mix)	% Air Voids	% VMA	% VFA	Measured Stability, kN	Adjusted Stability, kN	Flow, 0.25 mm
3.5–A		1240.6	726.4	1246.3	519.9	2.386					10.9	10.9	8
3.5–B		1238.7	723.3	1242.6	519.3	2.385					10.8	10.8	7
3.5–C		1240.1	724.1	1245.9	521.8	2.377					11.2	11.2	7
Average						2.383	2.570	7.3	14.0	48.0		10.9	7
4.0–A		1244.3	727.2	1246.6	519.4	2.396					9.7	9.7	9
4.0–B		1244.6	727.0	1247.6	520.6	2.391					10.1	10.1	9
4.0–C		1242.6	727.9	1244.0	516.1	2.408					10.3	10.3	8
Average						2.398	2.550	6.0	13.9	57.1		10.0	9
4.5–A		1249.3	735.8	1250.2	414.4	2.429					10.8	10.8	9
4.5–B		1250.8	728.1	1251.6	523.5	2.389					10.7	10.3	9
4.5–C		1251.6	735.3	1253.1	517.8	2.417					10.4	10.4	9
Average						2.412	2.531	4.7	13.9	66.1		10.5	9
5.0–A		1256.7	739.8	1257.6	517.8	2.427					10.2	10.2	9
5.0–B		1258.7	742.7	1259.3	516.6	2.437					9.7	9.7	8
5.0–C		1258.4	737.5	1259.1	521.6	2.413					10.0	10.0	9
Average						2.425	2.511	3.4	13.8	75.2		10.0	9
5.5–A		1263.8	742.6	1264.3	521.7	2.422					9.8	9.8	9
5.5–B		1258.8	741.4	1259.4	518.0	2.430					10.2	10.2	10
5.5–C		1259.0	742.5	1259.5	517.0	2.435					9.8	10.0	9
Average						2.429	2.493	2.5	14.1	82.1		10.0	9

Notes: AC-20 binder, $G_b = 1.030$, $G_{sb} = 2.674$, Absorbed AC of aggregate: 0.6%, $G_{se} = 2.717$, Compaction: 75 blows

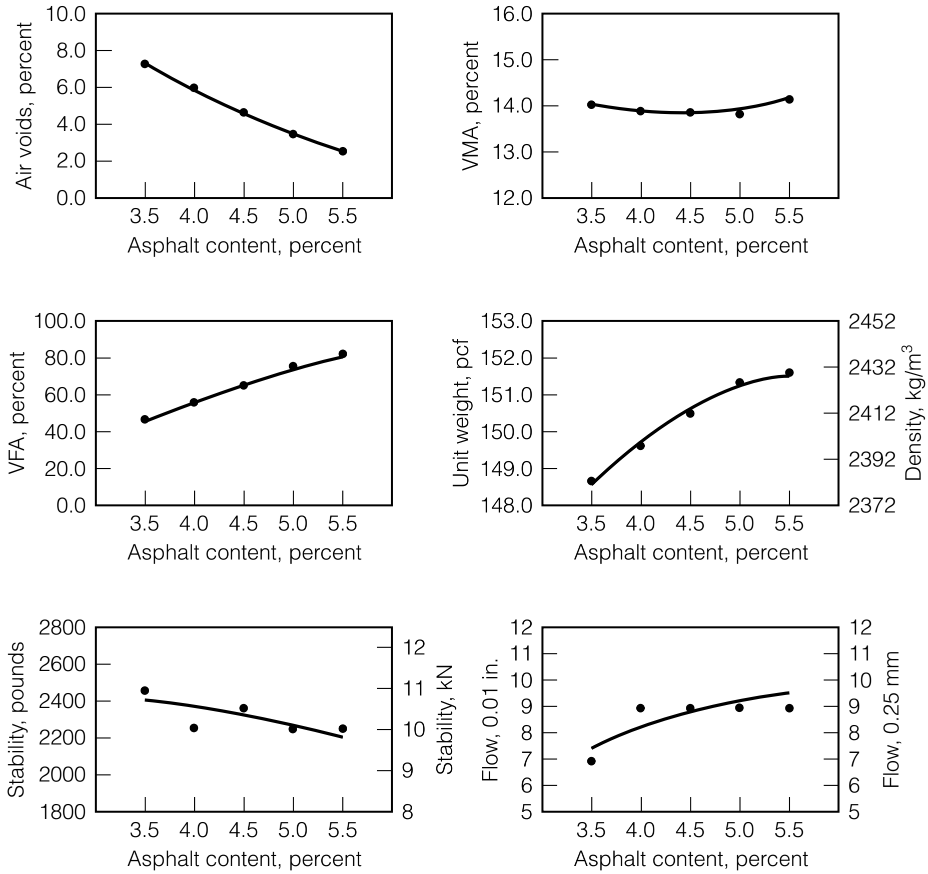

FIGURE 9.33 Graphs used for Marshall mix design analysis. (See Table 9.12) (The Asphalt Institute 1995).

The design asphalt content is usually the most economical one that will satisfactorily meet all of the established criteria. Different criteria are used by different agencies. Table 9.13 and 9.14 depict the mix design criteria recommended by The Asphalt Institute. Figure 9.34 shows an example of the narrow range of acceptable asphalt contents. The asphalt content selection can be adjusted within this narrow range to achieve a mix that satisfies the requirements of a specific project. Other agencies, such as the National Asphalt Paving Association, use the asphalt cement content at 4% air voids as the design value, and then check that the other factors meet the criteria. If the Marshall stability, Marshall flow, VMA, or VFA fall outside the allowable range, the mix must be redesigned using an adjusted aggregate gradation or new material sources.

The laboratory-developed mixture design forms the basis for the initial job mix formula (JMF). The initial JMF should be adjusted to reflect the slight differences between the laboratory-supplied aggregates and those used in the field.

TABLE 9.13 Asphalt Institute Criteria for Marshall Mix Design (The Asphalt Institute, 1995)

	Traffic Level					
	Light		Medium		Heavy	
Compaction (blows)	35		50		75	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
Stability, kN	3.34	—	5.34	—	8.01	—
Flow, 0.25 mm	8	18	8	16	8	14
Air Voids, %	3	5	3	5	3	5
VMA, %	Use the criteria in Table 9.14					
VFA, %	70	80	65	78	65	75

TABLE 9.14 Minimum Percent Voids in Mineral Aggregate (VMA) (The Asphalt Institute, 1995)

	Minimum VMA, Percent		
Nominal Maximum	Design Air Voids[2]		
Particle Size[1]	3.0	4.0	5.0
2.36 mm (No. 8)	19.0	20.0	21.0
4.75 mm (No. 4)	16.0	17.0	18.0
9.5 mm (3/8 in.)	14.0	15.0	16.0
12.5 mm (1/2 in.)	13.0	14.0	15.0
19.0 mm (3/4 in.)	12.0	13.0	14.0
25.0 mm (1.0 in.)	11.0	12.0	13.0

[1]The nominal maximum particle size is one size larger than the first sieve to retain more than 10 percent.

[2]Interpolate minimum VMA for design air void values between those listed.

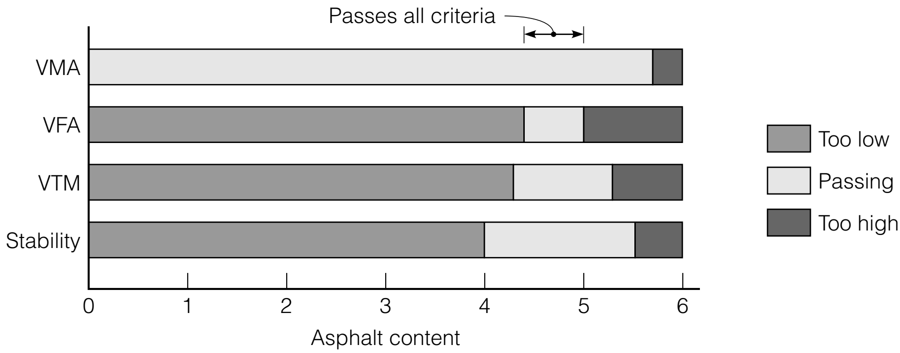

FIGURE 9.34 An example of the narrow range of acceptable asphalt contents. (The Asphalt Institute 1995).

Sample Problem 9.6

The Marshall method was used to design an asphalt concrete mixture. An AC-30 asphalt cement with a specific gravity (G_b) of 1.031 was used. The mixture contains a 9.5 mm nominal maximum particle size aggregate with a bulk specific gravity (G_{sb}) of 2.696. The theoretical maximum specific gravity of the mix (G_{mm}) at asphalt content of 5.0% is 2.470. Trial mixes were made with average results as shown in the following table:

Asphalt Content (P_b) (% by Weight of Mix)	Bulk Specific Gravity (G_{mb})	Corrected Stability (kN)	Flow (0.25 mm)
4.0	2.360	6.3	9
4.5	2.378	6.7	10
5.0	2.395	5.4	12
5.5	2.405	5.1	15
6.0	2.415	4.7	22

Determine the design asphalt content using the Asphalt Institute design criteria for medium traffic. (Table 9.13). Assume a design air void content of 4% when using Table 9.14.

Solution

Analysis steps:

1. Determine the effective specific gravity of the aggregates G_{se}, using Equation 9.6:

$$G_{se} = \frac{P_s}{\left(\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}\right)} = \frac{100 - 5.0}{\left(\dfrac{100}{2.470} - \dfrac{5.0}{1.031}\right)} = 2.666$$

The calculations in steps 2–5 are for 4.0% asphalt content as an example. Repeat for other asphalt contents.

2. Use G_{se} to determine G_{mm} for the other asphalt contents, using Equation 9.5:

$$G_{mm} = \frac{100}{\left(\frac{P_s}{G_{se}} + \frac{P_b}{G_b}\right)} = \frac{100}{\left(\frac{100-4.0}{2.666} + \frac{4.0}{1.031}\right)} = 2.507$$

3. Compute voids in the total mix for each asphalt content, using Equation 9.8:

$$VTM = 100\left(1 - {}^{G_{mb}}/_{G_{mm}}\right) = 100\left(1 - {}^{2.360}/_{2.507}\right) = 5.9$$

4. Compute voids in mineral aggregate, using Equation 9.9:

$$VMA = 100 - \left({}^{G_{mb}P_s}/_{G_{mm}}\right) = 100 - \left(2.360 \times {}^{(100-4.0)}/_{2.696}\right) = 16.0$$

5. Compute voids filled with asphalt, using Equation 9.10:

$$VFA = 100\frac{(VMA - VTM)}{VMA} = 100\frac{(16.0 - 5.9)}{16.0} = 63.3$$

6. A summary of all calculations is given in the following table:

P_b (%)	G_{mb}	Corrected Stability (kN)	Flow, (0.25 mm)	G_{mm}	G_{se}	VTM (%)	VMA (%)	VFA (%)
4.0	2.360	6.3	9	2.507		5.9	16.0	63.3
4.5	2.378	6.7	10	2.488		4.4	15.8	71.9
5.0	2.395	5.4	12	2.470	2.666	3.0	15.6	80.5
5.5	2.405	5.1	15	2.452		1.9	15.7	87.8
6.0	2.415	4.7	22	2.434		0.8	15.8	95.0

7. Plot stability, flow and volumetric parameters versus P_b. (See Figure SP9.6.)
8. Determine the asphalt content that corresponds to VTM = 4% and the corresponding parameters.
   Compare with criteria:

	From Graphs	Criteria
P_b @ 4%	4.6	
Stability (kN)	6.6	5.34 (min)
Flow (0.25 mm)	10.5	8 to 16
G_{mb}	2.383	NA
VMA (%)	15.7	15.0 (min)
VFA (%)	75	65 to 78

Therefore, the design asphalt content is 4.6%.

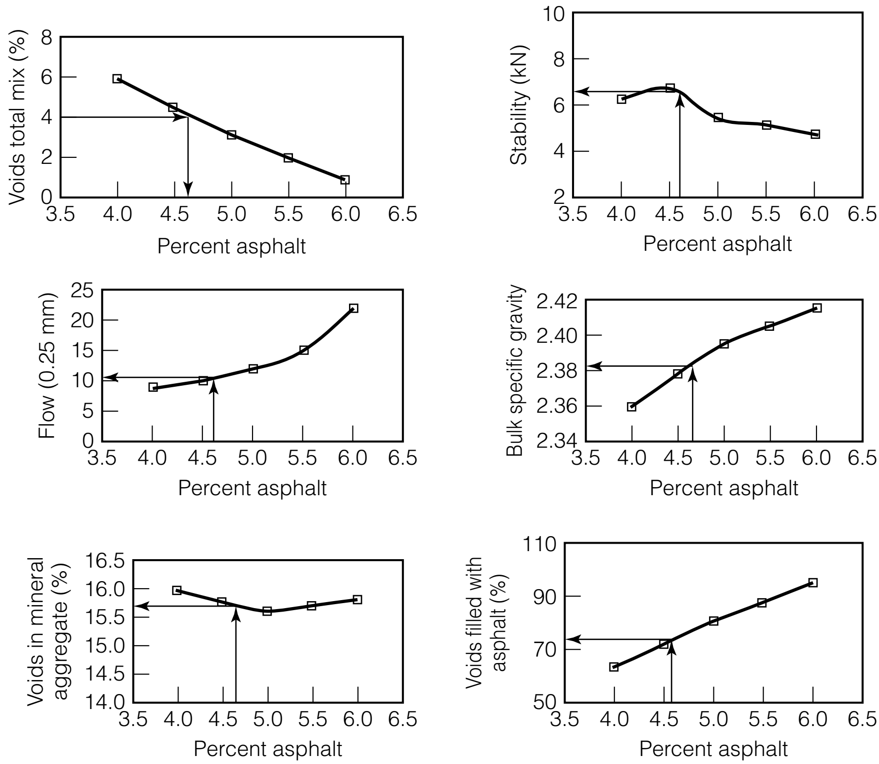

FIGURE SP9.6

9.9.6 Hveem Method of Mix Design

The basic steps required for performing Hveem mix design are the following (The Asphalt Institute 1995):

1. aggregate evaluation
2. asphalt cement evaluation
3. evaluation of centrifuge kerosene equivalent of fine aggregate
4. evaluation of surface capacity of coarse aggregate
5. estimation of optimum asphalt content
6. specimen preparation
7. measurement of the Hveem stability
8. density and voids analysis
9. determination of design asphalt content.

The evaluation of aggregate and asphalt cement is performed as in the Marshall method of mix design. The Hveem method requires measuring aggregate properties and using a series of charts to estimate the design asphalt content (The Asphalt Institute 1995).

Three cylindrical specimens 102 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high are prepared, using the California kneading compactor (Figure 9.25) according to ASTM D1561. Three asphalt contents near the estimated design value are used to fabricate the specimens. The Hveem stability of the specimens is determined using the Hveem stabilometer (Figure 9.35), according to ASTM D1560. The Hveem stabilometer is a device that allows for the application of a lateral pressure on the specimen while applying vertical load using a compression machine.

As in the Marshall method, the bulk specific gravity, theoretical maximum specific gravity, percent air voids (VTM), and density of all specimens are determined. The Hveem stability, density, and air voids are tabulated and plotted versus asphalt content. The optimum asphalt content for the design mix should be the highest asphalt content the mix will accommodate without reducing the stability or void content below the minimum values required by the design criteria.

The laboratory-developed mixture design forms the basis for the initial JMF. The initial JMF should be adjusted to consider the slight differences between the laboratory-supplied aggregates and those used in the field.

9.9.7 Evaluation of Moisture Susceptibility

Since loss of bond between asphalt and aggregates (*stripping*) has become a significant form of asphalt pavement distress, several methods have been developed for evaluating the susceptibility of a mix to water damage. Most of the popular methods require the specimens to be at the optimum asphalt content and mix gradation.

FIGURE 9.35 Hveem stabilometer.

The specimens are divided into two lots: reference specimens and conditioned specimens. A strength test is used to evaluate the strength before and after conditioning; the retained strength, the ratio of conditioned strength to reference strength, expressed in percent, is computed. Criteria are used to determine if the retained strength is adequate. The different techniques for evaluating moisture susceptibility vary, depending on the specimen preparation, conditioning procedures, and strength.

The immersion–compression test (ASTM D1075) has been used to evaluate moisture susceptibility. The method evaluates the retained compressive strength after vacuum saturation. Other methods use Marshall specimens, freezing and water soaking to condition the samples, and determining diametral strength and modulus values to evaluate the retained strength. Freezing the samples greatly increases the severity of the test.

There are several ways to alter asphalt concrete's susceptibility to water damage. Methods identified by the Asphalt Institute include the following:

1. increasing asphalt content
2. using a higher viscosity asphalt cement
3. cleaning aggregate of any dust and clay
4. adding antistripping additives
5. altering aggregate gradation

In addition, portland cement and lime have been used by some agencies as antistripping agents. Generally, when water damage susceptibility is a problem, the additive is added to the mix at three levels, and the water damage test is performed to determine the minimum amount of additive that can be used to increase the retained strength to an acceptable level. If an acceptable mix can be developed, Marshall or Hveem specimens are prepared, and the mix is tested to determine whether it meets the design criteria.

9.10 Characterization of Asphalt Concrete

Tests used to characterize asphalt concrete are somewhat different from those used to characterize other civil engineering materials, such as steel, portland cement concrete, and wood. One of the main reasons for this difference is that asphalt concrete is a nonlinear viscoelastic or viscoelastoplastic material. Thus, its response to loading is greatly affected by the rate of loading and temperature. Also, asphalt pavements are typically subjected to dynamic loads applied by traffic. Moreover, asphalt pavements do not normally fail due to sudden collapse under the effect of vehicular loads, but due to accumulation of permanent deformation in the wheel path (rutting), cracking due to repeated bending of the asphalt concrete layer (fatigue cracking), thermal cracking, excessive roughness of the pavement surface, migration of asphalt binder at the pavement surface (bleeding or flushing), loss of flexibility of asphalt binder due to aging and oxidation (raveling), loss of bond between the asphalt binder and aggregate particles due to moisture (stripping), or other factors. Therefore, most of the tests used to characterize asphalt concrete try to simulate actual field conditions.

Many laboratory tests have been used to evaluate asphalt concrete properties and to predict its performance in the field. These tests are performed on either laboratory-prepared specimens or cores taken from in-service pavements. These tests measure the response of the material to load, deformation, or environmental conditions, such as temperature, moisture, or freeze and thaw cycles. Some of these tests are based on empirical relations, while others evaluate fundamental properties. All tests on asphalt concrete are performed at accurately controlled test temperatures and rates of loading, since asphalt response is largely affected by these two parameters.

The Superpave tests used for mix design, as well as Marshall or Hveem tests discussed earlier, have been used to characterize asphalt concrete mixtures. Other tests are also being used, some of which are standardized by ASTM or AASHTO, while others have been used mostly for research. The next several sections discuss some of the common tests.

9.10.1 Indirect Tensile Strength

When traffic loads are applied on the pavement surface, tension is developed at the bottom of the asphalt concrete layer. Therefore, it is important to evaluate the tensile strength of asphalt concrete for the design of the layer thickness. In this test, a cylindrical specimen 102 mm (4 in.) in diameter and 64 mm (2.5 in.) high is typically used. A compressive vertical load is applied along the vertical diameter, using a loading device similar to that shown in Figure 9.36. The load is applied by using two curved loading strips moving with a rate of deformation of 51 mm/min. (2 in./min.). Tensile stresses are developed in the horizontal direction, and when these stresses reach the tensile strength, the specimen fails in tension along the vertical diameter. The test is performed at a specified temperature. With 12.5 mm (0.5 in.) loading strips, the indirect tensile strength is computed as

$$\sigma_t = \frac{2P}{\pi t D} \tag{9.24}$$

where

σ_t = tensile strength, MPa (psi)
P = load at failure, N (lb)
t = thickness of specimen, mm (in.)
D = diameter of specimen, mm (in.)

9.10.2 Diametral Tensile Resilient Modulus

To evaluate the structural response of the asphalt pavement system, the modulus of asphalt concrete material is needed. Since asphalt concrete is not a linear viscoelastic material, the modulus of elasticity, Young's modulus, is not applicable. The diametral tensile resilient modulus test (ASTM

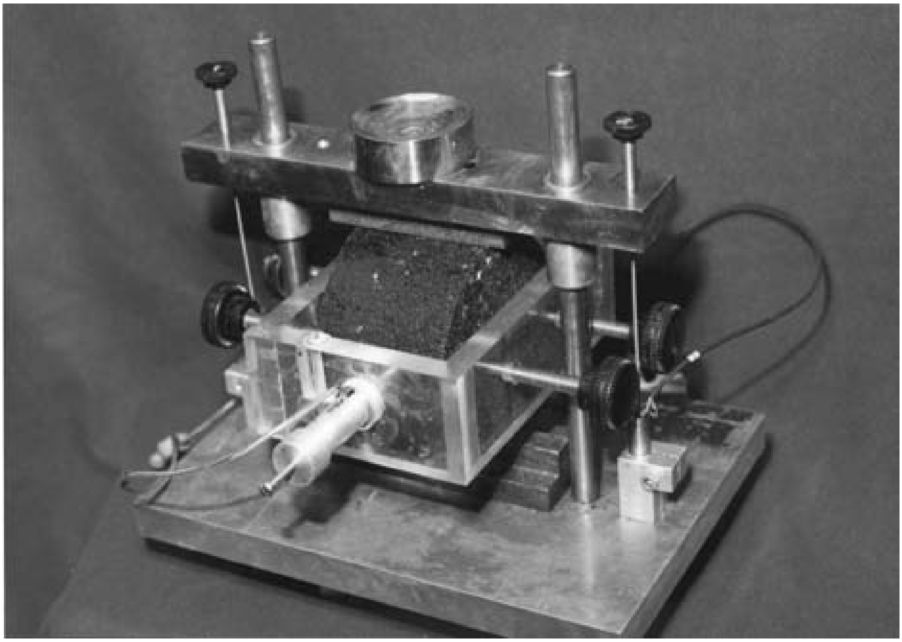

FIGURE 9.36 Diametral loading for indirect tensile strength and resilient modulus tests.

D4123) provides an analogous modulus, known as the resilient modulus. The test uses a cylindrical specimen 102 mm (4 in.) in diameter and 63.5 mm (2.5 in.) high. A pulsating load is applied along the vertical diameter, using a load guide device similar to that shown in Figure 9.36. The load is commonly applied with a duration of 0.1 second and a rest period of 0.9 second. After a few hundred repetitions, the recoverable horizontal deformation is measured using two linear variable differential transducers (LVDTs). Figure 9.37 shows typical load and horizontal deformation versus time relationships. Since the test is nondestructive, the test is repeated on the same specimen after rotating it 90°. The test is commonly performed at three temperatures: 5°C, 25°C, and 40°C (41°F, 77°F, and 104°F). The diametral tensile resilient modulus is computed as

$$M_R = \frac{P(0.27 + \nu)}{t.\Delta H} \tag{9.25}$$

where

M_R = indirect tensile resilient modulus, MPa (psi)
P = repeated load, N (lb)
ν = Poisson's ratio, typically 0.3, 0.35, and 0.4 at temperatures of 5°C, 25°C, and 40°C, respectively
t = thickness of specimen, mm (in.)
ΔH = sum of recoverable horizontal deformations on both sides of specimen, mm (in.)

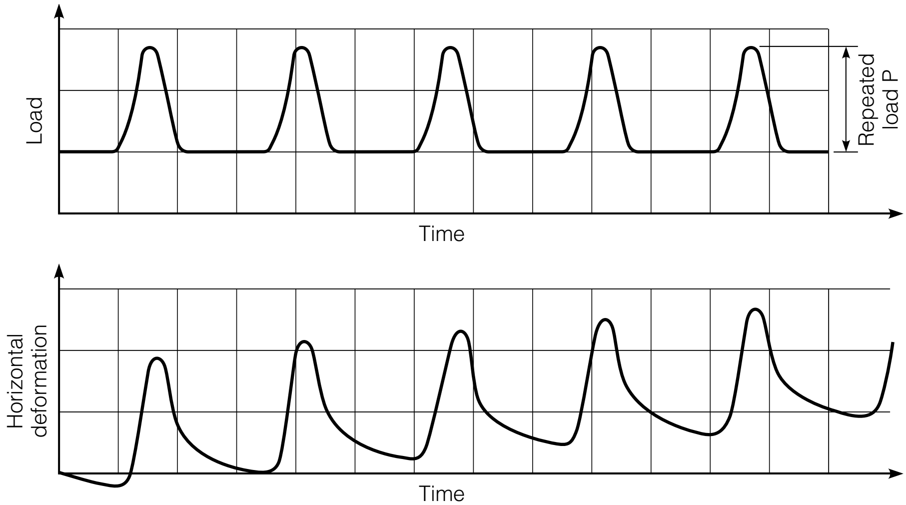

FIGURE 9.37 Typical load and horizontal deformation versus time during the resilient modulus test.

Typical resilient modulus values of asphalt concrete are 6.89 GPa, 4.13 GPa, and 1.38 GPa (1000 ksi, 600 ksi, and 200 ksi) at temperatures of 5°C, 25°C, and 40°C, respectively. The diametral tensile resilient modulus test is very sophisticated, because it measures very small deformations. Therefore, extreme caution must be exercised to align the specimen between the loading trips and to reduce possible rocking. Also, the load magnitude must be small enough to reduce the possibility of permanent deformation in the specimen, yet large enough to obtain measurable deformation.

Sample Problem 9.7

The resilient modulus test was performed on an asphalt concrete specimen and the following data were obtained:

diameter = 4.000 in.
thickness = 2.523 in.
repeated load = 559 lb
sum of recoverable horizontal deformations = 254×10^{-6} in.

Assuming a Poisson's ratio of 0.35, calculate the resilient modulus.

Solution

$$M_R = \frac{P(0.27 + \upsilon)}{t.\Delta H} = 559 \times (0.27 + 0.35)/(2.523 \times 254 \times 10^{-6})$$
$$= 541{,}000 \text{ psi}$$

9.10.3 Freeze and Thaw Test

The freeze and thaw test is performed to evaluate the effect of freeze and thaw cycles on the stiffness properties of asphalt concrete. Cylindrical specimens 102 mm (4 in.) in diameter and 64 mm (2.5 in.) high are used. Three specimens are tested for resilient modulus as discussed earlier, while the other three specimens are subjected to cycles of freeze and thaw, after which the resilient modulus is determined. The tensile strength ratio is computed by dividing the average resilient modulus of conditioned specimens by the average resilient modulus of unconditioned specimens, expressed in percent. A minimum tensile strength ratio is usually required to identify mixes that are not severely affected by freeze and thaw cycles.

9.10.4 Use of Rheological Models to Analyze Time-Dependent Response

Asphalt concrete is a viscoelastic material exhibiting a time-dependent response under load. Rheological models consisting of combinations of Hookean (spring) and Newtonean (dashpots) elements have been used to analyze the response of time-dependent materials, as discussed in Chapter 1. The Burgers model illustrated in Figure 1.12 can closely approximate the response of asphaltic mixtures (Mamlouk 1984). Laboratory tests, such as the creep test, are used to obtain the parameters of the Burgers model using a curve-fitting procedure. Once these parameters are determined, the model can be used to predict the response of the material under different loading conditions. For example, Burgers model has been used to predict rutting of asphalt concrete pavement under the action of traffic loads.

9.11 Asphalt Concrete Production

Asphalt concrete is produced in either a batch plant or a continuous (drum) plant (The Asphalt Institute 1989). In the United States, batch plants were used extensively in the past; however, more energy efficient continuous plants are now preferred.

In continuous plants (Figure 9.38), aggregates of different gradations are placed in cold bins. The gradation proportions needed are taken from the cold bins by a cold feed elevator. Aggregates are transferred to the first part of the drum, where they are dried and heated. Hot asphalt cement is introduced in the last one-third of the drum; then aggregates and asphalt are mixed. Since asphalt concrete is produced continuously in this type of plant, it is transferred to a storage silo until placed in a truck and transported to the job site.

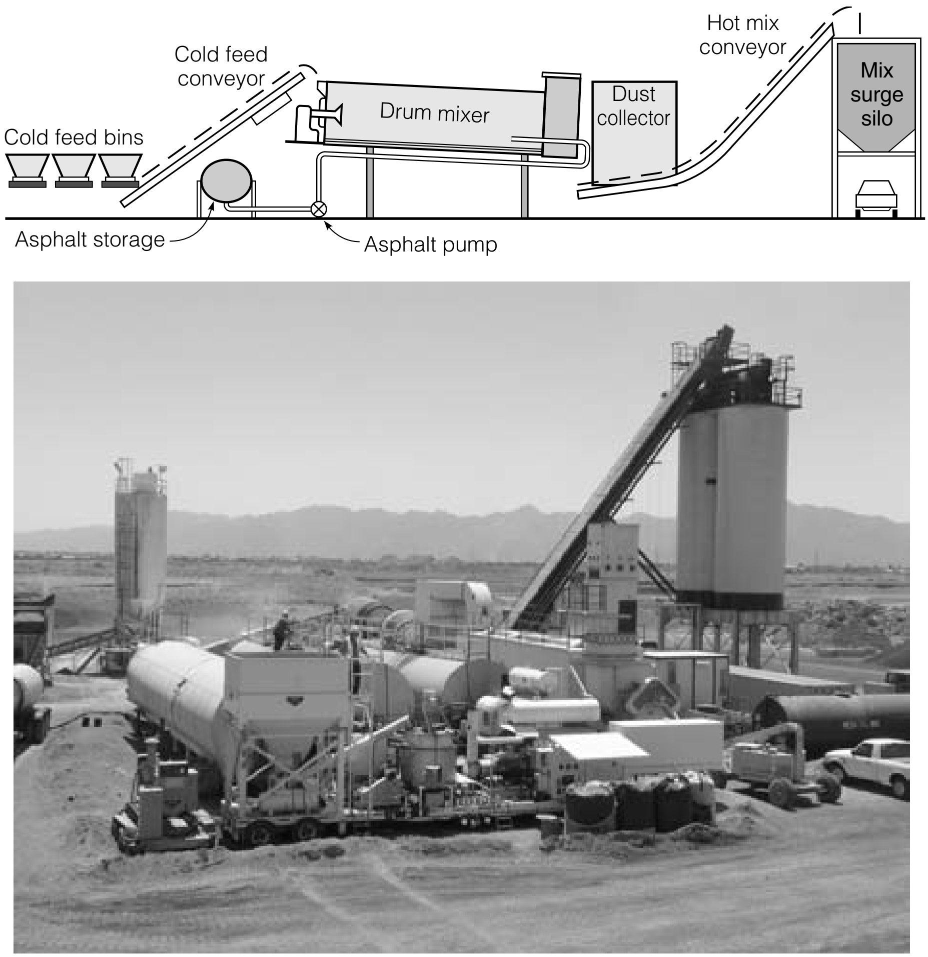

FIGURE 9.38 Continuous (drum) mix asphalt concrete plant.

9.12 Recycling of Asphalt Concrete

Recycling pavement materials has a long history. However, recycling became more important in the mid-1970s, after the oil embargo, due to the increase in asphalt prices. In an effort to efficiently use available resources, there was a need to recycle or reuse old pavement materials (The Asphalt Institute 1989). Although the pavement could be badly deteriorated, the old asphalt concrete materials could be successfully reused in new pavements. Currently, recycling of old pavement materials is becoming a normal practice due to the following advantages:

1. economic saving of about 25% of the price of materials
2. energy saving in manufacturing and transporting raw materials

3. environmental saving by reducing the amount of required new materials and by eliminating the problem of discarding old materials
4. eliminating the problem of reconstruction of utility structures, curbs, and gutters associated with overlays
5. reducing the dead load on bridges due to overlays
6. maintaining the tunnel clearance, compared with overlays

Recycling can be divided into three types: *surface recycling, central plant recycling,* and *in-place recycling.*

9.12.1 Surface Recycling

Surface recycling is defined as the reworking of the top 25 mm (1 in.) of the pavement surface using a heater–scarifier. The heater planing machine heats the pavement surface, which repairs minor cracks and roughness. Usually, a rejuvenating agent is added after heating, followed by slight scratching of the surface and compaction.

9.12.2 Central Plant Recycling

Central plant recycling is performed by milling the old pavement (Figure 9.39) and sending the reclaimed asphalt pavement (RAP) to a central asphalt concrete plant, where it is mixed with some form of rejuvenating agent or soft asphalt and aggregates to produce hot-mixed asphalt concrete. If the RAP materials are mixed with the aggregates in a conventional asphalt concrete plant, they will burn and produce smoke, causing significant

FIGURE 9.39 Milling old pavement.

environmental problems. Therefore, the RAP materials are added to the pugmill (mixer) in the batch plant or added at midlength in the drum at the drum mix plant. The amount of recycled materials varies from 20% to 70%. The gradation of new aggregates is selected to correct any deficiency in the gradation of recycled aggregate. Typically, the grade of new asphalt cement is soft so that, when it is mixed with the old, hard asphalt, an appropriate consistency will result. The mix design of asphalt concrete, including recycled materials, is usually performed using either the Marshall or Hveem procedure.

In addition to hot-mix central plant recycling, cold central plant recycling can use new emulsified or cutback asphalt. However, the cold process will not have the quality of the hot-mixed material.

9.12.3 In-Place Recycling

In-place recycling is performed by ripping and pulverizing the old pavement surface and adding new aggregate, water, and asphalt emulsion. The old and new materials are mixed together in place, graded, and compacted. The surface is left to cure and is then used as a surface layer for low-volume roads. The recycled layer can also be used as a stabilized base, covered by an asphalt concrete surface.

9.13 Additives

Many types of additives (modifiers) are used to improve the properties of asphalt or to add special properties to the asphalt concrete mixtures (Roberts et al. 1996). Laboratory tests are usually performed and field performance is observed in order to evaluate the effect of the additives and to justify their cost. The effects of using additives should be carefully evaluated; otherwise premature pavement failure might result. The recyclability of modified asphalt mixtures is still being evaluated.

9.13.1 Fillers

Several types of fillers, such as crushed fines, portland cement, lime, fly ash, and carbon black can be added to asphalt concrete. Fillers are used to satisfy gradation requirements of materials passing the 0.075 mm (No. 200) sieve, to increase stability, to improve bond between aggregates and asphalt, or to fill the voids and thus reduce the required asphalt.

9.13.2 Extenders

Extenders such as sulfur and lignin are used to reduce the asphalt requirements, thus reducing the cost.

9.13.3 Rubber

Rubber has been used in asphalt concrete mixture in the form of natural rubber, styrene–butadiene (SBR), styrene–butadiene–styrene (SBS), or recycled tire rubber. Rubber increases elasticity and stiffness of the mix and increases the bond between asphalt and aggregates. Scrap rubber tires can be added to the asphalt cement (wet method) or added as crumb rubber to the aggregates (dry method).

9.13.4 Plastics

Plastics have been used to improve certain properties of asphalt. Plastics used include polyethylene, polypropylene, ethyl–vinyl–acetate (EVA), and polyvinyl chloride (PVC). They increase the stiffness of the mix, thus reducing the rutting potential. Plastics also may reduce the temperature susceptibility of asphalt and improve its performance at low temperatures.

9.13.5 Antistripping Agents

Antistripping agents are used to improve the bond between asphalt cement and aggregates, especially for water susceptible mixtures. Lime is the most commonly used antistripping agent and can be added as a filler or a lime slurry and mixed with the aggregates. Portland cement can be used as an alternative to lime.

9.13.6 Others

Other additives, such as fibers, oxidants, antioxidants, and hydrocarbons, have been used to modify certain asphalt properties' tensile strength and stiffness.

SUMMARY

Asphalt produced from crude oil is a primary road-building material. The civil engineer is directly involved with the specification and requirements for both the asphalt cement binder and the asphalt concrete mixtures. There are several methods for grading asphalt cements. The current trend is toward the use of the performance-grading method, used in the Superpave process developed through the Strategic Highway Research Program. This grading method directly ties the binder properties to pavement-performance parameters. Similarly, the Superpave mix design method uses performance tests to evaluate the mixture characteristics relative to expected field performance. This method will continue to be refined by highway agencies. With the support of the Federal Highway Administration, there is a concerted effort being placed on replacing the traditional Marshall and Hveem mix design methods with the Superpave procedures.

QUESTIONS AND PROBLEMS

9.1 What is the difference between tar and asphalt cement?

9.2 Discuss the main uses of asphalt.

9.3 Define what is meant by temperature susceptibility of asphalt. Discuss the effect on the performance of asphalt concrete pavements. Are soft asphalts used in hot or cold climates?

9.4 Temperature has a large effect on the asphalt viscosity. On one graph, plot the relationship between asphalt viscosity (logarithmic) and temperature for two cases: (a) a low-temperature susceptible asphalt and (b) a high-temperature susceptible asphalt. Label all axes and relations.

9.5 Briefly discuss the chemical composition of asphalt.

9.6 What is the significance of each one of these tests:

a. flash point test
b. RTFO procedure
c. rotational viscometer test
d. dynamic shear rheometer test
e. penetration test.

9.7 Discuss the aging that occurs in asphalt cement during mixing with aggregates and in service. How can the different types of aging of asphalt cement be simulated in the laboratory?

9.8 Show how various Superpave tests used to characterize the asphalt binder are related to pavement performance.

9.9 Define the four methods used to grade asphalt binders. Which method is used in your state?

9.10 To select an asphalt binder for a specific location, the mean seven-day maximum pavement temperature is estimated at 61°C with a standard deviation of 1.5°C. The mean minimum pavement temperature is −8°C with standard deviation of 3.2°C. What grade asphalt (PG) is needed at 98% reliability.

9.11 As a materials engineer working for a highway department, what standard PG asphalt binder grade would you specify for each of the conditions shown in Table P9.11 (show all calculations and fill in the table)?

Table P9.11

	Seven-Day Pavement	Maximum Temperature, °C	Minimum Pavement Temperature, °C		Recommended PG Grade	
Case	Mean, °C	Std. Dev., °C	Mean, °C	Std. Dev., °C	50% Reliability	98% Reliability
1	46	1.5	−34	2		
2	56	2	−15	2.5		
3	66	2	8	1.5		

9.12 For the following temperature conditions, calculate the proper Superpave grade for both 50% and 98% reliabilities (show your calculation).

Seven-day maximum pavement temperature of 46°C with a standard deviation of 2°C.

Minimum pavement temperature of 19°C with a standard deviation of 4°C.

9.13 What are the differences between CRS-2 and SS-1 emulsions?

9.14 Discuss how asphalt emulsions work as a binder in asphalt mixtures.

9.15 What are the components of hot-mix asphalt? What is the function of each component in the mix?

9.16 What are the objectives of the asphalt concrete mix-design process?

9.17 Why is it important to have an optimum binder content in HMA? What would happen if a less-than-optimum binder content is used? What would happen if more than the optimum value is used? What is the typical range of binder content in HMA?

9.18 Explain why the strength of asphalt concrete is not necessarily the most important property of the material.

9.19 An asphalt concrete specimen has a mass in air of 1249.3 g, mass in water of 735.8 g, and saturated surface-dry mass of 1250.2 g. Calculate the bulk specific gravity of the specimen.

9.20 As part of mix design, a laboratory-compacted cylindrical asphalt specimen is weighed for determination of bulk specific gravity. The following numbers are obtained:

Dry Mass in Air = 1204.5 grams
Mass when submerged in water = 689.4 grams
Mass of Saturated Surface Dry (SSD) = 1211.3 grams

a. What is the bulk specific gravity of the compacted specimen (G_{mb})
b. If the maximum theoretical specific gravity of the specimen (G_{mm}) is 2.531, what would be the air void content of the specimen in percent?

9.21 For asphalt concrete, define
a. air voids
b. voids in the mineral aggregate
c. voids filled with asphalt

9.22 An aggregate blend is composed of 53% coarse aggregate by weight (Sp. Gr. 2.702), 43% fine aggregate (Sp. Gr. 2.621), and 4% filler (Sp. Gr. 2.779). The compacted specimen contains 6% asphalt binder (Sp. Gr. 1.052) by weight of total mix, and has a bulk density of 145.2 lb/ft^3. Ignoring absorption, compute the percent voids in total mix, percent voids in mineral aggregate, and the percent voids filled with asphalt.

9.23 An asphalt concrete mixture includes 94% aggregate by weight. The specific gravities of aggregate and asphalt are 2.7 and 1.0, respectively. If the bulk density of the mix is 145 pcf, what is the percent voids in the total mix?

9.24 After two years of traffic, cores were recovered from the roadway which has severe rutting and bleeding. The bulk specific gravity and the maximum theoretical specific gravity were measured on these cores and are as follows:

Bulk specific gravity = 2.498
Maximum theoretical specific gravity = 2.545

a. Calculate the air voids
b. If the design air void content was 4%, explain what effect the calculated air voids had on the rutting and bleeding noted.

9.25 An asphalt concrete specimen has the following properties:

Asphalt content = 5.3% by total weight of mix
Bulk specific gravity of the mix = 2.442
Theoretical maximum specific gravity = 2.535
Bulk specific gravity of aggregate = 2.703

Calculate the percents VTM, VMA, and VFA.

9.26 Briefly describe the volumetric mix design procedure of Superpave.

9.27 Based on the data shown in Table P9.27, select the blend for a Superpave design aggregate structure.

Table P9.27

	Blend		
Data	**1**	**2**	**3**
G_{mb}	2.451	2.465	2.467
G_{mm}	2.585	2.654	2.584
G_b	1.030	1.030	1.030
P_b	5.9	5.5	5.8
P_s	94.1	94.5	94.2
P_d	4.5	4.5	4.5
G_{sb}	2.657	2.667	2.705
H_{ini}	127	135	124
H_{des}	113	114	118

9.28 Based on the data in Table P9.28, determine the design binder content for a Superpave mix design for a 15 million ESAL.

Table P9.28

	Binder content (%)			
Data	**5.5**	**6.0**	**6.5**	**7.0**
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.579	2.558	2.538	2.518
G_b	1.025	1.025	1.025	1.025
P_s	94.5	94.0	93.5	93.0
P_d	4.5	4.5	4.5	4.5
G_{sb}	2.705	2.705	2.705	2.705
h_{ini}	129	131	131	128
h_{des}	112	113	116	115

9.29 Given the data in Table P9.29, select the blend and the design binder content for a Superpave design aggregate structure for a 5 million ESAL.

Table P9.29

	Blend		
Data	**1**	**2**	**3**
G_{mb}	2.457	2.441	2.477
G_{mm}	2.598	2.558	2.664
G_b	1.025	1.025	1.025
P_b	5.9	5.7	6.2
P_s	94.1	94.3	93.8
P_d	4.5	4.5	4.5
G_{sb}	2.692	2.688	2.665
H_{ini}	125	131	125
H_{des}	115	118	115

	Binder Content (%)			
Data	**5.4**	**5.9**	**6.4**	**6.9**
G_{mb}	2.351	2.441	2.455	2.469
G_{mm}	2.570	2.558	2.530	2.510
G_b	1.025	1.025	1.025	1.025
P_s	94.6	94.1	93.6	93.1
P_d	4.5	4.5	4.5	4.5
G_{sb}	2.688	2.688	2.688	2.688
h_{ini}	125	131	126	130
h_{des}	115	118	114	112

9.30 The Marshall method of mix design has been widely used by many highway agencies.
a. What are the steps of Marshall Mix Design?
b. What parameters are calculated?
c. Show the typical graphs that are plotted after tests are completed.
d. What is the purpose of plotting these graphs?

9.31 An asphalt concrete mixture is to be designed according to the Marshall procedure. An AC-20 asphalt cement with a specific gravity of 1.00 is to be used. A dense aggregate blend is to be used, with a maximum size of 19 mm and bulk specific gravity of 2.696. The theoretical maximum specific gravity of the mix is 2.470. Trial mixes were made, with the average results shown in Table P9.31. Using a spreadsheet program, plot the appropriate graphs necessary for the Marshall procedure, and select the optimum asphalt content using the Asphalt Institute design criteria for medium traffic. Assume a design air void content of 4% when using Table 9.14.

Table P9.31

Asphalt Content, % by Weight	Bulk Specific Gravity	Stability, N	Flow, 0.25 mm
4.0	2.303	7076	9
4.5	2.386	8411	10
5.0	2.412	7565	12
5.5	2.419	5963	15
6.0	2.421	4183	22

9.32 An asphalt concrete mixture is to be designed according to the Marshall procedure. An AC-30 asphalt cement with a specific gravity (G_b) of 1.00 is to be used. A dense aggregate blend is to be used, with a maximum aggregate size of 3/4 in. and a bulk specific gravity (G_{sb}) of 2.786. The theoretical maximum specific gravity of the mix (G_{mm}), at asphalt content of 5.0%, is 2.490. Trial mixes were made with average results as shown in Table P9.32. Using a spreadsheet program, plot the appropriate six graphs necessary for the Marshall procedure and select the optimum asphalt content, using the Asphalt Institute design criteria for medium traffic (see Table 9.13). Assume a design air void content of 4% when using Table 9.14.

Table P9.32

Asphalt Content (P_b) (% by Weight)	Bulk Specific Gravity (G_{mb})	Stability, lb	Flow, 0.01 in.
3.50	2.294	1600	8
4.00	2.396	1980	9
4.50	2.421	2130	11
5.00	2.416	1600	14
5.50	2.401	1280	20

9.33 The Marshall procedure was used to design an asphalt concrete mixture for a heavy-traffic road. Asphalt cement with a specific gravity of 1.025 is to be used. The mixture contains a 19 mm nominal maximum particle size aggregate, with bulk specific gravity of 2.654. The theoretical maximum specific gravity of the mix is 2.480 at 4.5% asphalt content. Trial mixes were made, with the average results shown in Table P9.33. Determine the optimum asphalt content using the Asphalt Institute design criteria. Assume a design air void content of 4% when using Table 9.14.

Table P9.33

Asphalt Content, % by Weight	Bulk Specific Gravity	Stability, kN	Flow, 0.25 mm
3.5	2.367	8.2	7.3
4.0	2.371	8.6	9.4
4.5	2.389	7.5	11.5
5.0	2.410	7.2	12.5
5.5	2.422	6.9	13.2

9.34 Describe the process for determining the stripping potential for an asphalt concrete mix.

9.35 Briefly discuss how the indirect tensile resilient modulus is determined in the lab.

9.36 The resilient modulus test was performed on an asphalt concrete specimen and the following data were obtained:

Diameter = 4.029 in.
Height = 2.497 in.
Repeated load = 559 lb
Sum of recoverable horizontal deformations = 254×10^{-6} in.

Assuming a Poisson's ratio of 0.35, calculate the resilient modulus.

9.37 State six advantages of recycling asphalt pavement materials. Why can we not mix the RAP materials with aggregates in a conventional hot-mix asphalt concrete plant? Show the proper ways of recycling the RAP materials in the two types of hot-mix asphalt plants.

9.38 State four different asphalt modifiers that can be added to asphalt or asphalt mixtures, and indicate the effect of each.

9.39 When is portland cement used in asphalt concrete?

9.40 During construction of the HMA layer of asphalt pavement, cores were taken at random after compaction to detect any construction problem and to ensue that the density is within the specification limits. The target value was set at 148.2 pcf and the upper and lower specification limits were set at 148.9 pcf and 147.5 pcf, respectively. The density data shown in Table P9.40 were collected.

Table P9.40

Core No.	Density (pcf)	Core No.	Density (pcf)
1	148.3	11	147.8
2	147.8	12	148.1
3	148.2	13	147.3
4	148.7	14	147.7
5	148.2	15	147.3
6	147.7	16	147.0
7	148.4	17	147.1
8	147.8	18	146.7
9	147.7	19	146.9
10	148.6	20	146.8

a. Using a spreadsheet program, create a control chart for these data showing the target value and the upper and lower specification limits. Are the density data within the specification limits?
b. Comment on any trend and possible reasons.

9.14 References

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