Chapter 6
Asphalt

The word "asphalt" is usually used in the US, while Europe uses the word "bitumen" more often. Asphalt is used mostly in pavement construction. It is also used as a sealing and waterproofing agent for roofs and foundations. Around 5000 years ago, ancient Babylonians built an asphalt block road. In the 1830s, the British started to build the tarmacadam, which is to pour asphalt on an aggregate layer. The US built its first asphalt pavement with natural asphalt in the 1870s and the first pavement with petroleum asphalt in 1913. In 1925, China built its first asphalt pavement with petroleum asphalt on the Zhongshan Road in Nanjing.

6.1 Classification and Production

6.1.1 Classification

According to its different origins, asphalt can be divided into two categories: asphalt and tar. Asphalt can either be found in natural deposits or refined from petroleum. Natural asphalt refers to the product of petroleum under natural conditions, and it is formed by long-term geophysical actions. Trinidad lake asphalt and rock asphalt are two typical natural asphalt. Petroleum asphalt is the sticky, black, and highly viscous liquid or semi-solid residue during the refining of petroleum by steaming and other processes. Tar is a by-product or distillation of coke production from coal. Tar was used to pave roads, but is now primarily used for waterproofing membranes. Asphalt is soluble in petroleum products while tar is insoluble in a petroleum solvent.

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6.1.2 Production

The quantity and quality of asphalt depend on the crude petroleum sources and the refining methods. Some crude sources produce little asphalt, while others have high asphalt contents. For nearly 1500 types of petroleum, those from the Middle East and South America produce good-quality asphalt. A petroleum refinery's job is to break crude oil hydrocarbon molecule chains down into slightly different groupings of molecules of molecular weight, through distillation with heat, chemical reactions, and changes in pressure. Different products including gas, gasoline, kerosene, diesel, lubricating oil, heavy gas oil, and asphalt are separated at different temperatures. Since asphalt is a less valuable product than other components of crude oil, refineries are set up to produce more valuable fuels such as gasoline, kerosene, and diesel oil, at the expense of asphalt production. The production methods of asphalt include:

1. Distillation

Crude oil is separated into gasoline, kerosene, diesel oil, and other components according to their different boiling points through the atmospheric distillation tower and vacuum distillation tower, and then the residue is obtained as asphalt. The one-stage and two-stage distilled asphalt fractions are referred to as straight-run asphalt.

2. Oxidation

The oxidation process involves the reaction and rearrangement of asphalt molecules in the presence of oxygen at elevated temperatures. In the oxidation tower, the straight-run asphalt is heated, the air is blown and sometimes the catalyst is added. The reactions include dehydrogenation, oxidation, and polycondensation. The low molecular weight hydrocarbons in the asphalt are transformed into high molecular weight hydrocarbons. This type of asphalt is mostly used for industrial applications such as roofing and pipe coatings, and low penetration grade pavement asphalt.

3. Semi-oxidation

The straight-run asphalt may be too soft and have high-temperature susceptibility. The over-oxidized asphalt may be too still and have poor low-temperature performance. Semi-oxidation is improved oxidation with lower oxidation temperature, longer time, and lower air volume to obtain the asphalt with better high- and low-temperature performance.

4. Solvent deasphalting

Solvent deasphalting is a pretreatment process for inferior residual oil. It uses heavy oil such as vacuum residue as raw materials, and extracts hydrocarbons such as propane and butane as solvents. The extract is deasphalted oil, which is usually used as a heavy lubricating oil raw material or a cracking raw material. The residue is the asphalt.

5. Blending

Asphalt with different viscosities or other properties can be blended to produce blended asphalt that meet the requirements. The proportions of asphalts with different viscosities can be determined through laboratory tests, calculation methods, or component adjustment methods according to the requirements.

6. Cutback

Cutback asphalt is produced by dissolving asphalt in a lighter molecular weight hydrocarbon solvent such as kerosene, gasoline. When the cutback is sprayed on pavements or mixed with aggregates, the solvent evaporates, leaving the asphalt residue as the binder. Depending on the evaporation rate of the solvent, we have slow, medium, and rapid curing cutbacks. In the past, cutbacks were widely used for pavement construction. However, the cost of expensive solvents increases, and cutbacks are hazardous materials due to the volatility of the solvents. Therefore, the use of cutbacks is limited and is mainly for pothole patching now.

7. Emulsification

An alternative to dissolving the asphalt in a solvent is to physically break asphalt down into micron-sized globs that are mixed into water containing an emulsifying agent. Emulsified asphalt typically consists of about 60–70% asphalt, 30–40% water, and a fraction of a percentage of emulsifying agents. As shown in Fig. 6.1, an 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 same charges of emulsifying molecules repel, pushing asphalt globs apart, and force asphalt and water, two kinds of immiscible liquids, to combine into a suspension substance. When the water in asphalt emulsions evaporates, the asphalt globs are allowed to come together, and form the binder. The separation between the asphalt residue and water is referred to as breaking or setting. According to the type of emulsifier, emulsified asphalt can be divided into cationic emulsified asphalt, anionic emulsified asphalt, and non-ionic emulsified asphalt. Depending on the rate of emulsion setting, we also have slow, medium, and rapid setting asphalt emulsion.

8. Modification

Modified asphalt is the asphalt prepared by mixing rubber, resin, high polymer, natural asphalt, ground rubber powder, or other additives or modifiers. The typical

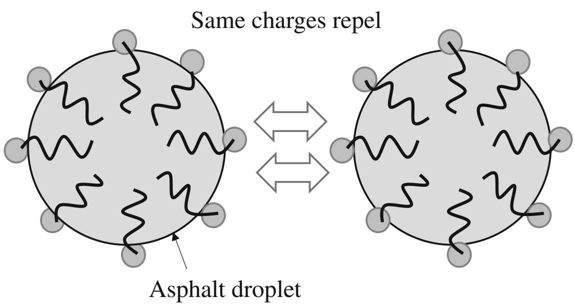


Fig. 6.1 Asphalt emulsification

polymer modifiers are either plastics or rubber and alter the strength and viscoelastic properties of the asphalt by increasing its elastic response, improving its cohesive and fracture strength, and providing greater ductility.

6.2 Composition and Structure

6.2.1 Composition

Asphalt contains various hydrocarbons and their nonmetallic derivatives (oxygen, sulfur, nitrogen). The percentages of the chemical components and the molecular structure, vary depending on the source of crude oil. Its components are mainly carbon (80–87%), hydrogen (10–15%), and non-hydrocarbon elements such as oxygen, sulfur, nitrogen (< 3%), and very few metals. The constituents of asphalt are determined based on whether or not they are soluble in different solvents. It may be firstly divided into two main fractions: asphaltenes which are insoluble in a light aliphatic hydrocarbon solvent such as *n*-heptane and maltenes which are soluble in *n*-heptane. The maltenes can be further divided into resins which are highly polar hydrocarbons and oil which can be subdivided into aromatics and saturates.

1. Three fractions

The three fractions of asphalt include asphaltenes, resins, and oil. Table 6.1 summarizes the characteristics of the three fractions. 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 oil. The resin's viscosity is largely affected by temperature. When the resins are oxidized, for example during aging, they yield asphaltene-type molecules. Resins can be divided into neutral resin and acid resin. Neutral resin increases the asphalt's plasticity, fluidity, and bonding. Acidic resin is the most active component in asphalt. It can improve the infiltration of asphalt to mineral materials, especially improve the adhesion with the carbonate rock, and increase the emulsification of asphalt. Oil is a clear or white liquid and is soluble in most solvents, allowing asphalt to flow.

2. Four fractions

When dividing oil into aromatics and saturates, we have the four fractions of asphalt, including asphaltenes, resins, aromatics, and saturates, which account for approximately 15, 20, 50, and 15% of the total. The aromatic oil is oily and yellow or brown in appearance, and includes naphthenoaromatic type rings. The saturated oil is made up mainly of long straight saturated chains and appears as a highly viscous colorless oil. Table 6.2 summarizes the characteristics of the four fractions.

Table 6.1 Characteristics of the three fractions in asphalt (Huang et al., 2020)

Fractions	Appearance	Molecular weight	Ratio of carbon/hydrogen	Content (%)	Specific gravity	Physicochemical characteristics
Oil	Clear or white liquid	200–700	0.5–0.7	45–60	0.7–1.0	Almost soluble in most organic solvents and has optical activity
Resin	Brown gel or solid	800–3000	0.7–0.8	15–30	1.0–1.1	High-temperature susceptibility, melting point is less than 100 °C
Asphaltene	Dark brown, friable solid	1000–5000	0.8–1.0	5–30	1.1–1.5	Does not melt but carbonized when heated

Table 6.2 Characteristics of the four fractions in asphalt (Huang et al., 2020)

Fractions	Appearance	Specific gravity	Molecular weight	Chemical structures
Saturate	Colorless liquid	0.89	625	Alkane and cycloalkane
Aromatic	Yellow liquid	0.99	730	Aromatic hydrocarbon
Resin	Brown gel	1.09	970	Multiple-ring structure
Asphaltene	Dark brown solid	1.15	3400	Condensed structure

6.2.2 Colloidal Structure

Asphalt is a colloidal system consisting of high molecular weight asphaltene micelles dispersed or dissolved in a lower molecular weight oily medium or the maltenes (Fig. 6.2). Asphaltenes can cluster together to form large particles. 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.

According to the different chemical composition and relative content of each component in asphalt, the colloidal structure of asphalt can be classified into three types, which are sol type, sol–gel type, and gel type (Fig. 6.3). The sol type includes less than 10% asphaltene, in which the asphaltenes particles are complete dispersal. It is usually the refinery asphalt. It is soft at low temperatures and is highly temperature-dependent. The gel type includes higher than 30% asphaltenes in which the asphaltene particles are connected, forming a comb-shaped network. It is usually the oxidized asphalt. It is very brittle at low temperatures and its temperature dependency is low. Between the two types is the sol–gel type containing 15–25% asphaltenes. Its low-temperature brittleness and temperature susceptibility are both moderate.

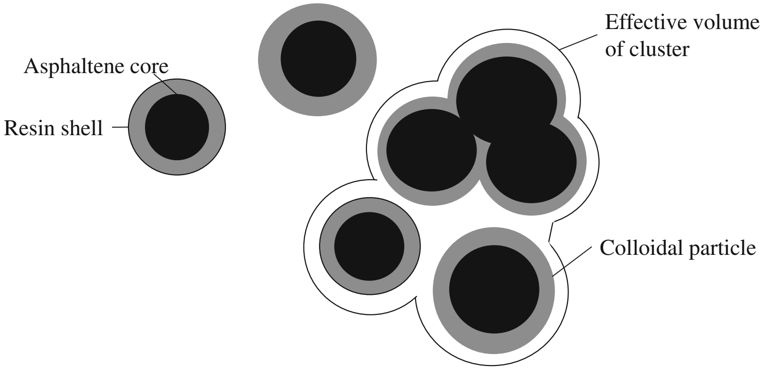


Fig. 6.2 Asphalt colloidal structure, redrafted after Lesueur (2009)

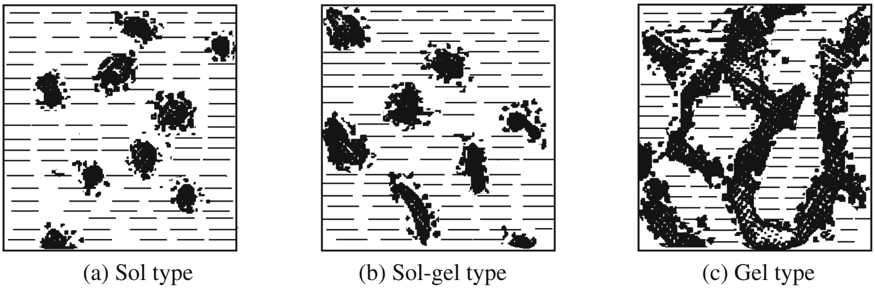


Fig. 6.3 Colloidal structure of asphalt, reprinted after Huang et al. (2020)

6.3 Properties

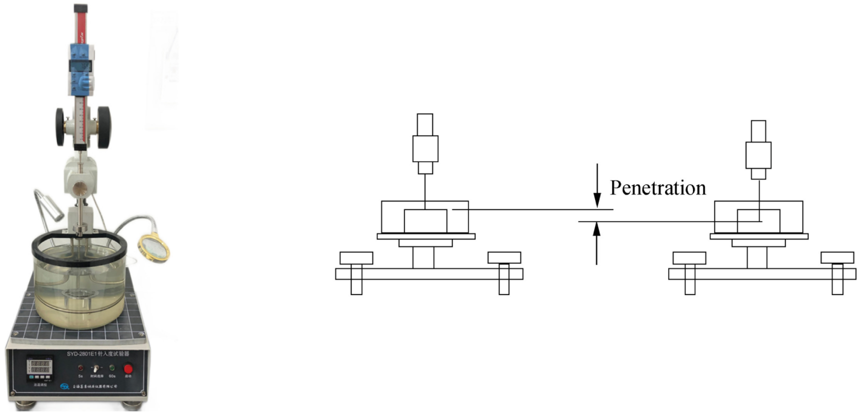
6.3.1 Physical Properties

1. Density

The density of asphalt varies with the change in temperature. At 15–25 °C, the density of asphalt ranges from 0.96 to 1.04. The density of asphalt is a key parameter for asphalt mixture design. Preservation and transportation of asphalt also need to consider the density and thermal expansion of asphalt.

2. Coefficient of expansion

The change in volume of asphalt at a temperature rise of 1 °C is the coefficient of bulk expansion. The coefficient of expansion of asphalt can be determined based on the density at different temperatures as shown in Eq. (6.1), it is the change of density over the change of temperature.



(a) Asphalt penetration tester (b) Test method

Fig. 6.4 Penetration test of asphalt

$$A = \frac{D_2 - D_1}{D_1} \times \frac{1}{T_1 - T_2} \tag{6.1}$$

where

D_2 density of asphalt at temperature T_2 (g/cm^3);
D_1 density of asphalt at temperature T_1 (g/cm^3);
T_1, T_2 two testing temperatures (°C).

6.3.2 Penetration

According to specification JTG E20-2011 T0604 (2011), penetration refers to the penetration depth of a 100 g needle into asphalt in 5 s at 25 °C (Fig. 6.4). The depth of penetration, in units of 0.1 mm, is recorded and reported as the penetration value. The penetration is inversely related to viscosity, as the test measures the viscous resistance to the penetration of a needle into a container of asphalt. Penetration is the most widely used method to measure the relative viscosity or consistency of asphalt in the world and is still used to determine the asphalt grade in many countries.

6.3.3 Viscosity

In 1930, Physicist Thomas Parnell set up the Pitch Drop Experiment (Fig. 6.5). It is one of the longest-running lab tests. At room temperature, asphalt still exhibits viscosity over a very long period. The asphalt drops formed slowly and approximately fell a drop every ten years. Viscosity is a very important property of asphalt. It is a measure of the resistance of a material to deformation at a given rate. Viscosity can be measured directly in various ways. The penetration, softening point and different viscosity tests can all be used to evaluate the viscosity or consistency of asphalt. The different types of viscosities include relative viscosity, absolute or dynamic viscosity, and kinematic viscosity.

1. Relative viscosity

Relative viscosity can be measured by the Ford viscosity cup (Fig. 6.6). It is also called the standard viscosity. Relative viscosity refers to the time that 50 mL asphalt needs to flow through a nozzle. Penetration and softening point tests are also relative viscosity tests.

2. Absolute or dynamic viscosity

The absolute or dynamic viscosity can be measured by three different but similar types of equipment, including the capillary viscometer, the rotational viscometer, and the sliding plate viscometer.

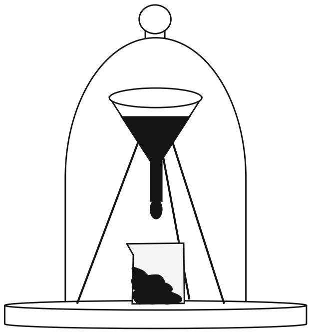
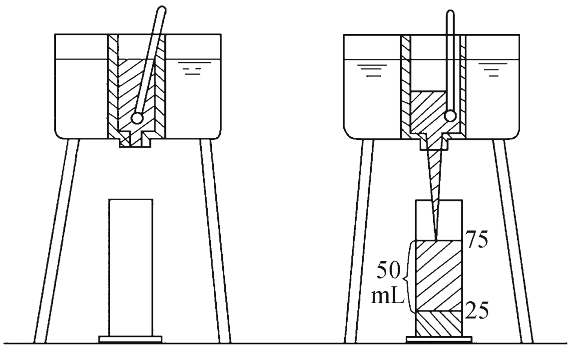


Fig. 6.5 Pitch drop experiment



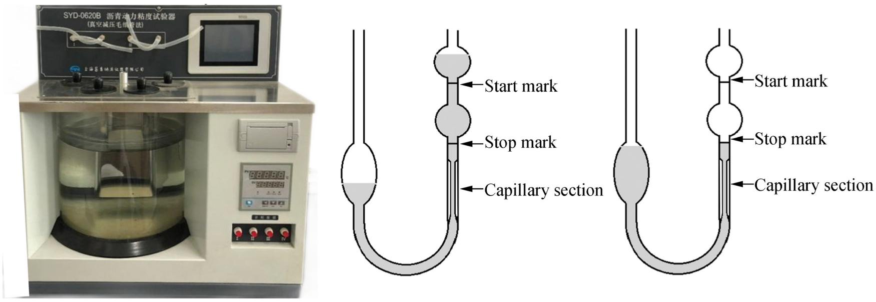
(a)Initial state (b)50 mL asphalt flow out

Fig. 6.6 Relative viscosity test using the Ford viscosity cup, redrafted after Huang et al. (2020)

(1) The capillary viscometer measures the time for asphalt to flow through a small tube under a controlled vacuum (Fig. 6.7). According to specification JTG E20-2011 T0620 (2011), the time required for the leading edge of the asphalt to pass between the start and stop mark is recorded and then the absolute viscosity can be calculated.

(2) The rotational viscometer (RV) measures the torque required to rotate a spindle immersed in the sample fluid (Fig. 6.8). It is mainly used to test the workability of asphalt at 135 °C. According to specification JTG E20-2011 T0625 (2011), the test uses a rotational or Brookfield viscometer with a spindle placed in the asphalt sample and rotated at a specified speed. The material between the inner cylinder and the outer cylinder is analogous to the thin asphalt film in the sliding plate viscometer. The viscosity is determined by the amount of torque required to rotate the spindle at the specified speed.

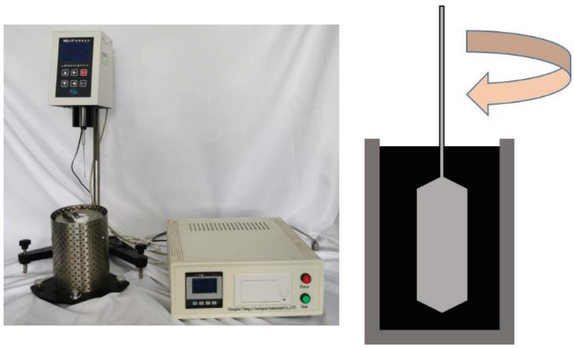
(3) The sliding plate or microfilm viscometer measures the ratio of shear stress over the shear strain in a unit of time (Fig. 6.9). It is mainly used to evaluate the viscosity of asphalt at low temperatures ranging from 10 to 55 °C.



(a) Capillary viscometer (b) Test method

Fig. 6.7 Capillary viscosity test, redrafted after Read and Whiteoak (2003)

Fig. 6.8 Rotational viscosity test, redrafted after Huang et al. (2020)



(a) Rotational viscometer (b) Test method

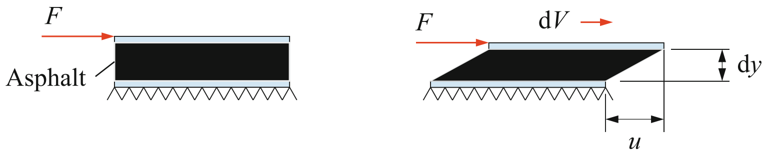


Fig. 6.9 Sliding plate viscosity test

3. Kinematic viscosity

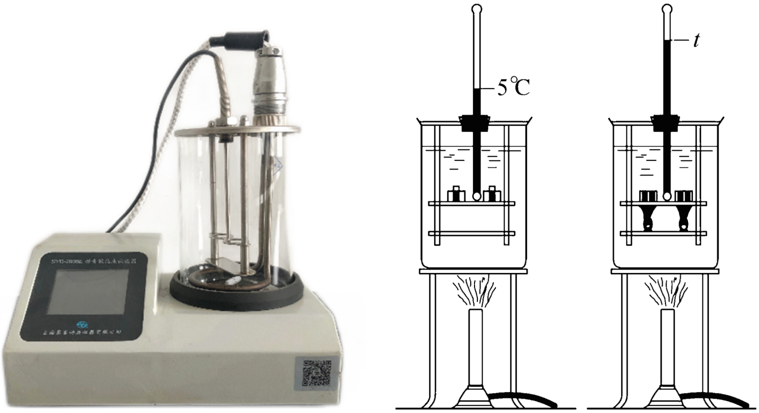
Kinematic viscosity is the ratio of absolute or dynamic viscosity divided by the density of the liquid at the temperature of measurement. Dynamic viscosity is a measurement of how difficult it is for a fluid to flow. The dynamic viscosity can be regarded as the friction inside asphalt, and the kinematic viscosity is the friction per unit weight.

6.3.4 Softening Point

Evaluation of the viscosity of asphalt requires a wide range of operating conditions including temperature, loading rate, stress, and strain. To simplify the test, the mechanical behavior and rheological properties of asphalt have been described using empirical tests and equations. The penetration test and the softening point test are the two most widely used empirical tests. According to specification JTG E20-2011 T0606 (2011), the softening point refers to the temperature at which the asphalt sample, heated at a controlled rate in a liquid bath, is soft enough to allow a 3.5 g ball to fall a distance of 25 mm. It uses a copper ring and the heating rate is 5 °C/min (Fig. 6.10). If the softening point is higher than 80 °C, we use an oil bath instead of a water bath. The penetration at softening point is around 800 (0.1 mm) and the viscosity at softening point is around 1200 Pa s. The softening point, in combination with the penetration, can be used to evaluate the temperature susceptibility of asphalt.

6.3.5 Brittle Point

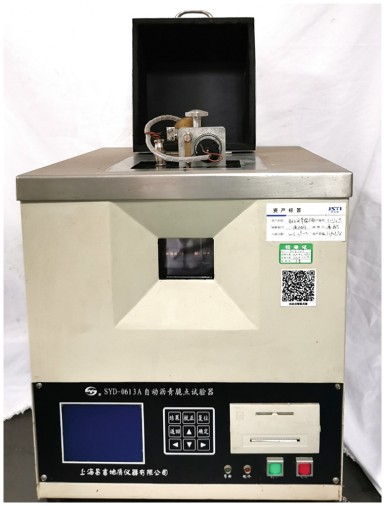
According to specification JTG E20-2011 T0613 (2011), the brittle point is tested by brushing asphalt film on a piece of metal sheet and decreasing temperature (Fig. 6.11). It refers to the temperature when asphalt film breaks. The penetration at the brittle point is around 1.2 (0.1 mm), therefore it is also called equivalent brittle point.



(a) Ring-and-ball apparatus (b) Test method

Fig. 6.10 Softening point test of asphalt, redrafted after Huang et al. (2020)

Fig. 6.11 Asphalt brittle point tester



6.3.6 Ductility

According to specification JTG E20-2011 T0605 (2011), the ductility of asphalt refers to the stretched distance (cm) at breaking. Ductility is tested by stretching a standard-sized dog-bone shape asphalt sample at the speed of 50 mm/min at 25 °C to its breaking point (Fig. 6.12). Ductility at low temperatures such as 5 or 10 °C is an indicator of the resistance of the asphalt to cracking. Therefore, the ductility is also related to the viscosity of asphalt.

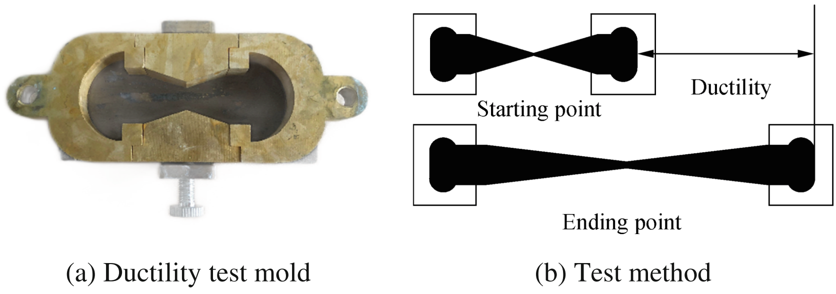


Fig. 6.12 Ductility test of asphalt, redrafted after Huang et al. (2020)

6.3.7 Adhesion

According to specification JTG E20-2011 T0616 (2011), two tests can be used to evaluate the adhesion between asphalt and aggregates. The boiling method involves selecting five cubic shape particles with a size of 13.2–19 mm, coating the aggregate with asphalt, and boiling them in distilled water for 3 min. Adhesion is evaluated by 5 grades according to the situation of asphalt film peeling. The water immersion method involves mixing 100 g aggregates with a size of 9.5–13.2 mm and 5.5 g asphalt at a specified temperature to prepare the sample. After cooling, the mixture is immersed in distilled water at 80 °C for 30 min, and then the adhesion is evaluated according to the percentage of peeling area.

The adhesion of asphalt to an aggregate is dependent on complex factors. Carbonate aggregates such as limestone have a better affinity with asphalt than granite. The main characteristics of asphalt affecting its adhesion to aggregates are its viscosity, surface tension, and polarity. The viscosity and surface tension will govern the extent to which asphalt is absorbed into the pores at the surface of the aggregate particles. The absorption of asphalt into the aggregates also depends on the total volume of permeable pore space and the size of the pore openings.

6.3.8 Durability

Durability includes several different phenomena such as aging, moisture damage, frost damage, etc.

1. Aging

Aging refers to the change of properties and chemical composition during construction and the service life period of asphalt. It can be classified into short-term aging which occurs during the mixing, transportation, and paving process, and long-term aging which occurs in the field as a result of exposure to traffic and climatic conditions during its service life. Typical aging distress is cracking. Factors influencing aging include temperature, aggregate absorption of asphalt, and the presence of oxygen. Aging occurs most severely in hot climates, near pavement surfaces, in mixtures with more voids and high-absorptive aggregates.

2. Moisture damage

Moisture damage is mainly due to the stripping of asphalt and aggregates when encountering water. It is essential to dry aggregates thoroughly during asphalt mixture production to ensure good asphalt adhesion, and sometimes to add additives to facilitate the development of adhesion. The asphalt-aggregate interface is vulnerable to damage. The high density and low permeability of the mixture can improve moisture damage. Moisture damage will cause the rutting potential and material loss distress such as raveling and potholes on the asphalt pavement.

3. Frost damage

Frost damage is that the water in the voids of asphalt mixtures expands to form ice, causing pressure within the matrix of an asphalt mixture. In addition, the water in the voids during freezing and thawing also causes unusual dynamic pressures under traffic loading. Those pressures cause a fracture at aggregate particle contacts and the material loss distress in the asphalt pavement. Practices suggest that a dense asphalt mixture with low content of air voids has a sufficiently low permeability to avoid severe moisture damage or frost damage.

6.3.9 Safety

At high temperatures, asphalt can flash or ignite in the presence of an open flame or spark. The flash point test is a safety test that measures the temperature at which asphalt flashes. The flash point is the lowest liquid temperature at which the application of the test flame causes the vapors of the sample to ignite. The Cleveland open cup method is usually adopted. According to specification JTG E20-2011 T0611 (2011), it is to partially fill the standard brass cup with an asphalt binder. 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 Fig. 6.13. The flash point of asphalt is the temperature that can sustain a flame for a short time after the volatile fumes come off the sample. The minimum temperature at which the volatile fumes are sufficient to sustain a flame for an extended time is the fire point. The flash point of an asphalt binder should be higher than 230 °C.

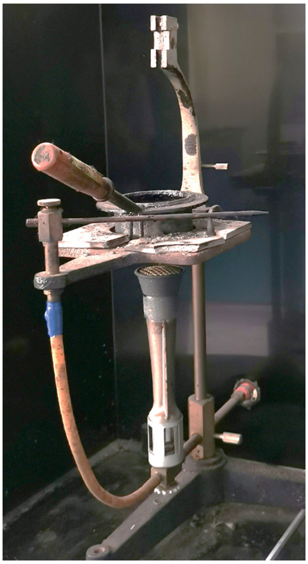


Fig. 6.13 Cleveland open cup method

6.4 Temperature Susceptibility

6.4.1 Temperature Dependency

The viscosity or consistency of asphalt is greatly affected by temperature. The asphalt gets hard and brittle at low temperatures and soft at high temperatures. The viscosity of asphalt decreases with the increase of temperature. There is an optimum range of viscosity for asphalt in the field at the annual temperature range. If the viscosity of asphalt is too high, the mixture will be too brittle and susceptible to low-temperature cracking. If the viscosity is too low, the mixture will flow readily, resulting in permanent deformation. There are also optimum viscosity ranges for mixing and paving. During construction, the asphalt needs to be heated to specific temperature ranges to meet the viscosity requirements for mixing and paving. As shown in Fig. 6.14, the proper viscosity for mixing and paving mixture are 0.17 and 0.25 Pa s, respectively. The proper mixing temperature is 155–160 °C and the proper compaction temperature is 145–150 °C. Warm mix additives can reduce the viscosity of asphalt and therefore the mixing and compaction can be done at a temperature lower than traditional hot mix asphalt mixture.

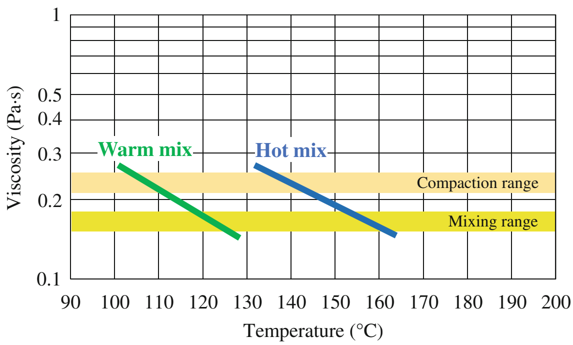


Fig. 6.14 Viscosity for asphalt compaction and mixing, redrafted after Huang et al. (2020)

6.4.2 Time–Temperature Equivalency

Asphalt is the typical time–temperature equivalent material. The time–temperature equivalent principle refers to the phenomenon that the time-dependent mechanical properties of materials rely on variations of temperatures. For example, under the same load, the deformation of asphalt at 60 °C in 1 h may be the same as the deformation of that at 25 °C in 10 h. Based on the time–temperature equivalency principle, the test results obtained at a higher temperature and shorter load duration are equivalent to tests performed at a lower temperature and longer load duration. Therefore, it is reasonable to predict the material's long-time mechanical properties based on their relationships with rising temperatures. It is also reasonable to shift creep curves at different temperatures into a master curve at a reference temperature.

6.4.3 Penetration Index

There is a simple way to evaluate the temperature susceptibility of asphalt using the penetration at different temperatures. The softening points are different temperatures measured at the same force and same deformation, while penetrations are the different deformations measured at the same force and same temperature. The results of the two tests are related. The penetration at the softening point is around 800 (0.1 mm). As shown in Fig. 6.15, penetration has a near exponential relationship with temperature and the logarithm of penetration ($\lg P$) has a linear relationship with temperature (T).

$$\lg P = AT + K \tag{6.2}$$

where

A slope;
K intercept (constant).

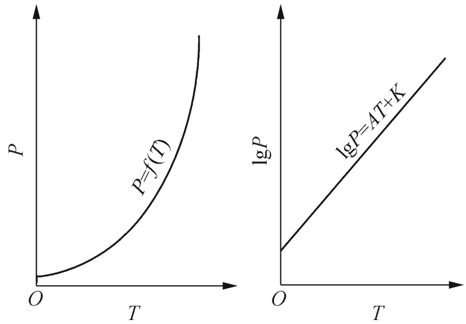


Fig. 6.15 Asphalt penetration versus temperature, redrafted after Huang et al. (2020)

The slope A represents the rate of change of penetration with temperature, indicating the temperature susceptibility. Therefore, A is called penetration-temperature susceptibility coefficient and can be calculated as

$$A = \frac{\lg 800 - \lg P_{(25\,^{\circ}\mathrm{C}, 100\,\mathrm{g}, 5\,\mathrm{s})}}{T_{\mathrm{R\&B}} - 25} \tag{6.3}$$

where

$P_{(25\,^{\circ}\mathrm{C}, 100\,\mathrm{g}, 5\,\mathrm{s})}$ the penetration at 25 °C (0.1 mm);
$T_{\mathrm{R\&B}}$ softening point (°C).

For a very high-temperature susceptibility, A is close to infinity. For a very low-temperature susceptibility, A is close to zero. Based on Eq. (6.3), we can also obtain the equivalent softening point temperature T_{800} at P = 800, and the equivalent brittle point temperature $T_{1.2}$ at P = 1.2, as shown in Eqs. (6.4) and (6.5), respectively.

$$T_{800} = \frac{\lg 800 - K}{A} = \frac{2.9031 - K}{A} \tag{6.4}$$

$$T_{1.2} = \frac{\lg 1.2 - K}{A} = \frac{0.0792 - K}{A} \tag{6.5}$$

The penetration index (PI) is defined as a function of the slope A, calculated as Eq. (6.6).

$$PI = \frac{30}{1 + 50A} - 10 \tag{6.6}$$

PI of −10 means a very high-temperature susceptibility, while PI of 20 means a very low-temperature susceptibility. PI can be used not only to evaluate the temperature sensitivity of asphalt, but also to classify the colloidal structure of asphalt. PI lower than −2 indicates a sol-type colloidal structure. PI between −2 and 2 indicates a sol–gel-type colloidal structure. PI higher than 2 indicates a gel-type colloidal structure.

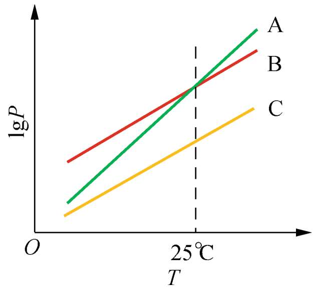


Fig. 6.16 Asphalt with different penetration-temperature relationship

PI is a very important property of asphalt. When designing asphalt mixtures for a climatic region, we should not only select asphalt of proper penetration, but also check the penetration index. As shown in Fig. 6.16, for a cold region, we want a higher penetration value, and asphalt A and B meet this requirement. However, asphalt A has a higher temperature susceptibility or a lower penetration index. Therefore, asphalt B with a lower temperature susceptibility or a higher penetration index is preferred.

6.5 Aging and Modification

6.5.1 Aging

Aging is the change of properties and chemical composition of asphalt during construction and its service life period. There are two mechanisms of aging. Short-term aging occurs during mixing, transportation, placing, and compacting while the asphalt is at an elevated temperature, and is responsible for a significant change in properties. Long-term aging occurs gradually, and thus asphalt becomes ever harder during its service life. In general, as an asphalt binder ages, its viscosity increases and it becomes stiffer and more brittle, and causes cracking. Aging is a process including complicated physical changes and chemical reactions.

(1) Oxidation is the reaction of oxygen with asphalt. Many of the polar molecules within the asphalt are readily able to combine with any free oxygen they find. The result is additional cross-linking between molecules and a general stiffening and increase of viscosity.
(2) Volatilization is the evaporation of the lighter constituents of asphalt. It is primarily a function of temperature and occurs principally during hot mixed asphalt production.
(3) Polymerization is the combining of likely molecules to form larger molecules, causing a progressive hardening.
(4) Separation is the absorption of lighter saturate fractions by porous aggregates.

(5) Thixotropy is the setting of the asphalt when unagitated. Thixotropic effects can be somewhat reversed by heat and agitation. Asphalt pavements with little or no traffic are generally associated with thixotropic hardening.
(6) Syneresis is the separation of less viscous liquid from the more viscous asphalt molecular network. The liquid loss hardens asphalt and is caused by shrinkage or rearrangement of the asphalt structure due to physical or chemical changes. Syneresis is a form of bleeding.

As shown in Fig. 6.17, short-term aging can be simulated by thin-film oven (TFO) and rolling thin-film oven (RTFO). The TFO simulates short-term aging by heating a film of asphalt for 5 h at 163 °C (JTG E20-2011 T0609, 2011). For the RTFO aging, the asphalt binder is poured into the glass jars placed on a rack in a forced-draft oven at a temperature of 163 °C for 75 min (JTG E20-2011 T0610, 2011). The rack rotates vertically, continuously exposing the fresh asphalt. The asphalt in the rotating bottles is also subjected to an air jet to speed up the aging process. Long-term aging can be simulated by using the pressure-aging vessel (PAV). It consists of a temperature and pressure-control chamber. A specified thickness of residue from the RTFO is placed in the PAV pans and then aged at the specified aging temperature for 20 h under 2.1 MPa air pressure (JTG E20-2011 T0630, 2011). Since the pressure aging procedure forces oxygen into the sample, it is necessary to use a vacuum oven to remove any air bubbles from the sample before testing.

The effects of aging can be evaluated by the change of composition and properties. With the increase of aging time, the amount of asphaltenes increases, and the amount of aromatics significantly decreases. A portion of the resins changes into asphaltenes while part of the aromatics changes into the resins. Because of the change of composition, the asphalt becomes stiffer. In terms of the change of asphalt properties, large differences in properties before and after aging indicate severe aging affects. The aging index refers to the ratio of asphalt properties after aging over those before aging. As shown in Fig. 6.18, short-term aging is the aging that occurs during mixing, transportation, and application. It accounts for the majority of aging. Therefore, the warm-mix technology reduces the mixing temperature and can greatly reduce short-term aging.

(a) TFO (b) RTFO (c) PAV

Fig. 6.17 Asphalt aging equipment

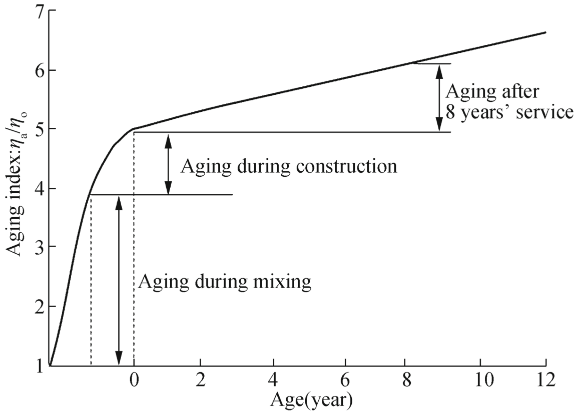


Fig. 6.18 Asphalt aging process, redrafted after Read and Whiteoak (2003)

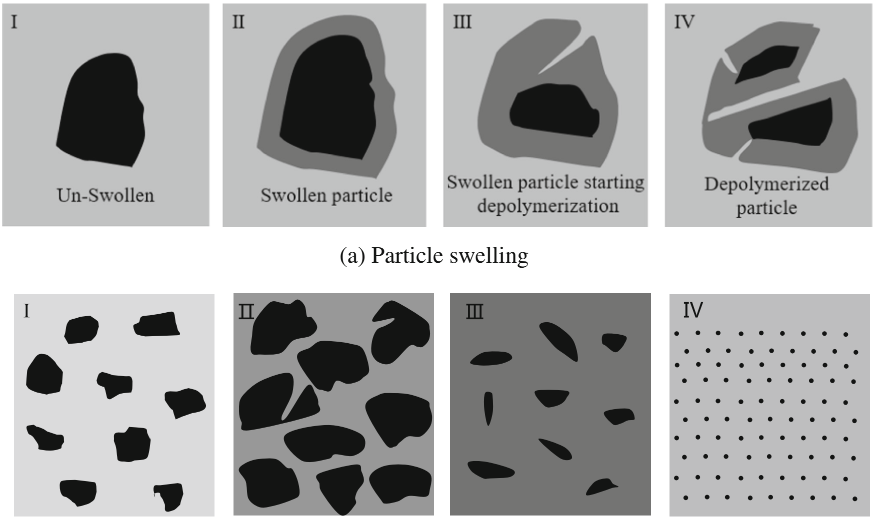
6.5.2 Modification

Unmodified asphalt may be too soft at high temperatures, fracture at low temperatures, and lose adhesion under a combination of aging and moisture damage. Modification of asphalt is to add modifiers or additives into asphalt to improve the properties of asphalt or to add special properties to asphalt mixtures. Table 6.3 summarizes the major types of modifiers. Some modifiers such as fillers, fibers, and antistripping additives can be directly added during mixing. However, many polymer modifiers need to be evenly mixed with asphalt to produce modified asphalt first. A shear mill can grind asphalt and modifier down to particles of between 1 and 3 μm during blending.

Styrene–butadiene–styrene (SBS), styrene-butadiene rubber (SBR), and ethyl vinyl acetate (EVA) are three of the most commonly used polymer modifiers. Polymer modification improves both the coating of the aggregates for improved durability and the elasticity of the binder, which benefits the rutting, fatigue, and thermal cracking resistance of the binder. Polymer blended with asphalt forms two phases, which are the polymer-rich phase and the asphalt-rich phase, depending on the content of the polymer. The key of rubber modified asphalt is the rubber-asphalt interaction including particle swelling and dissolution (Fig. 6.19). Particle swelling is the light

Table 6.3 Major types of asphalt modifiers

Types	Purposes	Examples
Filler	Reduce binder content, increase stability and bond	Mineral, fines, lime, cement, fly ash, carbon black
Extender	Reduce binder content	Sulfur, lignin
Polymer	Increase stiffness at high temperature, increase elasticity at medium temperature, reduce stiffness at low temperature	Latex, SBS, SBR, crumb rubber, polyethylene



(a) Particle swelling

(b) Particle dissolution

Fig. 6.19 Mechanism of asphalt polymer modification, redrafted after Abdelrahman (2006)

fractions such as aromatic oil of the asphalt are absorbed into the polymer chains of crumb rubber, forming a gel-like material that is double to triple the original volume. Dissolution is the chemical breakdown of polymer chains of rubber into small molecules and dissolving into the liquid phase of asphalt.

6.6 Penetration and Viscosity Grading

The asphalt binder is produced in several grades or classes. There are several grading systems for classifying asphalt binders, including penetration grading, viscosity grading, and performance grading.

6.6.1 Penetration Grading

The penetration grading system was developed in the early 1900s to characterize the consistency of asphalt. It is the most widely used asphalt grading system. It classifies asphalt mainly based on its penetration test results. The specification JTG F40-2004 (2004) defines seven grades of asphalt based on the penetration test results at 25 °C (Table 6.4). The grades range from 30 to 160. Among them, grades 50–130 are the most widely used, while grade 30 is for the extremely hot area and grade 160 is for the extremely cold area. In each grade, there are three quality classes of asphalt from Class A to Class C. Class A is the best quality asphalt and can be used in any pavements and any courses. Class B can be used in the bottom course of expressways and Class I roads, or all courses of lower-class roads. Class B can also be used as the base asphalt for modification, emulsification, and cutback asphalt. Class C can only be used in all courses of the Class III and lower-class roads.

In addition to testing the penetration of the original sample at 25 °C, the specification JTG F40-2004 (2004) requires testing several other properties of asphalt. The penetration is to test the medium temperature performance. The penetration index is to test the temperature susceptibility. The softening point and kinematic viscosity at 60 °C are to test the high-temperature stability. Ductility at 15 and 10 °C is for low temperature cracking resistance. Wax content and solubility are for asphalt quality. The flash point is for safety reasons. Further, the weight change, penetration, and ductility of the short-term aged residues should also be tested to examine if the asphalt still has sufficient medium temperature penetration and low temperature cracking resistance after mixing and paving.

Climatic condition is a very important factor in selecting asphalt grade. According to specification JTG F40-2004 (2004), climatic regions are classified based on three criteria: the highest average temperature of one month, the lowest temperature, and the annual precipitation (Table 6.5). Based on the highest average temperature of one month, there are three climatic regions, i.e. very hot, hot, and cool. Based on the lowest temperature, there are four climatic regions, i.e. very cold, cold, cool, and warm. Based on the annual precipitation, there are four climatic regions, i.e. very humid, humid, dry, and very dry.

The climatic regions for selecting asphalt grade are named by two numbers. The first number is the high-temperature classification and the second number is the low-temperature classification. In addition to the temperature regions, rainfall regions are also defined. For example, the highest average temperature of one month in Jiangsu province is higher than 30 °C, the lowest temperature is between −21.5 and −9 °C, and the annual precipitation is 500–1000 mm. According to Table 6.5, Jiangsu province is at '1–3' temperature region and '2' rainfall region, meaning it is very hot in summer and cool in winter, and it is a humid area.

6.6.2 Viscosity Grading

In the early 1960s, an improved asphalt grading system was developed based on a rational scientific viscosity test. This scientific test replaced the empirical penetration test as the key asphalt characterization and is now still used in areas like Canada. Viscosity grading can be done on original asphalt samples, called AC grading, or aged residue samples, called AR grading. The AR grading system is an attempt to simulate asphalt properties after short-term aging and thus, it should be more representative of how asphalt behaves in HMA pavements.

Table 6.4 Penetration grades in China (JTG F40-2004, 2004)

Test items	Unit	Classes	Grades																	Test methods
			160	130	110			90					70					50	30	
Penetration (25 °C, 5 s, 100 g)	0.1 mm		140–200	120–140	100–120			80–100					60–80					40–60	20–40	T 0604
Climate region					2–1	2–2	3–2	1–1	1–2	1–3	2–2	2–3	1–3	1–4	2–2	2–3	2–4	1–4		
Penetration index (PI)		A	−1.5 to +1.0																	T 0604
		B	−1.8 to +1.0																	
Min. softening point (R&B)	°C	A	38	40	43			45			44		46		45			49	55	T 0606
		B	36	39	42			43			42		44		43			46	53	
		C	35	37	41			42					43					45	50	
Min. dynamic viscosity at 60 °C	Pa s	A	–	60	120			160			140		180		160			200	260	T 0620
Min. ductility at 10 °C	cm	A	50	50	40			45	30	20	30	20	20	15	25	20	15	15	10	T 0605
		B	30	30	30			30	20	15	20	15	15	10	20	15	10	10	8	
Min. ductility at 15 °C	cm	A, B	100															80	50	
		C	80	80	60			50					40					30	20	
Min. wax content	%	A	2.2																	T 0615
		B	3.0																	
		C	4.5																	

(continued)

Table 6.4 (continued)

Test items	Unit	Classes	Grades							Test methods
			160	130	110	90	70	50	30	
Min. flash point	°C		230			245	260			T 0611
Min. solubility	%		99.5							T 0607
Density	g/cm³		–							T 0603
Residue of TFO (or RTFO) test										T 0610 or T 0609
Max. weight change	%	±0.8								
Max. penetration ratio	%	A	48	54	55	57	61	63	65	T 0604
		B	45	50	52	54	58	60	62	
		C	40	45	48	50	54	58	60	
Max. ductility at 10 °C	cm	A	12	12	10	8	6	4	–	T 0605
		B	10	10	8	6	4	2	–	
Max. ductility at 15 °C	cm	C	40	35	30	20	15	10	–	T 0605

Table 6.5 Climatic regions for asphalt applications (JTG F40-2004, 2004)

Hot climate region	Regions	1	2	3	–
	High temperature	Very hot	Hot	Cool	–
	Highest average temperature of one month (°C)	> 30	20–30	< 20	–
Cold climate region	Regions	1	2	3	4
	Low temperature	Very cold	Cold	Cool	Warm
	Lowest temperature (°C)	< −37	−37 to 21.5	−21.5 to 9	> −9
Rainfall region	Regions	1	2	3	4
	Rainfall	Very humid	Humid	Dry	Very dry
	Annual precipitation (mm)	> 1000	500–1000	250–500	< 250

In the viscosity grading system, the 60 °C viscosity is to test the asphalt performance at high pavement temperature. The 135 °C viscosity is to test the asphalt performance at mixing temperature. Penetration and ductility at 25 °C are to test the asphalt performance at medium pavement temperature. The flash point is to test asphalt safety. Solubility is to test asphalt quality. Besides, the viscosity and ductility test the performance of short-term aged asphalt at high and medium pavement temperature, respectively.

Details of the AC and AR viscosity grading can be found in specifications AASHTO M 226-80 (1980) and the ASTM D3381/D3381M-13 (2013). As shown in Table 6.6, for AC viscosity grading, AC-5 grade means the viscosity at 60 °C is 500 ± 100 poise, which is less viscous than the AC-40 grade whose 60 °C viscosity is 4000 ± 800 poise. Typical grades used for hot mixed asphalt paving in the US are AC-10, AC-20, AC-30, AR-4000, and AR-8000.

Figure 6.20 shows the relationships between penetration grading, AC viscosity grading, and AR viscosity grading. Asphalt with the same viscosity might be classified into three penetration grades. Therefore, the empirical penetration grading sometimes may not effectively differentiate asphalt. The viscosity grading is more rational.

6.7 Performance Grading

6.7.1 Equipment

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials and designing asphalt mixes. The SHRP produced the Superpave, which is the superior performing asphalt pavements mix

Table 6.6 Viscosity grading (AASHTO M 226-80, 1980; ASTM D3381/D3381M-13, 2013)

Specifications	Grading based on original asphalt (AC)						Grading based on aged residue (AR)				
AASHTO M 226-80	AC-2.5	AC-5	AC-10	AC-20	AC-30	AC-40	AR-10	AR-20	AR-40	AR-80	AR-160
ASTM D3381/D3381M-13	AC-2.5	AC-5	AC-10	AC-20	AC-30	AC-40	AR-1000	AR-2000	AR-4000	AR-8000	AR-16000

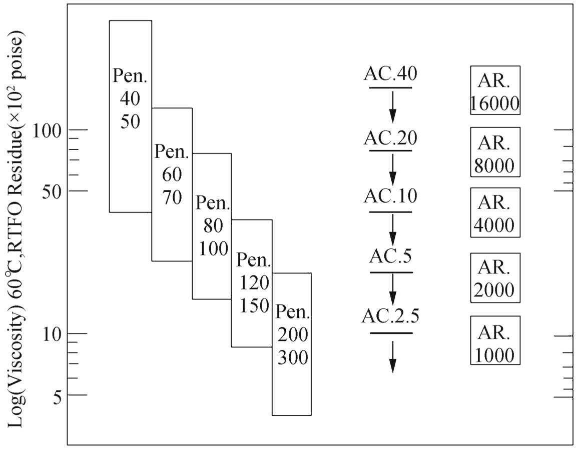


Fig. 6.20 Relationships between penetration, AC viscosity, and AR viscosity grading, redrafted after Ray et al. (2009)

design method for asphalt concrete and the performance grading method for asphalt specification (AASHTO M 320-10, 2010). The performance grade, or PG grade, classifies asphalt by performance at the applicable temperature of the region where the road is located. It involves a grading discipline and a series of new testing methods.

1. RTFO

The RTFO is used to simulate short-term aging. It is not newly developed equipment. In the test, asphalt is poured into the glass jars placed on a rack in a forced-draft oven at a temperature of 163 °C for 75 min. The rack rotates vertically. The asphalt in the rotating jars is subjected to an air jet to speed up the aging process.

2. PAV

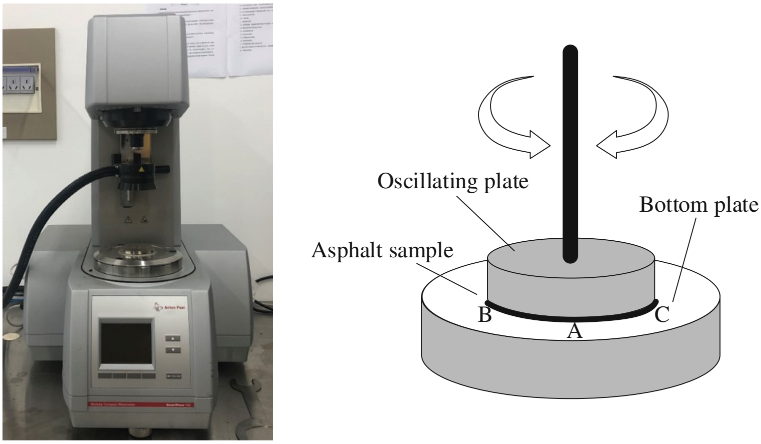
The PAV can simulate 5–10 years of long-term aging. It consists of a temperature and pressure-control chamber. A specified thickness of residue from the RTFO is placed in the PAV pans and then aged at the specified aging temperature for 20 h under 2.1 MPa. The aging temperature ranges from 90 to 110 °C and is selected according to the grade of the asphalt.

3. RV

The RV tests the workability for mixing and paving of original asphalt at 135 °C (Fig. 6.8). 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.

4. Dynamic shear rheometer (DSR)

The DSR is used in all stages of the PG tests to evaluate both rutting and fatigue potentials of asphalt (Fig. 6.21). According to specification JTG E20-2011 T0628 (2011), one of the parallel plates oscillates with respect to the other at preselected



(a) DSR (b) Test method

Fig. 6.21 Dynamic shear rheometer (DSR), redrafted after Read and Whiteoak (2003)

frequencies and rotational deformation amplitudes or torque amplitudes. For evaluating rutting potential, the test temperature is equal to the high temperature for the grade of asphalt and the sample size is 25 mm in diameter and 1 mm in thickness. For evaluating fatigue potential, the intermediate temperature is used and the sample size is 8 mm in diameter and 2 mm in thickness.

Complex shear modulus (G^*) and phase angle (δ) can be obtained from the DSR test results. Based on the shear stress-time and the shear strain–time curves, G^* is the ratio of the maximum shear stress ($\tau_{\max}$) over the maximum shear strain ($\gamma_{\max}$). The complex shear modulus (G^*) can be considered as the sample's total resistance to deformation when repeatedly sheared. The phase angle (δ), is the lag between the applied shear stress and the resulting shear strain (Fig. 6.22). The larger the phase angle, the more viscous the material. For pure elastic material, phase angle equals 0.

$$G^* = \frac{\tau_{\max}}{\gamma_{\max}} \tag{6.7}$$

where

$\tau_{\max}$ the maximum shear stress (MPa);
$\gamma_{\max}$ the maximum shear strain.

Because of the viscoelastic nature of asphalt, G^* is composed of two parts: the elastic or storage modulus (G') and the viscous or lost modulus (G'') (Fig. 6.23).

$$G' = G^* \cos \delta \tag{6.8}$$

$$G'' = G^* \sin \delta \tag{6.9}$$

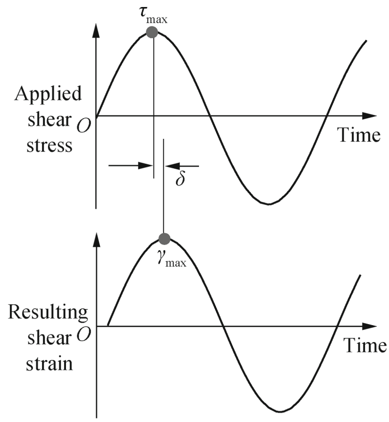


Fig. 6.22 Shear stress and strain during the DSR test, redrafted after Read and Whiteoak (2003)

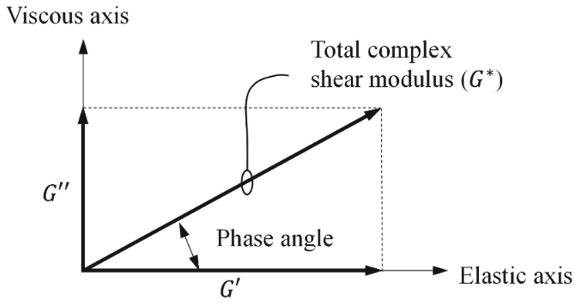


Fig. 6.23 Elastic (storage) modulus and viscous (lost) modulus, redrafted after Read and Whiteoak (2003)

where

G' elastic (storage) modulus (MPa);
G'' viscous (lost) modulus (MPa).

G^* and δ are used as predictors for HMA rutting and fatigue cracking. Rutting is the main concern in the early service life of pavement, while fatigue cracking becomes the major concern in the late service life. Table 6.7 presents the PG requirements of the DSR test results, which are mainly to address two kinds of distress.

(1) To address rutting, $G^*/\sin\delta$ should be no less than 1 and 2.2 kPa for the original binder and RTFO residue, respectively. To resist rutting, the asphalt should be stiffer and more elastic. Intuitively, high G^* means the asphalt is stiffer. Low

Table 6.7 PG criteria of the DSR test results

Materials	Value	Criteria (kPa)	Distress
Original binder	$G^*/\sin\delta$	≥ 1.0	Rutting
RTFO residue	$G^*/\sin\delta$	≥ 2.2	Rutting
PAV residue	$G^*\sin\delta$	≤ 5000	Fatigue cracking

δ means the asphalt is elastic and able to recover its original shape after being deformed by a load. Therefore, a minimum value of $G^*/\sin\delta$ is specified.

(2) To address fatigue cracking, $G^*\sin\delta$ should be no less than 5000 kPa. To resist fatigue cracking, the asphalt should be elastic and able to dissipate energy by rebounding and not cracking. But it should not be too elastic or too stiff which may cause cracking rather than deform-then-rebound. As shown in Fig. 6.23, $G^*\sin\delta$ is the viscous modulus G''. A smaller viscous modulus means less deformation or damage at each load and therefore is good for preventing fatigue cracking. Therefore, when fatigue cracking is of greatest concern late in a pavement's life, a maximum value for the viscous component of the complex shear modulus is specified.

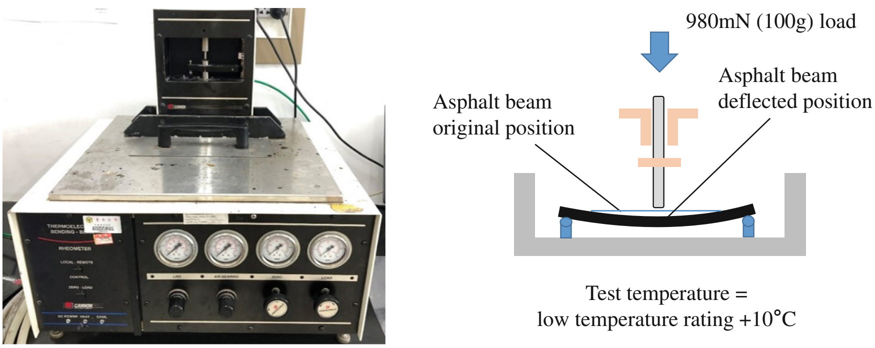
5. Bending beam rheometer (BBR)

The BBR is used to test the thermal cracking potential of short-term and long-term aged asphalt. According to specification JTG E20-2011 T0627 (2011), it measures the midpoint deflection of a simply supported prismatic beam of asphalt subjected to a constant load applied to its midpoint (Fig. 6.24). The test temperature equals the low-temperature rating plus 10 °C. The flexural creep stiffness of the beam is calculated based on the load magnitude, deflection, and dimensions of the beam specimen.

$$S(t) = \frac{PL^3}{4bh^3\Delta(t)} \tag{6.10}$$

where

P load, 100 g;
L support span, 102 mm;
b beam width, 12.7 mm;



(a) BBR (b) Test method

Fig. 6.24 The BBR test

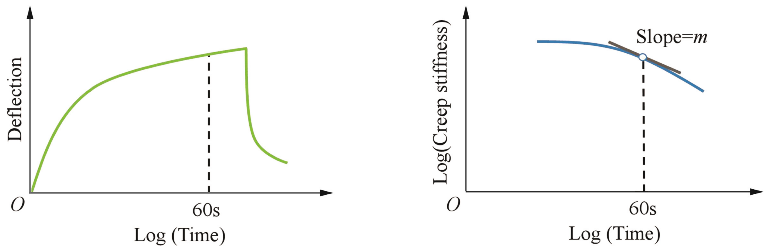


Fig. 6.25 The creep stiffness and slope, redrafted after Huang et al. (2020)

h beam thickness, 6.35 mm;
$\Delta(t)$ deflection at 60 s (mm).

As the beam creeps, the midpoint deflection is monitored. The flexural creep stiffness of the beam is calculated by dividing the maximum stress by the maximum strain for each of the specified loading times. The low-temperature thermal-cracking performance of asphalt mixtures is related to the creep stiffness and the slope of the logarithm of the creep stiffness versus the logarithm of the time curve of the asphalt (Fig. 6.25).

(6) Direct tension tester (DDT)

The DDT measures the failure strain and failure stress of a dog-bone shape asphalt specimen under direct tension (Fig. 6.26). According to specification JTG E20-2011 T0629 (2011), the test temperature is between −36 and 0 °C. The strain at failure is a measure of the amount of elongation that the asphalt can sustain without cracking. It is used as a criterion for specifying the low-temperature properties of the binder.

$$\varepsilon_{\mathrm{f}} = \frac{\Delta L}{L_{\mathrm{e}}} \tag{6.11}$$

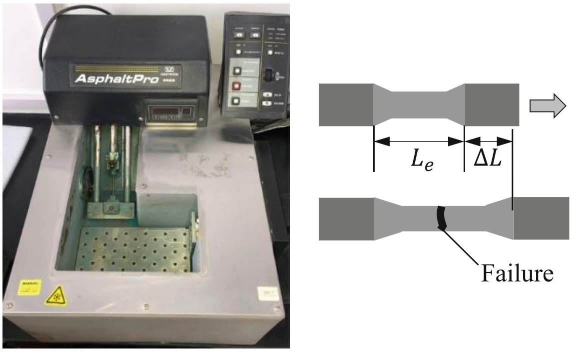
where

ε_{f} failure strain;
L_{e} gauge length (mm);
ΔL change in length (mm).

6.7.2 Testing Temperature

Unlike previous specifications that require performing the test at a fixed temperature and varying the requirements for different grades of asphalt, the PG specifications require performing the test at different critical pavement temperatures and fixing the criteria for all grades. The binder performance grading and Superpave mix design methods include performance-based specifications for asphalt binders and mixtures

Fig. 6.26 The DDT test



(a) DTT (b) Test method

to primarily control three types of distress: rutting, thermal cracking and fatigue cracking. Three pavement design temperatures are required for the PG grade: a maximum, a minimum, and an intermediate temperature (Table 6.8). The maximum pavement design temperature is the highest successive 7-day average maximum pavement temperature. The minimum pavement design temperature is the minimum surface temperature expected over the life of a pavement. The intermediate pavement design temperature is the average of maximum and minimum pavement design temperature +4 °C. The maximum and intermediate test temperatures are the same as their pavement design temperatures. The minimum test temperature is the minimum pavement design temperature +10 °C to reduce the testing time.

Table 6.8 PG testing temperatures

Distress	Temperature levels	Pavement design temperatures	Determinations	Test temperatures
Rutting	Maximum	Highest successive 7-day average maximum pavement temperature	20 mm below the pavement surface	Same
Thermal cracking	Minimum	Minimum surface temperature expected over the life of a pavement	Pavement surface	+10 °C
Fatigue	Intermediate	Average of maximum and minimum pavement design temperature +4 °C	Average temperature + 4 °C	Same

Table 6.9 Binder grades in the PG specifications (AASHTO M 320-10, 2010)

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

6.7.3 PG Grades

For the performance grade, the names of grades start with PG, which means performance graded, followed by two numbers representing the maximum and minimum pavement design temperatures in Celsius. Table 6.9 shows the binder grades in the PG specifications. The high-temperature grade ranges from 46 to 82 °C while the low-temperature grades range from −46 to −10 °C. It is noted that PG asphalt binders are specified in increments of 6 °C.

Generally, if the difference between the high and low temperature grades are less than 86 °C, the asphalt can be directly obtained from a crude oil refinery. If the difference between the high and low temperature grades are between 86 and 92 °C, the asphalt are from high-quality crude oil. PG binders that differ in the high- and low-temperature specifications by 90 °C or more generally require some sort of modification.

6.7.4 PG Tests

The main PG grading tests of asphalt can be classified into three stages (Table 6.10). The first stage is to test the workability of the original binder using RV. The second stage is to evaluate the rutting, fatigue, and thermal cracking risks of the short-term aged RTFO asphalt residue, using DSR, BBR, and DTT. The third stage is to evaluate the fatigue and thermal cracking performance of long-term aged PAV asphalt residue, using DSR, BBR, and DTT. It is noted that DSR can be used to evaluate both rutting and fatigue and is the most important equipment in the Superpave PG grading system.

Figure 6.27 summarizes the PG grading tests of asphalt and addressed properties including workability, rutting, fatigue cracking, and thermal cracking potentials. Firstly, the rotational viscosity at 135 °C of the unaged sample is tested to evaluate the workability of a sample. Secondly, the rutting potential of the RTFO aged sample is tested using DSR at the highest pavement design temperature. The fatigue cracking

Table 6.10 PG grading tests of asphalt

Tests	Original binder	Short-term aged (RTFO)	Long-term aged (PAV)
Rotational viscometer (RV)	Workability	–	–
Dynamic shear rheometer (DSR)	–	Rutting and fatigue	Fatigue
Bending beam rheometer (BBR)	–	Thermal cracking	Thermal cracking
Direct tension tester (DTT)	–	Thermal cracking	Thermal cracking

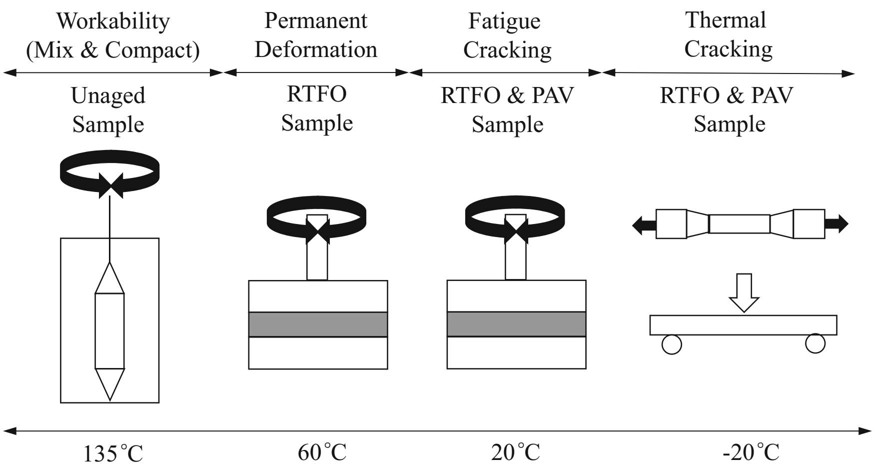


Fig. 6.27 Tests in PG grading, redrafted after Ghani et al. (2022)

potential of both RTFO and PAV aged samples is tested using DSR at the intermediate pavement design temperature. The thermal cracking potential of both RTFO and PAV aged samples is tested using BBR and DTT at the minimum pavement design temperature plus 10 °C.

Table 6.11 summarizes the specifications or test temperatures for the PG grading system. The physical properties or 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.

6.7.5 Selection of PG Grades

The proper asphalt PG grade can be selected based on the distribution of local air temperatures. As shown in Fig. 6.28a, we firstly determine the average 7-day

Table 6.11 Specifications and test temperatures for the PG grading system (AASHTO M 320-10, 2010)

Performance grades	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 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																					
Min. flash point temperature (°C)	230																				
Viscosity, ASTM D4402: Maximum, 3 Pa s, test temperature (°C)	135																				
Min. $G^*/\sin\delta$, 1 kPa test temperature at 10 rad/s (°C)	46			52							58					64					
RTFO residue																					
Max. weight loss (%)	1																				
Min. $G^*/\sin\delta$, 2.2 kPa Test temperature at 10 rad/s (°C)	46			52							58					64					

(continued)

Table 6.11 (continued)

Performance grades	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Pressure-aging vessel residue																					
PAV aging temperature (°C)	90			90							100					100					
Max. $G^* \sin \delta$, 5000 kPa test temperature at 10 rad/s (°C)	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Max. creep stiffness: S, 300 MPa, Min. m-value, 0.3 test temperature at 60 s (°C)	−24	−30	−36	0	−6	−12	−18	−24	−30	−36	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	−30
Min. direct tension: failure strain, 1.0% test temperature at 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

(continued)

Table 6.11 (continued)

Performance grades	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																
Min. flash point temperature (°C)	230															
Viscosity, ASTM D4402: Maximum, 3 Pa s, test temperature (°C)	135															
Min. $G^*/\sin\delta$, 1 kPa test temperature at 10 rad/s (°C)	70						76					82				
RTFO residue																
Max. weight loss (%)	1															
Min. $G^*/\sin\delta$, 2.2 kPa Test temperature at 10 rad/s (°C)	70						76					82				
Pressure-aging vessel residue																
PAV aging temperature (°C)	100 (110)						100 (110)					100 (110)				
Max. $G^*\sin\delta$, 5000 kPa test temperature at 10 rad/s (°C)	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Max. creep stiffness: S, 300 MPa, Min. m-value, 0.3 test temperature at 60 s (°C)	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	0	−6	−12	−18	−24
Min. direct tension: failure strain, 1.0% test temperature at 1.0 mm/min (°C)	0	−6	−12	−18	−24	−30	0	−6	−12	−18	−24	0	−6	−12	−18	−24

maximum (51 °C) and the average 1-day minimum (−17 °C) pavement temperatures based on air temperatures. There is a 50% probability that the maximum pavement temperature exceeds 51 °C or the minimum pavement temperature is lower than −17 °C. Therefore, the PG grade of asphalt should be larger than this temperature range. As shown in Fig. 6.28b, for a 98% reliability, the asphalt must cover the range from −23 to 55 °C. Since PG asphalt binders are specified in increments of 6 °C, the closest PG asphalt binder grade is PG 58-28. Therefore, a PG 58-28 is selected to meet 98% reliability requirements, as shown in Fig. 6.28c.

Questions

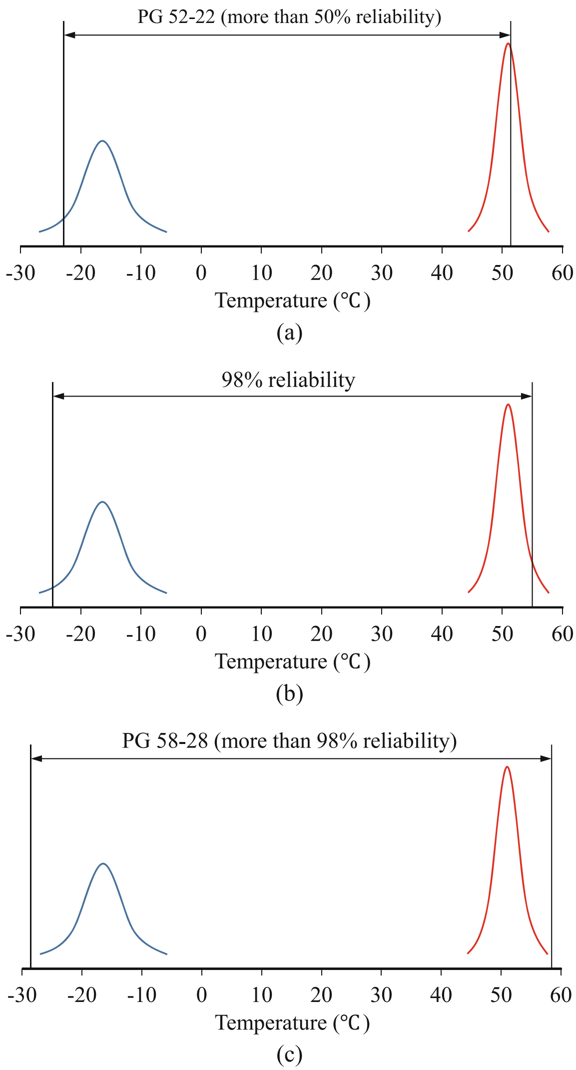


Fig. 6.28 Selection of asphalt PG grades

1. Discuss three asphalt production methods and discuss if the produced asphalt can be directly used in pavement?
2. What are mechanisms of asphalt cutbacks and asphalt emulsions, respectively?
3. Discuss the chemical composition and colloidal structure of asphalt.
4. Briefly discuss the procedures of penetration test and softening point test.
5. Define the temperature susceptibility of asphalt and draw a graph showing viscosity versus temperature of asphalt?
6. Discuss how to obtain the penetration index of an asphalt sample based on the results of the penetration and softening point test.
7. Discuss the aging of asphalt during mixing and in service. How to simulate the different types of aging of asphalt in the laboratory?
8. What is the purpose of each of these tests?

    (1) Flash point test
    (2) RTFO procedure
    (3) RV test
    (4) DSR test
    (5) BBR test

9. Define the three methods used to grade asphalt. Which method is used in your country?
10. What tests and samples (original, short-term aged, and long-term aged) are used in the PG grade to evaluate the workability, rutting, fatigue cracking and thermal cracking, respectively?

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