Bitumen Tests for the Design of Flexible Pavements

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Section 1 — Background

Overview

Pavement design is one of the most important parts of transportation engineering. To carry traffic from one place to another place comfortably, economically, and safely, an engineering design of pavements is essential. In this handout, the required background knowledge about the structural design of pavements is discussed. The pavement structure should be able to provide an acceptable riding quality, satisfactory skid resistance, favorable light-reflecting characteristics, and low noise. The aim is to ensure that the transmitted wheel loads are sufficiently reduced, so that they do not exceed the capacity of all the layers of pavement including the subgrade.

A highway pavement is a structure consisting of layers of natural and processed materials above the natural ground (often called subgrade). A pavement's primary function is to distribute the vehicle loads from the top of the pavement to a larger area of the subgrade without causing any damage to the subgrade. The pavement structure should be able to provide an acceptable riding quality, satisfactory skid resistance, favorable light-reflecting characteristics, and low noise. The aim is to ensure that the transmitted wheel loads are sufficiently reduced, so that they do not exceed the capacity of all the layers of pavement including the subgrade

A pavement is expected to meet the following requirements:

- Sufficient thickness to distribute the wheel-induced stresses to a reduced value on the subgrade soil.
- Structurally adequate to keep the cracking and deformation within tolerable limits.
- Structurally strong to withstand all types of stresses imposed upon it.
- Adequate coefficient of friction to prevent skidding of vehicles.
- Smooth surface to provide comfort to road users even at the expected speed.
- Produces least noise from moving vehicles.
- Dust and waterproof surface for avoiding reduced visibility.
- Drains water laterally or vertically without washing layer particles.

- Long service life with a desirable level of comfort considering the economy.

Two types of pavements are generally recognized: Flexible Pavements and Rigid Pavements, as shown in Figure 1.

A combination of these two pavements is also possible, and is termed Composite Pavement as shown in Figure 2.

Flexible pavement Rigid pavement

Figure 1 Flexible and Rigid Pavements

Figure 2 Composite Pavement Black Topping (top) and White Topping (bottom)

Section 2 — Pavement Types

Flexible Pavements

- Flexible pavements are usually surfaced with Asphalt Materials. These pavements are called flexible because the pavement structures can flex or bend under a traffic loading.

- A flexible pavement structure requires several layers of materials because these layers are not stiff enough to distribute the wheel load to a large area (Figure 3).

- Beneath the asphalt layer, a crushed aggregate base layer is commonly seen. Below the base layer, a subbase layer is also used based on the subgrade strength.

- The natural subgrade soil can be improved by compaction or mixing of some improved soil, asphalt millings, low-quality aggregate based on the availability of these materials, and degree of improvement required.

- Superpave, which is an acronym for Superior Performing Asphalt Pavements, is a performance-based specification for asphalt binder and volumetric mixture design. The idea was to allow asphalt pavement designs that could handle the unique weather and traffic conditions of a given site in any geographic area of the U.S. The system consisted of three components:

    - Asphalt binder specification: a system of classifying asphalt binder based on its performance response to temperatures and aging characteristics.

    - A design system grounded in traffic loading and environmental conditions.

    - Mix design system and analysis tests for performance prediction models.

- Superpave leverages modern asphalt paving technology to develop mixtures more resistant to cracking from low temperature and fatigue factors and reduce permanent deformation. Superpave means mix designs can be tailored for better performance and longer life based on a geographical area's temperature extremes, traffic loads, and utilization of the road or highway.

Rigid Pavements

- Rigid pavements are composed of reinforced or non-reinforced Portland cement concrete (PCC) surface course.
- Such pavements are stiffer than flexible pavements due to the high modulus of elasticity [typically 3000–4000 Ksi (21–28 GPa) for PCC and 500–1000 Ksi (3.4–6.9 GPa) for asphalt layer] of the PCC material.

- These pavements can have reinforcing steel to reduce thermal cracking or eliminate joints. Each of these pavement types distributes load over the subgrade in a different fashion.

- Rigid pavements, because of PCC's high elastic modulus, tend to distribute the load over a relatively wide area of a subgrade (Figure 3).

- The concrete slab itself supplies most of a rigid pavement's structural capacity. On the other hand, a flexible pavement having a low modulus distributes loads over a smaller area. It requires a thicker pavement, which is achieved through a combination of thin layers due to field compaction difficulty of constructing a thicker layer.

- Compared to flexible pavements, rigid pavements are placed either directly over the prepared subgrade or over a single layer of granular or stabilized material called base course.

- In rigid pavements, the load is distributed by the slab action, in which the pavement behaves like an elastic plate resting on an elastic medium.

- Rigid pavements should be analyzed by the "plate theory" instead of the "layer theory", assuming an elastic plate resting on an elastic foundation.

- The "plate theory" assumes the concrete slab as a medium thick plate that is plane before loading and remains plane after loading.

- Bending of the slab due to wheel load and temperature variation causes tensile and flexural stresses within the pavement layers.

- Layered elastic models assume that each pavement structural layer is homogeneous, isotropic, and linearly elastic. In other words, it is the same everywhere and will rebound to its original form once the load is removed.

- The plate theory is a simplified version of layer theory that assumes the concrete slab as a medium thick plate which is plane before loading and to remain plane after loading. Bending of the slab due to wheel load and temperature variation and the resulting tensile and flexural stress.

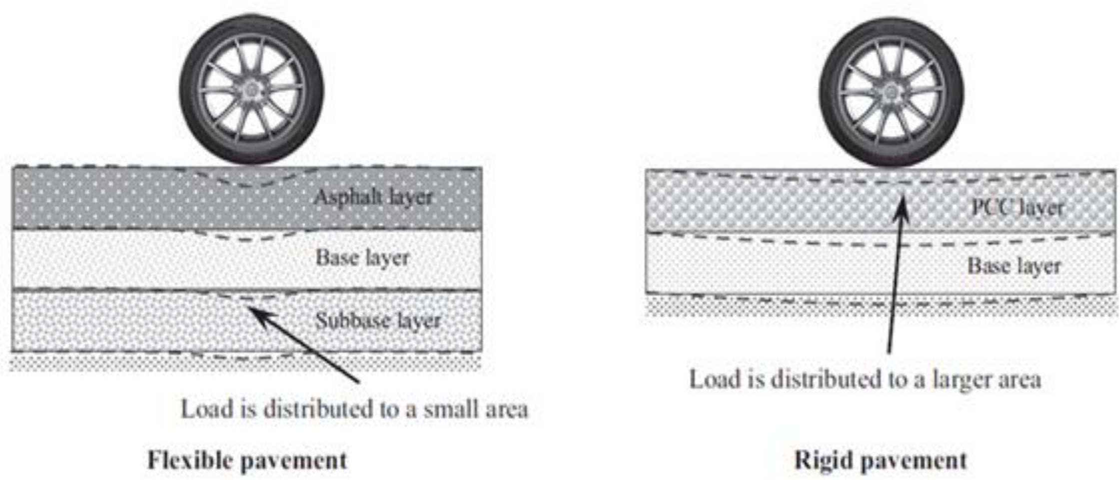


Figure 3 Deformation Behaviour of Flexible and Rigid Pavements

Three levels are available for determining the input values for most of the materials and traffic parameters: Level 1 from site specific and actual tests, resulting in higher accuracy, Level 2 from less than optimal testing or by correlations, and Level 3 from the agency database, user selected default values.

Section 3 — Asphalt Materials (Flexible Pavements)

Asphalt concrete (AC) is the general name of hardened asphaltic material mixed with coarse aggregate, fine aggregate, and some additives. The term "asphalt concrete (AC)" is interchangeably used as "hot-mix asphalt (HMA)" in the pavement industry. However, AC not only is HMA, but also includes some other forms (discussed later in this handout). AC is the material of most concern for asphalt pavement. In Section 3, the physical and mechanical characteristics of soil and aggregates have been discussed in certain pavement projects for suitability. In the current section, the asphalt materials, their properties, grading systems, and characterization methods are discussed. All these discussions are targeted at the AASHTOWare pavement ME design procedure in addition to the AASHTO 1993 design system. The construction of a typical flexible pavement roadway is shown in Figure 4.

In the United States, there are about 4,000 asphalt processing plants which generate about 525 million tons of asphalt with the total value of over $3 billion. There are about 300,000 employees working in this industry. There is a total of 8.7 million miles of pavement (2.5% interstate and 97.5% non-interstate), of which interstates comprise 65% concrete and 35% asphalt pavements. Non-interstates have 94% asphalt and 6% concrete (Rivera et al., 2017). Thus, the importance of asphalt binder is huge. The United States used approximately 130 million barrels (23 million tons) of asphalt binder and road oil in 2011, worth $7.7 billion, according to the U.S. Energy Information Administration. In the recent peak years of 1999 and 2005, nearly 200 million barrels were consumed (EIA, 2011). In 2001, the United States produced almost 35 million tons of asphalt at a rough value of around $6 billion. Roads and highways constitute the largest sole use of asphalt at 85% of the total (Asphalt Institute, 2001).

Approximately 83% of asphalt binder used in the United States in 2011 was used for paving purposes (Grass, 2012). In the United States, more than 92% of all paved roads and highways are surfaced with asphalt products. The United States has about 4,000 plants producing asphalt mixtures, with total production of about 452 million tons in 2007 (NAPA, 2012) and about 396 million tons in 2010 (Hansen and Newcomb, 2011). The value of asphalt paving mixtures produced in the United States was estimated at $11.5 billion in 2007 (U.S. Census Bureau, 2007). Asphalt usually accounts for between 4% and 8% of the AC mix by weight and about 30% of the cost of AC pavement structure depending on type and quantity.

Figure 4 Construction of Typical Flexible pavement Roadway

Grading of Asphalt Binders

Asphalt binder can be graded broadly into three systems:

1. Penetration grading
2. Viscosity grading
3. Performance grading

Penetration Grading

Penetration grading (in short named as *pen grade*) is primarily developed based on the penetration test on asphalt binder following ASTM D 946. In this test, a standard needle penetrates an asphalt binder specimen when placed under a 100-g (0.22-lb) load for 5 s as shown in Figure 5.

Figure 5 Asphalt Penetration Testing

The test does not measure any fundamental parameter and characterizes asphalt binder only at 77°F. Penetration grades are classified as a range of penetration units (one penetration unit = 0.1 mm) as 40–50 if the penetration ranges from 4 to 5 mm. Four other types of penetration grading are 60–70, 85–100, 120–150, and 200–300 as listed in Table 1. The higher is the penetration, the softer is the asphalt binder. Thus, the 200–300 pen grade is the softest grade and the 40–50 pen grade is the hardest grade. Typical asphalt binders used in the United States are 65–70 pen grade and 85–100 pen grade. The penetration requirements of different penetrating grading of binder are listed in Table 1.

Penetration grade	Penetration at 77°F (25°C)	
	Minimum penetration	Maximum penetration
40–50	40	50
60–70	60	70
85–100	85	100
120–150	120	150
200–300	200	300

Table 1 Penetration grading for Asphalt Binder

Viscosity Grading

Viscosity grading is a better grading system than penetration grading, but it does not test asphalt binder rheology at low temperatures. This grading system is based on the absolute viscosity of virgin binder and rolling thin-film oven (RTFO) test aged binder. RTFO test simulates the effects of short-term aging during mixing and compaction of the asphalt mixture by heating the asphalt binder film in an oven at 163°C (325°F) for 5 hours. The absolute viscosity or simply viscosity is defined as the resistance to flow of a fluid. The absolute viscosity test is discussed later in this section.

Grading on virgin binder is denoted by two letters (AC) and a number. AC means asphalt cement and the numerical value indicates viscosity at 60°C (140°F) in hundreds of poises. Both ASTM D 3381 and AASHTO M 226 use this procedure. Table 2 lists the AC grading for virgin binder where the AC grades are listed in hundreds of poises (cm-g-s = dyne-s/cm^2). The less is the number of poises, the lower is the viscosity and thus the easier is the flow of a substance. Thus, AC-5 [viscosity is 500 ± 100 poise at 60°C (140°F)] is less viscous than AC-40 [viscosity is 4,000 ± 800 poise at 60°C (140°F)].

Grading on RTFO aged binder is denoted by the two-letter, AR grading and a number. AR means asphalt residue and the numerical value indicates viscosity at 60°C (140°F) in hundreds of poises by AASHTO M 226 (ASTM D 3381 uses the viscosity in poise) as listed in Table 2. For example, according to AASHTO M 226, AR-40 means the RTFO aged binder has an absolute viscosity of 4,000 poise. ASTM D 3381 expresses this binder as AR-4000. Common asphalt binders used in the United States are AC-10, AC-20, AC-30, AR-4000, and AR-8000.

AASHTO M 226	ASTM D 3381	Absolute viscosity at 60°C (140°F) (poises)
Grading Based on Original Asphalt (AC)		
AC-2.5	AC-2.5	250 ± 50
AC-5	AC-5	500 ± 100
AC-10	AC-10	1,000 ± 200
AC-20	AC-20	2,000 ± 400
AC-30	AC-30	3,000 ± 600
AC-40	AC-40	4,000 ± 800
Grading Based on Aged Residue (AR)		
AR-10	AR-1000	1,000 ± 200
AR-20	AR-2000	2,000 ± 400
AR-40	AR-4000	4,000 ± 800
AR-80	AR-8000	8,000 ± 1,600
AR-160	AR-16000	16,000 ± 3,200

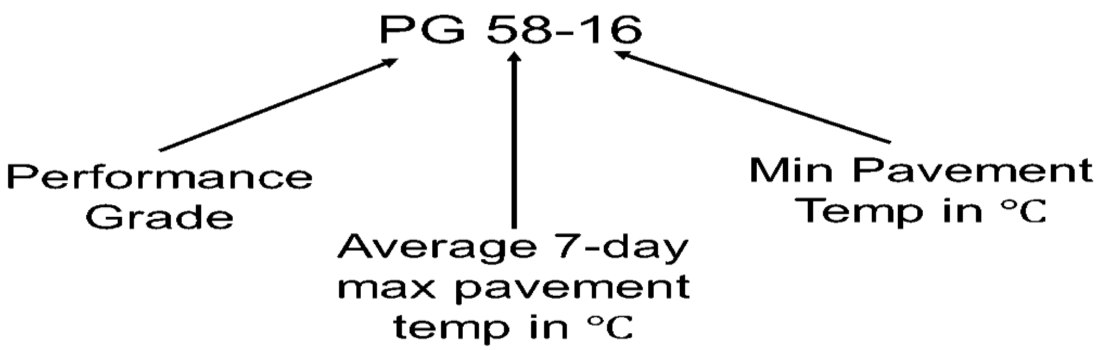
Table 2 Standard Viscosity Graded Binder

Performance Grading (PG)

As part of the Superpave research effort, the PG system was developed to more accurately and fully characterize asphalt binders for use in asphalt pavements. This method tests asphalt binder at an upper performance temperature and at a lower performance temperature. This method also tests virgin asphalt, RTFO aged asphalt residue, and pressure-aging vessel (PAV) aged asphalt residue. The PAV provides simulated long-term aged asphalt binder for in-service aging over a period of 7 to 10 years. The basic PAV procedure takes RTFO aged asphalt binder specimens, places them in stainless steel pans, and then ages them for 20 hours in a heated vessel pressurized to 305 psi (2.10 MPa) at 90°C or 100°C.

The PG system is based on the idea that the properties of an asphalt binder should be related to the conditions under which it is used. For asphalt binders, this involves expected climatic conditions as well as aging considerations. Therefore, the PG system uses a common battery of tests (as the older penetration and viscosity grading systems do) but specifies that an asphalt binder must pass these tests at specific temperatures that are dependent upon the specific climatic conditions in the area of intended use. Therefore, a binder used in Colorado would be different than a binder used in, say, New York.

Superpave performance grading is reported using two numbers: the first being the average 7-day maximum pavement temperature in degrees Celsius and the second being the single-day minimum pavement design temperature likely to be experienced in degrees Celsius. Thus, a PG 58-16 is intended for use where the average 7-day maximum pavement temperature is 58°C and the expected single-day minimum pavement temperature is −16°C. Notice that these numbers are pavement temperatures and not air temperatures. The PG system requirement is listed in Table 3.



Performance Grade (PG)	PG 52							PG 58					PG 64				
	10	-16	-22	-28	-34	-40	-46	-16	-22	-28	-34	-40	-16	-22	-28	-34	-40
Average 7-Day Maximum Pavement Design Temperature, °C	<52							<58					<64				
Minimum Pavement Design Temperature, °C	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-16	>-22	>-28	>-34	>-40
Original Binder																	
Flash Point Temp, T48: Minimum °C	230																
Viscosity, ASTM D 4402: Maximum, 3 Pa-s (3,000 cP), Test Temp, °C	135																
Dynamic Shear, TP5: G*/sinδ, Minimum, 1.00 kPa Test Temperature @ 10 rad/sec, °C	52							58					64				
Rolling Thin Film Oven (T 240) or Thin Film Oven (T 179) Residue																	
Mass Loss, Maximum, %	1.00																
Dynamic Shear, TP5: G*/sinδ, Minimum, 2.20 kPa Test Temp @ 10 rad/sec, °C	52							58					64				

Pressure Aging Vessel (PAV) Residue																	
PAV Aging Temperature, °C	90							100					100				
Dynamic Shear, TP5: G*/sinδ, Minimum, 5,000 kPa Test Temp @ 10 rad/sec, °C	25	22	19	16	13	10	7	25	22	19	16	13	28	25	22	19	16
Physical Hardening	Report																
Creep Stiffness, TP1: S, Maximum, 300 MPa m-value, Minimum, 0.300 Test Temp, @ 60 sec, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30
Direct Tension, TP3: Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-36	-12	-18	-24	-30

Table 3 Performance Grading (PG) Binder System

Example 1: PG Binder Selection

In a region, the average 7-day maximum pavement design temperature is 50°C, and the minimum pavement design temperature is −12°C. Determine the recommended asphalt binder for this region.

Solution

From Table 3, for the average 7-day maximum pavement design temperature less than 52°C, PG 52 is recommended. Then, for the minimum pavement design temperature −12°C, the second number is −16 (as −12 > −16). Therefore, the PG 52-16 binder is recommended.

Example 2: Test Temperature

For the asphalt binder PG 64-28, determine the recommended design temperature for the direct tension test of the binder.

Solution

From Table 3, the recommended design temperature for PG 64-28 binder for direct tension test is −18°C.

Other Tests on Asphalt Binders

The following tests are required for the PG system.

Temperature

The pavement surface temperatures for the pavement site can be determined using the LTPPBind 3.0/3.1 software. The software is available at the FHWA website.

Rolling Thin-Film Oven Test

The rolling thin-film oven (RTFO) procedure following the AASHTO T 240 and ASTM D 28723 provides simulated short-term aged asphalt binder for physical property testing. This test simulates the aging during mixing and placement. It also provides a quantitative measure of the volatiles lost during the aging process. This test takes virgin (unaged) asphalt binder specimens in cylindrical glass bottles and places these bottles in a rotating carriage within an oven (Figure 6). The carriage rotates within the oven while the 325°F (163°C) temperature ages the specimens for 85 minutes.

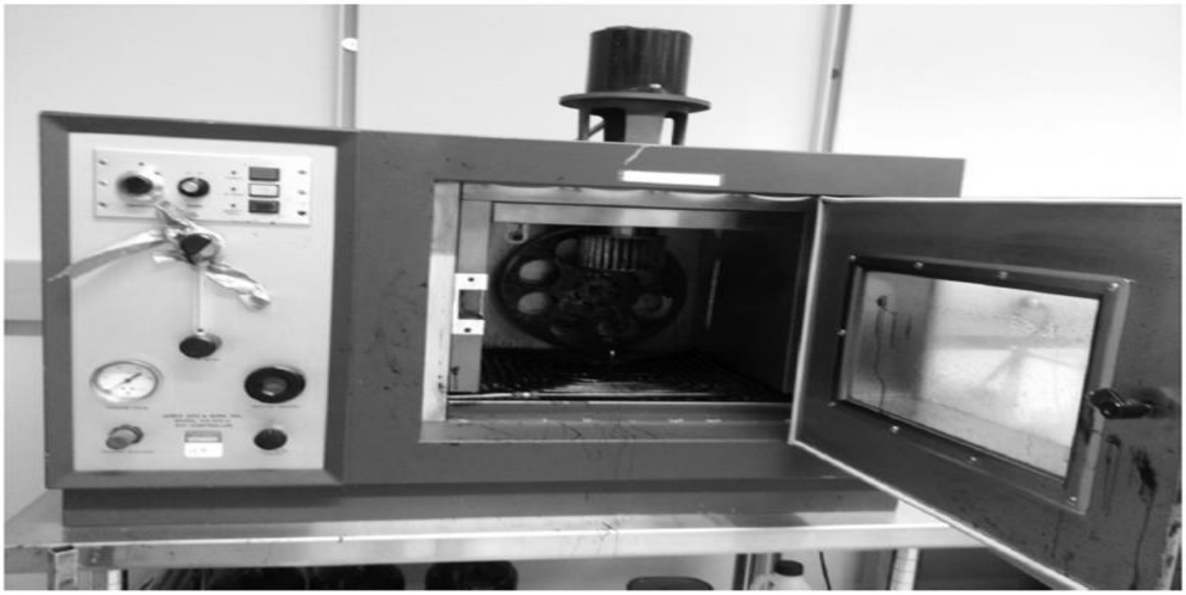


Figure 6 Rolling Thin-Film Oven Test Apparatus

Pressure Aging Vessel (PAV)

The pressure aging vessel (PAV) provides simulated long-term aged asphalt binder for physical property testing. This test simulates the aging that occurs during in-service life. The asphalt binder is exposed to heat and pressure to simulate in-service aging over a span of 7 to 10 years. The basic PAV procedure takes RTFO aged asphalt binder specimens, places them in stainless steel pans, and then ages them for 20 hours in a heated vessel pressurized to 305 psi (2.10 MPa or 20.7 atmospheres) at 90°C or 100°C as shown in Figure 7. The specimens are then stored for use in physical property tests. The standard PAV procedure is AASHTO R 28.

Figure 7 Pressure Aging Vessel Test Apparatus

Dynamic Shear Test

The dynamic shear rheometer (DSR) is used to characterize the viscous and elastic behavior of asphalt binders at medium to high temperature. This characterization is used in the Superpave PG asphalt binder specification. The actual temperatures anticipated in the area where the asphalt binder will be placed determine the test temperatures used. The standard dynamic shear rheometer test is AASHTO T 315. The DSR measures a specimen's complex shear modulus (G^*) and phase angle (δ). The complex shear modulus (G^*) can be considered the specimen's total resistance to deformation when repeatedly sheared, while the phase angle (δ) is the lag between the applied shear stress and the resulting shear strain. The larger is the phase angle (δ), the more viscous is the material. A zero-degree phase angle means an elastic material, whereas a 90-degree phase angle means a pure viscous material. Asphalt material is in between these two values. The shear modulus (G^*) is defined mathematically as the ratio of peak shear stress (τ_o) and the peak recoverable shear strain (γ_o), which is presented by the equation:

$$|E^*| = \frac{\text{Peak stress}}{\text{Peak strain}} = \frac{\sigma_o}{\varepsilon_o}$$

where $|E^*|$ = Dynamic modulus
σ_o = Peak dynamic stress, applied by loading frame
ε_o = Peak recoverable axial strain, measured upon loading

A small specimen of asphalt binder is sandwiched between two plates shown in Figure 8. The test specimen is kept at near constant temperature as desired. The top plate oscillates at 10 rad/s (1.59 Hz) in a sinusoidal waveform while the equipment measures the maximum applied shear stress, the resulting maximum shear strain, and the time lag between them. The software then automatically calculates the complex modulus (G*) and phase angle (δ) as shown in Figure 9.

To prepare the test specimen, cylindrical specimens of 6 inches (150 mm) in diameter and 7 inches (170 mm) in height are compacted using the gyratory compactor following the AASHTO T 312 test standard. The compacted specimens are then cored and sawed to a diameter of 4 inches (100 mm) and height of 6 inches (150 mm) as shown in Figure 10. Air voids of the finished specimens should be within the design limits, commonly between 4% and 6%.

Figure 8 Dynamic Shear Rheometer Testing

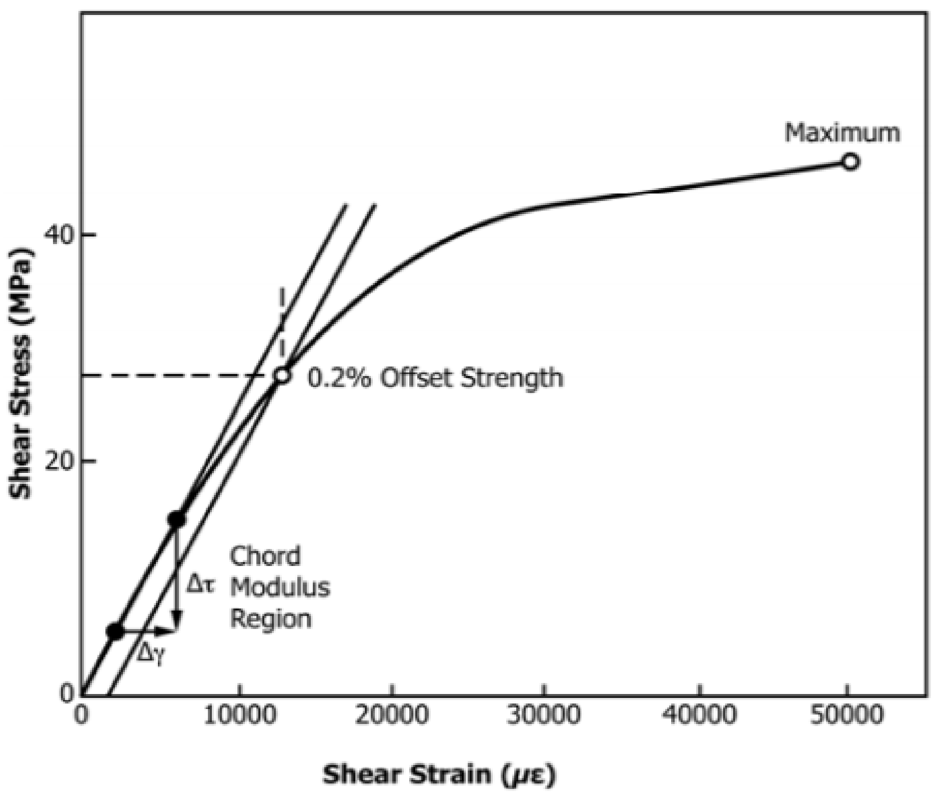


Figure 9 Dynamic Shear Rheometer Testing Results

(a) Cored specimen

(b) Edge cut specimen

Figure 10 Specimen Preparation for Dynamic Modulus Testing

Two empirical equations can be used to determine the dynamic modulus |E*|

The viscosity-based model:

$$\log\left|E^*\right| = 3.750063 + 0.02932\,\rho_{200} - 0.001767\,(\rho_{200})^2 - 0.002841\,\rho_4 - 0.058097\,V_a - 0.802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{3.871977 - 0.0021\,\rho_4 + 0.003958\,\rho_{38} - 0.000017(\rho_{38})^2 + 0.00547\,\rho_{34}}{1 + e^{(-0.603313 - 0.313351\log(f_r) - 0.393532\log(\eta))}}$$

where $|E^*|$ = Dynamic modulus of asphalt concrete, psi

ρ_{34} = Cumulative % retained on the ¾ in. sieve

ρ_{38} = Cumulative % retained on the ⅜ in. sieve

ρ_4 = Cumulative % retained on the No. 4 sieve

ρ_{200} = Passing through the No. 200 sieve, %

η = Viscosity of binder at the temperature of interest, 10^6 poise

V_{beff} = Effective binder content, % by volume

V_a = Air void content, %

f_r = Reduced frequency, Hz

and the shear-based model:

$$\log E^{*} = -0.349 + 0.754\left(\left|G_b^{*}\right|^{-0.0052}\right) \times \begin{pmatrix} 6.65 - 0.032\rho_{200} + 0.0027(\rho_{200})^2 + 0.011\rho_4 \\ -0.0001(\rho_4)^2 + 0.006\rho_{38} - 0.00014(\rho_{38})^2 - 0.08V_a \\ -1.06\left(\dfrac{V_{beff}}{V_a + V_{beff}}\right) \end{pmatrix}$$

$$+ \frac{2.56 + 0.03V_a + 0.71\left(\dfrac{V_{beff}}{V_a + V_{beff}}\right) + 0.012\rho_{38} - 0.0001(\rho_{38})^2 - 0.01\rho_{34}}{1 + e^{(-0.7814 - 0.5785\log\left|G_b^{*}\right| + 0.8834\log\delta_b)}}$$

where E^* = Dynamic modulus of asphalt concrete, psi
ρ_{200} = Aggregates (by weight of the total aggregates) passing through No. 200 sieve, %
ρ_{34} = Cumulative % retained on the ¾ in. sieve
ρ_{38} = Cumulative % retained on the ⅜ in. sieve
ρ_4 = Cumulative % retained on the No. 4 sieve
V_{beff} = Effective binder content, % by volume
V_a = Air void content, %
δ_b = Phase angle of binder associated with $|G^*|$, degree
$|G^*|$ = Dynamic shear modulus of binder, psi

Example 3: Dynamic Modulus

In a dynamic modulus test of AC specimen, a sinusoidal stress of 390sin(ωt) is applied and a sinusoidal strain of 0.001sin(ωt+80) is obtained; where the stress is in psi, *ω* is the angular frequency, and *t* is the time. What is the dynamic modulus of the asphalt specimen?

Solution

$$|E^*| = \frac{\text{Peak stress}}{\text{Peak strain}} = \frac{\sigma_0}{\varepsilon_0} = \frac{390\text{ psi}}{0.001} = 390{,}000\text{ psi}$$

Answer *The dynamic modulus of the asphalt specimen is 390 ksi.*

Bending Beam Rheometer (BBR) Creep Stiffness

Creep stiffness means the stiffness or modulus measured using creep (sustained) loading. Creep is defined as the deformation in a material to the point of sudden fracture or its loss of usefulness due to the need to support a load for an extended period (Hibbeler 2017). The BBR test provides a measure of low-temperature creep stiffness and relaxation properties of asphalt binders. These parameters give an indication of an asphalt binder's ability to resist low-temperature cracking.

According to the ASTM D 6648 or AASHTO T 313 standard, an asphalt beam of 4.0 in. (102 mm) long, 0.5 in. (12.5 mm) wide, and 0.25 in. (6.25 mm) high is prepared by pouring the heated binder in a mold. After cooling, the beam is then kept in the test bath for an hour. Then, a load of 100 g (980 mN) is applied at the center of the beam for a total of 240 s, as shown in Figure 11. The deflection of the beam is recorded during this loading period. Using the classical strength of materials equation for a center-point loaded beam, the stiffness after 60 s of loading is calculated as:

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

where $S(t)$ = Creep stiffness at time t, the standard is 60 s
P = Applied constant load, the standard is 100 g (980 mN)
L = Beam span, the standard is 4.0 in. (102 mm)
b = Beam width, the standard is 0.5 in. (12.5 mm)
h = Beam height, the standard is 0.25 in. (6.25 mm)
$\delta(t)$ = Deflection at time $t = 60$ s

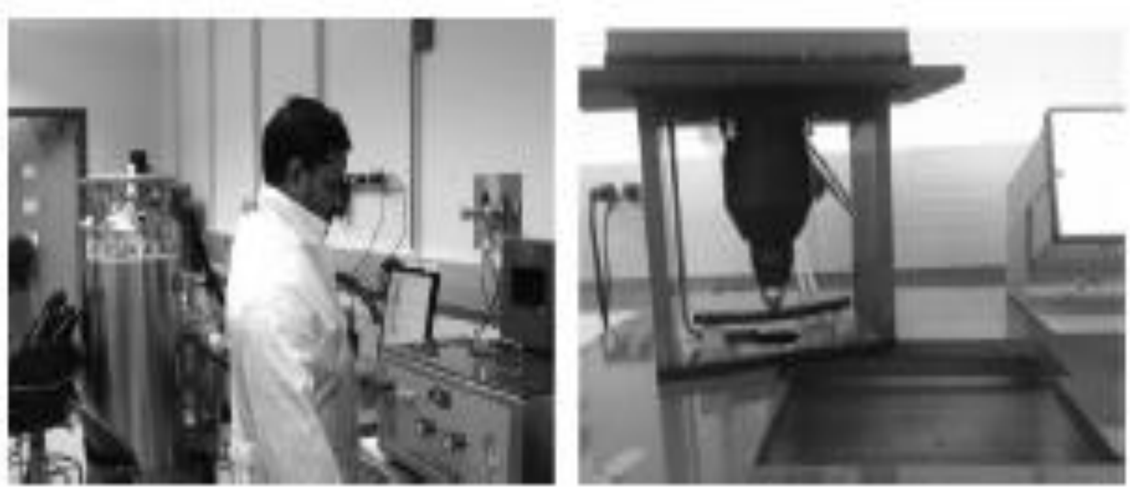


Figure 11 Bending Beam Rheometer Testing

The *m*-value is the slope of "Log*S* versus Log-time" curve at 60 s of loading. The *m*-value indicates the rate of change of stiffness with loading time (Figure 12). The PG binder specification requires the *m*-value to be equal or greater than 0.30 at 60 s of loading (McGennis et al., 1994).

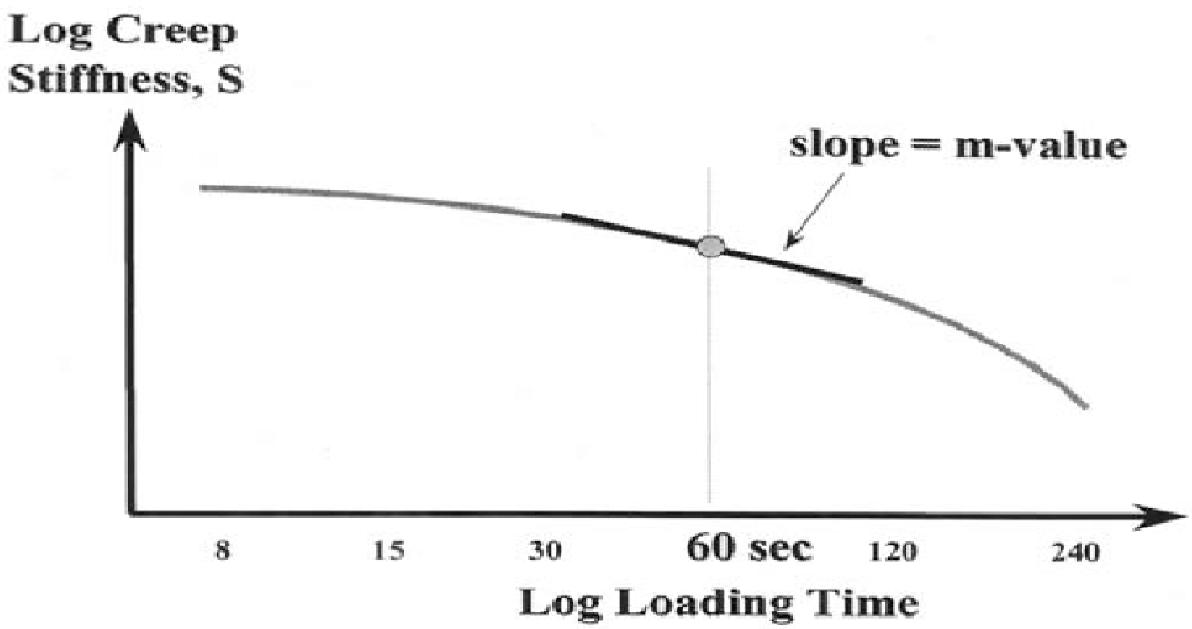


Figure 12 Creep Stiffness vs. Loading Time

Example 4: Creep Stiffness

In a BBR testing, an asphalt beam 12.5 mm wide, 6.25 mm high, and 102 mm long is used. After applying a load of 100 g for 60 s, a deflection of 0.9545 mm is recorded. Calculate the creep stiffness of the specimen.

Solution

P = Applied constant load, 100 g = 0.1 kg (9.81 m/s^2) = 0.981 N

L = Beam span = 102 mm = 0.102 m

b = Beam width = 12.5 mm = 0.0125 m

h = Beam height = 6.25 mm = 0.00625 m

$\delta(t)$ = Deflection 60 s = 0.9545 mm = 0.0009545 m

$$\begin{aligned}\text{Creep stiffness, } S(t) &= \frac{PL^3}{4bh^3\delta(t)} = \frac{0.981\,\mathrm{N}\,(0.102\,\mathrm{m})^3}{4(0.0125\,\mathrm{m})(0.00625\,\mathrm{m})^3(0.0009545\,\mathrm{m})} \\ &= 89{,}347{,}732\ \mathrm{Pa} \;=\; 89.3\ \mathrm{MPa}\end{aligned}$$

Answer *The creep stiffness is 89.3 MPa.*

Creep compliance is defined as the time-dependent strain divided by the applied stress. This test is conducted following the AASHTO T 322-07 test protocol on compacted cylindrical HMA specimen with a diameter of 6 in. (150 mm) and a thickness of 1.5 to 2 in. (38–50 mm). A static load is imposed along a diametric axis of the temperature-controlled specimen for 100 seconds, as shown in Figure 13. During the loading period, vertical and horizontal deformations are measured on the two parallel faces of the specimen using four extensometers. Using these displacements (or strains) and the applied stress, creep compliance can be calculated.

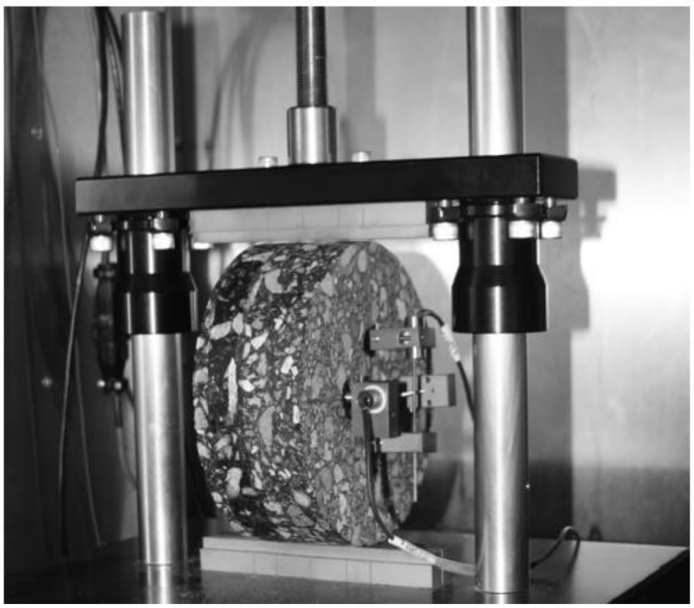


Figure 13 Creep Compliance Test Setup

Direct Tension Test (DTT)

The direct tension test (DTT), which measures the tensile strength of asphalt binder at a critical cracking temperature, is conducted following the AASHTO T 314 test protocol at different temperatures (commonly 10°C higher than the low temperature grade of the binder). The effective length of the specimen is slightly more than 1.5 in. (38 mm) and the effective area of the cross section of the specimen is about 1.5 in. (38 mm) square. The specimen is pulled at a constant strain rate of 3% per minute. The variations of stress and strain are recorded. The DTT setup is shown in Figure 14.

Indirect Tensile Strength Test (ITS)

Indirect tensile strength (ITS) is an important input parameter in the transverse cracking model used in the AASHTOWare pavement ME design guide. Due to decrease in temperature, AC contracts and tensile stress develops in the asphalt mixture. Once the developed tensile stress exceeds the tensile capacity of asphalt mixture, thermal cracks develop. Testing an asphalt specimen in tension similar to a steel rod is quite impossible. Thus, tensile test is conducted with ease in indirect mode and is called the ITS. It is a measure of resistance capacity to low-temperature cracking of asphalt pavement (Islam et al.,

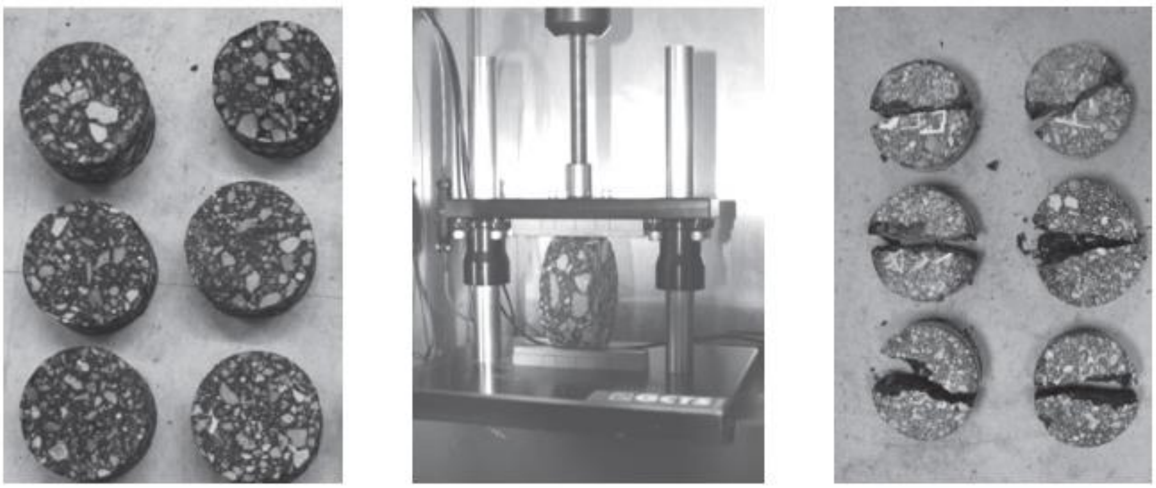
Figure 14 Direct Tension Test Setup

2015b). This value is usually calculated in the laboratory using indirect tensile (IDT) specimens and used in the design guide as an input parameter.

To prepare specimen, cylindrical specimens of 7-in. height and 6-in. diameter are prepared as a first step, using a Superpave gyratory compactor following AASHTO T 312 test standard. The specimens are then cut into thin circular specimens of 4-in. diameter and 2-in. thickness. The ITS of asphalt specimens at different temperatures are determined using an IDT test device in an environmental chamber, as shown in Figure 15. Uniform compressive load of 50 mm/min was applied until failure. The peak load was recorded to determine the ITS of the asphalt specimens. The ITS is calculated using the equation:

$$\text{ITS} = \frac{2P}{\pi D t}$$

where ITS = Indirect tensile strength, psi
P = Peak load required to crack the specimen diagonally, lb
D = Diameter of the specimen, in.
t = Length or thickness of the specimen, in.



Prepared specimens Test setup Few failed specimens

Figure 15 Indirect Tensile Strength Test

Example 5: Indirect Tensile Strength Test

In an ITS test, the peak load required to fail a specimen of 4.2-in. diameter and 2.25-in. thickness is 3,200 lb. Calculate the ITS of the specimen.

Solution

Given:

Peak load, P = 3,200 lb

Diameter, D = 4.2 in.

Thickness, t = 2.25 in.

$$\text{Indirect tensile strength, ITS} = \frac{2P}{\pi D t} = \frac{2(3{,}200\,\text{lb})}{\pi(4.2\,\text{in.})(2.25\,\text{in.})} = 216\,\text{psi}$$

Answer *The indirect tensile strength is 216 psi.*

Absolute Viscosity (Dynamic Viscosity)

The dynamic viscosity (also known as absolute viscosity) is the measurement of the fluid's internal resistance to the flow, while kinematic viscosity refers to the ratio of the dynamic viscosity to the density. Asphalt flow value is a (unit: mm) measure of the flexibility of the asphalt mix. It indicates [during testing] the change in diameter of the sample in the direction of the load application between the start of loading and at the time of maximum load. The more the flow value, the more is the flexibility (i.e. the lesser the stiffness of asphalt).

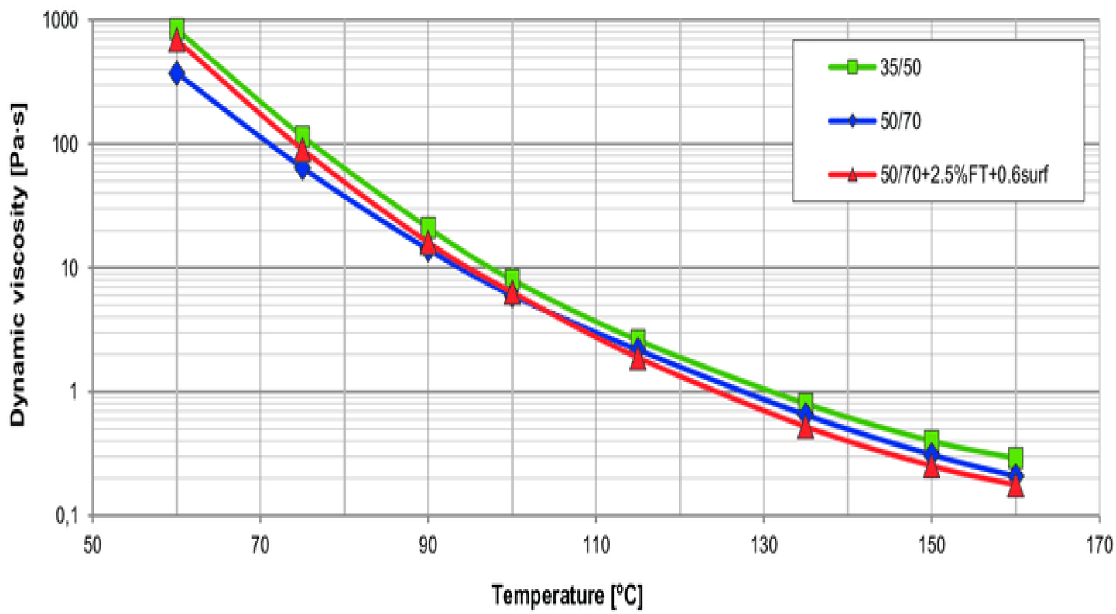
The basic absolute viscosity test (ASTM D 2171 and AASHTO T 202) measures the time it takes for a fixed volume of asphalt binder to be drawn up through a capillary tube as shown in Figure 16 and typical results are shown in Figure 17.

Figure 16 Capillary Viscometer

Asphalt drawing is carried out by means of closely controlled vacuum suction conditions at 60°C. This temperature is selected because it is approximately the maximum asphalt pavement surface temperature during summer in most areas. The viscometer is a U-tube with a reservoir where the asphalt is introduced and a section with a calibrated diameter and timing marks.

Vacuum is applied at one end and the time during which the asphalt flows between two timing marks on the viscometer is measured. The flow time in seconds is multiplied by the calibration factor of the viscometer in order to obtain the absolute viscosity in poises.

Although the absolute viscosity is an improvement over the penetration test, it still only measures the viscosity at one temperature, and thus does not fully characterize an asphalt binder's consistency over the expected range of construction and service conditions.



Type of bitumen binder	Dynamic viscosity std. dev.							
	60°C	75°C	90°C	100°C	115°C	135°C	150°C	160°C
35/50	27.2386	4.0365	0.8001	0.2792	0.1001	0.0269	0.0156	0.0106
50/70	14.2821	2.0649	0.4967	0.1872	0.0736	0.0217	0.0113	0.0076
50/70 + 2.5%FT + 0.6surf	37.1371	5.3100	0.9984	0.4451	0.1073	0.0313	0.0188	0.0125

Figure 17 Dynamic Viscosity vs. Temperature

Kinematic Viscosity

The kinematic viscosity of a liquid is the absolute (or dynamic) viscosity divided by the liquid's density at the temperature of measurement. The 135°C (275°F) measurement temperature is selected to simulate the mixing and compaction temperatures commonly used during asphalt pavement construction.

The basic kinematic viscosity test (ASTM D 2170 and AASHTO T 201-03) measures the time it takes for a fixed volume of asphalt binder to flow through a Zeitfuchs Cross-Arm Viscometer under closely controlled conditions of head and temperature as shown in Figure 18 and typical results are shown in Figure 19. The kinematic viscosity in centistoke is obtained by multiplying the time taken by the calibration factor of the viscometer provided by the manufacturer (Brown et al., 2009). The absolute viscosity can be obtained from this kinematic viscosity by multiplying it by the density of asphalt binder as follows:

$$\textit{Absolute viscosity (poises)} = \textit{Kinematic viscosity (stokes)} \times \textit{Specific gravity}$$

Figure 18 Zeitfuchs Cross-Arm Viscometer

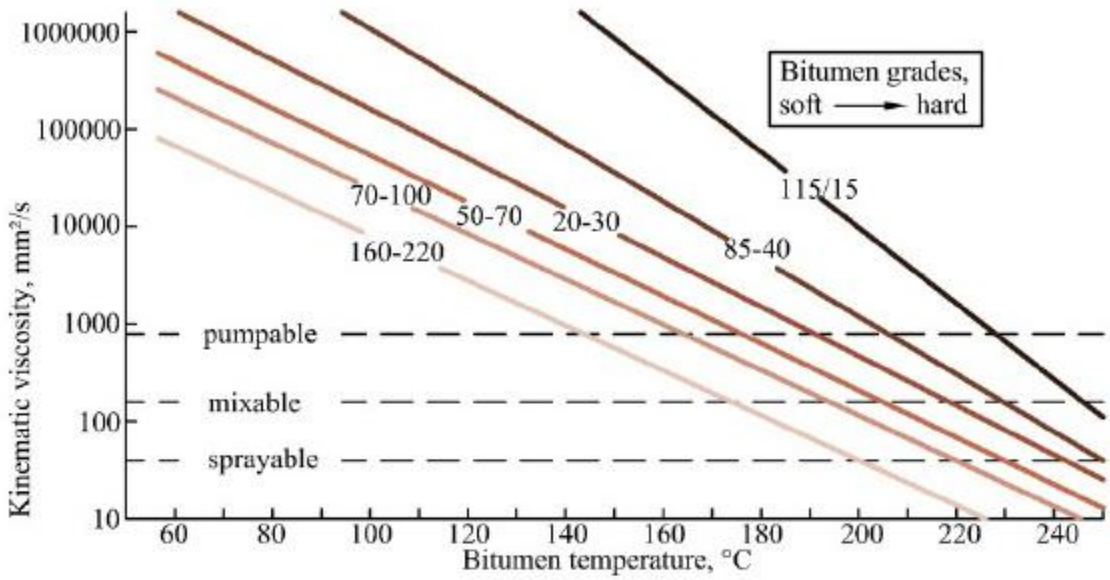


Figure 19 Kinematic Viscosity vs. Temperature

Example 6: Absolute Viscosity

An asphalt binder has a kinematic viscosity of 1,200 centistokes. If its specific gravity is 0.98, determine its absolute viscosity in poise.

Solution

1 stoke = 100 centistokes, therefore 1,200 centistokes = 12 stokes.

Absolute viscosity (poises) = kinematic viscosity (stokes) x specific gravity = 12 x 0.98 = 11.76 poises.

Answer *The absolute viscosity is 11.76 poises.*

Brookfield Viscosity

The Brookfield Dial Reading Viscometer measures the fluid viscosity at given shear rates. Brookfield viscometers employ the well-known principle of rotational viscometry; they measure the viscosity by sensing the torque required to rotate a spindle at constant speed while immersed in the sample fluid. The rotational viscometer (RV) or Brookfield viscometer is used to determine the viscosity of asphalt binders in the high-temperature range of manufacturing and construction (Figure 20).

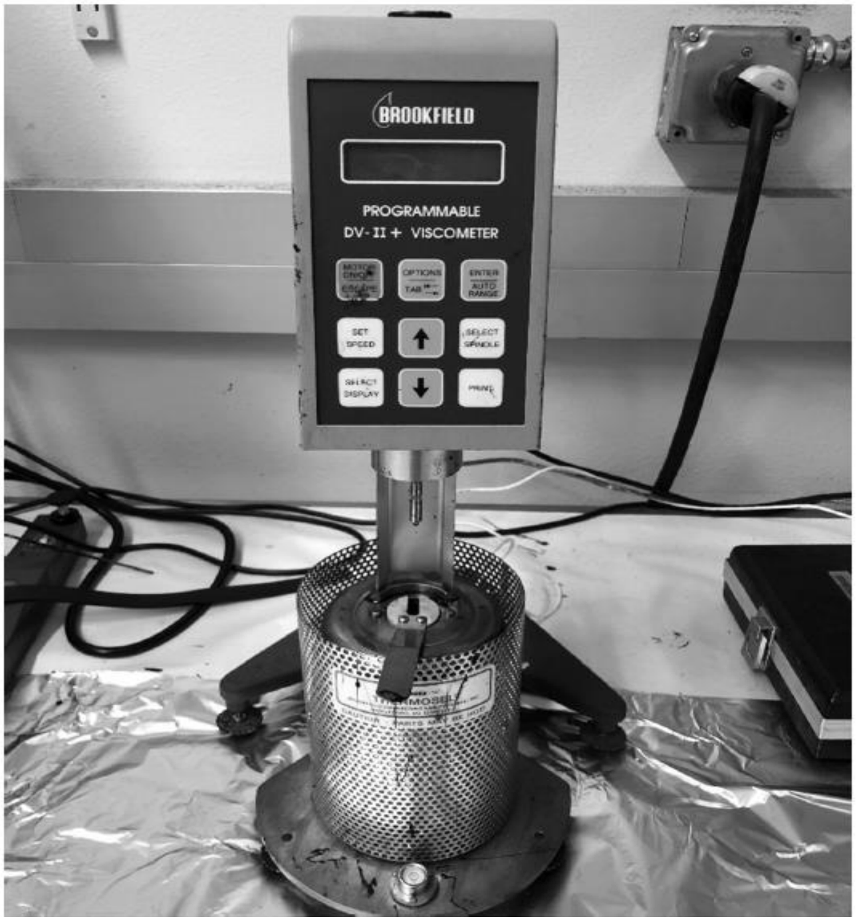


Figure 20 Rotational (Brookfield) Viscometer

The RV test can be conducted at various temperatures, but since manufacturing and construction temperatures are similar regardless of the environment, the test for Superpave PG asphalt binder specification is always conducted at 275°F (135°C). About 11 g of asphalt binder is poured into the chamber. The test measures the torque required to maintain a cylindrical spindle's rotational speed (20 rpm) while submerged in the asphalt binder. This torque is then converted to a viscosity and displayed automatically by the RV in the unit of centipoise (1,000 centipoise = 1 Pa·s). The standard test methods are ASTM D 4402 and AASHTO T 316.

Specific Gravity

Specific gravity is the ratio of the mass of a given volume of bitumen to the mass of an equal volume of water, both taken at a recorded/specified temperature. The primary aim of this test is, it can simply convert the material weight to volume and vice versa.

Since the specific gravity of the asphalt binder varies with temperature, specific gravity tests are useful in making volume corrections based on temperature. Refer to Figure 21. The specific gravity at 15.6°C (60°F) is commonly used when buying/selling asphalt cements. For good pavement construction, the range of specific gravity of the bitumen should be 0.97-1.10. A typical specific gravity for asphalt is around 1.03. The standard test method is AASHTO T 228-09 and ASTM D-70.

Figure 21 Measuring Specific Gravity

Ring and Ball Softening Point

The ring-and-ball softening point is defined as the temperature at which a disk of the sample held within a horizontal ring is forced downward a distance of 1 in. (25.4 mm) under the weight of a steel ball as the sample is heated at a prescribed rate in a water or glycerol bath.

The ring and ball test is used to determine the softening point of bitumen, asphalt, and coal tar. This test consists of two brass rings and two steel balls, using which the softening point of various bituminous materials are determined.

The ring and ball softening point is measured following the ASTM D 36 and AASHTO T 53 test standards. The temperature at which an asphalt binder cannot support the weight of a steel ball and starts to flow is known as the softening point.

Two horizontal disks of bitumen, cast in shouldered brass rings, are heated at a controlled rate in a liquid bath while each supports a steel ball.

The softening point is reported as the mean of the temperatures at which the two disks soften enough to allow each ball, enveloped in bitumen, to fall 1.0 in. (25 mm). Refer to Figures 22, 23, 24, and 25.

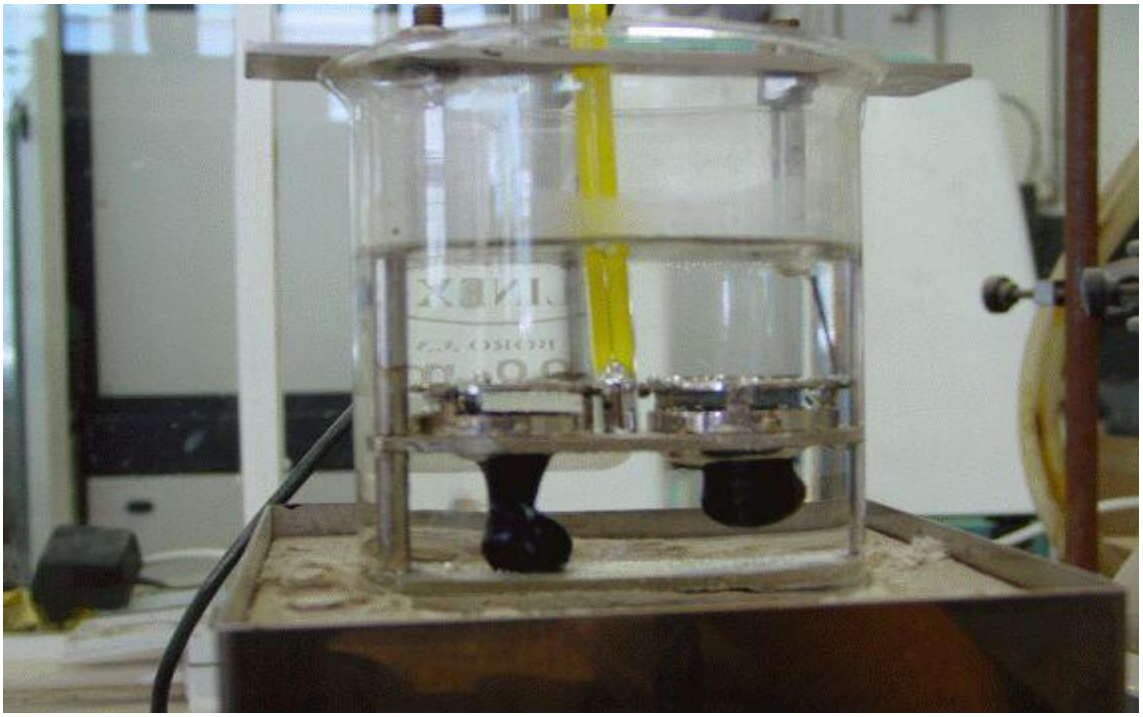


Figure 22 Apparatus Measuring Ring and Ball Softening Point

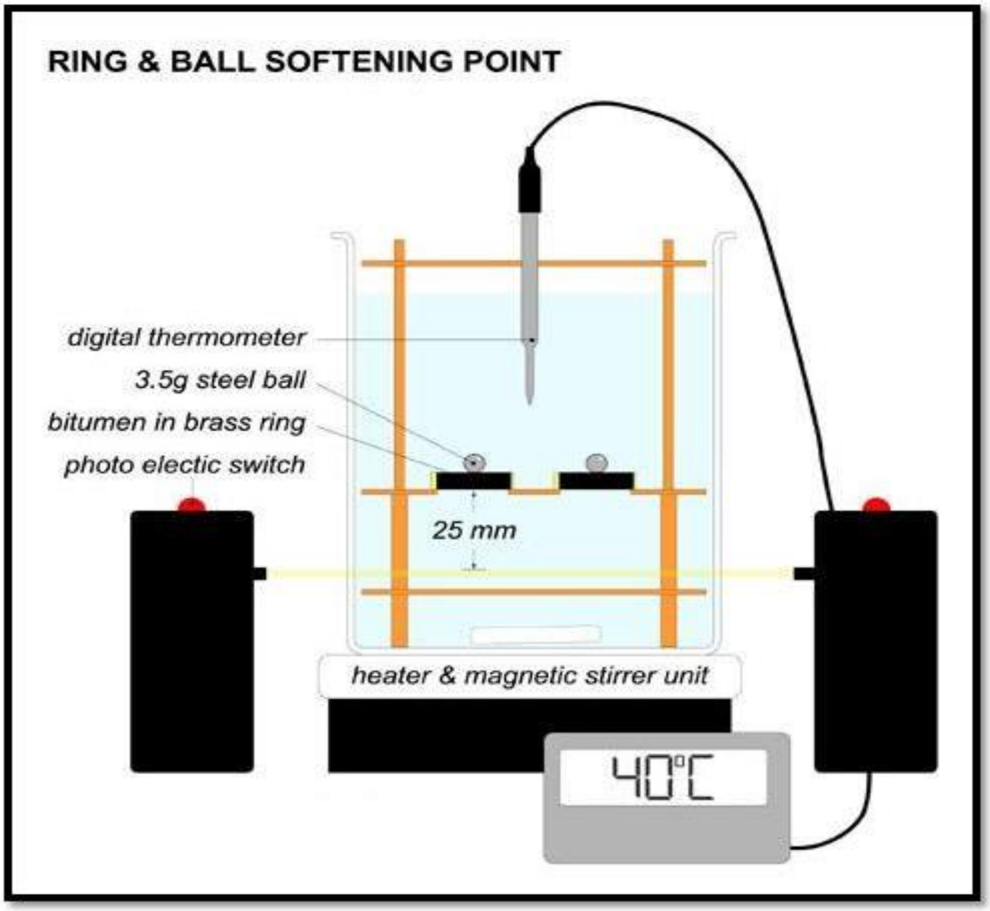


Figure 23 Apparatus Measuring Ring and Ball Softening Point (Detailed)

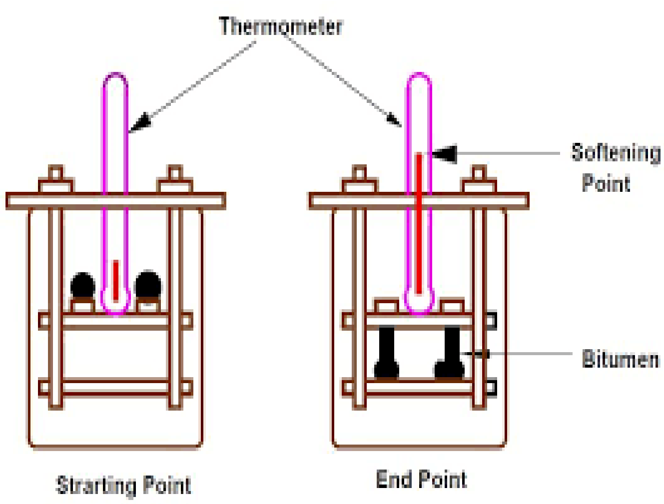


Figure 24 Measuring Ring and Ball Softening Point Procedure

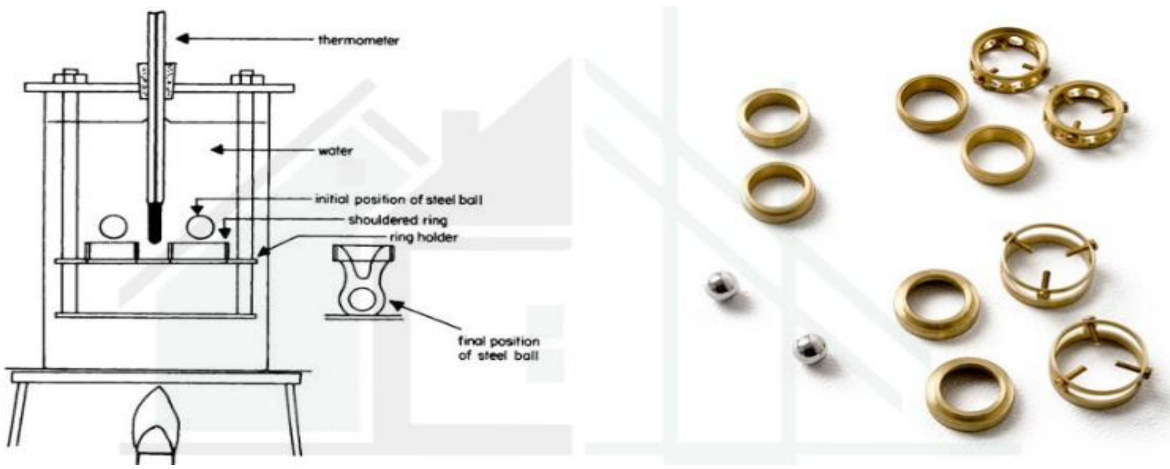


Figure 25 Measuring Ring and Ball Softening Point Procedure (Detailed)

Flash Point Temperature

A typical flash point test involves heating a small specimen of asphalt binder in a test cup. The temperature of the specimen is increased and at specified intervals a test flame is passed across the cup. The flash point is the lowest liquid temperature at which the test flame causes the specimen's vapors to ignite. Approximately 80 g of product is placed into a brass cup and heated at the rate of 5°C per minute. Every 2°C a flame is moved across the top of the cup until the flash point is reached. This test is used for cutback bitumens according to AASHTO T 79 and ASTM D92. Refer to Figure 26.

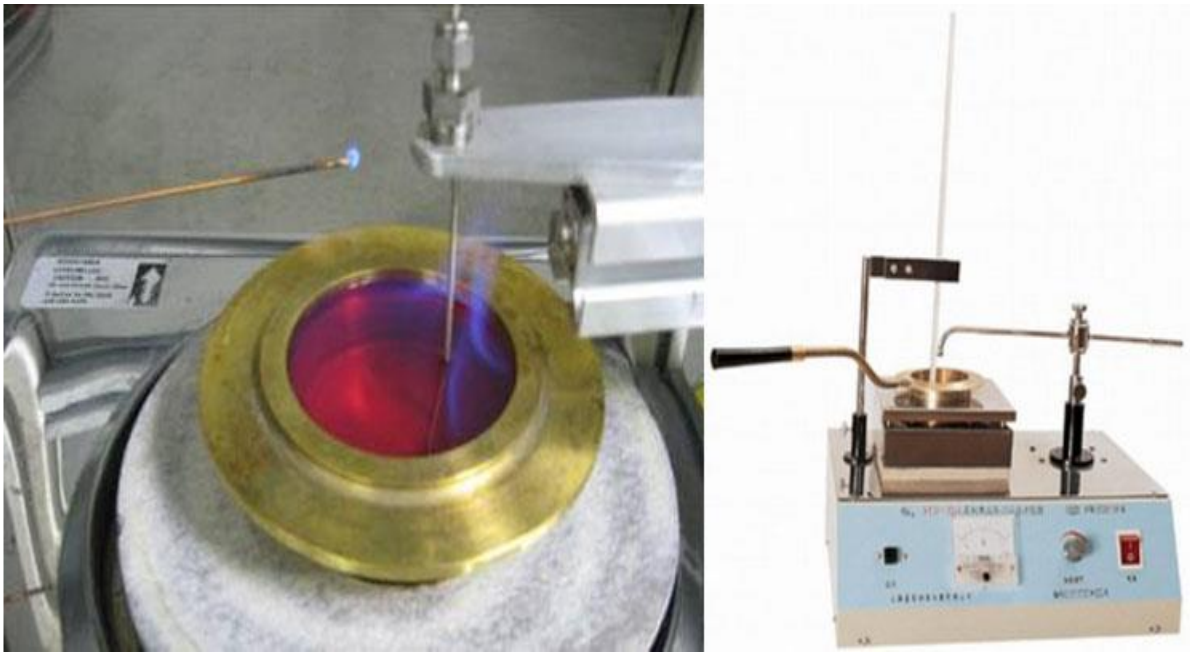


Figure 26 Measuring Flash Point Temperature

Ductility

The ductility test is measure of cohesiveness and elasticity of the bitumen. Bitumen is a binder of pavement construction, which binds from ductility thin films around the aggregates. This serves as satisfactory binders in improving the physical interlocking the aggregates. The ductility of a bituminous material is measured by the distance in centimeters to which it will elongate before breaking when two ends of standard briquette specimen of material are pulled apart at a specified speed and specified temperature. The ductility at 25°C (77°F) test measures asphalt binder ductility by stretching a standard-sized briquette of asphalt binder to its breaking point, according to AASHTO T 51 and ASTM D 113. The stretched distance at break in centimeters is then reported as ductility. Like the penetration test, this test has limited use since it is empirical and conducted only at 25°C (77°F). Refer to Figures 27 and 28.

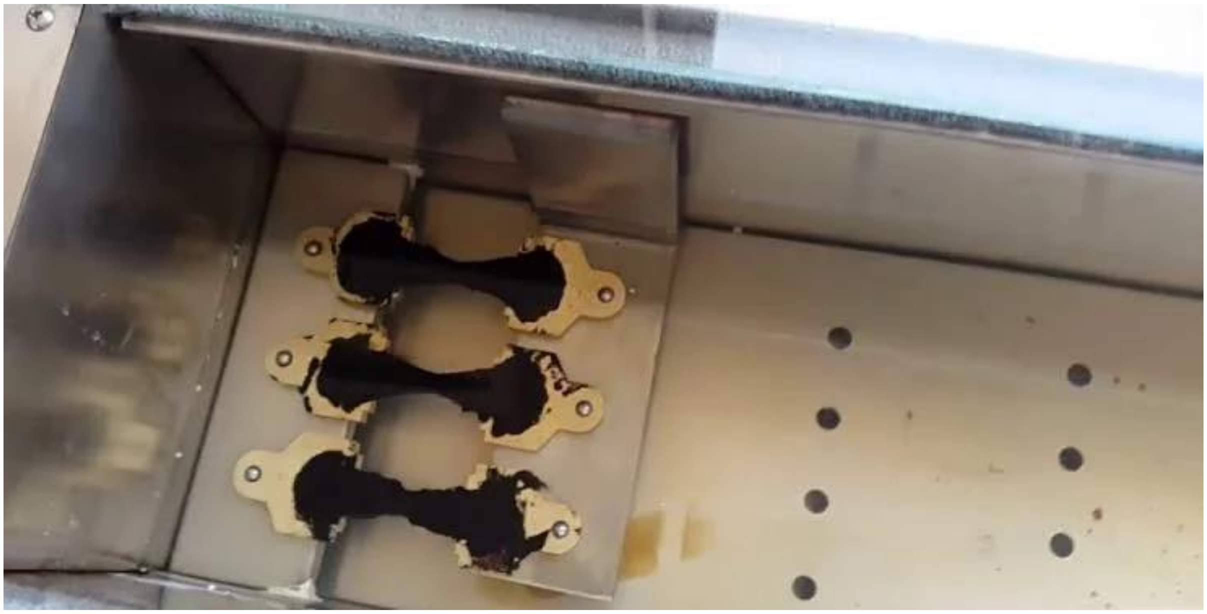


Figure 27 Measuring the Ductility

Figure 28 Bitumen Ductility Test

Solubility in Trichloroethylene

This test method covers the determination of the degree of solubility in trichloroethylene of asphalt materials having little or no mineral materials, according to AASHTO T 44 and ASTM D2042 (Figure 29). This method is not applicable to tars and their distillation residues or highly cracked petroleum products. For methods covering tars, pitches, and other highly cracked petroleum products, and the use of other solvents, see Test Methods D4, D2318, and D2764.

Figure 29 Measuring Solubility in Trichloroethylene

Asphalt cement, as used for asphalt paving, should consist of almost pure bitumen. Impurities are not active constituents of cement and may be detrimental to the asphalt cement performance. Mineral impurities can be quantified by dissolving a specimen of asphalt cement in trichloroethylene or 1,1,1 trichloroethane through a filter mat. Anything remaining on the mat is considered an impurity.

Fatigue Endurance Limit (FEL)

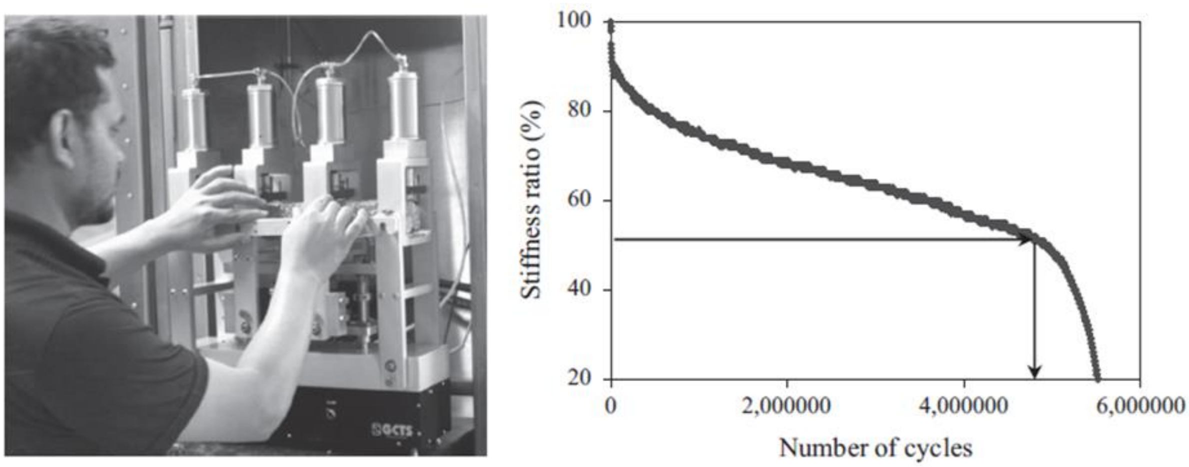
Fatigue endurance limit is an input parameter in the AASHTOWare pavement ME design software. Fatigue damage occurs in asphalt pavement due to repeated tensile strain at the bottom of AC under traffic loading (Islam, 2015). If the developed strain value is below the threshold value (called FEL), no fatigue damage occurs. Beam fatigue test is conducted according to the AASHTO T 321 standards. To prepare specimen, as a first step, beam slabs of 18 × 6 × 3 in. (450 × 150 × 75 mm) are prepared as shown in Figure 30 and then each slab is cut into two beams of 15 × 2.5 × 2 in. (375 × 63 × 50 mm) using a laboratory saw.

The specimen is clamped as shown in Figure 30 and loading is done using a sinusoidal waveform at a frequency of 5 or 10 Hz at a fixed temperature of 68°F (20°C). The test is conducted at different strain levels and different fatigue lives are obtained. The material is

(a) Compaction of mixture (b) Cutting the slabs (c) Prepared specimens

Figure 30 Beam Specimen Preparation

considered failed when the stiffness ratio decreases by 50%, as shown in Figure 31. This is because soon after decreasing the stiffness by 50%, microcracks form into macrocracks and the stiffness drops dramatically. Stiffness ratio is the ratio of the stiffness at current cycle of loading to the stiffness at the initial cycle of loading.



(a) Test setup (b) Test results

Figure 31 Beam Fatigue Test

The specimen's flexural stiffness (E) is calculated using the resulting maximum stress (σ_o) and the applied maximum strain (ε_o) data are recorded from each cycle, as depicted in the below equation:

$$E = \frac{\sigma_o}{\varepsilon_o}$$

The maximum strain and stress in the specimen are calculated using the below equations, respectively. These equations can be derived using the classical mechanics of materials principle.

$$\sigma_o = \frac{PL}{bh^2} \qquad \varepsilon_o = \frac{12h\delta}{3L^2 - 4a^2}$$

where ε_o = Maximum applied strain
σ_o = Maximum developed stress
P = Load applied by actuator at time t
b = Average specimen width (commonly 2.5 in.)
h = Average specimen height (commonly 2.0 in.)
δ = Deflection at center of beam at time t
a = Distance between inside clamps (commonly $L/3$)
L = Distance between outside clamps

Refer to Figure 32.

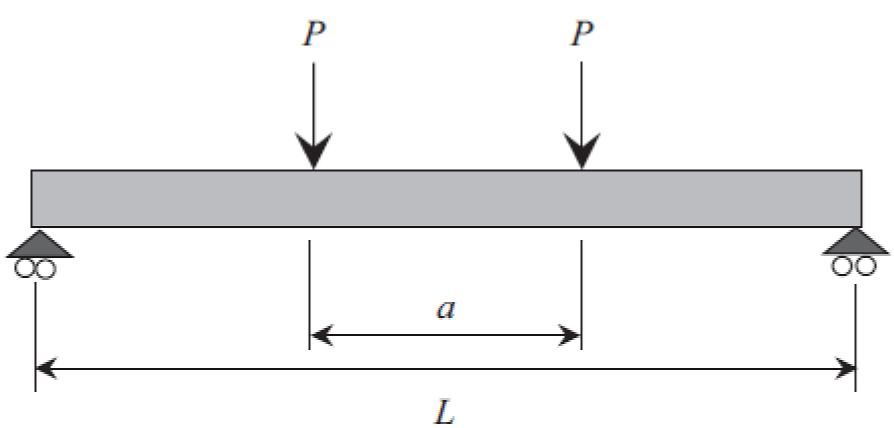


Figure 32 Schematic of Beam Fatigue Testing on Asphalt Beam

Then, a curve (shown in Figure 33) is drawn considering applied strain (ε) and fatigue life (N). From the regression model, the strain value (ε) is calculated at which the fatigue life (N) is 50 million. This strain value is referred to as FEL. The reason for choosing 50 million is that if a specimen can withstand 50 million load cycles in the laboratory, it is expected to be able to withstand more than that in field, which can be considered perpetual.

Creep is permanent deformation over time; fatigue is crack propagation over time. Both occur due to applied loads and both can lead to failure. Creep is characterized by looking at the elongation of the sample; fatigue, by elongation of the crack.

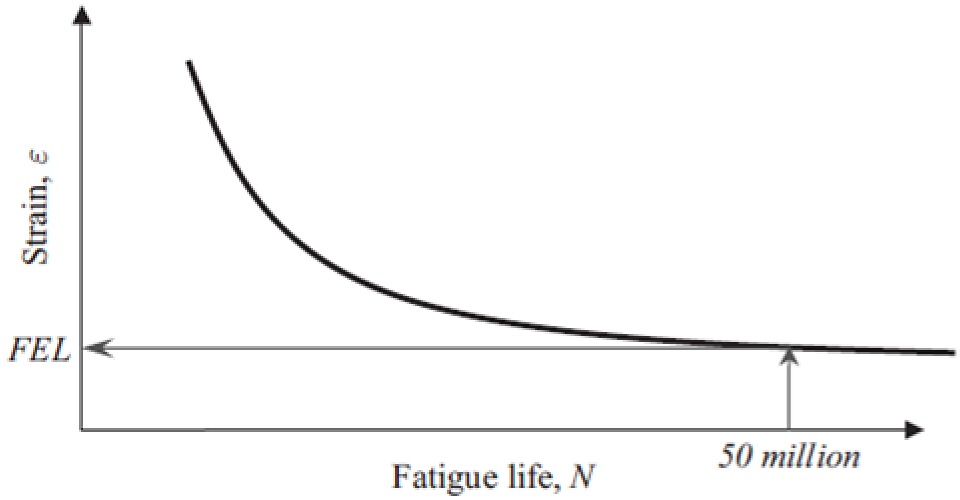


Figure 33 Determining FEL from ε-N Curve

Example 7: Determining the FEL

In a fatigue test of asphalt specimen, the data shown in Figure 34 are obtained:

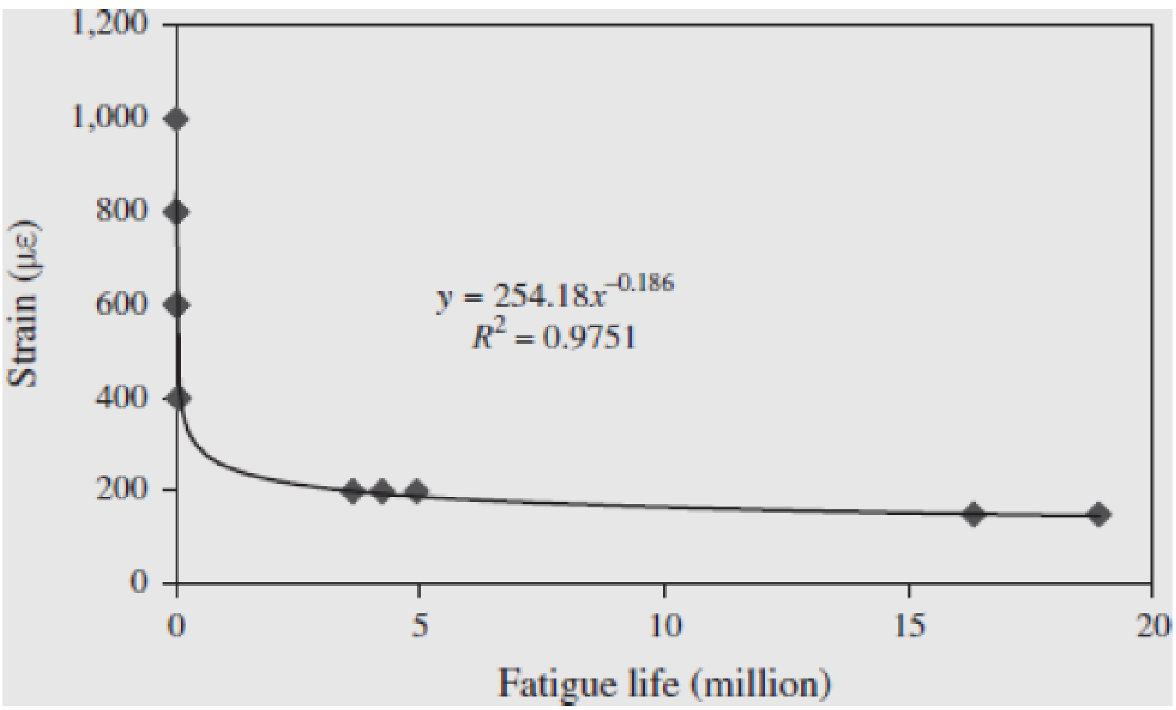


Figure 34 Fatigue Test Results for an Asphalt Mix

Determine the fatigue endurance limit (FEL) if the perpetual pavement takes 50 million cycles of load.

Solution

Obtained regression curve, $y = 254.18x^{-0.186}$

If x = 50 million, then $y = 254.18x^{-0.186} = 254.18(50)^{-0.186} = 123\ \mu\varepsilon$

Answer *The fatigue endurance limit is 123 microstrains.*

Poisson's Ratio Test

Poisson's ratio of AC can be determined by laboratory test or by using the AASHTOWare pavement ME design default relationship. It is an important property necessary to determine the lateral deformation of a material when it is loaded. Very often, a flat value of 0.35 is assumed, although Poisson's ratio varies with loading frequency and temperature according to equation below:

$$\mu = 0.15 + \frac{0.35}{1 + e^{a+b(E^*)}}$$

where the coefficients a and b are −1.63 and 3.84×10^{-6} respectively, and E^* is the dynamic modulus in psi. The above equation is recommended for new HMA mixes. For existing, age-hardened HMA layers, use the typical values listed in Table 4.

Temperature (°F)	Dense-graded HMA	Open-graded HMA
Less than 0	0.15	0.35
0–40	0.20	0.35
41–70	0.25	0.40
71–100	0.35	0.40
101–130	0.45	0.45
More than 130	0.48	0.45

Table 4 Common Poisson's Ratio of HMA

Example 8: Poisson's Ratio

The dynamic modulus of an asphalt mixture is 500 ksi. Calculate the Poisson's ratio.

Solution

Given:

E = 500,000 psi

Known:

$a = -1.63$ and

$b = 3.84 \times 10^{-6}$

$$\text{Poisson's ratio, } \mu = 0.15 + \frac{0.35}{1 + e^{a+b(E^*)}} = 0.15 + \frac{0.35}{1 + e^{-1.63+3.84x10^{-6}(500{,}000\,\text{psi})}} = 0.299$$

Answer *The Poisson's ratio is 0.30.*

Surface Shortwave Absorptivity

The surface shortwave absorptivity of pavements depends on the pavement's composition, color, and texture. It directly correlates with the amount of solar energy absorbed by pavement surface. The lower the value, the lower the solar energy is absorbed. The AASHTOWare pavement ME design default value is 0.85. No test standard is still available.

Thermal Conductivity

The thermal conductivity is defined as the rate at which heat passes through a specified material, expressed as the amount of heat that flows per unit time through a unit area with a temperature gradient of one degree per unit distance. It can be measured using the ASTM E 1952 test standard. If the test facility is not available, typical values for HMA ranging from 0.44 to 0.81 Btu/(ft)(h)(°F) can be used. The AASHTOWare pavement ME design default value set in program is 0.67 Btu/(ft)(h)(°F).

Heat Capacity

The heat capacity of a substance is the amount of heat required to change its temperature by one degree and has units of energy per degree. It can be measured using the ASTM D 2766 test standard. If the test facility is not available, typical values for HMA ranging from 0.22 to 0.40 Btu/(lb)(°F) can be used. The AASHTOWare pavement ME design default value set in program is 0.23 BTU/lb·°F.

Coefficient of Thermal Contraction (CTC)

The coefficient of thermal contraction (CTC) is defined as the fractional decrease in dimensions (x, y, z direction) due to unit change in temperature. There is no standard test standard yet but the AASHTOWare default value may be used. The AASHTOWare pavement ME design software calculates CTC internally using the mix volumetric properties such as proportions of aggregates according to equation below:

$$L_{\text{mix}} = \frac{VMA \times B_{ac} + V_{agg} \times B_{agg}}{3V_{\text{total}}}$$

where L_{mix} = Linear CTC of the AC mix (typically, 2.2 to 3.4 × 10^{-5} per degree Celsius)
B_{ac} = Volumetric CTC of the asphalt cement (binder) in the solid state (typically, 3.5 to 4.3 × 10^{-4} per degree Celsius)
B_{agg} = Volumetric CTC of the aggregate
VMA = % volume of voids in mineral aggregates
V_{agg} = % volume of aggregate in the mixture (typically, 21 to 37 × 10^{-6} per degree Celsius)
V_{total} = 100%

Air Voids

Air voids refer to the total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture as shown in equation below. The amount of air voids in a mixture is extremely important and closely related to stability and durability. AASHTO T 166 test protocol is to be used to determine the air void. The air void is calculated according to the equation below:

$$\text{Air void} = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100$$

where
G_{mm} = Theoretical maximum specific gravity determined using the AASHTO T 209 test protocol
G_{mb} = Bulk specific gravity determined using the AASHTO T 166 test protocol

Effective Asphalt Content by Volume

It is defined as the total asphalt binder content of the HMA less the portion of asphalt binder that is lost by absorption into the aggregate. AASHTO T 308 test protocol is to be used to determine the Effective Asphalt Content by volume.

Aggregate Specific Gravity

Specific gravity is a measure of a material's density as compared to the density of water at 73.4°F (23°C). Therefore, by definition, water at a temperature of 73.4°F (23°C) has a specific gravity of 1. Specific gravities of fine and coarse aggregate can be determined using the AASHTO T 84 and T 85 test protocols.

Unit Weight

Unit weight means the weight of HMA per unit bulk volume. The AASHTO T 166 test protocol is to be used to determine the bulk specific gravity (G_{mb}) of the material, which is to be multiplied by the water unit weight to determine the unit weight of the material.

Voids Filled with Asphalt (VFA)

The portion of the voids in the mineral aggregate that contains asphalt binder is called VFA. This represents the volume of the effective asphalt content. It can also be described as the percent of the volume of the void in mineral aggregate (VMA) that is filled with asphalt cement.

Asphalt Mixtures

Asphalt mixture consists broadly of three types:

Hot-Mix Asphalt (HMA), Warm-Mix Asphalt (WMA), Cold-Mix Asphalt (CMA).

Hot-Mix Asphalt (HMA)

Asphalt pavement refers to any paved road surfaced with asphalt. Hot mix asphalt (HMA) is a combination of approximately 95% stone, sand, or gravel bound together by asphalt cement, a product of crude oil. Asphalt cement is heated, combined, and mixed with the aggregates at an HMA facility. The resulting hot mix asphalt is loaded into trucks for transport to the paving site. The trucks dump the hot mix asphalt into hoppers located at the front of paving machines. The asphalt is placed, and then compacted using a heavy roller, which is driven over the asphalt. Traffic is generally permitted on the pavement as soon as the pavement has cooled.

Asphalt mixture is commonly known as hot-mix asphalt (HMA), although it is one kind of AC mixture. HMA is an AC mixture that is produced by heating the mixture at a certain level of temperature, commonly 285 to 325°F (140 - 160°C). It should be noted that AC mixture can also be produced by heating up or even at ambient air temperature without heating. Refer to Figure 35. Hot mix asphalt should only be installed when the outside temperature is 40 degrees F or higher.

HMA consists of five types:

Dense Graded Mix, Open-Graded Friction Course, Asphalt-Treated Permeable Bases, Sand-Asphalt Mix, and Stone Matrix Asphalt.

Figure 35 Hot Mix Asphalt Application

Dense-Graded Mix

A dense-graded mix is a well-graded HMA intended for typical use (Figure 35). When properly designed and constructed, a dense-graded mix is relatively impermeable. Dense-graded mixes are generally referred to by their nominal maximum aggregate size and can further be classified as either fine-graded or coarse-graded. Fine-graded mixes have more fines and sand-sized particles than coarse-graded mixes. It is suitable for all pavement layers and all traffic conditions and works well for structural, friction, leveling, and patching needs.

Open-Graded Friction Course (OGFC)

Open-graded mix uses only crushed stone and a small percentage of manufactured sands with about 15% air voids. It is used for surface courses only to provide good friction and drain-infiltrated water laterally. Figure 36 shows a comparison between dense-graded and open-graded mixes. OGFC reduces tire splash/ spray in wet weather and typically results in smoother surfaces than dense-graded HMA. It also results in smoother surfaces than dense-graded HMA and reduces the thermal cracking in asphalt pavement. Figures 37, 38, and 39 show samples of open-graded mixes compared with dense-graded mixes.

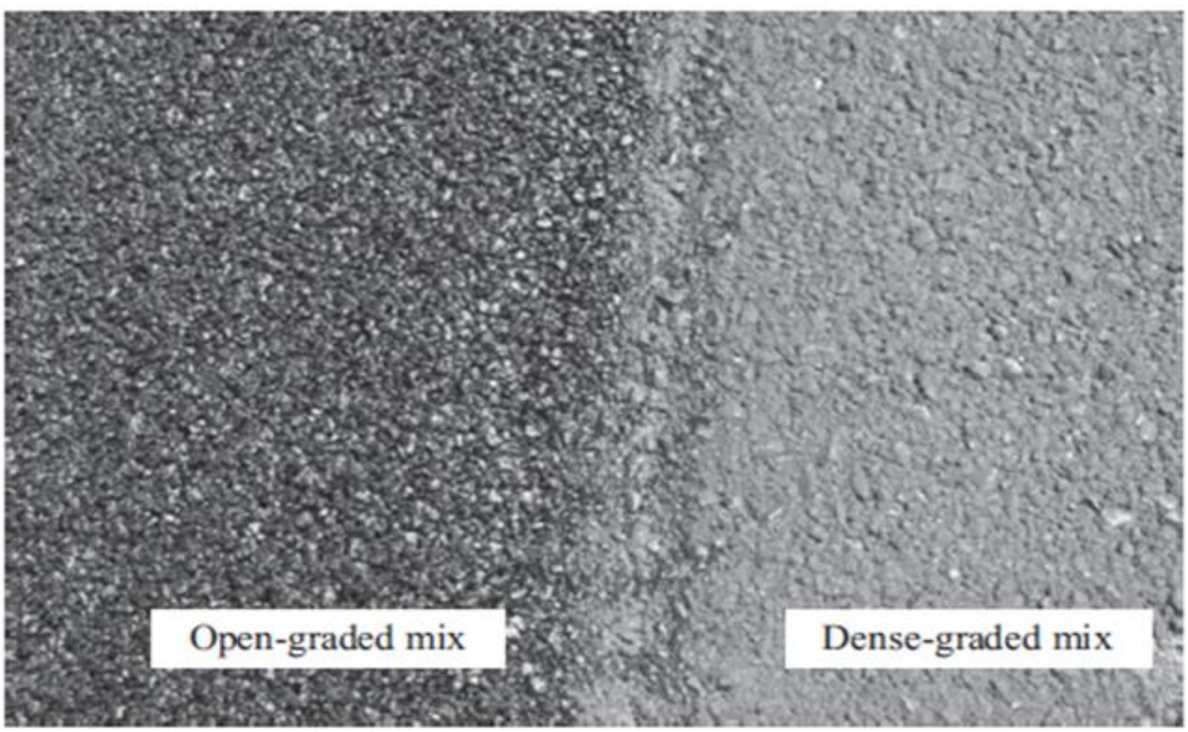


Figure 36 Comparison of Open-Graded and Dense-Graded AC

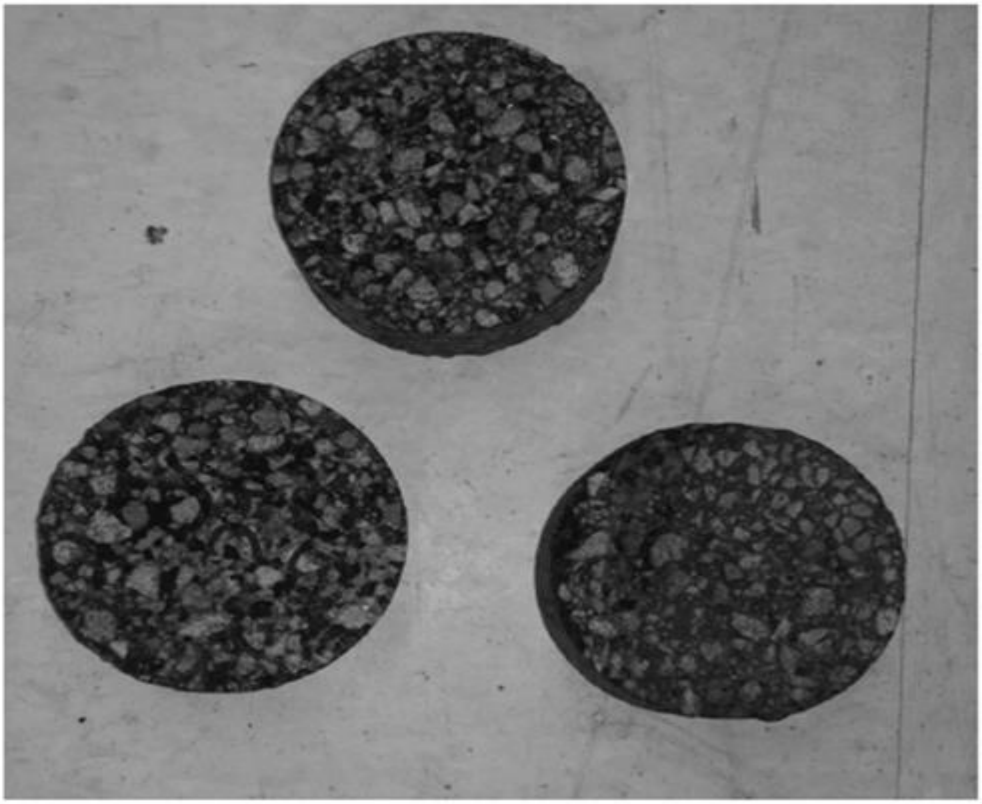
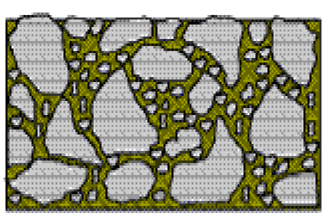


Figure 37 OGFC Specimens



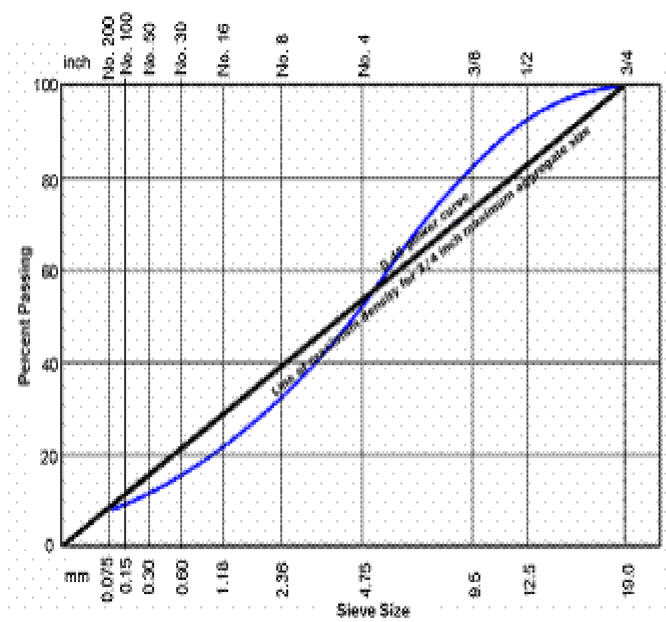


Figure 38 Sieve Analysis for Aggregates in DGM

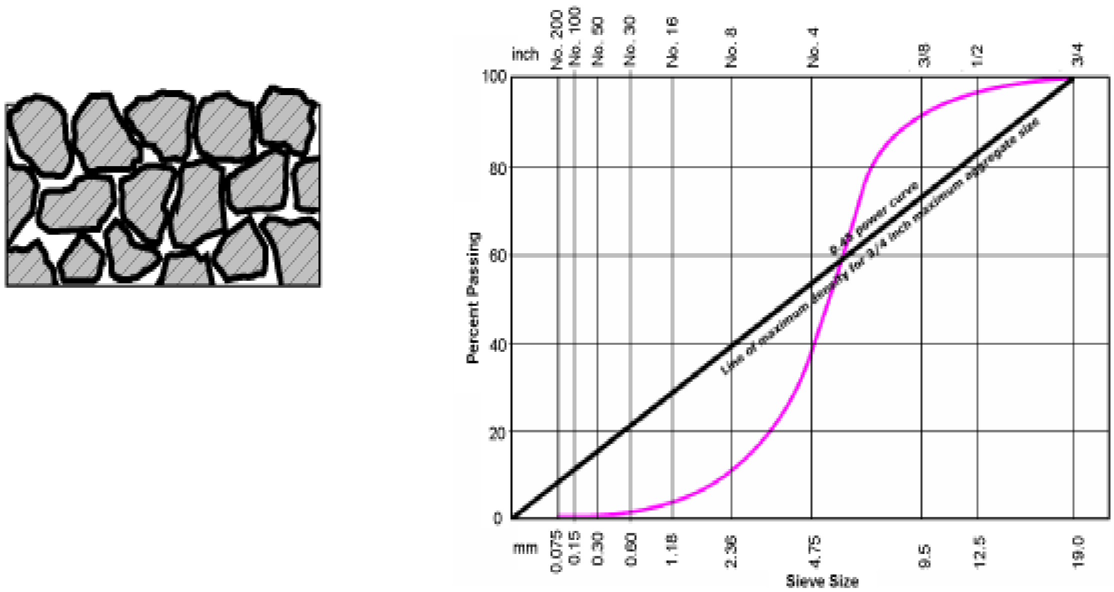


Figure 39 Sieve Analysis for Aggregates in OGFC

Asphalt-Treated Permeable Bases (ATPBs)

Asphalt-treated permeable base is placed under concrete or hot mix asphalt pavement and provides a highly permeable drainage layer within the pavement structure as shown in Figure 40. Asphalt-treated permeable base has less stringent specifications than OGFC since it is used only under dense-graded HMA, stone-matrix asphalt (SMA), or Portland Cement Concrete for drainage. ATPB is used as a drainage layer below dense-graded HMA, SMA, or PCC.

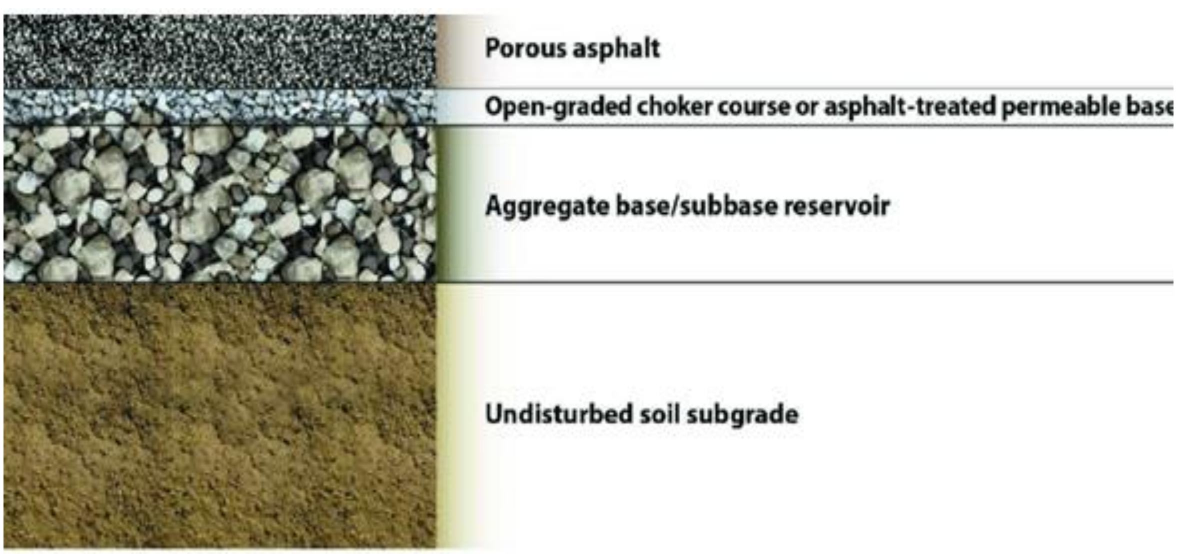


Figure 40 Asphalt-Treated Permeable Base

Sand-Asphalt Mix

Asphalt sand is a coarse washed sand used primarily in the production of asphalt mixes. This sand is the result of crushing and is typically the most angular of sands available as shown in Figures 41 and 42. Because it compacts, it works well as a leveling course under pavers or flagstones. Sand-asphalt mix is a dense-graded mix of asphalt and sand with nominal maximum aggregate size less than 3/8 in. (9.5 mm).

Figure 41 Sand- Asphalt Mix

Figure 42 Sand- Asphalt Mix Application

Stone Matrix Asphalt (SMA)

Stone matrix asphalt, sometimes called stone mastic asphalt, is a gap-graded HMA originally developed in Europe to maximize rutting resistance and durability (Figure 43). The goal of the mix design is to establish stone-on-stone contact within the mixture. Since aggregates do not deform as much as asphalt binder under load, this stone-on-stone contact greatly reduces rutting. SMA is generally more expensive than a typical dense-graded HMA because it requires more durable aggregates, higher asphalt content, and modified asphalt binder and fibers as shown in Figure 44. It is used to improve rut resistance and durability. SMA is almost exclusively used for surface courses on high-volume interstate pavements and roads. Gap-graded aggregate, modified asphalt binder, fiber filler, etc. are commonly used to produce this.

Figure 43 Stone Matrix Asphalt

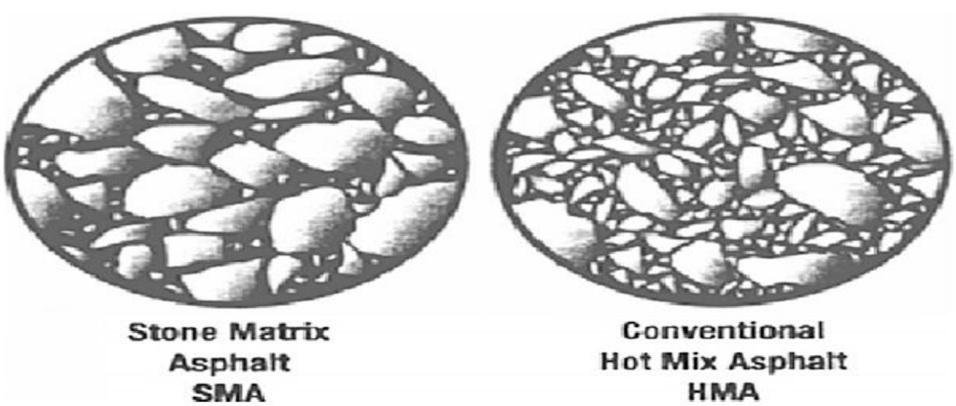


Figure 44 SMA vs. HMA

Warm-Mix Asphalt (WMA)

Warm mix asphalt (WMA) is the generic term for a variety of technologies that allow producers of hot mix asphalt (HMA) pavement material to lower temperatures at which the material is mixed and placed on the road as shown in Figure 45. It is a proven a technology that can: reduce paving costs and extend the paving season.

Asphalt workers' benefits: Energy and resource savings due to lower mixing temperatures, improved working conditions for lower emissions of fumes, aerosols and odour at both plant and work site, quicker reopening of new traffic surfaces, and lower production and laying temperatures reduce thermal aging of the bitumen.

Warm mix asphalt technology allows the producers of asphalt pavement material to lower the temperatures at which the material is mixed and placed on the road. Reductions of 30 to 120°F (17– 67°C) have been documented. Such drastic reductions have the obvious advantages of reducing fuel consumption and decreasing greenhouse gas production. Fuel consumption during WMA manufacturing is typically reduced by 20%. In addition, engineering benefits include better compaction on the road, the ability to haul paving mix for longer distances, and extending the paving season by being able to pave at lower temperatures.

WMA technologies reduce the asphalt binder's viscosity (thickness) so asphalt aggregates can be coated at lower temperatures. The key is the addition of additives (water-based, organic, chemical, or hybrid) to the asphalt mix (Bonaquist, 2011). The additives allow the asphalt binders and asphalt aggregates to be mixed at the lower temperatures. Reducing the viscosity also makes the mixture easier to manipulate and compact at the lower temperature.

Figure 45 Warm Mix Asphalt Application

Cold-Mix Asphalt (CMA)

Cold mix asphalt is, just like hot mix asphalt, a combination of aggregates and cutback or bitumen emulsion, commonly used on low traffic roads or rural roads as shown in Figure 46. Cold mix asphalt also works along flexible pavements and can be produced, either on-site or at mixing plants.

Cold mix asphalt concrete is formed by emulsifying asphalt with (essentially) soap in water before mixing with the aggregates. While in its emulsified state, the asphalt is less viscous, and the mixture is easy to work and compact. The emulsion will break after enough water evaporates and the cold mix will, ideally, take on the properties of HMA. Cold mix is commonly used as a patching material and on lesser trafficked service roads. Typically, it is based on two types of processing location: central plant processed and cold-in-place recycling.

In central plant processing, milled asphalts are transported to a plant, screened, and emulsifying agents are mixed. The produced mix is transported back to the site, placed, and compacted. The cold-in-place recycling is produced on site. The milled asphalts are screened in a large truck on site, mixed, placed, and compacted with emulsifying agents. Compacted cold mixes look similar to the conventional mixes.

A cold-mix asphalt ready to be paved is shown in Figure 47. Proper usages of cold mix asphalt are very competitive with the conventional mixes when used for low-traffic roads (Islam et al., 2018a). Refer to Figure 48 that shows the energy consumption vs HMA, WMA, and CMA.

Figure 46 Cold Mix Asphalt Application

Figure 47 Cold Mix Asphalt

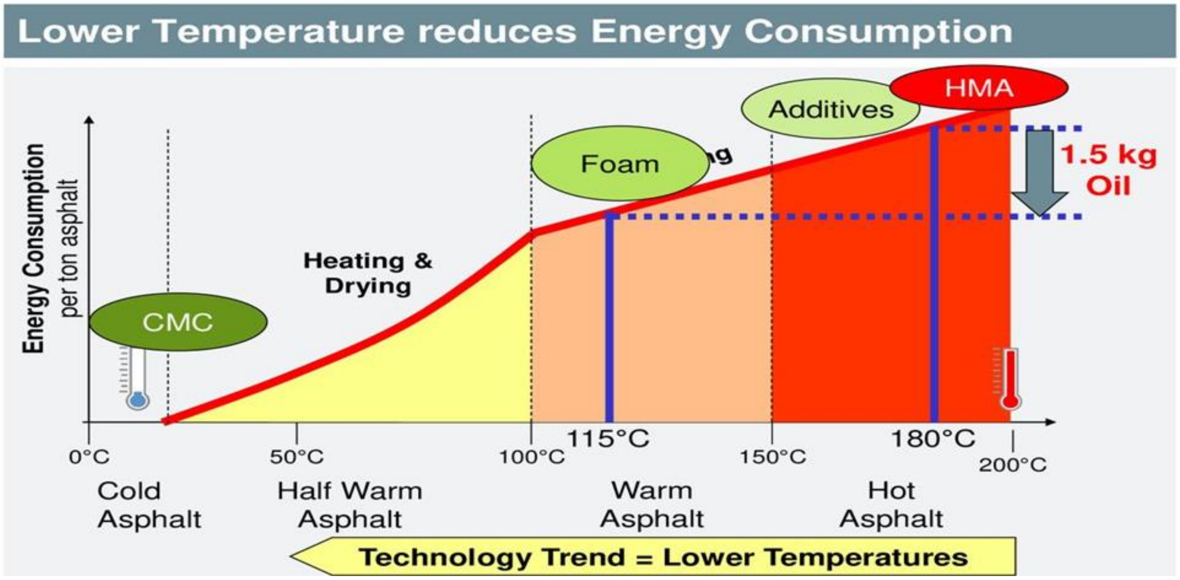


Figure 48 Energy Consumption vs. Asphalt Types

Recycled Asphalt Materials

Recycled asphalt materials consist mainly of four types: Reclaimed Asphalt Pavement (RAP), Reclaimed Asphalt Shingles (RAS), Rubberized Asphalt Concrete (RAC), Reclaimed Asphalt Pavement (RAP) in Base and Subbase.

Reclaimed Asphalt Pavement (RAP)

Reclaimed or recycled asphalt pavement is the milling of asphalt surface layer from old pavement containing aggregates and asphalt binder as shown in Figure 49 and Figure 50. Nowadays, up to 40% by weight of the whole mixture is being used in the United States (Hasan et al., 2018; Islam et al., 2014). The amount of RAP used in asphalt mixtures was 66.7 million tons in 2011, a 19% increase over 2009 (56 million tons) and about a 7% increase over 2010 (62.1 million tons). Assuming 5% liquid asphalt in RAP, this represents approximately 3.6 million tons of virgin asphalt binder conserved, or about 12% of the total binder used in 2011. Looking at 2011 U.S. data, approximately 87 million tons of RAP that was milled from existing pavements was run through asphalt mixing plants that year, with approximately 74 million tons of the 81 million tons of RAP (92%) recycled into new AC materials. For the years 2009 through 2011, RAP that was not recycled into AC was used for aggregate base (less than 10% annually) and cold mix (less than 3% annually) and less than 0.1% was landfilled (Hansen and Copeland, 2013).

RAP has been used to replace virgin materials in dense AC by up to 50%. However, where mixture performance is most critical, such as in asphalt surface layers, the level of replacement is often lower.

In general, replacement at up to 15% is considered to have minimal effects on properties. Most state highway agencies allow up to 15% or 30% replacement for structural layers, and some also allow those amounts for surface layers. The average RAP content in AC mixtures in the United States in 2009–2010 was about 13% for U.S. Department of Transportation (DOT) mixtures, 15% for other agency mixtures, and 18% for commercial and residential paving mixtures (Hansen and Newcomb, 2011).

Figure 49 Reclaimed Asphalt Pavement

Figure 50 Milling of Reclaimed Asphalt Surface

Reclaimed Asphalt Shingles (RAS)

Reclaimed asphalt shingles are collected from roof tear-offs and reused to the pavement and in many cases may improve the quality (Figure 51). Shingles can contain between 20% and 36% asphalt. This asphalt can be used to bind aggregates like the conventional asphalt. Shingle wastes either from the manufacturer or roof tear-off can be used to save virgin asphalt and avoid shingle landfills. The literature reports that RAS has increased resistance to rutting, reduced cracking and requires less compaction effort (Roque et al. 2018).

Figure 51 Reclaimed Asphalt Shingles

Rubberized Asphalt Concrete (RAC)

Rubberized asphalt concrete (commonly known as RAC) is a road paving material made by blending ground-up recycled tires with asphalt to produce a binder which is then mixed with conventional aggregate materials as shown in Figure 52. This mix is then placed and compacted into a road surface. Rubberized asphalt concrete, also known as asphalt rubber or just rubberized asphalt, is noise-reducing pavement material that consists of regular AC mixed with crumb rubber from recycled tires. Approximately 2.4 million tires are recycled

Figure 52 Rubberized Asphalt Concrete

every year as asphalt rubber and are expected to grow. RAC is a cost-effective, sustainable, safe, and environmentally friendly alternative to traditional road paving materials. The performances of RAC is very similar to conventional asphalt materials but it needs production machinery as shown in Figure 53.

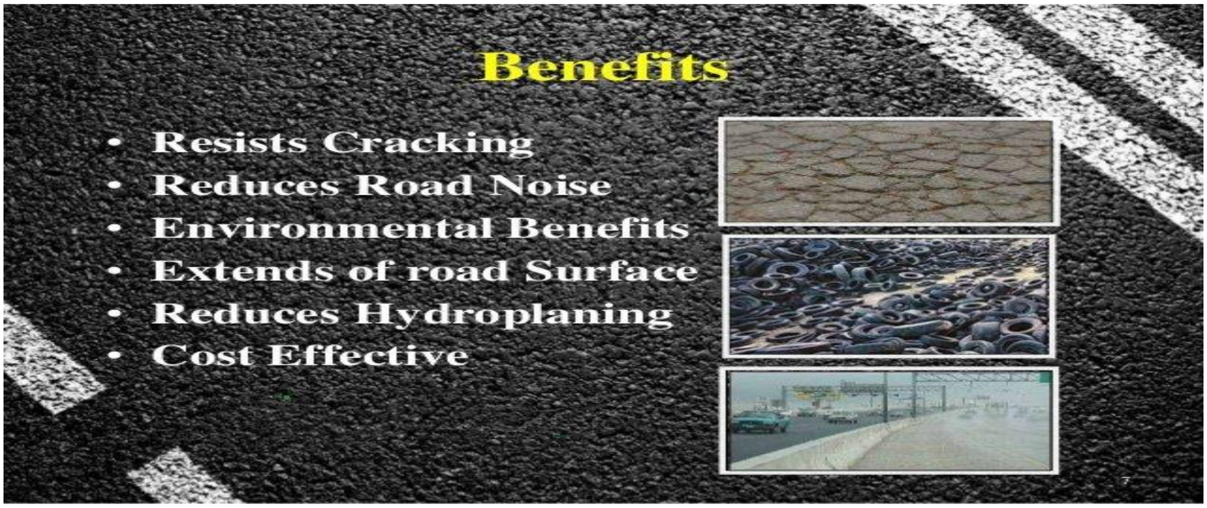


Figure 53 Benefits of Rubberized Asphalt Concrete

Reclaimed Asphalt Pavement (RAP) in Base and Subbase

In addition to the asphalt mix, the usages of RAP in aggregate base or subbase are also becoming popular nowadays (Hasan et al., 2018; Islam et al., 2014; Tarefder and Islam, 2015). There are different ways RAP can be used in base and subbase layers. One approach is plant processing where friction is transported, crushed, and screened to a central plant. The better quality RAP is used with the new asphalt mix production as discussed earlier. The inferior RAP is then added with the virgin base or subgrade materials. It improves the base and subgrade strength and saves the RAP from being dumps. An example of RAP-mixed base course used in an interstate highway in New Mexico is shown in Figure 54. About 50% of RAP is mixed with virgin aggregates to produce this base layer. In this interstate highway, RAP is also mixed with subgrade to produce a subbase layer.

Figure 54 RAP-Mixed Base Course in an Interstate Pavement in New Mexico

Section 4 — References

• AASHTO 2015 Mechanistic-Empirical Pavement Design Guide: A Manual of Practice. Washington, DC: American Association of State and Highway Officials.