Soil Tests for Flexible and Rigid Pavements Design

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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.

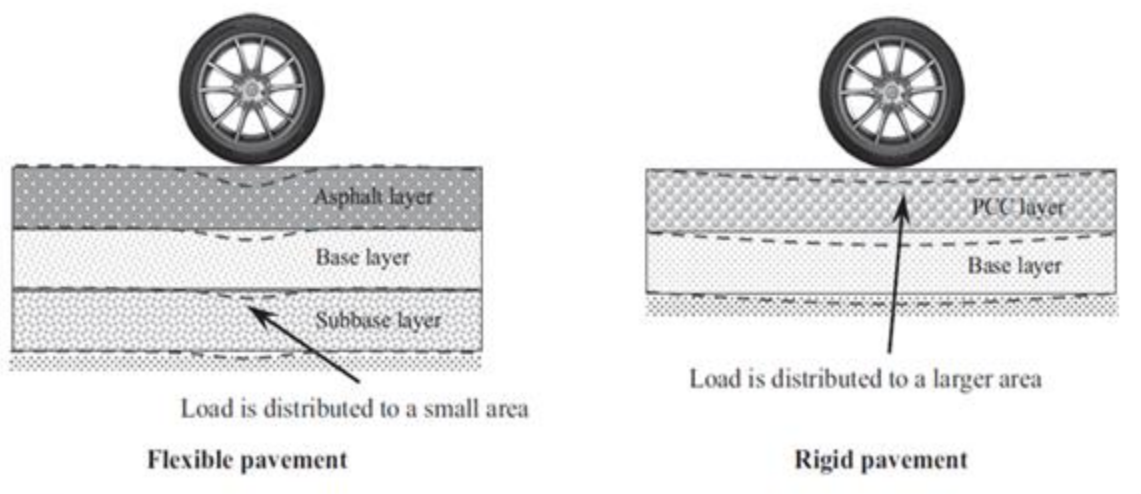


Figure 3 Deformation Behaviour of Flexible and Rigid Pavements

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 — Soils and Aggregates

Soils and aggregates are essential ingredients of highway pavements. Generally, the pavement is constructed over the natural soil (subgrade). On the top of subgrade, aggregate subbase and base layers are provided, which consist of crushed-stone coarse aggregates and fine aggregates that are highly compacted at the optimum moisture content. The surface layer, asphalt concrete (AC), or Portland cement concrete (PCC) is applied on top of the base layer. The surface, base and subbase layers together provide the structural integrity and strength to the pavement. The base and subbase layers also drain moisture if any moisture infiltrates through the surface layer or occurs due to ice melting or capillary action. As such, soils and aggregates are used in all parts of a pavement, and its strength and performance depend on the quality and properties of those soils and aggregates. Therefore, prior to construction, proper characterization of soils and aggregates is essential to understand its performance during the service life of a pavement.

Physical Properties

The physical properties of soils and aggregates are determined prior to applications. These properties are not necessarily indicators of material strength or stiffness, but these physical properties help select whether materials to borrow or use existing materials. For example, subgrade cut and fill is a common approach where soft clay or soft soil is encountered. Materials characterization to determine physical properties includes, but not limited to, the following:

Sieve Analysis, Atterberg Limits, Soil Classification, Proctor Test, Flat and Elongated Particles, Fine and Elongated Particles, Coarse Aggregate Angularity, Clay Content, Los Angeles (LA) Abrasion, Soundness, and Deleterious Materials.

Sieve Analysis

Sieve Analysis Procedure

A typical sieve analysis involves separating soils or aggregates by a nested column of sieves with wire mesh (screen), as shown in Figure 4 and Table 1. A typical gradation analysis is shown in Figure 5.

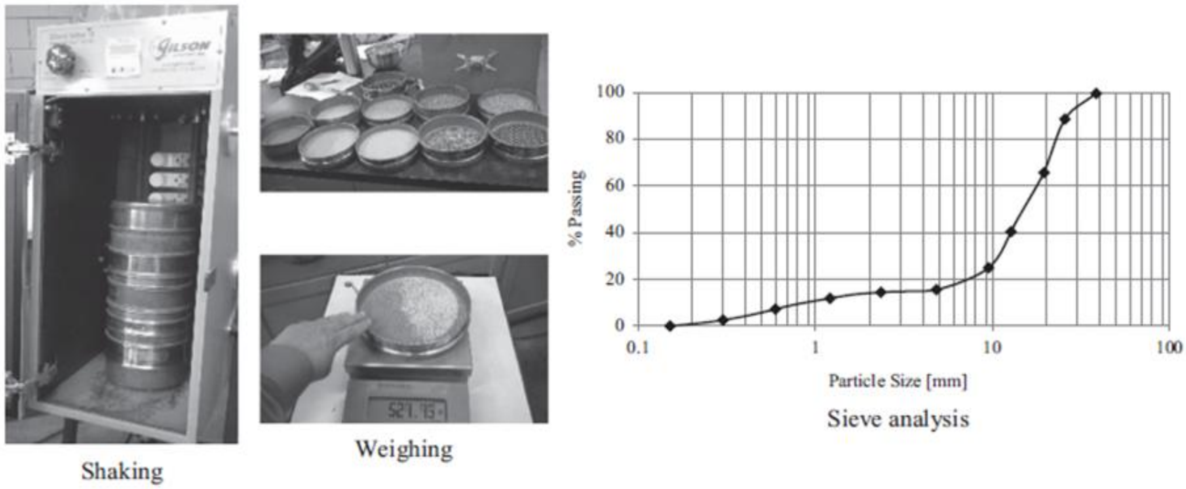


Figure 4 Sieve Analysis Procedure and Sieve Analysis Curve

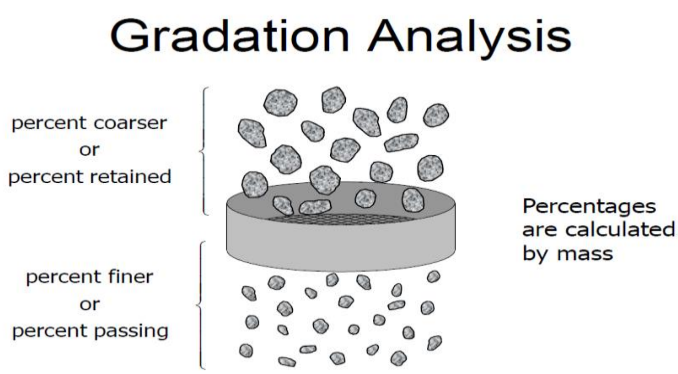


Figure 5 Gradation Analysis

Maximum Aggregate Size

ASTM C 125 defines the Maximum Aggregate Size in one of the following two ways:

Nominal Maximum Aggregate Size (NMAS): The traditional definition of the NMAS is the largest sieve that retains some of the aggregate particles but generally not more than 10% by weight. Superpave defines NMAS as one sieve size larger than the first sieve to retain more than 10% of the material. Each stock of aggregate is represented by its NMAS. For example, a 19-mm aggregate batch represents a batch of aggregate whose NMAS is 19 mm.

Maximum Size: The traditional definition of the maximum size is the smallest sieve through which 100% of the aggregate specimen particles pass. Superpave defines the maximum aggregate size as one sieve larger than the nominal maximum aggregate size (NMAS).

Sieve number	Size (mm)
1.5 in.	37.5
1.0 in.	25.0
¾ in.	19.0
½ in.	12.5
No. 4	4.75
No. 10	2.00
No. 20	0.85
No. 40	0.425
No. 100	0.15
No. 200	0.075

Table 1 Some Common Sieves with Sieve Openings

Example 1: Aggregate Sizes

After performing the sieve analysis, the masses of materials retained on each sieve are shown in Table 2.

Draw the Sieve Analysis Curve (% Passing versus Sieve size).

First, calculate the percent retained in each sieve as follows:

Then, the "Cumulative % Retained" is calculated by summing up the "% Retained" in the corresponding sieve plus that in the larger sieves. For example, "Cumulative % Retained" in No. 8 = 12.83 + 4.55 + 2.15 + 0.25 +0 = 19.78, as shown in Table 3.

"% Passing" is calculated as 100 – "Cumulative % Retained".

Plot the "% Passing" versus "Sieve Size" curve, as shown in Figure 6.

Sieve number	Sieve size (mm)	Mass retained (g)
1.0 in.	25	0
0.75 in.	19	3
0.375 in.	9.5	26
No. 4	4.75	55
No. 8	2.38	155
No. 16	1.19	189
No. 30	0.6	315
No. 50	0.297	255
No. 100	0.149	110
No. 200	0.075	78
Pan	Pan	22

Table 2 Sieve Data for Example 1

Sieve number	Sieve size (mm)	Mass retained (g)	% Retained	Cumulative % retained	% Passing
1.0 in.	25	0	0.00	0.00	100.0
0.75 in.	19	3	0.25	0.25	99.8
0.375 in.	9.5	26	2.15	2.40	97.6
No. 4	4.75	55	4.55	6.95	93.0
No. 8	2.38	155	12.83	19.78	80.2
No. 16	1.19	189	15.65	35.43	64.6
No. 30	0.6	315	26.08	61.51	38.5
No. 50	0.297	255	21.11	82.62	17.4
No. 100	0.149	110	9.11	91.72	8.3
No. 200	0.075	78	6.46	98.18	1.8
Pan	Pan	22	1.82	100.0	0.0
		Total = 1,208			

Table 3 Analysis of Sieving Data for Example 1

Determine the NMAS based on the Superpave Criteria.

Superpave defines the NMAS as one sieve size larger than the first sieve to retain more than 10% of the material. Here the first sieve to retain more than 10% of the material is No. 8 (2.4 mm). Therefore, NMAS = 4.75 mm.

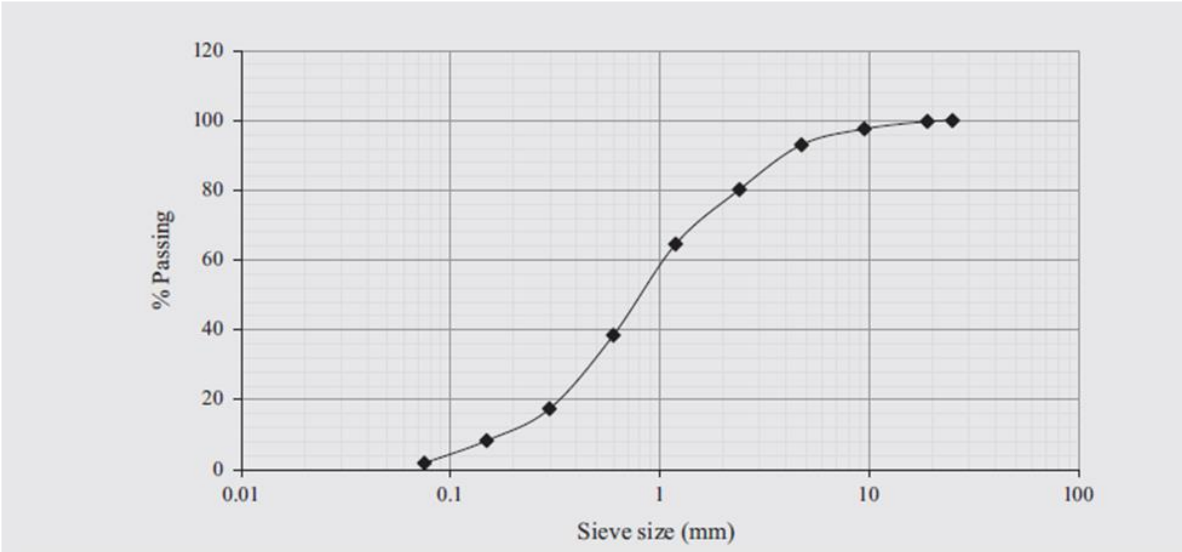


Figure 6 Sieve Analysis Curve of Example 1

Determine the Maximum Aggregate Size based on the SuperPAVE Criteria.

Superpave defines the Maximum Aggregate Size as one sieve larger than the Nominal maximum size. As, NMAS = 4.75 mm, the maximum size is 0.375 in. (9.5 mm).

Determine the NMAS based on the Traditional Criteria.

The traditional definition of the NMAS is the largest sieve that retains some aggregates but generally not more than 10% by weight. Therefore, the NMAS is 0.75 in. (19 mm) as it retains 0.25% materials.

Determine the Maximum Aggregate Size based on the Traditional Criteria.

The traditional definition of the Maximum Aggregate Size is the smallest sieve through which 100% of aggregate specimen particles pass. Therefore, the maximum size is 1.0 in. (25 mm), as it is the smallest size which passes all aggregates.

Desired Gradation

The optimum gradation depends on the material (AC or PCC), its desired characteristics, anticipated loading, environment, and mix properties. The optimum gradation produces the maximum density. This creates more particle-to-particle contact, which in AC would increase stability and reduce water infiltration. In PCC, the reduced void space reduces the amount of cement paste required. However, some minimum amount of void space is necessary to: (a) provide adequate volume for the binder, and (b) promote rapid drainage and resistance to frost action for base and subbases.

Fuller’s Curve modified by FHWA:

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

where P = % finer (% passing) than the sieve
d = Aggregate size being considered (in. or mm)
D = Maximum aggregate size to be used (in. or mm)
n = Parameter which adjusts the fineness or coarseness, 0.45 by the Federal Highway Administration (FHWA)

Example 2: Calculation for 0.45 Exponential Power Curve

Table 4 shows an example of calculating the densest gradation curve for a 19-mm (NMAS) aggregate. A number of smaller sieves are considered, such as 0.5 in. (12.5 mm), 0.375 in. (9.50 mm), etc. Then, the sieve size is raised to the exponential power 0.45, as listed in Column 2. Then, the percent passing is calculated using the Fuller's equation modified by FHWA, as listed in Columns 3 and 4. The 0.45 power curve is drawn in plain graph with Column 2 in the x-axis and Column 4 in the y-axis. If this gradation can be achieved for a 19-mm aggregate batch, then that would be the densest gradation. It can be noted that the densest gradation is not commonly desired; some amount of void space is essential so that materials have some room to be compacted under repeated traffic loading avoiding instability. Thus, the optimum gradation is sought for designing mix. The procedure to find out the optimum gradation is discussed in both concrete mix design and asphalt mix design as each of these mix designs has different requirements.

Particle size (mm)	(size, mm) 0.45	% Passing Calculation	% Passing
Column 1	**Column 2**	**Column 3**	**Column 4**
19.0	3.762	$P = \left(\frac{19.0}{19.0}\right)^{0.45} \times 100$	100
12.50	3.116	$P = \left(\frac{12.5}{19.0}\right)^{0.45} \times 100$	82.8
9.50	2.754	$P = \left(\frac{9.5}{19.0}\right)^{0.45} \times 100$	73.2
2.00	1.366	$P = \left(\frac{2.00}{19.0}\right)^{0.45} \times 100$	36.3
0.30	0.582	$P = \left(\frac{0.30}{19.0}\right)^{0.45} \times 100$	15.5
0.075	0.312	$P = \left(\frac{0.075}{19.0}\right)^{0.45} \times 100$	8.3

Table 4 Calculations for 0.45 Exponential Power Curve Using a 19-mm NMAS

Gradation Types

Dense or Well-Graded: Dense gradations are near the 0.45 power curve but may not be right on it. It consists of materials of all sizes. Refer to Figure 7.

Gap Graded: Gap graded refers to a gradation that contains only a small percentage of aggregate particles in the mid-size range. The curve is flat in the mid-size range. Refer to Figure 8.

Open Graded: Open gradation contains only a small percentage of aggregate particles in the small range. This results in more air voids because there are not enough small particles to fill the voids between the larger particles. Refer to Figure 9.

One-Sized or Uniformly Graded: One-sized gradation contains most of the particles in a very narrow size range. All the particles have similar size. The curve is steep and only occupies the narrow size range specified. Refer to Figure 10. Refer to Figure 11 for the different types of aggregates gradation.

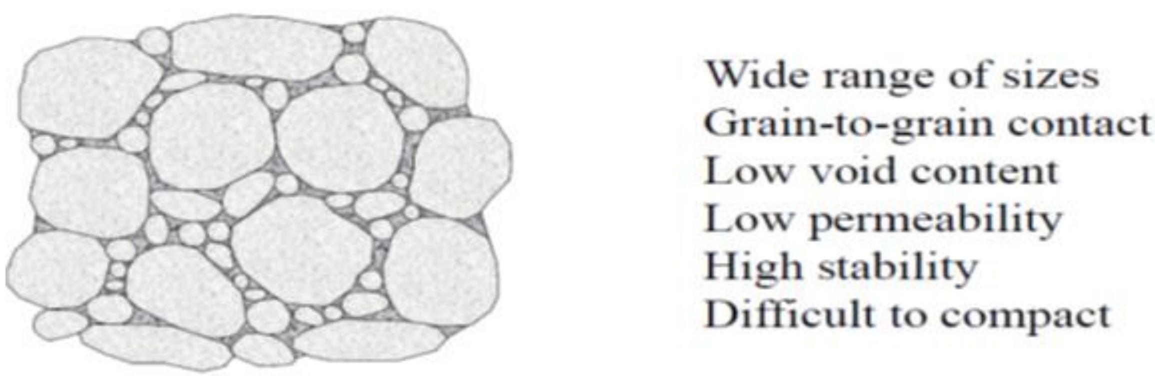


Figure 7 Dense Graded Aggregate

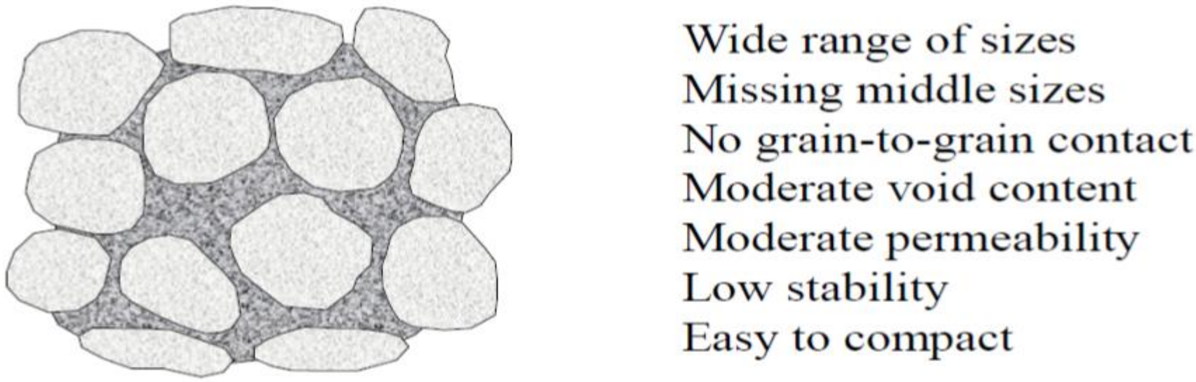


Figure 8 Gap Graded Aggregate

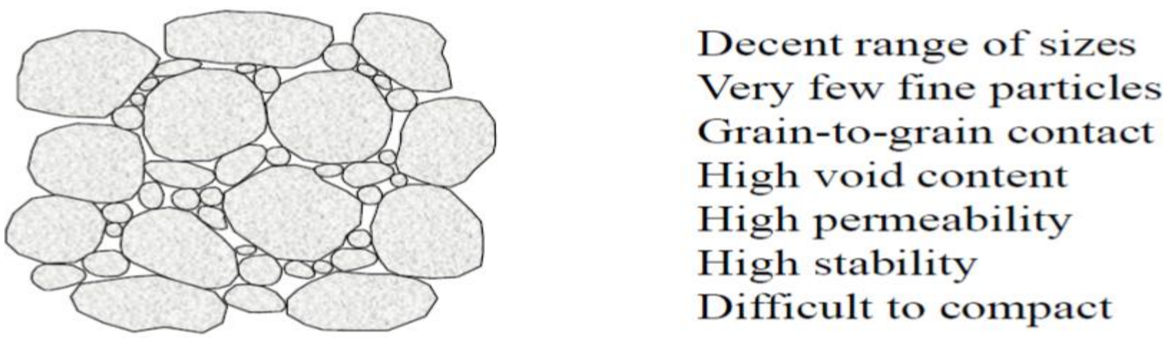


Figure 9 Open Graded Aggregate

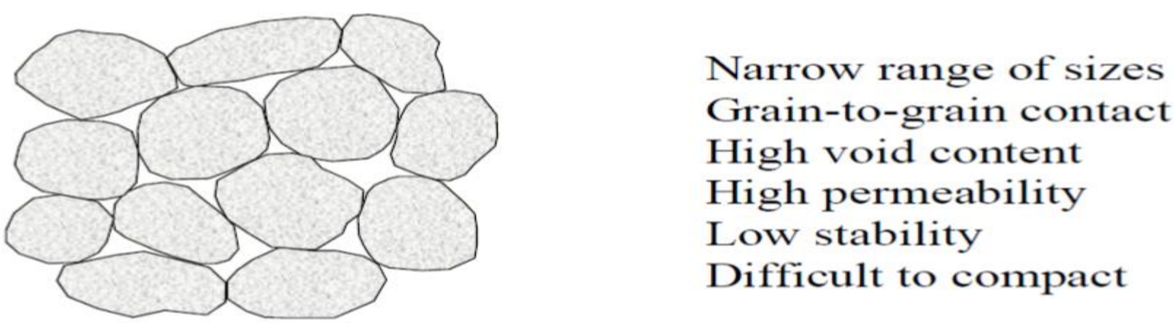


Figure 10 Uniformly Graded Aggregate

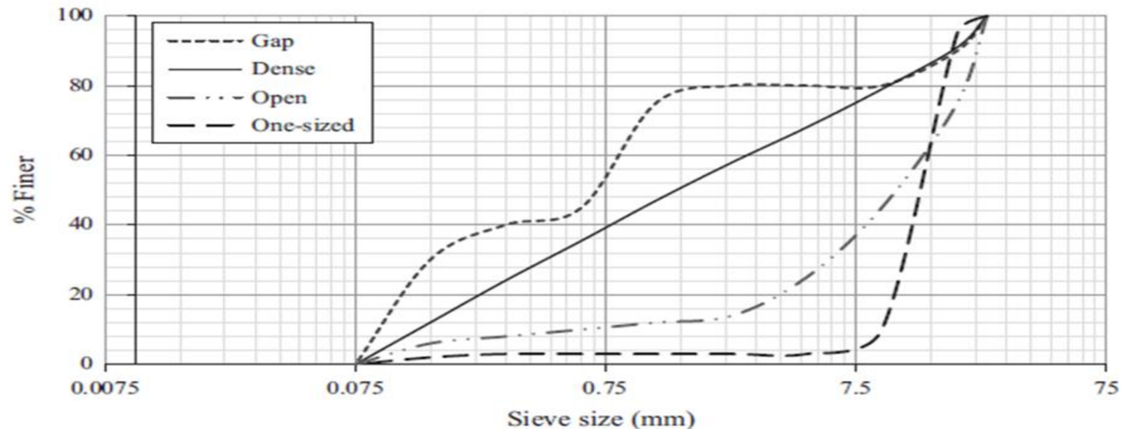


Figure 11 Different Types of Aggregate Gradations

Atterberg Limits

Atterberg limits is a basic measure of the critical water content of fine-grained soils. These tests include shrinkage limit, plastic limit, and liquid limit, which are outlined in ASTM D4318. Depending on the water content of a soil, it may appear in four states: solid, semi-solid, plastic, and liquid.

Liquid Limit

The liquid limit test, defined in ASTM Standard D4318, determines the water content at which the behavior of a clayey soil changes from plastic to liquid. However, the transition from plastic to liquid behavior is gradual over a range of water contents, and the shear strength of the soil is not actually zero at the liquid limit. The precise definition of the liquid limit is based on standard test procedures.

Plastic Limit

Plastic limit is a test that involves rolling out a thread of the fine portion of a soil on a flat, non-porous surface. The procedure is defined in ASTM Standard D4318. If the soil is at a moisture content where its behavior is plastic, this thread will retain its shape down to a very narrow diameter. The sample can then be remolded and the test repeated. As the moisture content falls due to evaporation, the thread will begin to break apart at larger diameters. The plastic limit is defined as the moisture content where the thread breaks apart at a diameter of 3.2 mm (about 1/8 inch). A soil is considered non-plastic if a thread cannot be rolled out down to 3.2 mm at any moisture possible.

Shrinkage Limit

Shrinkage limit is a test that evaluates the water content of a soil where further loss of moisture will not result in an additional volume reduction. The test to determine the shrinkage limit is ASTM D4943. The shrinkage limit is much less commonly used than the liquid and plastic limits.

Different Stages of Liquid Limit Testing

For fine soils, the liquid limit (LL) and plastic limit (PL) are of interest for soil classification. These two limits are also called the Atterberg limits. The LL is the water content at the plastic-liquid boundary. It is determined using the Casagrande device (Figure 12).

The basic steps of determining LL are mentioned below:

Take roughly 200 g of soil passing #40 sieve.

Mix water thoroughly and let it cure.

Spread the soil in the cup to a depth of 10 mm at the deepest point of the device.

Form a groove in the soil by drawing the tool perpendicular to the surface.

Lift and drop the cup about two drops per second.

Record the number of blows required to close the groove.

After several trials, determine the water content which needs 25 blows to close the groove by ½ in. (12.5 mm). This water content is called the LL.

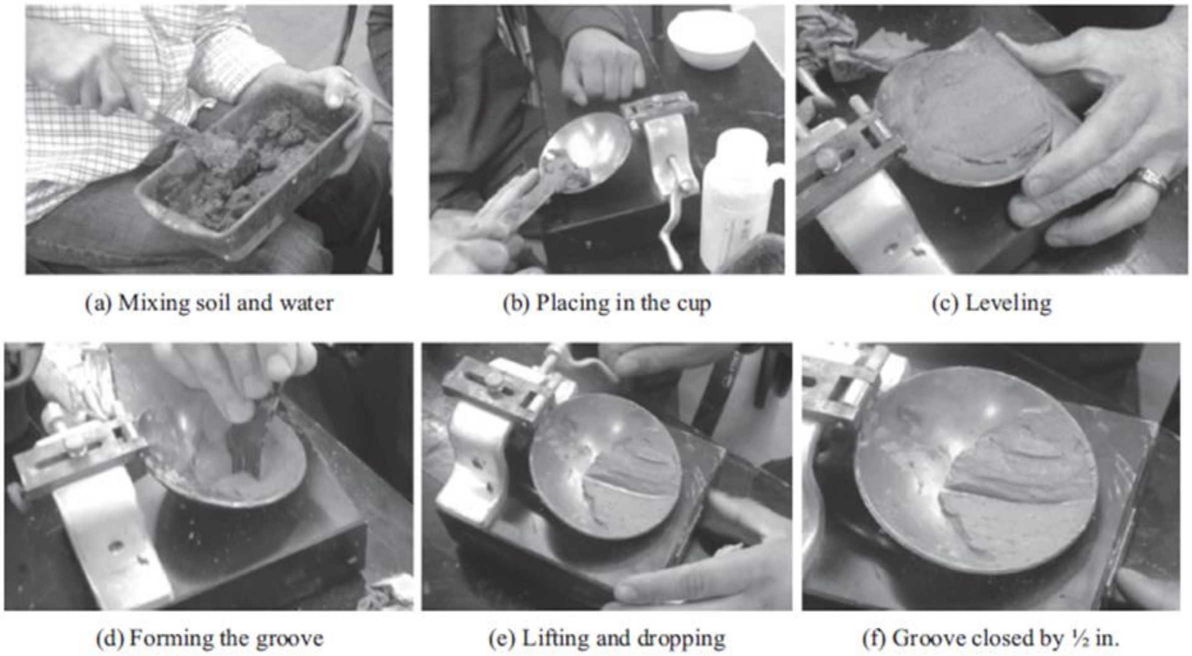


Figure 12 Different Stages of Liquid Limit Testing

Different Stages of Plastic Limit Testing

PL is the water content at the plastic-semisolid boundary. It is determined by rolling a soil specimen to a thread of 1/8 in. (3.2 mm) diameter as shown in Figure 13. The basic steps are as follows:

Take about 20 g of soil passing #40 sieve.

Thoroughly mix small amount of water and cure it for at least 16 minutes.

Take about 2 g of wet soil.

Roll by hand between palm and a glass plate.

If the thread breaks before reaching the diameter of 1/8 in. (3.2 mm), mix more water and repeat steps 1 through 4.

If the thread diameter can be reached easily, break it into several pieces and reroll.

Determine the water content at which the diameter can be barely reached. This water content is called the PL.

Plasticity index (PI) is the difference between the LL and the PL, i.e., PI = LL – PL.

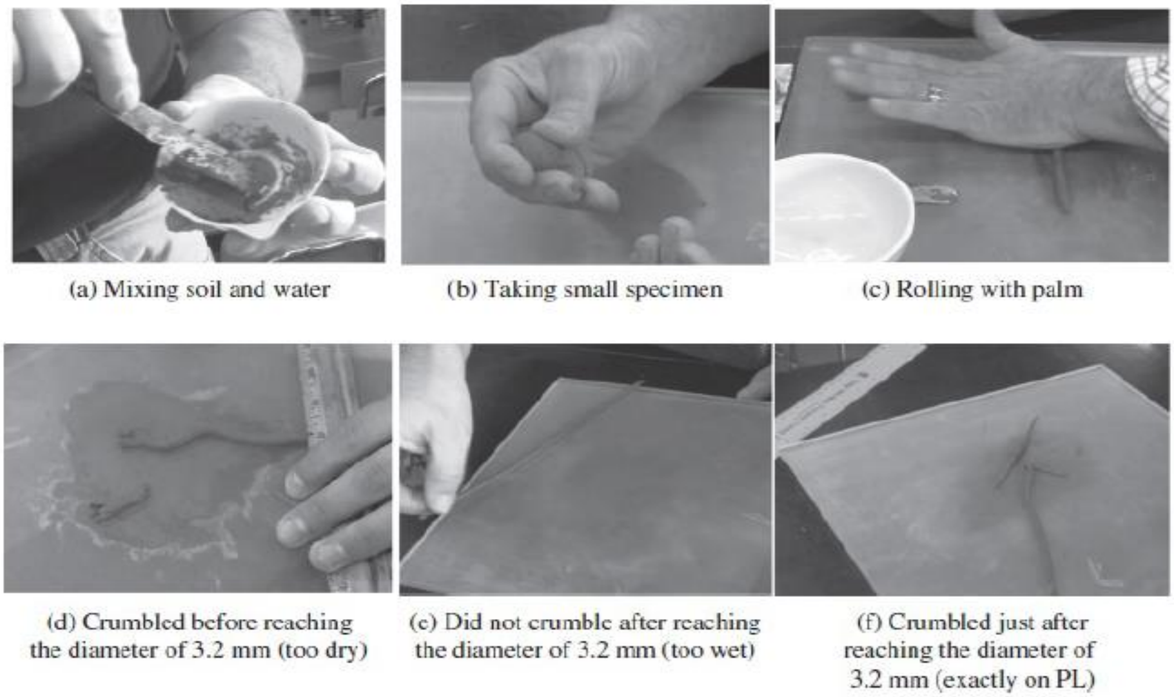


Figure 13 Different Stages of Plastic Limit Testing

The various stages of soils are clearly displayed in Figure 14.

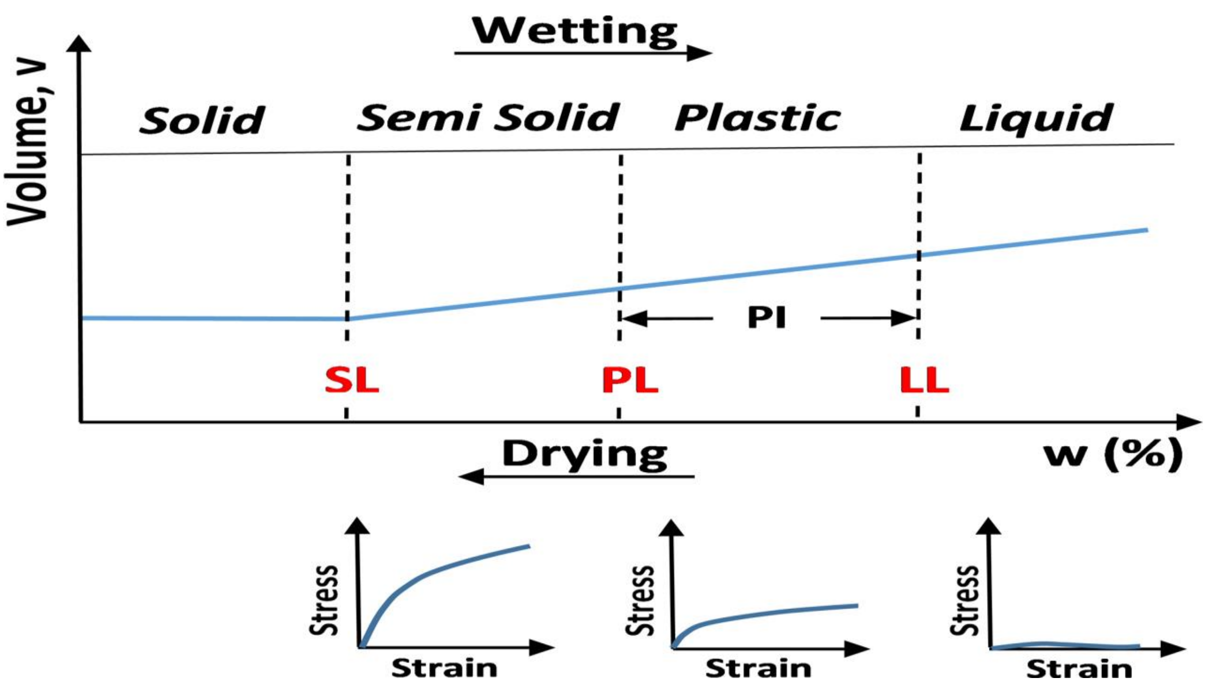


Figure 14 Various Stages of Soils

Soil Classification

Soil can be classified in many different ways, such as the Unified Soil Classification System (USCS), the MIT Soil Classification, the International Classification, the American Association of State Highway and Transportation Officials (AASHTO) Soil Classification, etc. The following two parameters are found to be useful for soil classification after years of researches and experience with wide-ranging soils all over the world:

The particle size distribution of soils and the liquid limit and the plasticity index.

The soil in nature consists of a mixture of particles of different sizes and cohesions. Thus, every soil consists of clay, silt, sand, and gravel in some proportions. Hence, to classify a given soil, it is necessary to determine its grain (or particle) size distribution and the Atterberg limits.

For roads and highways, the AASHTO Soil Classification system is commonly used. It classifies soil from A-1 to A-8, where A-1 is the best engineering soil and A-8 is the worst engineering soil (organic soil). A-8 soil is never recommended for engineering purpose, although it has a good value in agriculture and gardening. The AASHTO Soil Classification chart is shown in Table 5.

General Classification	*Granular Materials (35% or Less Passing No. 200)*							*Silt-Clay Materials (More than 35% Passing No. 200)*			
	A-1			*A-2*							*A-7*
Group Classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*	*A-4*	*A-5*	*A-6*	*A-7-5, A-7-6*
Sieve analysis											
Percent passing											
No. 10	−50 max.	—	—	—	—	—	—	—	—	—	
No. 40	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

*Plasticity index of A-7-5 subgroup ≤ LL − 30. Plasticity index of A-7-6 subgroup > LL − 30.
SOURCE: Adapted from *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 27th ed., Washington, D.C., The American Association of State Highway and Transportation Officials, copyright 2007. Used with permission.

Table 5 AASHTO Soil Classification Chart

AASHTO Soil Classification System

At the beginning, the soil passing the 0.075-mm sieve (No. 200 sieve) is looked at. If 35% or less than 35% of soil passes through the No. 200 sieve, then it is a granular soil, either A-1, A-2, or A-3. If more than 35% passes through the No. 200 sieve, then it is silt or clay materials, and falls under the AASHTO soil classes A-4 to A-7. As mentioned earlier, A-8 is an organic soil, and is excluded from the chart as it is not suitable for engineering purposes. A-1 group consists of stone fragments, gravel, and sand with a maximum of 25% fines. A-3 group consists of a fine sand with a maximum of 10% fines. Silty or clayey soil belongs to A-2, whereas pure silty soils with minimum of 36% fines fall within the A-4 and A-5 groups. A-6 and A-7 groups consist of clayey soils with minimum of 36% fines.

For A-1-a group, the percentages of soil passing through the 2-mm and 425-μm sieves are the additional criteria. For classifying as the A-1-b and A-3 groups, the percentage of soil passing through the 425-μm sieve is the additional criterion. For all other groups, liquid limit and plasticity index of the soil are the additional criteria.

In AASHTO classification system, the soil type is further grouped by computing an index called group index (GI). GI is used to describe the relative performance of soils, when used for highway construction, and is defined by the equation:

$$GI = 0.2\,(F - 35) + 0.005\,(F - 35)\,(LL - 40) + 0.01\,(F - 15)\,(PI - 10)$$

where F = Percentage passing through the 0.075 mm or No. 200 sieve
LL = Liquid limit
PI = Plasticity index

The group index is rounded off to the nearest whole number and appended in parentheses. If the computed group index is either zero or negative, the number zero is used as the group index. In determining the GI for A-2-6 and A-2-7 subgroups, the LL part of the GI equation is not used.

The Group Index range is provided in Table 6.

Type of subgrade soil	Group index range of subgrade
Good	0 – 1
Fair	2 – 4
Poor	**5 – 9**
Very poor	10 – 20

Table 6 Group Index Range

Example 3: AASHTO Soil Classification

A specimen of soil was tested in the laboratory, and the results of the laboratory tests were as follows: Liquid Limit = 39, Plastic Limit = 19
The sieve analysis data is presented in Table 7.
Determine the AASHTO classification of this soil (symbol plus Group Index).

Sieve size	Percent passing
No. 4	100
No. 10	85
No. 40	75
No. 200	45

Table 7 Sieve Analysis Data for Example 3

Solution
Step 1. Passing No. 200 sieve is more than 35%; therefore, soil is within A-4 to A-7.
Step 2. LL = 39 and PI = 39 – 19 = 20; therefore, soil is A-6 as per Table 5.
Step 3. GI = 0.2 (F – 35) + 0.005 (F – 35) (LL – 40) + 0.01 (F – 15) (PI – 10)
= 0.2 (45 – 35) + 0.005 (45 – 35) (39 – 40) + 0.01 (45 – 15) (20 – 10)
= 2 – 0.05 + 3
= 4.95 use 5, Poor soil as per Table 6.
Answer: The soil type is A-6(5).

Proctor Test

Proctor test is performed to determine the optimum moisture content (OMC) and the maximum dry density (MDD) of soils as illustrated in Figures 15 and 16. The dry density of a soil increases with the increase in moisture content. After the optimum moisture content, the dry density decreases with the increase in moisture content.

Compaction is quantified considering soil's dry density, which can be expressed as shown in the equation:

$$\gamma_d = \frac{\gamma}{1 + w}$$

where γ_d = Dry density of soil
γ = Moist density of soil
w = Moisture content of soil

Figure 15 Proctor Compaction Test

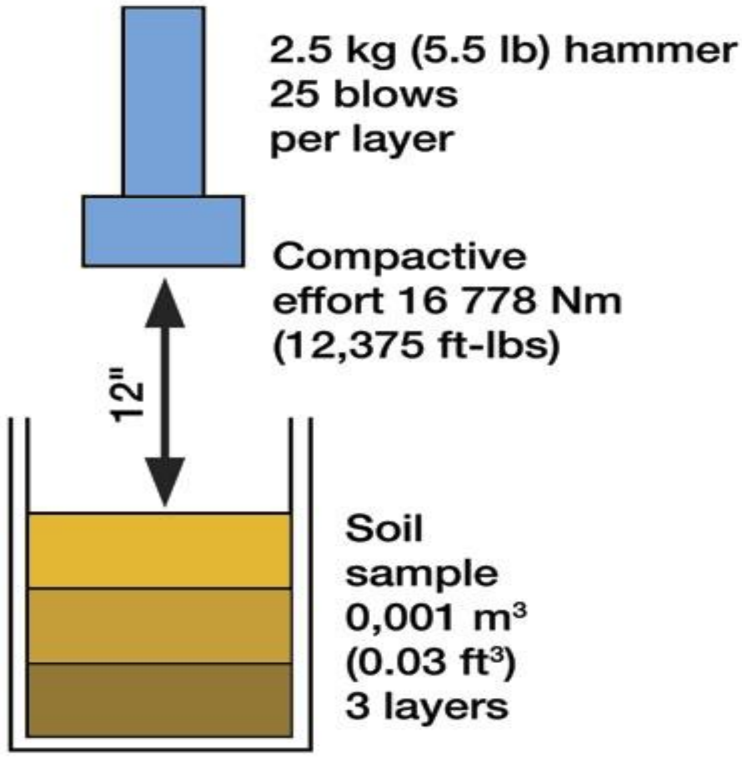


Figure 16 Proctor Compaction Test Procedure

The unit weight of moist soil can be determined using the equation:

$$\gamma = \frac{W}{V}$$

where W = Weight of moist soil
V = Volume of moist soil

Compaction Test Results and Compaction Curve

At least three specimens of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak, as shown in Figure 17. Such inverted "V" curves are obtained for cohesive soils (or soils with fines) and are known as compaction curves.

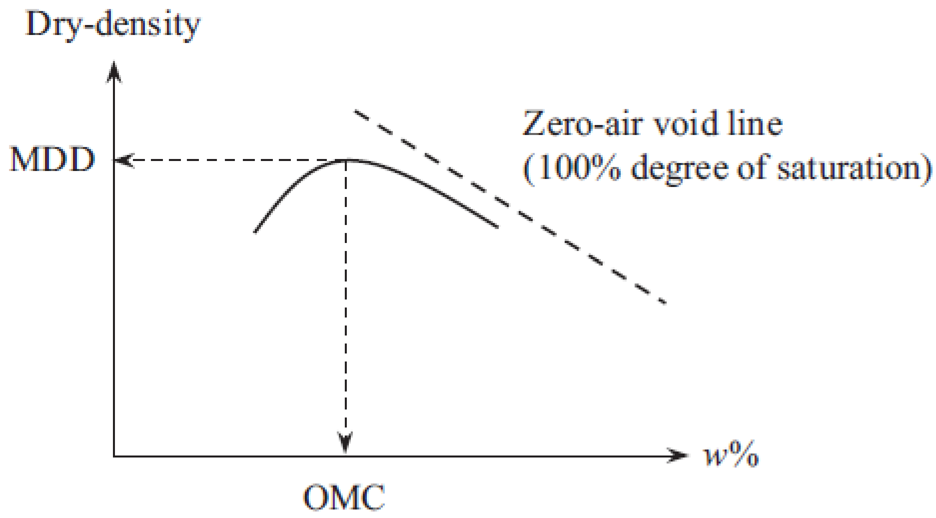


Figure 17 Compaction Test Result

The relation between the moisture content and the dry unit weight for a saturated soil is the zero air-voids line. It is not feasible to expel all the air completely by compaction, no matter how much compactive effort is used and in whatever manner. As water is added to a soil at a low moisture content, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced, and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them. This increase in dry density continues until a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus, the water at this stage hinders the closer packing of grains and reduces the dry unit weight. The MDD occurs at an OMC and their values can be obtained from the plot.

Laboratory Compaction Test

Laboratory compaction test is conducted in the laboratory using the ASTM D 698 test protocol, known as the standard Proctor test. The soil is usually compacted inside a standard mold in three equal layers, each receiving a number of blows from a standard weighted hammer at a specified height. This process is then repeated for various moisture contents and the dry density is determined for each specimen. The graphical relationship of the dry density to moisture content is then plotted to establish the compaction curve. The peak of the curve gives the OMC and MDD.

Example 4: Proctor Test

A compaction test was conducted in a soils laboratory, and the standard Proctor compaction procedure (ASTM D 698) was used. The weight of a compacted soil specimen plus mold was determined to be 4,805 g. The volume and weight of the mold were 1/30 ft^3 and 3,060 g, respectively. The water content of the specimen was 8%. Compute both the wet and dry unit weights of the compacted specimen.

Solution

Weight of wet soil = 4,805 – 3,060 g = 1,745 g

Volume of soil = 1/30 ft³

Unit weight of wet soil, $\gamma = \frac{W}{V} = \frac{1{,}745 \text{ g}}{\frac{1}{30}\text{ft}^3} = 52{,}350\frac{\text{g}}{\text{ft}^3} = 115\frac{\text{lb}}{\text{ft}^3}$ [1 g = 0.0022 lb]

Moisture content, $w = 8\% = 0.08$

Unit weight of dry soil, $\gamma_d = \frac{\gamma}{1+w} = \frac{115 \text{ pcf}}{1+0.08} = 107 \text{ pcf}$

Answers:

Wet unit weight = 115 lb/ft³

Dry unit weight = 107 pcf

Flat and Elongated Particles

Flat and elongated particles (ASTM D 4791) are a consensus aggregate property requirement in the Superpave asphalt mix design process and it is performed on coarse aggregate larger than No. 4 sieve (4.75 mm). ASTM D 4791 is used to determine the flat and elongated particles and defined as follows (Figures 18, 19, and 20):

- A flat particle is defined as one where the ratio of the middle dimension to the smallest dimension of the particle exceeds 3:1.
- An elongated particle is defined as one where the ratio of the largest dimension to the middle dimension of the particle exceeds 3:1.
- Particles are classified as flat and elongated if the ratio of the largest dimension to the smallest dimension exceeds 5:1.

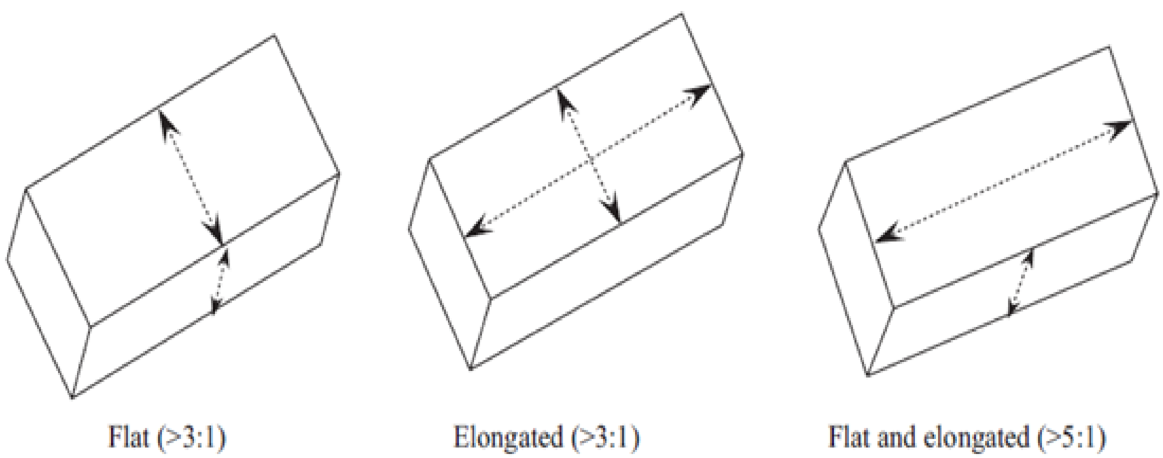


Figure 18 Concept of Flakiness Test for Coarse Aggregates

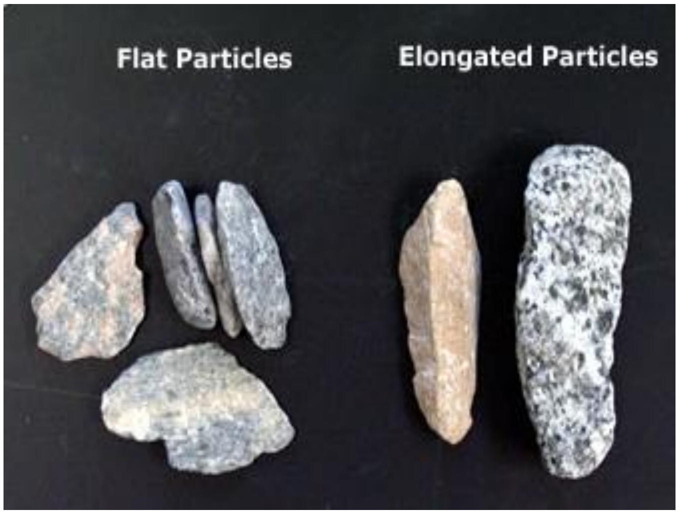


Figure 19 Flat and Elongated Particles

Figure 20 Sizes of Flat and Elongated Particles

Elongated particles are undesirable because they have a tendency to break during construction and under traffic. The maximum percentage of flat and elongated particles is commonly used as 10% by weight for total equivalent single axle of load of 0.3 million or more.

Fine Aggregate Angularity

Fine aggregate angularity (FAA) is also a consensus aggregate property requirement in the Superpave asphalt mix design process and is defined as the percent air voids present in a loosely compacted aggregates smaller than No. 8 sieve (2.38 mm). This property ensures enough fine aggregate internal friction and rutting resistance. In the test by the AASHTO T 304 or ASTM C 1252 (Uncompacted Void Content of Fine Aggregate), a specimen of fine aggregate is poured into a small calibrated cylinder by flowing through a standard funnel. By determining the weight of fine aggregate in the filled cylinder of known volume, void content can be calculated as the difference between the cylinder volume and fine aggregate volume collected in the cylinder. The uncompacted void content is calculated using the equation:

$$\text{Air void} = \frac{V - \left(\frac{M}{G}\right)}{V} \times 100$$

where V = Volume of the cylindrical measure (cm^3, cc, or mL, no other unit is possible)
M = Specimen mass (g, no other unit is possible)
G = Dry bulk specific gravity of the fine aggregate (discussed in the next subsection)

This value is also known as the FAA. The higher the FAA, the higher the angularity and rough surface. The fine aggregate bulk specific gravity is used to compute the fine aggregate volume. The required minimum value for fine aggregate angularity is a function of traffic level and depth within pavement. These requirements apply to the final aggregate blend, although estimates can be made on the individual aggregate stockpiles. Higher void contents mean more fractured faces. The minimum percentage of air voids in loosely compacted fine aggregate is commonly used as 40% to 45% by weight for total equivalent single axle of load of 0.3 million or more.

The FAA test estimates fine aggregate angularity by measuring the loose uncompacted void content of a fine aggregate sample. On a sample of known gradation, the loose uncompacted void content is indicative of the relative angularity and surface texture of the sample. The higher the void content, the higher the assumed angularity and rougher the surface. This test is used to ensure that the blend of fine aggregate has sufficient angularity and texture to resist permanent deformation (rutting) for a given traffic level (Figures 21 and 22).

Figure 21 FAA Test

The FAA test indirectly measures the angularity of fine aggregate using the aggregate's uncompacted void content. Angularity is a description of the degree of roughness, surface irregularities or sharp angles of the aggregate particles. Angular particles do not compact as readily as rounded particles because their angular surfaces tend to lock up with one another and resist compaction, while smoother, more rounded surfaces tend to pass by one another allowing for easier compaction. Therefore, the higher the measured uncompacted void content, the more angular the material.

Angular materials are desirable in paving mixtures because they tend to lock together and resist deformation after initial compaction, whereas rounded materials may not produce sufficient inter-particle friction to prevent rutting. The measured uncompacted voids are affected by the shape, angularity and texture of the fine aggregate, the aggregate grading and specific gravity.

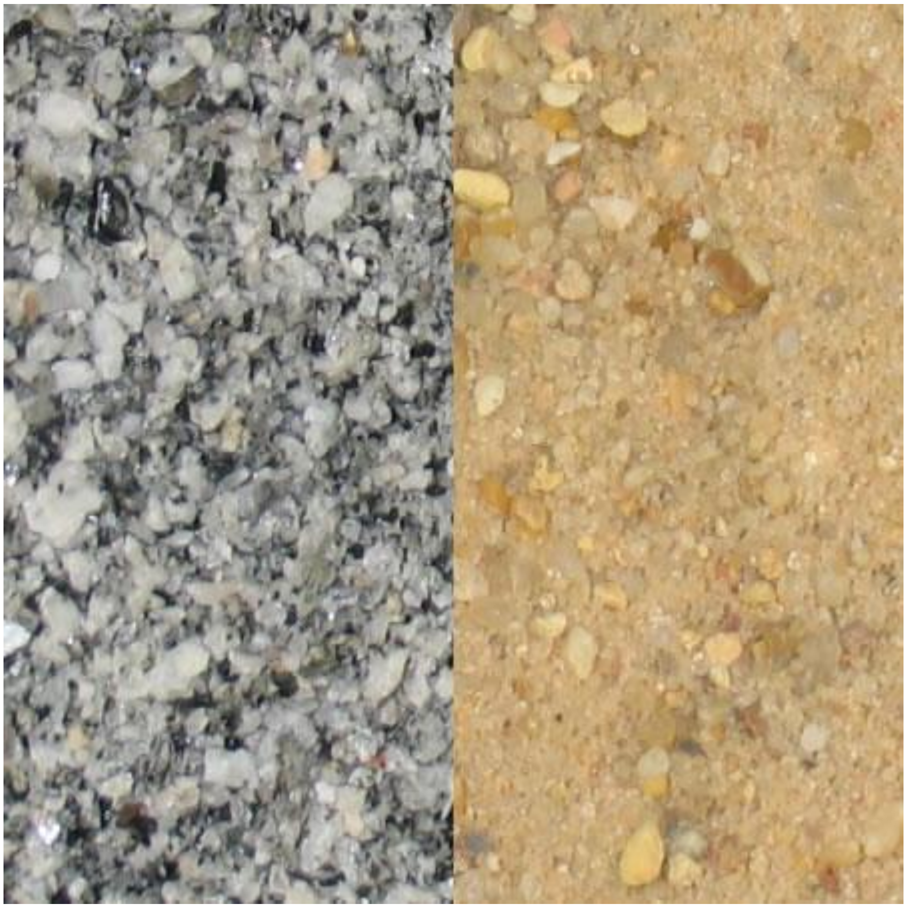


Figure 22 FAA Test Results

FAA can indicate the effect of the fine aggregate portion of the mixture on the overall properties of the mixture. It is generally believed that low FAA values will result in an unstable HMA mixture (e.g., potential for rutting and shoving) and high FAA values will result in a stable HMA mixture if all other mixture properties are satisfactory.

The National Aggregates Association (NAA) developed the original test upon which the FAA test is based. The FAA test has been adopted as a Superpave consensus aggregate requirement.

Example 5: Air Void of a Loosely-Filled Soil

A 100-cm^3 container is loosely filled with 225 g of fine soil with specific gravity of 2.45. Determine the air void of the loosely filled soil.

Solution

Given:

Volume of the cylindrical measure, V = 100 cm^3

Specimen mass, M = 225 g

Specific gravity, G = 2.45

The uncompacted void content is calculated as:

$$\begin{aligned} \text{Air void} &= \frac{V-\left(\frac{M}{G}\right)}{V}\times 100 \\ &= \frac{100\,\text{cm}^3-\left(\frac{225\,\text{g}}{2.45}\right)}{100\,\text{cm}^3}\times 100 \\ &= 8.16\% \end{aligned}$$

Answer

Air void = 8.2%

Note: Air void is commonly rounded to the nearest 0.1%.

Coarse Aggregate Angularity

Coarse aggregate angularity (CAA) ensures a high degree of aggregate internal friction and mix rutting resistance. It is defined as the percent by weight of aggregates larger than No. 4 sieve (4.75 mm) with one or more fractured faces. ASTM D 5821 procedure involves manually counting particles to determine fractured faces. A fractured face is defined as any fractured surface that occupies more than 25% of the area of the outline of the aggregate particle visible in that orientation. The required minimum values for coarse aggregate angularity are a function of traffic level and position within the pavement and vary between 50% and 100%.

The coarse aggregate angularity (CAA) test is a method of determining the angularity of coarse aggregates (Figures 23 and 24). Coarse aggregate angularity is important to ensure adequate aggregate interlock and prevent excessive HMA deformation under load (rutting).

This test is used to help ensure that the resulting HMA mixture will be resistant to deformation under repeated loads. Specifying a minimum percentage of coarse aggregate angularity can also be used to obtain improved durability for aggregates used in surface treatments and to obtain increased friction and texture for aggregates used in pavement surface courses.

Figure 23 CAA Test

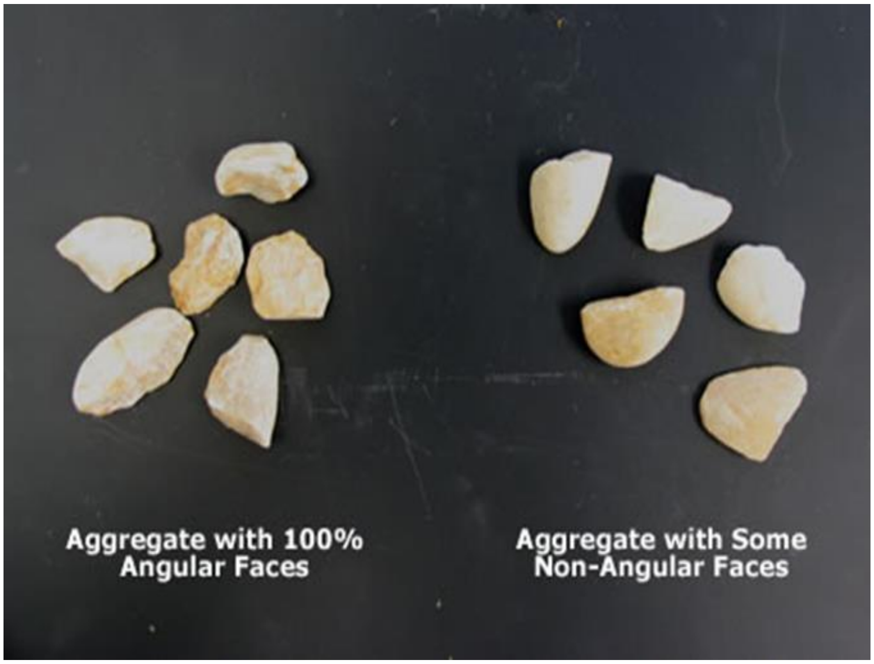


Figure 24 CAA Test Results

Clay Content

Clay content is the percentage of clay material contained in the aggregate fraction finer than No. 4 sieve (4.75 mm). It is measured following the AASHTO T 176 test protocol. In this test, a specimen of fine aggregate is placed in a graduated cylinder, as shown in Figures 25 and 26, with a flocculating solution and agitated to loosen clayey fines present in and coating the aggregate. The flocculating solution forces the clayey material into suspension above the granular aggregate. After a period that allows sedimentation, the cylinder height of suspended clay and sedimented sand is measured. The sand equivalent (SE) value is computed as a ratio of the sand to clay height readings expressed as a percentage. The required clay content values (40 – 50%) for fine aggregate are expressed as a minimum sand equivalent and are a function of traffic level. The ratio of the height of sand over the height of clay is considered the SE, as shown in the equation below. The larger is the sand equivalent value, the cleaner (less fine dust or clay-like materials) is the aggregate.

$$\mathrm{SE} = \frac{\text{Sand height}}{\text{Clay height}} \times 100$$

Figure 25 Testing for Silt Content in Sand

Figure 26 Sand Equivalent Testing

Example 6: Clay Content

In a sand equivalent test, a clay layer of 13 mm is deposited on the 23 mm of sand particles. Calculate the SE of the specimen.

Solution

Sand height = 23 mm

Clay height = 23 + 13 mm = 36 mm

$$SE = \frac{\text{Sand height}}{\text{Clay height}} \times 100 \frac{23\,\text{mm}}{36\,\text{mm}} \times 100 = 64\%$$

Answer The SE is 64%.

Note: Sand equivalent is commonly rounded to the nearest 1%.

Los Angeles (LA) Abrasion

Los Angeles (LA) abrasion test is a toughness test. This test simulates the resistance of coarse aggregate to abrasion and mechanical degradation during handling, construction, and in-service. The Los Angeles (LA) abrasion test is widely used as an indicator of the relative quality of aggregates. It measures the degradation of standard grading of aggregates when subjected to abrasion and impact in a rotating steel drum with an abrasive charge of steel balls. The percent loss of materials from an aggregate blend during the LA abrasion test (AASHTO T 96 or ASTM C131) is measured. The test is performed by subjecting the coarse aggregate and some steel spheres inside a large drum, as shown in Figure 27. After rotating the drum, the weight of the aggregate that is retained on a No. 12 (1.70 mm) sieve is subtracted from the original weight to obtain a percentage of the total aggregate weight that has broken down and passed through the No. 12 (1.70 mm) sieve. The test result is percent loss, which is the weight percentage of coarse material lost during the test as a result of the mechanical degradation. Maximum loss values typically range from approximately 35% to 45%.

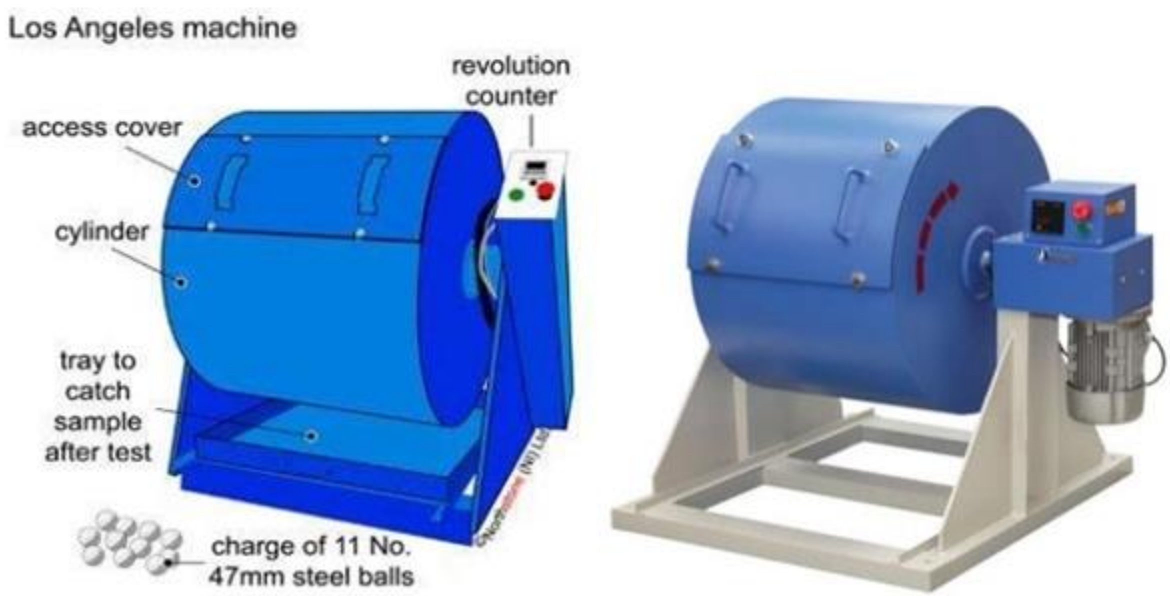


Figure 27 Los Angeles Abrasion Test Apparatus

The abrasion test is significant as it determines the relative quality, toughness, and durability of mineral aggregates subjected to impact and abrasion. This testing offers insight into how asphalt and concrete aggregates will stand up to wear and tear over time.

Table 8 presents the maximum permissible abrasion value in percent for each type of pavement.

Sl. No.	Type of Pavement	Max. permissible abrasion value in %
1	Water bound macadam sub base course	60
2	WBM base course with bituminous surfacing	50
3	Bituminous bound macadam	50
4	WBM surfacing course	40
5	Bituminous penetration macadam	40
6	Bituminous surface dressing, cement concrete surface course	35
7	Bituminous concrete surface course	30

Table 8 Maximum Permissible Abrasion Value in % for each Type of Pavement

How does the L.A. Abrasion Test Work?

1. A sample is prepared by separating into individual size fractions of the required masses.
2. The sample of specifically sized aggregates and the abrasive charge is placed in the L.A. Abrasion Machine and rotated at 30-33 rpm.
3. The sample is removed and washed over a No. 12 (1.70 mm) sieve and placed in an oven to dry.
4. The percent loss or the difference between the original mass and the final mass is calculated. For example, an L.A. Abrasion loss value of 40 indicates that 40% of the original sample mass passed through the sieve.

What is the Use of the Abrasion Test?

1. To find out what could be the Los Angeles abrasion value of aggregate before use in specific operations.
2. To determine the suitability of aggregates for use in road construction projects.
3. Ultimately, the Hardness of an aggregate can be determined by knowing the properties of the aggregate.

Soundness

Soundness test estimates the resistance of aggregates to weathering while in-service. It can be performed on both coarse and fine aggregates. This test measures the percent loss of materials from an aggregate blend during the sodium or magnesium sulfate soundness test according to the AASHTO T 104 standard as shown in Figure 28. The test is performed by alternately exposing an aggregate specimen to repeated immersions in saturated solutions of sodium or magnesium sulfate each followed by oven drying. One immersion and drying is considered one soundness cycle. During the drying phase, salts precipitate in the permeable void space of the aggregate. Upon reimmersion, the salt rehydrates and exerts internal expansive forces that simulate the expansive forces of freezing water. The test result is total percent loss over various sieve intervals for a required number of cycles. The maximum loss values range from approximately 10% to 20% for five cycles.

Figure 28 Soundness Test

Deleterious Materials

Deleterious materials are defined as the weight percentage of contaminants, clay lumps, and friable particles such as shale, wood, mica, and coal in the blended aggregate. The test is performed by wet sieving aggregate size fractions over prescribed sieves following the AASHTO T 112 standard as shown in Figure 29. The weight percentage of material lost as a result of wet sieving is reported as the percent of clay lumps and friable particles. A wide range of maximum permissible percentage of clay lumps and friable particles is evident. Values range from 0.2% to 10%, depending on the exact composition of the contaminant. Some of the adverse effects of several deleterious substances are listed below:

1. Organic impurities delay setting and hardening of concrete and reduce strength gain.
2. Coal, lignite, clay lumps, and friable particles increase popouts and reduce durability.
3. Fines passing No. 200 sieve (smaller than 0.075 mm) weaken the bond between asphalt and aggregates.

Six types of deleterious substances exist in aggregates: Organic Impurities, Clay, Silt and Crusher Dust, Salts, Unsound Particles, and Alkali- Aggregate Reactions.

Figure 29 Deleterious Materials in Aggregates

Mechanical Properties

In pavement design, each layer of pavement is treated in mechanistic fashion. More clearly, the stress-strain in each layer of pavement is determined using mechanistic analysis. Hence, the stiffness of soil and aggregate layers are essential design input parameters. In general, the more resistant to deformation a subgrade/base is, the more load it can support before reaching a critical deformation value. There are other factors involved when evaluating subgrade materials (such as shrink/swell in the case of certain clays and ash) which are minors. The methods of stiffness characterization are discussed here.

The mechanical properties of soils include: Resilient Modulus or Elasticity, California Bearing Ratio (CBR), Resistance, Dynamic Cone Penetration (DCP), Poisson's Ratio, Maximum Dry Density, Optimum Moisture Content, Specific Gravity, Saturated Hydraulic Conductivity, Soil-Water Characteristic Curve Parameters.

Resilient Modulus or Elasticity

The resilient modulus (MR) of unbound layer is the most desired input parameter in the AASHTOWare pavement ME design software. This test is conducted according to the AASHTO T 307 test sequence for base/subbase/subgrade materials. This method measures the elastic modulus of untreated base, subbase, and soil materials. A repeated axial cyclic stress of fixed magnitude is applied to a laboratory-prepared cylindrical specimen. During testing, the specimen is subjected to a dynamic cyclic stress and a static confining pressure by means of a pressure chamber. The total recoverable axial deformation of the specimen is measured and used to calculate the MR value as shown in the equation:

$$M_R = \frac{S_{\text{cyclic}}}{\varepsilon_r}$$

where S_{cyclic} is the applied cyclic axial stress and ε_r is the resilient axial strain as presented in the equations below:

$$S_{\text{cyclic}} = \frac{P_{\text{cyclic}}}{A}$$

where P_{cyclic} is the applied cyclic load and A is the cross-sectional area of the specimen.

$$\varepsilon_r = \frac{e_r}{L}$$

where e_r is the resilient axial deformation due to S_{cyclic} and L is the original specimen length.

Resilient Modulus Test

The specimen diameters are typically 6.0, 4.0, and 2.5 in. for the base, subbase, and subgrade materials, respectively. The height of the specimens is taken at least twice of the diameter. The specimen is compacted using dynamic impact compaction by the modified Proctor effort. Prior to the testing, the specimen is capped with gypsum to prepare a uniform top surface. A cyclic haversine–shaped load is applied in a triaxial pressure chamber. Each test sequence (1 s) has 0.1 s of load pulse and 0.9 s of rest period. The average stress-strain at the last five cycles is considered for M_R calculation for each loading condition. ASTM D7460 recommend the use of haversine loading, in which the equipment applies a one-way range displacement, bending the specimen in one direction only in relation to its neutral axis. A cyclic haversine - shaped load is applied in a triaxial pressure chamber. Each test sequence (1 s) has 0.1 s of load pulse and 0.9 s of rest period. The average stress-strain at the last five cycles is considered for M_R calculation for each loading condition. A prepared specimen and a test-ready specimen are shown in Figures 30 and 31.

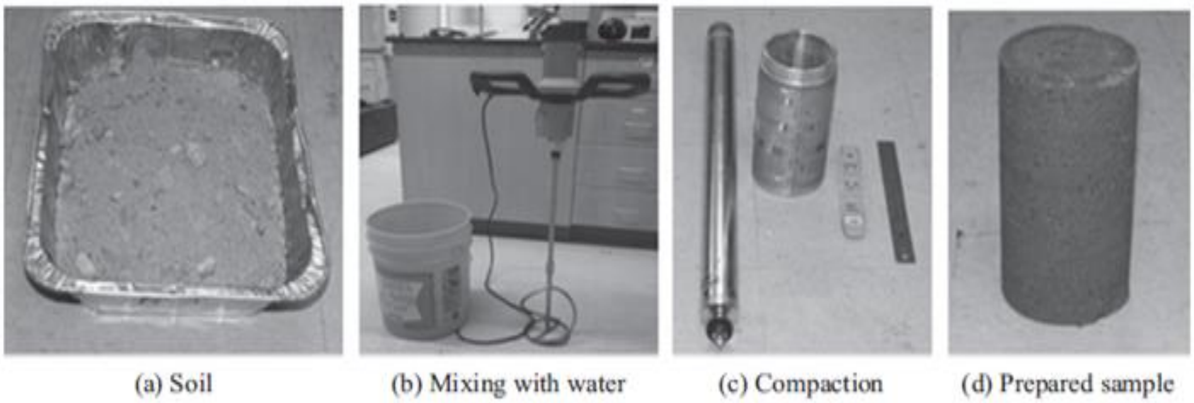


Figure 30 Specimen Preparation

Figure 31 Resilient Modulus Test Set-Up

Example 7: Resilient Modulus

In a resilient modulus test, a cylindrical specimen of 100-mm diameter and 200-mm height is used. The applied total axial force is 2,100 N. The resulting axial deformation in the specimen is 0.39 mm. Determine the resilient modulus of the specimen.

Solution

Specimen height, L = 200 mm = 0.2 m, Specimen diameter, D = 100 mm = 0.1 m.

Specimen cross-sectional area, $A = \frac{\pi D^2}{4} = \frac{\pi (0.1\ \mathrm{m})^2}{4} = 0.007854\ \mathrm{m}^2$

Applied stress, $S_{\mathrm{cyclic}} = \frac{P_{\mathrm{cyclic}}}{A} = \frac{2{,}100\,\mathrm{N}}{0.007854\,\mathrm{m}^2} = 267{,}380\ \mathrm{Pa}$

Resulting strain, $\varepsilon_r = \frac{e_r}{L} = \frac{0.39\,\mathrm{mm}}{200\,\mathrm{mm}} = 0.00195$

Resilient modulus, $M_R = \frac{S_{\mathrm{cyclic}}}{\varepsilon_r} = \frac{267{,}380\,\mathrm{Pa}}{0.00195} = 137{,}117{,}950\ \mathrm{Pa} = 137\ \mathrm{MPa}$

Answer *The resilient modulus is 137 MPa. Note: M_R is commonly rounded to nearest hundreds of psi or nearest MPa.*

California Bearing Ratio (CBR)

CBR in Laboratory

The California bearing ratio (CBR) test compares the bearing capacity of a material with that of a well-graded crushed stone. Thus, a high-quality crushed stone material should have a CBR at 100%. It is primarily intended for, but not limited to, evaluating the strength of cohesive materials having maximum particle sizes less than 19 mm (0.75 in.). It was developed by the California Division of Highways about the year 1930 and was subsequently adopted by numerous design agencies. This test can be performed on in situ base/subbase/soil (ASTM D 4429), or laboratory-compacted base/subbase/soil (ASTM D1883 or AASHTO T 193).

In the laboratory, the basic CBR test involves applying a load to a small penetration piston (Figures 32 and 33) at a rate of 1.3 mm (0.05 in.) per minute and recording the total load at penetrations ranging from 0.64 mm (0.025 in.) up to 7.62 mm (0.30 in.).

Figure 32 CBR Test Equipment and its Accessories

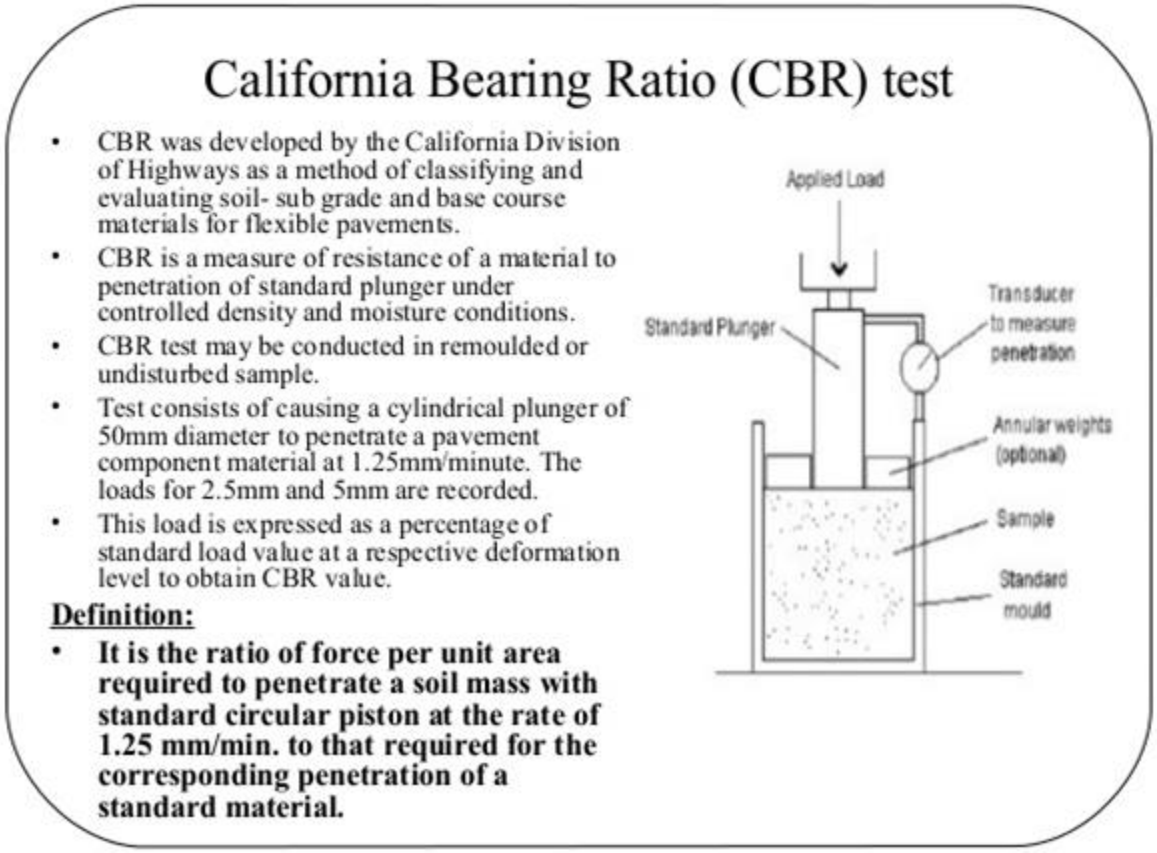


Figure 33 CBR Testing Procedure

The values obtained are inserted into the following equation to obtain the CBR value:

$$\text{CBR (\%)} = 100\left(\frac{x}{y}\right)$$

where x = Resistance pressure on the piston for 0.1 in. (2.54 mm) or 0.2 in. (5.08 mm) penetration

y = Standard pressure required for a well-graded crushed stone

• 1,000 psi (6.9 MPa) for 0.1 in. (2.54 mm) penetration

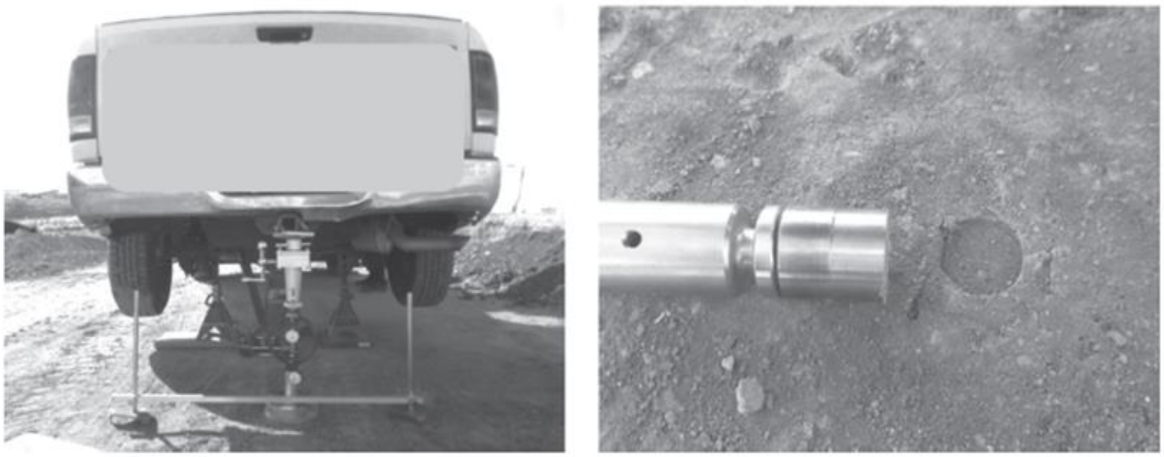
• 1,500 psi (10.3 MPa) for 0.2 in. (5.08 mm) penetration

The M_R can be obtained empirically from the CBR value using the equation:

M_R (psi) = $2555(\text{CBR})^{0.64}$ for CBR > 10, and = 1500(CBR) for CBR < 10

CBR in Field

The CBR can also be determined for in situ condition (ASTM D 4429) as per Figures 34 and 35. It is similar in nature to laboratory testing. The main difference is the soil compaction and loading frame. In the field, the in-situ soil being tested thus does not require any soil preparation. The loading frame is prepared by a heavy vehicle.



Testing setup　　　　Penetration after testing

Figure 34 In Situ CBR Testing Setup

Figure 35 In Situ CBR Testing

Example 8: CBR and Resilient Modulus

While conducted the CBR test on an aggregate layer, a load of 1,750 lb was required to penetrate 0.1 in. The diameter of the penetrating piston was 1.95 in.

Determine the CBR and the resilient modulus of the aggregate layer.

Solution

Piston diameter, D = 1.95 in.

Piston cross-sectional area, $A = \frac{\pi D^2}{4} = \frac{\pi(1.95\text{ in.})^2}{4} = 2.99\text{ in.}^2$

Applied stress, $x = \frac{\text{Load}}{\text{Area}} = \frac{1{,}750\,\text{lb}}{2.99\,\text{in.}^2} = 585.3\,\text{psi}$

Known: Stress required for a crushed stone to penetrate by 1.0 in., y = 1,000 psi

California bearing ratio, $\text{CBR}\,(\%) = 100\left(\frac{x}{y}\right) = 100\left(\frac{585.3\,\text{psi}}{1{,}000\,\text{psi}}\right) = 58.53 \approx 59$

Resilient modulus, $M_R(\text{psi}) = 2{,}555(\text{CBR})^{0.64} = 2{,}555\,(59)^{0.64} = 34{,}733\,\text{psi}$

Answers *The CBR is 59 and the resilient modulus is 34,700 psi.*

Resistance

The resistance value (*R*-value) test is a material stiffness test conducted by following the AASHTO T 190 or the ASTM D 2844 standard. The materials tested are assigned an *R*-value. The *R*-value test was developed by the California Division of Highways and first reported in the late 1940s. During this time rutting (or shoving) in the wheel tracks was a primary concern and the *R*-value test was developed as an improvement of the CBR test.

The test procedure to determine *R*-value requires that the laboratory-prepared specimens are fabricated to a moisture and density conditions representative of the worst possible in situ condition of a compacted subgrade. The *R*-value is calculated from the ratio of the applied vertical pressure to the developed lateral pressure, and is essentially a measure of the material's resistance to plastic flow, as shown in the equation below.

$$R = 100 - \left\{ \frac{100}{\left[\left(\frac{2.5}{D}\right)\left[\left(\frac{P_V}{P_H}\right) - 1\right] + 1\right]} \right\}$$

Where R = resistance value from 0 to 100, P_V = applied vertical pressure (160 psi), P_H = applied horizontal pressure at P_V = 160 psi, and D = displacement of stabilometer fluid necessary to increase the P_H from 0 to 100 psi.

The testing apparatus used in the R-value test is called a stabilometer and is shown in Figure 36

Figure 36 R-Value Test Equipment

The M_R can be obtained empirically from the R-value using the equation:

M_R (psi) = 1155 + 555R

Dynamic Cone Penetration (DCP)

The DCP test provides a measure of a material's in situ resistance to penetration following the ASTM D 6951. DCP testing execution is shown in Figure 37. It consists of a rod with a standard sliding weight called hammer attached to the top and a disposable cone tip to penetrate the soil on the bottom. The weight of the hammer is 8 kg (17.6 lb) and it slides on a 16-mm driving rod. The tip has an included angle of 60 degrees and a diameter at the base of 20 mm.

The hammer is lifted up and dropped from a standard height of 2.26 feet (575 mm), which causes the cone at the bottom of the device to be forced into the ground. The weight is dropped multiple times until there are enough blows to determine the soil characteristics or the cone has reached a depth of interest. With each blow the new depth of the device is recorded. The depths and corresponding blow numbers are then plotted in Excel where a best linear fit is applied. The slope is considered the DCP value and is usually measured in millimeters per blow or inches per blow.

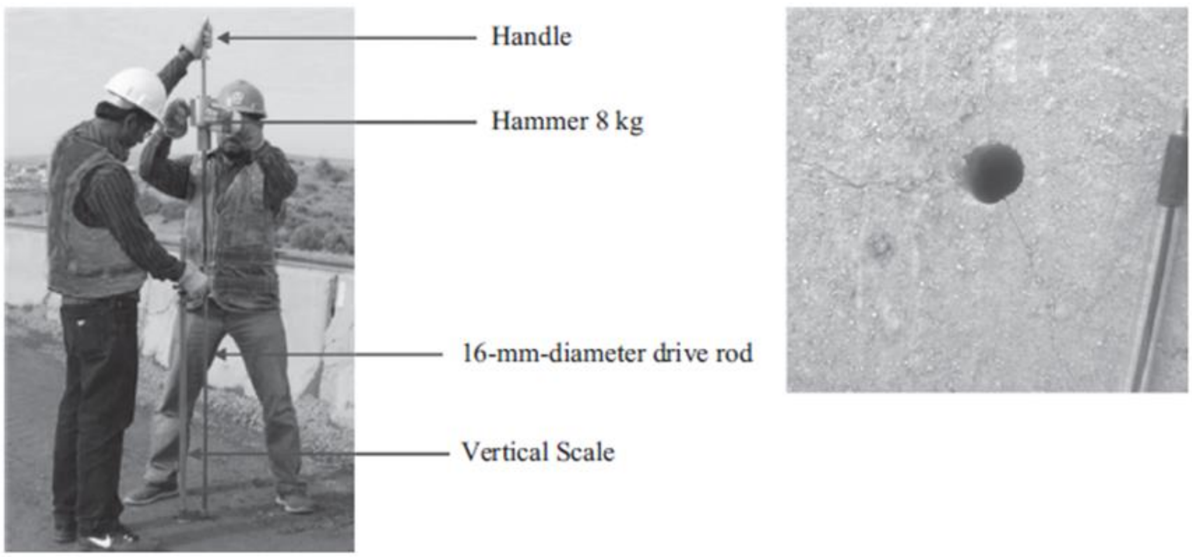


Figure 37 DCP Testing

The M_R can be obtained empirically from the *R*-value using the equation:

$$M_R(\text{psi}) = 2{,}555\left(\frac{292}{\text{DCP}^{1.12}}\right)^{0.64}$$

where the DCP value is in millimeters per blow. DCP index is related to the CBR, as in the equation below:

$$\text{CBR} = \frac{292}{\text{DCP}^{1.12}}$$

Example 9: Analysis of DCP Test Data

The DCP test readings on a pavement site are listed in Table 9. Determine the DCP, MR and CBR values.

Blow	Penetration reading (mm)
0	2.5
1	15.6
2	25.2
3	31.6
4	36.8
5	42.0
6	47.2
7	52.4
8	57.6
9	62.8
10	67.8

Table 9 Penetration Reading for Example 9

Solution

The penetration at each blow of loading is first calculated (Table 10).

Blow	Penetration reading (mm)	Penetration (mm)
0	2.5	0
1	15.6	15.6 – 2.5 = 13.1
2	25.2	25.2 – 15.6 = 9.6
3	31.6	31.6 – 25.2 = 6.4
4	36.8	36.8 – 31.6 = 5.2
5	42.0	42.0 – 36.8 = 5.2
6	47.2	47.2 – 42.0 = 5.2
7	52.4	52.4 – 47.2 = 5.2
8	57.6	57.6 – 52.4 = 5.2
9	62.8	62.8 – 57.6 = 5.2
10	67.8	67.8 – 62.8 = 5.0

Table 10 Penetration Calculation for Example 9

At the beginning, the DCP value is very high (13.1 mm, 9.6 mm, etc.) and then becomes constant. This may be due to the reason that the surface soil is commonly loose and becomes dense with increase in depth. The average DCP value can be taken as 5.2 mm/blow. The mathematical average penetration may also be considered neglecting the penetration by the first few blows.

Resilient modulus, $M_R(\text{psi}) = 2{,}555\left(\frac{292}{\text{DCP}^{1.12}}\right)^{0.64} = 2{,}555\left(\frac{292}{5.2^{1.12}}\right)^{0.64} = 29{,}650\,\text{psi} \approx$ 29,700 psi

$$\text{CBR} = \frac{292}{\text{DCP}^{1.12}} = \frac{292}{5.2^{1.12}} = 46$$

Answers: *DCP = 5.2 mm/blow, M_R = 29,700 psi, CBR = 46*

Resilient Modulus from Soil Physical Testing (Sieve Analysis)

The M_R can also be determined empirically if the Atterberg limits and percent of soil passing No. 200 sieve (0.075-mm opening) are known. The equation below shows the relationship that can be used to determine empirically the M_R value.

$$M_R(\text{psi}) = 2{,}555\left(\frac{75}{1 + 0.728(P_{200})(\text{PI})}\right)^{0.64}$$

where P_{200} = Passing No. 200 sieve (use the decimal value), and PI = Plasticity index = the difference between liquid limit and plastic limit.

Example 10: Resilient Modulus from Sieve Analysis

The LL and PL of a soil are 38 and 22, respectively. After sieve analysis, it was found that 7.5% of soil passes No. 200 sieve. Calculate the resilient modulus of the soil.

Solution

Given data: Plasticity index, PI = LL − PL = 38 − 22 = 16 Soil passing No. 200 sieve, P = 7.5% = 0.075.

$$\begin{aligned}\text{Resilient modulus, } M_R(\text{psi}) &= 2{,}555\left(\frac{75}{1+0.728(P_{200})(PI)}\right)^{0.64}\\ &= 2{,}555\left(\frac{75}{1+0.728(0.075)(16)}\right)^{0.64}\\ &= 27{,}096 \text{ psi}\end{aligned}$$

Answer The resilient modulus is 27,100 psi.

Resilient/Elastic Modulus of Chemically Stabilized Soil

There are some empirical relationships which can be used to determine the resilient/elastic modulus of chemically stabilized soil. For cement-treated aggregate, the modulus of elasticity (E) in psi can be determined as:

$$E = 57{,}000\sqrt{f_c'}$$

where f'_c is the compression strength in psi tested in accordance with AASHTO T 22 standard.

For soil cement, the modulus of elasticity (E) in psi can be determined as:

$$E = 1{,}200\, q_u$$

where q_u is the unconfined compression strength in psi tested in accordance with ASTM D 1633 standard.

For lime-cement fly ash, the modulus of elasticity (E) in psi can be determined as:

$$E = 500 + q_u$$

where q_u is the unconfined compression strength in psi tested in accordance with ASTM C 593 standard.

For lime-stabilized soils, the resilient modulus (M_R) in psi can be determined as:

$$M_R = 0.124\, q_u + 9.98$$

where q_u is the unconfined compression strength in psi tested in accordance with ASTM D 5102 standard.

Resilient Modulus Variations Due to Moisture

The AASHTOWare pavement ME design guide considers the M_R of unbound layer affected by the in situ moisture content. For example, the AASHTOWare pavement ME design guide uses the following model to determine the variation in M_R of unbound layer with the degree of saturation (AASHTO 2015):

$$\log\left(\frac{M_R}{M_{Ropt}}\right) = a + \frac{b-a}{1+\exp\left[\ln\frac{-b}{a} + k_m(S - S_{opt})\right]}$$

where M_{Ropt} = resilient modulus at optimum moisture content; a, b, k_m are regression parameters; $(S - S_{opt})$ = variation in degree of saturation expressed in decimal. For fine-grained materials, a, b, and k_m are – 0.5934, 0.4, and 6.1324, respectively. For coarse-grained materials, a, b, and k_m are – 0.3123, 0.3, and 6.8157, respectively.

The degree of saturation (S) can be determined using the following equation:

$$S = \frac{wG_s}{e}$$

where e = Void ratio, w = Gravimetric moisture content, G_s = Specific gravity of the material. Remember that the degree of saturation (S) and the water content (w) are not the same parameters.

Example 11: Degree of Saturation

A chunk of moist soil weighs 49 lb with a volume of 0.46 ft^3. After oven-drying, the weight of the soil decreases to 38 lb. The specific gravity and void ratio of the soil are 2.64 and 1.1, respectively. Calculate the degree of saturation of the soil.

Solution

Moist weight of soil = 49 lb

Dry weight of soil = 38 lb

Soil volume = 0.46 ft^3

Specific gravity, G = 2.64

Void ratio, e = 1.1

Moisture content, $w = \dfrac{\text{Weight of water}}{\text{Dry weight of soil}} \times 100 = \left(\dfrac{49\,\text{lb} - 38\,\text{lb}}{38\,\text{lb}}\right) \times 100\% = 28.9\%$

Now, $Se = wG_S$

Therefore, $S = \dfrac{wG_S}{e} = \dfrac{0.289(2.64)}{1.1} = 0.69 = 69\%$

Answer *The degree of saturation of the soil is 69%.*

Example 12: Variation of Resilient Modulus with Moisture

The resilient modulus of a fine-grained soil is 20 ksi at the optimum moisture content of 15.5%. If the field moisture contents of the soil are 20% and 10% in rainy season and dry season, respectively, calculate the resilient moduli of that soil in rainy season and dry season.

For fine-grained materials, *a*, *b*, and km are − 0.5934, 0.4, and 6.1324, respectively. The specific gravity of the soil is 2.6 and the void ratio is 1.1.

Solution

Resilient modulus at the optimum moisture content, M_{Ropt} = 20ksi

Regression constant a = − 0.5934

Regression constant, b = 0.4

Regression constant, k_m = 6.1324

Specific gravity, G = 2.6

Void ratio, e = 1.1

Degree of saturation at the optimum moisture content is:

$$S_{\text{opt}} = \frac{wG_S}{e} = \frac{0.155\,(2.6)}{1.1} = 0.3664$$

During the rainy season, the degree of saturation at 20% moisture content is:

$$S = \frac{wG_S}{e} = \frac{0.2\,(2.6)}{1.1} = 0.4727$$

$$\begin{aligned}
\text{Known: } \log\left(\frac{\mathrm{M_R}}{\mathrm{M_{Ropt}}}\right) &= a + \frac{b-a}{1+\exp\left[\ln\frac{-b}{a} + k_m(S-S_{opt})\right]} \\
\log\left(\frac{M_R}{20}\right) &= -0.5934 + \frac{0.4-(-0.5934)}{1+e^{\left[\ln\frac{-0.4}{-0.5934}+6.1324(0.4727-0.3664)\right]}} = -0.1603 \\
\frac{M_R}{20} &= 10^{-0.1603}
\end{aligned}$$

Therefore, the resilient modulus at 20% moisture content, M_R = 13.8ksi.

During the dry season, the degree of saturation at 10% moisture content is:

$$S = \frac{wG_S}{e} = \frac{0.1(2.6)}{1.1} = 0.2364$$

$$\log\left(\frac{M_R}{20}\right) = -0.5934 + \frac{0.4-(-0.5934)}{1+e^{\left[\ln\frac{-0.4}{-0.5934}+6.1324(0.2364-0.3664\right]}} = 0.1686$$

$$\frac{M_R}{20} = 10^{0.1686}$$

Answers **13.8 ksi, 29.5 ksi**

Resilient Modulus Variations Due to Stress Level

The resilient modulus is dependent not only on the moisture content but also on the applied stress level. The resilient modulus should be input in the AASHTOWare pavement ME design software for the average probable stress in field. However, it is always not possible to predict the probable stress in soil or aggregate layer in the field as the traffic-induced stress is mostly uncertain. This is why a generalized model is used in the AASHTOWare pavement ME design software, which is presented in the equation:

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3}$$

where $M_R = \text{Resilient modulus, psi}$

$\theta = \text{Bulk stress} = \sigma_1 + \sigma_2 + \sigma_3$

$\tau_{oct} = \text{Octahedral shear stress} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$

$\sigma_1 = \text{Major principal stress}$

$\sigma_2 = \text{Intermediate principal stress}$

$\sigma_3 = \text{Minor principal stress/confining pressure}$

$\mathrm{p}_a = \text{Normalizing stress}$

$k_1,\ k_2,\ k_3 = \text{Regression constants}$

The laboratory-tested M_R data is fitted using the linear or nonlinear regression analysis. Then the M_R can be determined for any stress possible in pavement.

Poisson's Ratio

Poisson's ratio is defined as the ratio of the lateral strain to the axial strain and is required to calculate the lateral stress-strain upon applying vertical pressure. For soil and aggregate layers, its effect on the pavement response is not very significant. There is no national test standard available so far and the default values included in the AASHTOW are pavement ME design software are mostly used. Depending on the soil type, this value varies from 0.15 to 0.45.

Maximum Dry Density

Maximum dry density is determined using the Proctor test following the AASHTO T 180 test standard.

Optimum Moisture Content

Optimum moisture content is determined using the Proctor test following the AASHTO T 180 test standard.

Specific Gravity

Specific gravity is a required parameter to analyze the phase relationship and calculate the overburden pressure. This parameter can be determined using the AASHTO T 100 test standard.

Saturated Hydraulic Conductivity

Saturated hydraulic conductivity is determined using the AASHTO T 215 test standard.

Soil-Water Characteristic Curve (SWCC) Parameters

SWCC parameters are required to determine the variations in ground moisture over the years. There are different methods of determining SWCC parameters such as pressure plate (AASHTO T 99), filter paper (AASHTO T 180), and Tempe cell (AASHTO T 100).

If Level 1 resilient modulus value cannot be obtained by any of the mentioned testing system, Level 2 or 3 data (available in the AASHTOWare pavement ME design software) listed in Table 11 may be used.

Soil type	Base/subbase for flexible and rigid pavements	Embankment and subgrade for flexible pavements	Embankment and subgrade for rigid pavements
A-1-a	40,000	29,500	18,000
A-1-b	38,000	26,500	18,000
A-2-4	32,000	24,500	16,500
A-2-5	28,000	21,500	16,000
A-2-6	26,000	21,000	16,000
A-2-7	24,000	20,500	16,000
A-3	29,000	16,500	16,000
A-4	24,000	16,500	15,000
A-5	20,000	15,500	8,000
A-6	17,000	14,500	14,000
A-7-5	12,000	13,000	10,000
A-7-6	8,000	11,500	13,000

Table 11 Recommended Levels 2 and 3 M_R (psi) at Optimum Moisture for Different Types of Soils

The performances of different types of soils, such as strength, frost susceptibility, compressibility, drainage performance, etc., are listed in Table 12.

Major divisions	Name	Strength when not subject to Frost action	Potential frost action	Compressibility and expansion	Drainage characteristics
Gravel and Gravelly Soils	Well-graded gravels or gravel-sand mixes, little to no fines; GW	Excellent	None to very slight	Almost none	Excellent
	Poorly graded gravels or gravel-sand mixes little or no fines; GP	Good to excellent	None to very slight	Almost none	Excellent
	Silty gravels, gravels and silt mixes; GM	Good to excellent	Slight to medium	Very slight	Fair to poor
	Very silty gravels, gravel-sand silt mixes; GM	Good	Slight to medium	Slight	Poor to impervious
	Clayey gravels, gravels, and clay mixes; GC	Good	Slight to medium	Slight	Poor to impervious
Sand and Sandy Soils	Well-graded sands or gravelly sands, little to no fines; SW	Good	None to very slight	Almost none	Excellent
	Poorly graded sands or gravelly sands, little or no fines; SP	Fair to Good	None to very slight	Almost none	Excellent
	Silty sands, sand-silt mixes; SP	Fair to Good	Slight to high	Very slight	Fair to poor
	Silty sands, sand-silt mixes; SM	Fair	Slight to high	Slight to medium	Poor to impervious
	Clayey sands, sand-clay mixes; SC	Poor to fair	Slight to high	Slight to medium	Poor to impervious
Silts and Clays with the Liquid Limit Less Than 50	Inorganic silts and very fine sand, rock flour, silty or clayey fine sand or clayey silts with slight plasticity; MG, MS, and ML	Poor to fair	Medium to very high	Slight to medium	Fair to poor
	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays; CG, CL, and CS	Poor to fair	Medium to high	Slight to medium	Practically impervious
	Organic silts and organic silt-clays of low plasticity; MSO and CLO	Poor	Medium to high	Medium to high	Poor
Silts and Clays with Liquid Limit Greater Than 50	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts; MH	Poor	Medium to very high	High	Fair to poor
	Inorganic clays of high plasticity, fat clays; CH	Poor to fair	Medium to very high	High	Practically impervious
	Organic clays of medium to high plasticity, organic silts; MHO and CHO	Poor to very poor	Medium	High	Practically impervious
Highly Organic Soils	Peat and other highly organic soils	Not suitable	Slight	Very High	Fair to poor

Table 12 Summary of Soil Performances as a Pavement Material

Section 4 — References

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