2

Engineering Properties of Soils

2–1 SOIL TYPES

Soils may be classified into three very broad categories: *cohesionless, cohesive,* and *organic* soils. In the case of cohesionless soils, the soil particles do not tend to stick together. Cohesive soils are characterized by very small particle size where surface chemical effects predominate. The particles do tend to stick together—the result of water-particle interaction and attractive forces between particles. Cohesive soils are therefore both sticky and plastic. Organic soils are typically spongy, crumbly, and compressible. They are undesirable for use in supporting structures.

Three common types of cohesionless soils are *gravel, sand,* and *silt.* Gravel has particle sizes greater than 2 millimeters (mm), whereas particle sizes for sand range from about 0.1 to 2 mm. Both gravel and sand may be further divided into "fine" (as fine sand) and "coarse" (as coarse sand). Gravel and sand can be classified according to particle size by sieve analysis. Silt has particle sizes that range from about 0.005 to 0.1 mm.

The common type of cohesive soil is clay, which has particle sizes less than about 0.005 mm. Clayey soils cannot be separated by sieve analysis into size categories because no practical sieve can be made with openings so small; instead, particle sizes may be determined by observing settling velocities of the particles in a water mixture.

Soils can also be categorized strictly in terms of grain size. Two such categories are *coarse-grained* and *fine-grained.* Gravel and sand, with soil grains coarser than 0.075 mm, or a No. 200 sieve size, are coarse-grained (also referred to as *granular* soils); silt and clay, with soil grains finer than 0.075 mm, are fine-grained.

Engineering properties of granular soils are affected by their grain sizes and shapes as well as by their grain-size distributions and their compactness (see Section 2–11). Granular soils, except for loose sand, generally possess excellent engineering properties. Exhibiting large bearing capacities and experiencing relatively small settlements, they make outstanding foundation materials for supporting roads and structures. Granular soils also make excellent backfill materials for retaining walls because they are easily compacted and easily drained, and because they exert small lateral pressures. In addition, as a result of high shear strengths and ease of compaction, granular soils make superior embankment material. One drawback, however, is that the high permeabilities of granular soils make them poor, or even unacceptable, for use alone as earthen dikes or dams.

Cohesive soils (mostly clays but also silty clays and clay–sand mixtures with clay being predominant) exhibit generally undesirable engineering properties compared with those of granular soils. They tend to have lower shear strengths and to lose shear strength further upon wetting or other physical disturbances. They can be plastic and compressible, and they expand when wetted and shrink when dried. Some types expand and shrink greatly upon wetting and drying—a very undesirable feature. Cohesive soils can *creep* (deform plastically) over time under constant load, especially when the shear stress is approaching its shear strength, making them prone to landslides. They develop large lateral pressures and have low permeabilities. For these reasons, cohesive soils—unlike granular soils—are generally poor materials for retaining-wall backfills. Being impervious, however, they make better core materials for earthen dams and dikes.

Silty soils are on the border between clayey and sandy soils. They are fine-grained like clays but cohesionless like sands. Silty soils possess undesirable engineering properties. They exhibit high capillarity and susceptibility to frost action, yet they have low permeabilities and low densities.

Any soil containing a sufficient amount of organic matter to affect its engineering properties is called *organic soil.* As mentioned previously, organic soils are typically spongy, crumbly, and compressible. In addition, they possess low shear strengths and may contain harmful materials. Organic soils are essentially unacceptable for supporting foundations.

More precise classifications of these soil types by particle size according to two systems—the American Association of State Highway and Transportation Officials (AASHTO) system and the Unified Soil Classification System (USCS)—are given in Table 2–1. It is clear from variations between these classifications that boundaries between soil types are more or less arbitrary.

In most applications in this book, soils are categorized as cohesionless or cohesive, with cohesionless generally implying a sandy soil and cohesive, a clayey soil. Some soils encountered in practice are mixtures of both types and therefore exhibit characteristics of both.

2–2 GRAIN-SIZE ANALYSIS

Never will a natural soil be encountered in which all particles are exactly the same size and shape. Both cohesionless and cohesive soils, as well as mixtures of the two, will always contain particles of varying sizes. Properties of a soil are greatly influenced by the sizes of its particles and distribution of grain sizes throughout the soil mass. Hence, in many engineering applications, it is not sufficient to know only that a given soil is clay, sand, rock, gravel, or silt. It is also necessary to know something about the distribution of grain sizes of the soil.

TABLE 2–1
Soil Classification Based on Grain Size[1]

	Coarse-Grained				Fine-Grained	
Agency	*Gravel*	*Coarse Sand*	*Medium Sand*	*Fine Sand*	*Silt*	*Clay*
AASHTO	75–2.00 (3-in.–No. 10 sieves)	2.00–0.425 (No. 10–No. 40 sieves)		0.425–0.075 (No. 40–No. 200 sieves)	0.075–0.002	<0.002
USCS	Coarse: 75–19.0 (3-in.–3/4-in. sieves) Fine: 19.0–4.75 (3/4-in.–No. 4 sieves)	4.75–2.00 (No. 4–No. 10 sieves)	2.00–0.425 (No. 10–No. 40 sieves)	0.425–0.075 (No. 40–No. 200 sieves)	Fines <0.075 (silt or clay)	

[1]All grain sizes are in millimeters.

TABLE 2–2
U.S. Standard Sieve Numbers and Their Sieve Openings

U.S. Standard Sieve Number	Sieve Opening (mm)
4	4.75
10	2.00
20	0.850
40	0.425
60	0.250
100	0.150
200	0.075

In the case of most cohesionless soils, distribution of grain size can be determined by sieve analysis. A sieve is similar to a cook's flour sifter. It is an apparatus containing a wire mesh with openings the same size and shape. When soil is passed through a sieve, soil particles smaller than the opening size of the sieve will pass through, whereas those larger than the opening size will be retained. Certain sieve-size openings between 4.75 and 0.075 mm are designated by U.S. Standard Sieve Numbers, as given in Table 2–2. Thus, grain sizes within this range can be classified according to U.S. Standard Sieve Numbers.

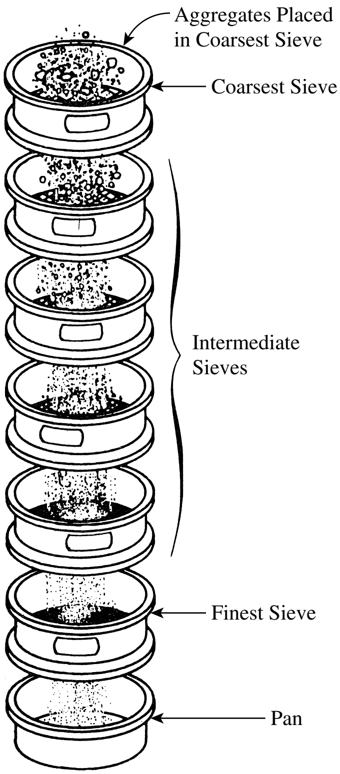


FIGURE 2–1 Sieve analysis.
Source: The Asphalt Handbook, Manual Series No. 4 (MS-4), Asphalt Institute, College Park, MD, 1989.

In practice, sieves of different opening sizes are stacked, with the largest opening size at the top and a pan at the bottom. Soil is poured in at the top, and soil particles pass downward through the sieves until they are retained on a particular sieve (see Figure 2–1). The stack of sieves is mechanically agitated during this procedure. At the end of the procedure, the soil particles retained on each sieve can be weighed and the results presented graphically in the form of a grain-size distribution curve. This is normally a semilog plot with grain size (diameter) along the abscissa on a logarithmic scale and percentage passing that grain size along the ordinate on an arithmetic scale. Example 2–1 illustrates the analysis of the results of a sieve test, including the preparation of a grain-size distribution curve.

EXAMPLE 2–1

Given

An air-dry soil sample weighing 2000 grams (g) is brought to the soils laboratory for mechanical grain-size analysis. The laboratory data are as follows:

U.S. Sieve Size	Size Opening (mm)	Mass Retained (g)
¾ in.	19.0	0
⅜ in.	9.50	158
No. 4	4.75	308
No. 10	2.00	608
No. 40	0.425	652
No. 100	0.150	224
No. 200	0.075	42
Pan	—	8

Required

A grain-size distribution curve for this soil sample.

Solution

To plot the grain-size distribution curve, one must first calculate the percentage retained on each sieve, the cumulative percentage retained, and the percentage passing through each sieve, then tabulate the results, as shown in Table 2–3.

$$\text{Total sample weight} = 2000 \text{ g}$$

1. The percentage retained on each sieve is obtained by dividing the mass retained on each sieve by the total sample mass. Thus,

$$\text{Percentage retained on 3/4-in. sieve} = \frac{0 \text{ g}}{2000 \text{ g}} \times 100\% = 0\%$$

$$\text{Percentage retained on 3/8-in. sieve} = \frac{158 \text{ g}}{2000 \text{ g}} \times 100\% = 7.9\%$$

$$\text{Percentage retained on No. 4 sieve} = \frac{308 \text{ g}}{2000 \text{ g}} \times 100\% = 15.4\% \quad \text{etc.}$$

TABLE 2–3
Sieve Analysis Data for Example 2–1

(1) Sieve Number	(2) Sieve Opening (mm)	(3) Mass Retained (g)	(4) Percentage Retained	(5) Cumulative Percentage Retained	(6) Percentage Passing
¾ in.	19.0	0	0	0	100.0
⅜ in.	9.50	158	7.9	7.9	92.1
No. 4	4.75	308	15.4	23.3	76.7
No. 10	2.00	608	30.4	53.7	46.3
No. 40	0.425	652	32.6	86.3	13.7
No. 100	0.150	224	11.2	97.5	2.5
No. 200	0.075	42	2.1	99.6	0.4
Pan	—	8	0.4	100.0	—

Therefore,

$$\text{Column } (4) = \frac{\text{Column } (3)}{\text{Total sample mass}} \times 100\%$$

2. The cumulative percentage retained on each sieve is obtained by summing the percentage retained on all coarser sieves. Thus,

$$\begin{aligned}
\text{Cumulative percentage retained on 3/4-in. sieve} &= 0\% \\
\text{Cumulative percentage retained on 3/8-in. sieve} &= 0\% + 7.9\% = 7.9\% \\
\text{Cumulative percentage retained on No. 4 sieve} &= 7.9\% + 15.4\% \\
&= 23.3\% \\
\text{Cumulative percentage retained on No. 10 sieve} &= 23.3\% + 30.4\% \\
&= 53.7\% \qquad \text{etc.}
\end{aligned}$$

3. The percentage passing through each sieve is obtained by subtracting from 100% the cumulative percentage retained on the sieves. Thus,

$$\begin{aligned}
\text{Percentage passing through 3/4-in. sieve} &= 100\% - 0\% = 100\% \\
\text{Percentage passing through 3/8-in. sieve} &= 100\% - 7.9\% = 92.1\% \\
\text{Percentage passing through No. 4 sieve} &= 100\% - 23.3\% \\
&= 76.7\% \qquad \text{etc.}
\end{aligned}$$

Therefore, column (6) = 100 − column (5).

4. Upon completion of these calculations, the grain-size distribution curve is obtained by plotting column (2), sieve opening (mm), versus column (6), percentage passing through, on semilog paper. The percentage passing is always plotted as the ordinate on the arithmetic scale and the sieve opening as the abscissa on the log scale (see Figure 2–2).

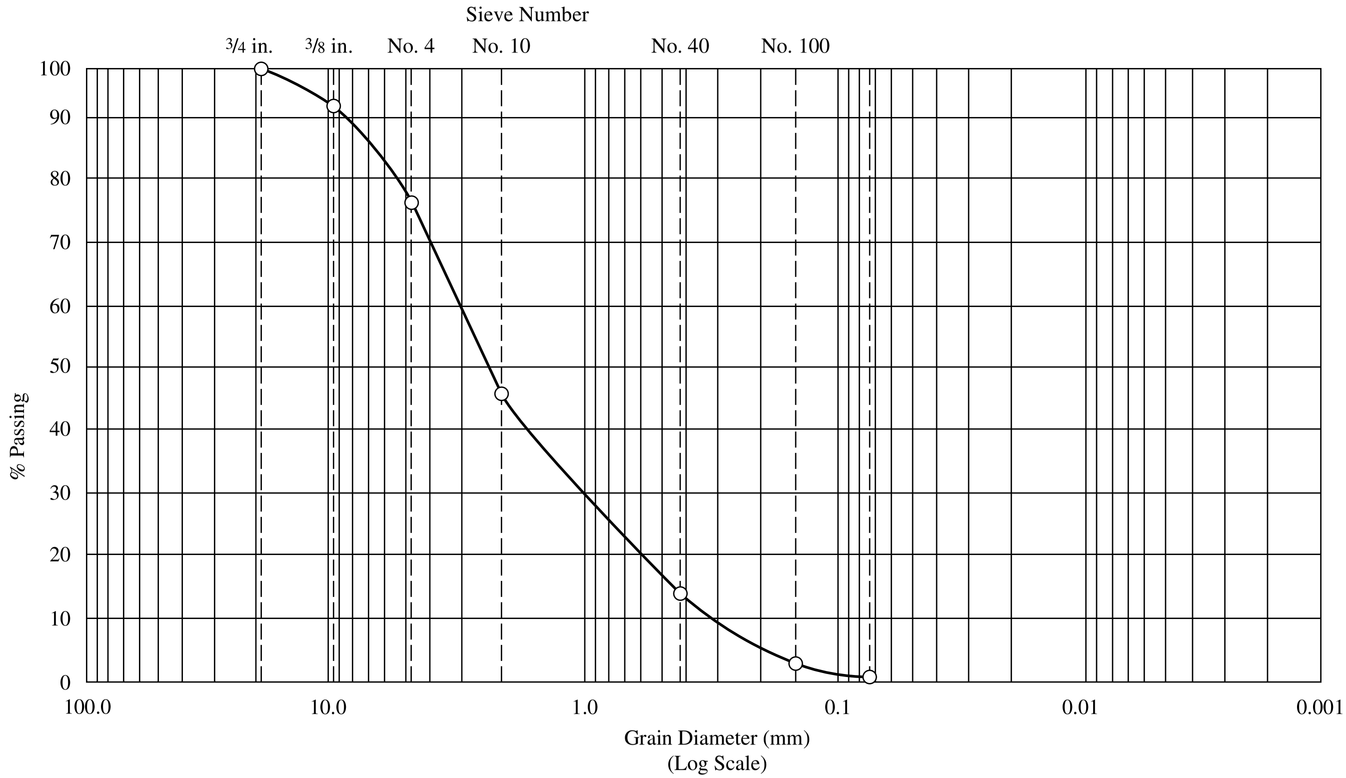


FIGURE 2–2 Grain-size distribution curve for Example 2–1.

Several useful parameters can be determined from grain-size distribution curves. The diameter of soil particles at which 50% passes (i.e., 50% of the soil by weight is finer than this size) is known as the *median size* and is denoted by D_{50}. The diameter at which 10% passes is called the *effective size* and is denoted by D_{10}. Two coefficients used only in the Unified Soil Classification System for classifying coarse-grained soils (see Section 2–4) are the *coefficient of uniformity* (C_u) and the *coefficient of curvature* (C_c), which are defined as follows:

$$C_u = \frac{D_{60}}{D_{10}} \tag{2–1}$$

$$C_c = \frac{(D_{30})^2}{D_{60}D_{10}} \tag{2–2}$$

where D_{60} and D_{30} are the soil particle diameters corresponding to 60 and 30%, respectively, passing on the cumulative grain-size distribution curve.

Median size gives an "average" particle size for a given soil sample; other parameters offer some indication of the particle size range. Effective size gives the maximum particle diameter of the smallest 10% of soil particles. It is this size to which permeability and capillarity are related. C_u and C_c have little or no meaning when more than 5% of the soil is finer than a No. 200 sieve opening (0.075 mm).

In the case of cohesive soils, distribution of grain size is not determined by sieve analysis because the particles are too small. Particle sizes may be determined by the hydrometer method, which is a process for indirectly observing the settling velocities of the particles in a soil–water mixture. Another valuable technique for analyzing cohesive soils is by use of *Atterberg limits,* which is described in the next section.

2–3 SOIL CONSISTENCY—ATTERBERG LIMITS

Atterberg (1911, 1912) defined four states of consistency for cohesive soils. (Consistency refers to their degree of firmness.) These states are *liquid, plastic, semisolid,* and *solid* (see Figure 2–3). The dividing line between liquid and plastic states is the *liquid limit,* the dividing line between plastic and semisolid states is the *plastic limit,* and the dividing line between semisolid and solid states is the *shrinkage limit* (Fig. 2–3). If a soil in the liquid state is gradually dried out, it will pass through the liquid limit, plastic state, plastic limit, semisolid state, and shrinkage limit and will reach the solid state. The liquid, plastic, and shrinkage limits are quantified, therefore, in terms of water content. For example, the liquid limit is reported in terms of the water content at which soil changes from the liquid state to the plastic state. The difference between the liquid limit (*LL*) and plastic limit (*PL*) is the *plasticity index* (*PI*); that is,

$$PI = LL - PL \tag{2–3}$$

The liquid, plastic, and shrinkage limits and the plasticity index are useful parameters in classifying soils and in making judgments as to their applications.

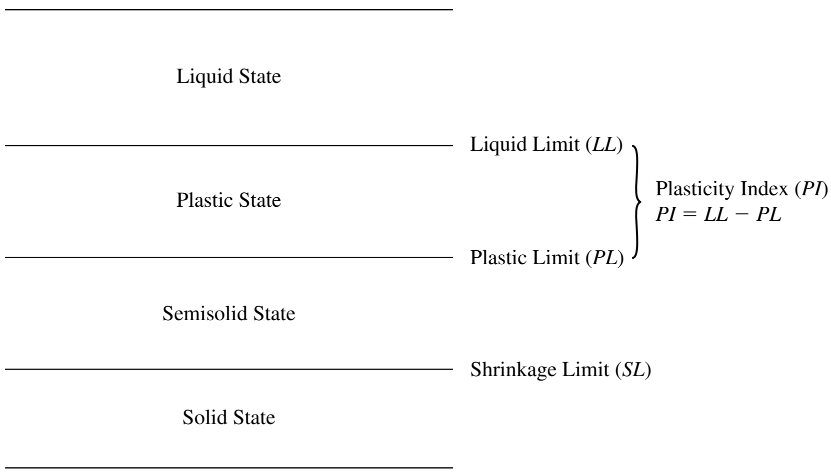


FIGURE 2–3 Atterberg limits.

Another "index," which provides an indication of a fine-grained soil's consistency, is known as the *liquidity index* (*LI*). It relates the natural (*in situ*) water content to plastic and liquid limits and is determined by

$$LI = \frac{w - PL}{LL - PL} \tag{2–4}$$

where w = natural (*in situ*) water content of the soil.

The liquidity index is useful for scaling the natural water content of a soil sample. Figure 2–4 gives a water content continuum. As indicated on Figure 2–4, if the liquid index (*LI*) is less than zero, the soil will have a brittle fracture if sheared. If the liquid index is between zero and one, the soil will behave like a plastic. A liquid index greater than one occurs when the natural (*in situ*) water content is greater than the liquid limit. In this situation, the soil will be a very viscous liquid when sheared.

Standard laboratory test procedures are available to determine Atterberg limits. Although Atterberg defined the four states of consistency for cohesive soils, his original consistency limit tests were somewhat arbitrary and did not yield entirely consistent results. Subsequently, Casagrande standardized the tests, thereby increasing reproducibility of test results.

Casagrande developed a *liquid limit device* for use in determining liquid limits. As shown in Figure 2–5, it consists essentially of a "cup" that is raised and dropped 10 mm by a manually rotated handle. In performing a liquid limit test, a standard groove is cut in a remolded soil sample in the cup using a standard grooving tool. The liquid limit is defined as that water content at which the standard groove will close a distance of 1/2 in. (12.7 mm) along the bottom of the groove at exactly 25 blows (drops) of the cup. Because it is difficult to mix the soil with the precise water content at which the groove will close 1/2 in. at exactly 25 blows, tests are usually run on samples with differing water contents, and a straight-line plot of

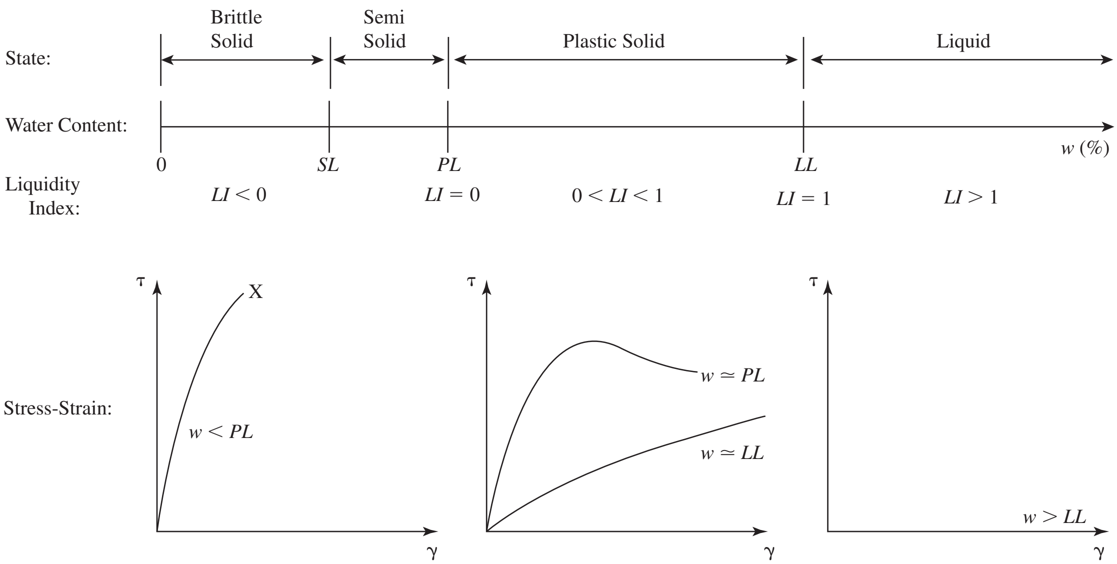
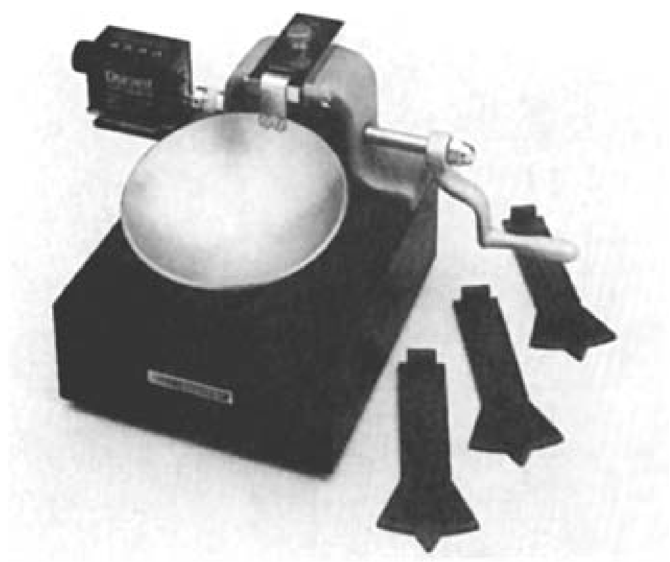


FIGURE 2–4 Water content continuum showing the various states of a soil as well as the generalized stress-strain response.
Source: R. D. Holtz and W. D. Kovacs, *An Introduction to Geotechnical Engineering*, 1981. Reprinted by permission of Pearson Education, Upper Saddle River, NJ.

FIGURE 2–5 Liquid limit device.
Source: Courtesy of Soiltest, Inc.



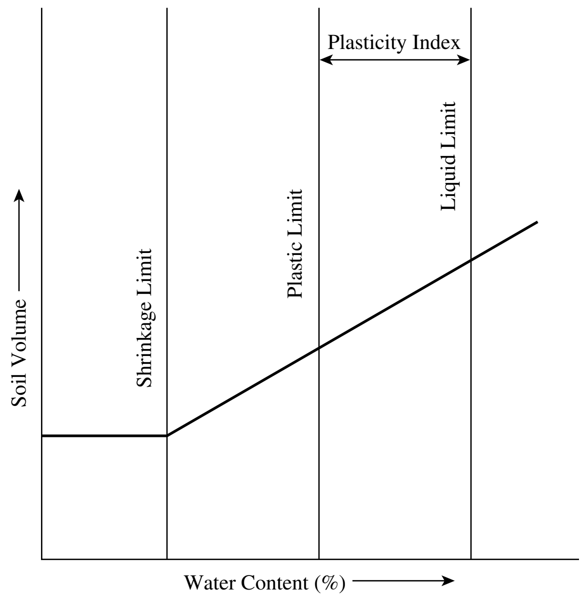
water content versus the logarithm of the number of blows required to close the groove 1/2 in. is prepared. From this plot, which is known as a *flow curve,* the particular water content corresponding to 25 blows is read and reported as the liquid limit.

The plastic limit is evaluated quantitatively in the laboratory by finding the water content at which a thread of soil begins to crumble when it is manually rolled out on a glass plate to a diameter of 1/8 in. and breaks up into segments about 1/8 to 3/8 in. (3 to 10 mm) in length. If threads can be rolled to smaller diameters, the soil is too wet (i.e., it is above the plastic limit). If threads crumble before reaching the 1/8-in. diameter, the soil is too dry and the plastic limit has been surpassed.

The shrinkage limit, the dividing line between the semisolid and solid states, is quantified for a given soil as a specific water content, and from a physical standpoint it is the water content that is just sufficient to fill the voids when the soil is at the minimum volume it will attain on drying. In other words, the smallest water content at which a soil can be completely saturated is called the *shrinkage limit.* Below the shrinkage limit, any water content change will *not* result in volume change; above the shrinkage limit, any water content change *will* result in an accompanying volume change (see Figure 2–6).

The general procedure for determining the shrinkage limit is begun by placing a sample in an evaporating dish and mixing it with enough distilled water to fill the soil voids completely. After a shrinkage dish is coated with petroleum jelly, wet soil is taken from the evaporating dish with a spatula and placed in the shrinkage dish. The placement should be done in three parts, with steps taken each time to drive all air out of the soil. After the shrinkage dish and wet soil are weighed, the soil is set aside to dry in air. It is then oven dried overnight, after which the shrinkage dish and dry soil are weighed. After the oven-dried soil pat is removed from the shrinkage dish, its volume can be determined by mercury displacement. In addition, the weight and volume of the empty shrinkage dish must be determined. The latter (i.e., the volume of the shrinkage dish) is also obtained by mercury displacement,

FIGURE 2–6 Definition of the shrinkage limit.



and it is the same as the volume of the wet soil pat. With these data known, the shrinkage limit can be determined by the following equation:

$$SL = w - \left[\frac{(V - V_o)\rho_w}{M_o}\right] \times 100 \qquad \textbf{(2–5)}$$

where SL = shrinkage limit (expressed as a percentage)
w = water content of wet soil in the shrinkage dish, %
V = volume of wet soil pat (same as volume of shrinkage dish), cm^3
V_o = volume of oven-dried soil pat, cm^3
ρ_w = approximate density of water equal to 1.0 g/cm^3
M_o = mass of oven-dried soil pat, g

The mass of the oven-dried soil pat, M_o, is determined by subtracting the mass of the dish coated with petroleum jelly from the mass of the dish coated with petroleum jelly plus the oven-dried soil. V and V_o should be expressed in cubic centimeters and M_o in grams.

Detailed procedures for laboratory determinations of liquid (ASTM D4318), plastic (ASTM D4318), and shrinkage (ASTM D427) limits are given in *Soil Properties: Testing, Measurement, and Evaluation,* 5th edition, by Liu and Evett (2003).

2–4 SOIL CLASSIFICATION SYSTEMS

In order to be able to describe, in general, a specific soil without listing values of its many soil parameters, it would be convenient to have some kind of generalized classification system. In practice, a number of different classification systems have evolved, most of which were developed to meet specific needs of the particular group that developed a given system. Today, however, only two such systems—the American Association of State Highway and Transportation Officials (AASHTO) system and the Unified Soil Classification System (USCS)—are widely used in engineering practice.

The AASHTO system is widely used in highway work and is followed by nearly all state departments of highways and/or transportation in the United States. Most federal agencies (such as the U.S. Army Corps of Engineers and the U.S. Department of the Interior, Bureau of Reclamation) use the Unified Soil Classification System; it is also utilized by many engineering consulting companies and soil-testing laboratories in the United States. Both of these classification systems are presented in detail later in this section.

Some years ago, the Federal Aviation Administration (FAA) had its own soil classification system, known appropriately as the FAA classification system, for designing airport pavements. Now, however, the FAA uses the Unified Soil Classification System. If one needs information about the FAA classification system, it can be found in the first two editions of this book.

AASHTO Classification System (AASHTO M-145)

Required parameters for classification by the AASHTO system are grain-size analysis, liquid limit, and plasticity index. With values of these parameters known, one enters the first (left) column of Table 2–4 and determines whether known

TABLE 2–4
Classification of Soils and Soil-Aggregate Mixtures by AASHTO Classification System (AASHTO M-145)

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silt–Clay Materials (more than 35% passing 0.075 mm)			
	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:											
2.00 mm (No. 10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (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 0.425 mm (No. 40):											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[1]
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to good					Fair to poor					

[1]Plasticity index of A-7-5 subgroup is equal to or less than *LL* minus 30. Plasticity index of A-7-6 subgroup is greater than *LL* minus 30.
Source: Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, *Specifications,* 13th ed., AASHTO, 1982.

parameters meet the limiting values in that column. If they do, then the soil classification is that given at the top of the column (A-1-a, if known parameters meet the limiting values in the first column). If they do not, one enters the next column (to the right) and determines whether known parameters meet the limiting values in that column. The procedure is repeated until the *first* column is reached in which known parameters meet the limiting values in that column. The soil classification for the given soil is indicated at the top of that particular column. (See Example 2–2.)

Once a soil has been classified using Table 2–4, it can be further described using a *group index*. This index utilizes the percent of soil passing a No. 200 sieve, the liquid limit, and the plasticity index. With known values of these parameters, the group index is computed from the following equation:

$$\text{Group index} = (F - 35)[0.2 + 0.005\,(LL - 40)] + 0.01(F - 15)(PI - 10) \qquad \textbf{(2–6)}$$

where F = percentage of soil passing a No. 200 sieve
LL = liquid limit
PI = plasticity index

The group index computed from Eq. (2–6) is rounded off to the nearest whole number and appended in parentheses to the group designation determined from Table 2–4. If the computed group index is either zero or negative, the number zero is used as the group index and should be appended to the group designation. If preferred, Figure 2–7 may be used instead of Eq. (2–6) to determine the group index.

As a general rule, the value of soil as a subgrade material is in inverse ratio to its group index (i.e., the lower the index, the better the material). Table 2–5 gives some general descriptions of the various classification groups according to the AASHTO system.

EXAMPLE 2–2

Given

A sample of soil was tested in the laboratory, and results of the laboratory tests were as follows:

1. Liquid limit = 42%.
2. Plastic limit = 16%.
3. The following sieve analysis data:

U.S. Sieve Size	Percentage Passing
No. 4	100.0
No. 10	93.2
No. 40	81.0
No. 200	60.2

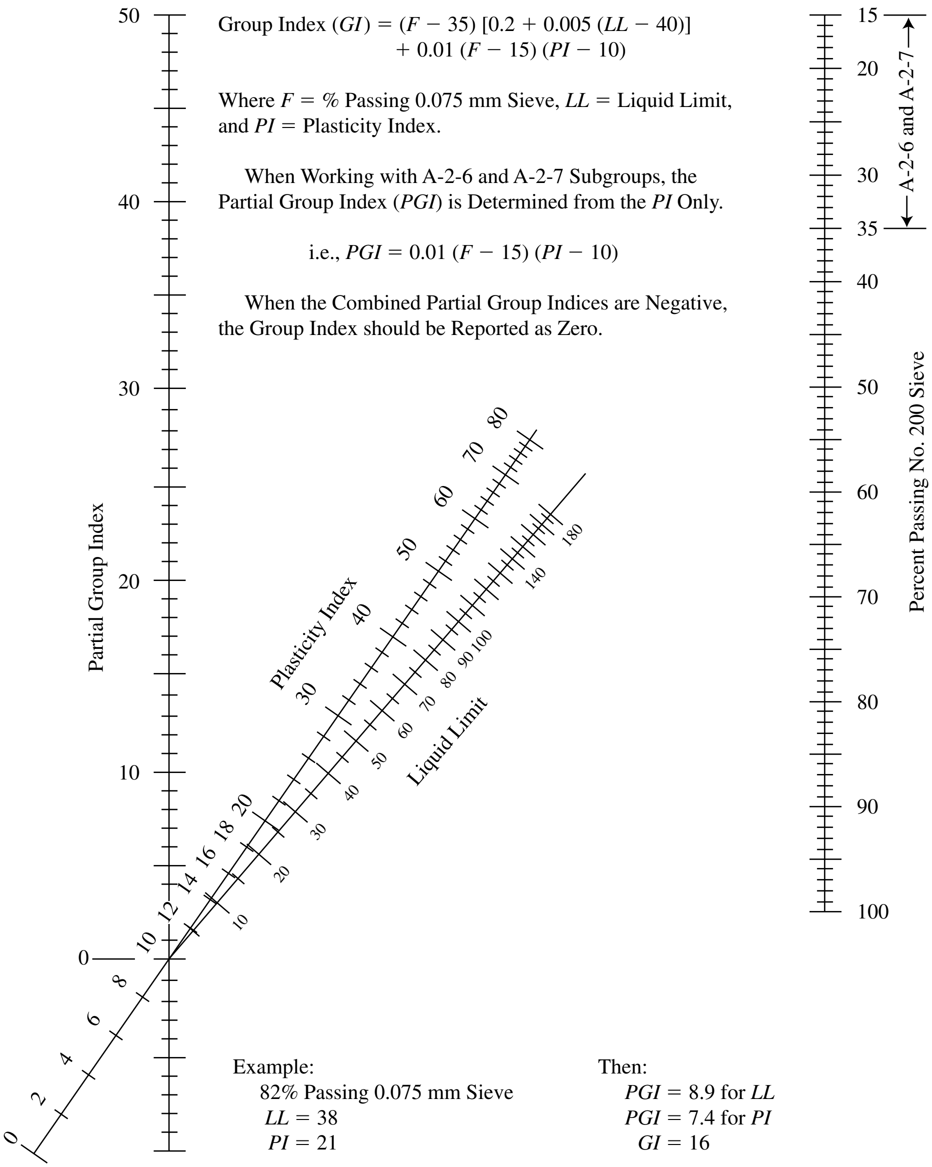


FIGURE 2–7 Group index chart (AASHTO M-145).
Source: Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, *Specifications,* 13th ed., AASHTO, 1982.

Required

Classify the soil sample by the AASHTO classification system.

Solution

By the AASHTO classification system:

$$PI = LL - PL \tag{2–3}$$

$$PI = 42\% - 16\% = 26\%$$

Using the given and computed values, enter the first (A-1-a) column of Table 2–4. The percent passing the No. 10 sieve of 93.2 is greater than 50; therefore, proceed

TABLE 2–5
Descriptions of AASHTO Classification Groups (AASHTO M-145)

(1) *Granular Materials.* Containing 35% or less passing 0.075-mm (No. 200) sieve, Note 1.

(1.1) *Group A-1:* The typical material of this group is a well-graded mixture of stone fragments or gravel, coarse sand, fine sand, and a nonplastic or feebly plastic soil binder. However, this group includes also stone fragments, gravel, coarse sand, volcanic cinders, etc. without soil binder.

(1.1.1) Subgroup A-1-a includes those materials consisting predominantly of stone fragments or gravel, either with or without a well-graded binder of fine material.

(1.1.2) Subgroup A-1-b includes those materials consisting predominantly of coarse sand either with or without a well-graded soil binder.

(1.2) *Group A-3:* The typical material of this group is fine beach sand or fine desert blow sand without silty or clay fines or with a very small amount of nonplastic silt. The group includes also stream-deposited mixtures of poorly graded fine sand and limited amounts of coarse sand and gravel.

(1.3) *Group A-2:* This group includes a wide variety of "granular" materials which are borderline between the materials falling in Groups A-1 and A-3 and silt–clay materials of Groups A-4, A-5, A-6, and A-7. It includes all materials containing 35% or less passing the 0.075-mm sieve which cannot be classified as A-1 or A-3, due to fines content or plasticity or both, in excess of the limitations for those groups.

(1.3.1) Subgroups A-2-4 and A-2-5 include various granular materials containing 35% or less passing the 0.075-mm sieve and with a minus 0.425-mm (No. 40) portion having the characteristics of the A-4 and A-5 groups. These groups include such materials as gravel and coarse sand with silt contents or plasticity indexes in excess of the limitations of Group A-1, and fine sand with nonplastic silt content in excess of the limitations of Group A-3.

(1.3.2) Subgroups A-2-6 and A-2-7 include materials similar to those described under Subgroups A-2-4 and A-2-5 except that the fine portion contains plastic clay having the characteristics of the A-6 or A-7 group.

Note 1: Classification of materials in the various groups applies only to the fraction passing the 75-mm sieve. Therefore, any specification regarding the use of A-1, A-2, or A-3 materials in construction should state whether boulders (retained on 3-in. sieve) are permitted.

(*continued*)

TABLE 2–5 *continued*

(2) *Silt–Clay Materials.* Containing more than 35% passing the 0.075-mm sieve.

- (2.1) *Group A-4:* The typical material of this group is a nonplastic or moderately plastic silty soil usually having 75% or more passing the 0.075-mm sieve. The group includes also mixtures of fine silty soil and up to 64% of sand and gravel retained on the 0.075-mm sieve.
- (2.2) *Group A-5:* The typical material of this group is similar to that described under Group A-4, except that it is usually of diatomaceous or micaceous character and may be highly elastic as indicated by the high liquid limit.
- (2.3) *Group A-6:* The typical material of this group is a plastic clay soil usually having 75% or more passing the 0.075-mm sieve. The group includes also mixtures of fine clayey soil and up to 64% of sand and gravel retained on the 0.075-mm sieve. Materials of this group usually have high volume change between wet and dry states.
- (2.4) *Group A-7:* The typical material of this group is similar to that described under Group A-6, except that it has the high liquid limits characteristic of the A-5 group and may be elastic as well as subject to high volume change.
    - (2.4.1) Subgroup A-7-5 includes those materials with moderate plasticity indexes in relation to liquid limit and which may be highly elastic as well as subject to considerable volume change.
    - (2.4.2) Subgroup A-7-6 includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change.

Note 2: Highly organic soils (peat or muck) may be classified as an A-8 group. Classification of these materials is based on visual inspection, and is not dependent on percentage passing the 0.075-mm (No. 200) sieve, liquid limit, or plasticity index. The material is composed primarily of partially decayed organic matter, generally has a fibrous texture, dark brown or black color, and odor of decay. These organic materials are unsuitable for use in embankments and subgrades. They are highly compressible and have low strength.

Source: Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, *Specifications,* 13th ed., AASHTO, 1982.

to the next column (A-1-b). The percent passing the No. 40 sieve of 81.0 is greater than 50; therefore, proceed to the next column (A-3). Although the percent passing the No. 40 sieve of 81.0 is greater than 51, the percent passing the No. 200 sieve of 60.2 is greater than 10. Therefore, proceed to the next column (A-2-4). Continue this process until column (A-7) is found to be the first column that meets all the limiting values in that column. Hence, this sample is classified as A-7. According to the AASHTO classification system, the plasticity index of the A-7-5 subgroup is equal to or less than the liquid limit minus 30, and the plasticity index of the A-7-6 subgroup is greater than the liquid limit minus 30 (see Note 1 under Table 2–4).

$$LL - 30\% = 42\% - 30\% = 12\%$$
$$[PI = 26\%] > [LL - 30\% = 12\%]$$

Hence, this is A-7-6 material.

From Figure 2–7 (group index chart), with $LL = 42\%$ and the percentage passing the No. 200 sieve = 60.2%, the partial group index for LL is 5.3. With $PI = 26\%$ and the percentage passing the No. 200 sieve = 60.2%, the partial group index for PI is 7.5. Hence,

$$\text{Total group index} = 5.3 + 7.5 = 12.8$$

Hence, the soil is A-7-6 (13), according to the AASHTO classification system.

Unified Soil Classification System (ASTM D 2487)

The Unified Soil Classification System was originally developed by Casagrande (1948) and is utilized by the Corps of Engineers and most nonhighway engineers. In this system, soils fall within one of three major categories: coarse-grained, fine-grained, and highly organic soils. These categories are further subdivided into 15 basic soil groups. The following group symbols are used in the Unified System:

G	Gravel
S	Sand
M	Silt
C	Clay
O	Organic
PT	Peat
W	Well graded
P	Poorly graded

Normally, two group symbols are used to classify soils. For example, SW indicates well-graded sand. Table 2–6 lists the 15 soil groups, including each one's name and symbol, as well as specific details for classifying soils by this system.

In order to classify a given soil by the Unified System, its grain-size distribution, liquid limit, and plasticity index must first be determined. With these values known, the soil can be classified by using Table 2–6 and Figure 2–8. The Unified Soil Classification System is published as ASTM D 2487.

EXAMPLE 2–3

Given

A sample of soil was tested in the laboratory with the following results:

1. Liquid limit = 30%.
2. Plastic limit = 12%.
3. Sieve analysis data:

U.S. Sieve Size	Percentage Passing
$\frac{3}{8}$ in.	100.0
No. 4	76.5
No. 10	60.0
No. 40	39.7
No. 200	15.2

TABLE 2–6
Soil Classification Chart by Unified Soil Classification System (ASTM D 2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse-grained soils: More than 50% retained on No. 200 sieve	*Gravels*: More than 50% of coarse fraction retained on No. 4 sieve	*Clean gravels:* Less than 5% fines[C]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[E]	GW	Well-graded gravel[E]
			$C_u < 4$ and/or $1 > C_c > 3$[E]	GP	Poorly graded gravel[E]
		Gravels with fines: More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	*Clean sands:* Less than 5% fines[D]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[E]	SW	Well-graded sand[I]
			$C_u < 6$ and/or $C_c < 1$ or $C_c > 3$[E]	SP	Poorly graded sand[I]
		Sands with fines: More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-grained soils: 50% or more passes the No. 200 sieve	*Silts and clays:* Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			$PI < 4$ plots below "A" line[J]	ML	Silt[K,L,M]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[K,L,M,N] / Organic silt[K,L,M,O]
	Liquid limit 50 or more	Inorganic	PI plots on or above "A" line.	CH	Fat clay[K,L,M]
			PI plots below "A" line.	MH	Elastic silt[K,L,M]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[K,L,M,P] / Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic color			PT	Peat

[A]Based on the material passing the 3-in. (75-mm) sieve.
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM, well-graded gravel with silt
GW-GC, well-graded gravel with clay
GP-GM, poorly graded gravel with silt
GP-GC, poorly graded gravel with clay
[D]Sands with 5 to 12% fines require dual symbols:
SW-SM, well-graded sand with silt
SW-SC, well-graded sand with clay
SP-SM, poorly graded sand with silt
SP-SC, poorly graded sand with clay

[E]$C_u = D_{60}/D_{10}$, $C_c = (D_{30})^2/(D_{10} \times D_{60})$
[F]If soil contains ≥ 15 % sand, add "with sand" to group name.
[G]If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[H]If fines are organic, add "with organic fines" to group name.
[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J]If Atterberg limits plot in hatched area, soil is a CL-ML silty clay.
[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L]If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
[M]If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N]$PI \geq 4$ and plots on or above "A" line.
[O]$PI < 4$ or plots below "A" line.
[P]PI plots on or above "A" line.
[Q]PI plots below "A" line.

Source: Annual Book of ASTM Standards, ASTM, Philadelphia, 1989. Copyright American Society for Testing and Materials. Reprinted with permission.

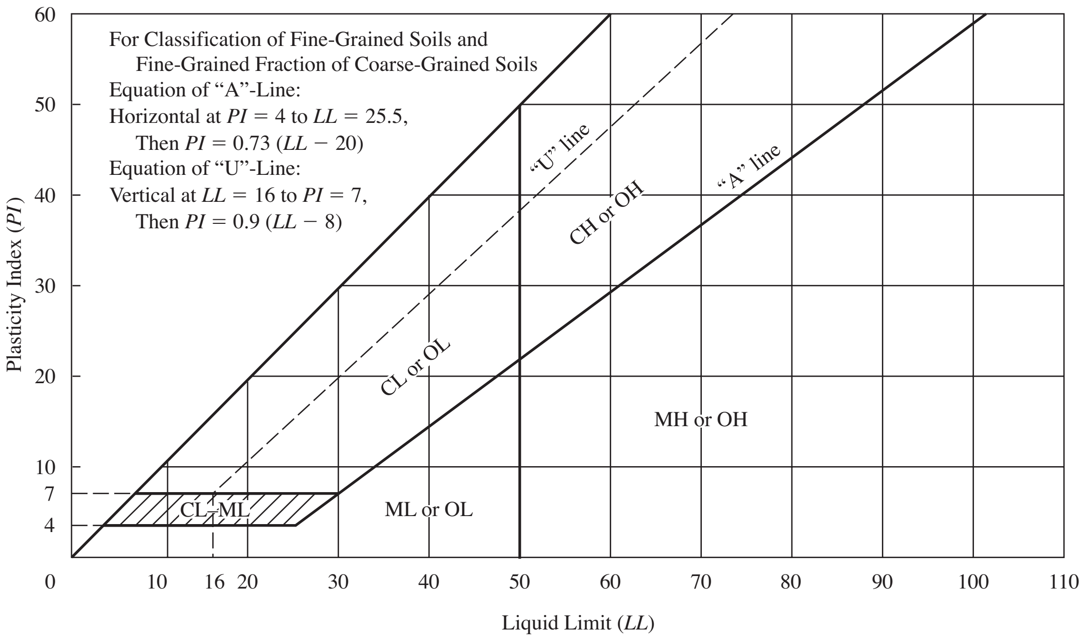


FIGURE 2–8 Plasticity chart (ASTM D 2487).
Source: Annual Book of ASTM Standards, ASTM, Philadelphia, 1989. Copyright American Society for Testing and Materials. Reprinted with permission.

Required

Classify the soil by the Unified Soil Classification System.

Solution

Because the percentage retained on the No. 200 sieve (100 − 15.2, or 84.8%) is more than 50%, go to the block labeled "Coarse-grained soils" in Table 2–6. The sample consists of 100 − 15.2, or 84.8%, coarse-grain sizes, and 100 − 76.5,or 23.5%, was retained on the No. 4 sieve. Thus, the percentage of coarse fraction retained on the No. 4 sieve is (23.5/84.8) (100), or 27.7%, and the percentage of coarse fraction that passed the No. 4 sieve is 72.3%. Because 72.3% is greater than 50%, go to the block labeled "Sands" in Table 2–6. The soil is evidently a sand. Because the sample contains 15.2% passing the No. 200 sieve, which is greater than 12% fines, go to the block labeled "Sands with fines: More than 12% fines." Refer next to the plasticity chart (Figure 2–8). With a liquid limit of 30% and plasticity index of 18% (recall that the plasticity index is the difference between the liquid and plastic limits, or 30 − 12), the sample is located above the "A" line, and the fines are classified as CL. Return to Table 2–6, and go to the block labeled "SC." Thus, this soil is classified SC (i.e., clayey sand) according to the Unified Soil Classification System.

EXAMPLE 2–4

Given

A sample of soil was tested in the laboratory with the following results:

1. Liquid limit = NP (nonplastic).
2. Plastic limit = NP (nonplastic).
3. Sieve analysis data:

U.S. Sieve Size	Percentage Passing
1 in.	100
¾ in.	85
½ in.	70
⅜ in.	60
No. 4	48
No. 10	30
No. 40	16
No. 100	10
No. 200	2

Required

Classify the soil by the USCS.

Solution

Because the percentage retained on the No. 200 sieve (100 − 2, or 98%) is more than 50%, go to the block labeled "Coarse-grained soils" in Table 2–6. The sample consists of 100 − 2, or 98%, coarse-grain sizes, and 100 − 48, or 52%, was retained on the No. 4 sieve. Thus, the percentage of coarse fraction retained on the No. 4 sieve is 52/98, or 53.1%. Because 53.1% is greater than 50%, go to the block labeled "Gravels" in Table 2–6. The soil is evidently a gravel. Because the sample contains 2% passing the No. 200 sieve, which is less than 5% fines, go to the block labeled "Clean gravels: Less than 5% fines." The next block indicates that the coefficients of uniformity (C_u) and curvature (C_c) must be evaluated.

$$C_u = \frac{D_{60}}{D_{10}} \tag{2–1}$$

$$C_c = \frac{(D_{30})^2}{D_{60}D_{10}} \tag{2–2}$$

Values of D_{60}, D_{30}, and D_{10} are determined from the grain-size distribution curve (see Figure 2–9) to be 9.5, 2.00, and 0.150 mm, respectively. Hence,

$$C_u = \frac{9.5 \text{ mm}}{0.150 \text{ mm}} = 63.3$$

$$C_c = \frac{(2.00 \text{ mm})^2}{(9.5 \text{ mm})(0.150 \text{ mm})} = 2.8$$

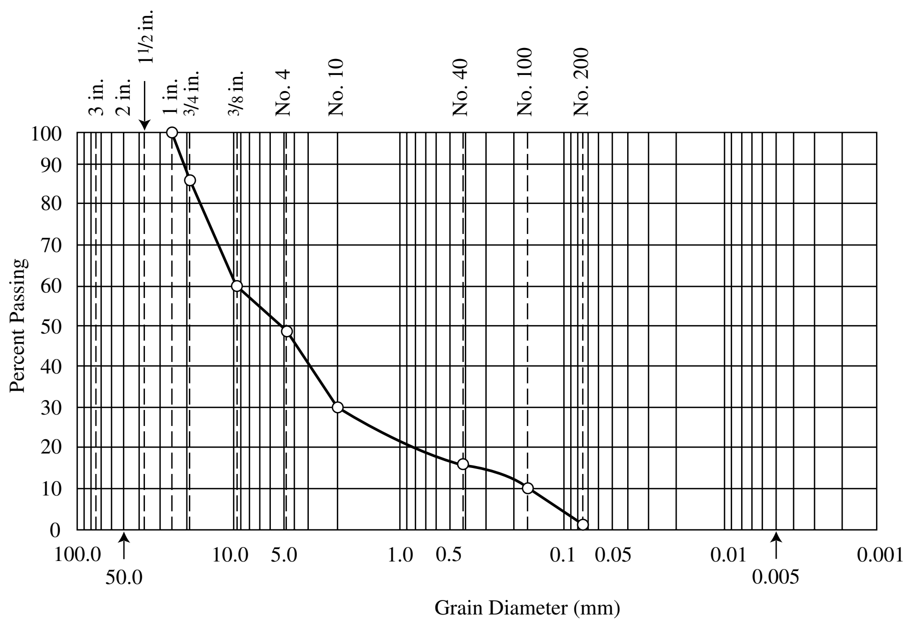


FIGURE 2–9 Grain-size distribution curve for Example 2–4.

Because C_u (63.3) is greater than 4, and C_c (2.8) is between 1 and 3, this sample meets both criteria for a well-graded gravel. Hence, from Table 2–6 the soil is classified GW (i.e., well-graded gravel) according to the Unified Soil Classification System.

EXAMPLE 2–5

Given

A sample of inorganic soil was tested in the laboratory with the following results:

1. Liquid limit = 42%.
2. Plastic limit = 16%.
3. Sieve analysis data:

U.S. Sieve Size	Percentage Passing
No. 4	100.0
No. 10	93.2
No. 40	81.0
No. 200	60.2

Required

Classify the soil sample by the Unified Soil Classification System.

Solution

Because the percentage passing the No. 200 sieve is 60.2%, which is greater than 50%, go to the lower block (labeled "Fine-grained soils") in Table 2–6. The liquid limit is 42%, which is less than 50%, so go to the block labeled "Silts and clays: Liquid limit less than 50." Now, because the sample is an inorganic soil, and the plasticity index is 42 − 16, or 26%, which is greater than 7, refer next to the plasticity chart (Figure 2–8). With a liquid limit of 42% and plasticity index of 26%, the sample is located above the "A" line. Return to Table 2–6 and go to the block labeled "CL." Thus, the soil is classified CL according to the Unified Soil Classification System.

2–5 COMPONENTS OF SOILS

Soils contain three components, which may be characterized as solid, liquid, and gas. The solid components of soils are weathered rock and (sometimes) decayed vegetation. The liquid component of soils is almost always water (often with dissolved matter), and the gas component is air. The volume of water and air combined is referred to as the *void.*

Figure 2–10 gives a block diagram showing the components of a soil. These components may be considered in terms of both their volumes and their weights/masses. In Figure 2–10, terms V, V_a, V_w, V_s, and V_v represent total volume and volume of air, water, solid matter, and voids, respectively. Terms W, W_a, W_w, and W_s stand for total weight and weight of air, water, and solid matter, respectively. Similarly, terms M, M_a, M_w, and M_s denote total mass and mass of air, water, and solid matter, respectively. The weight and mass of air (W_a and M_a) are both virtually zero.

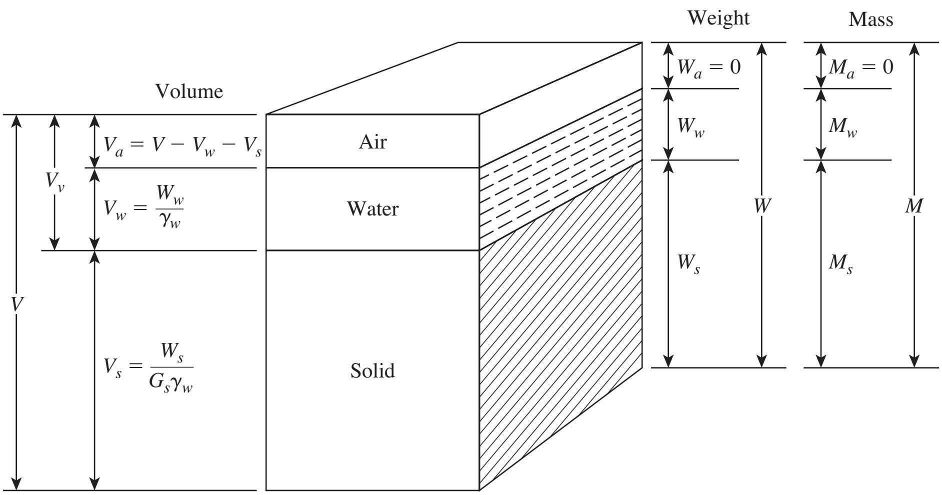


FIGURE 2–10 Block diagram showing components of soil.

2–6 WEIGHT/MASS AND VOLUME RELATIONSHIPS

A number of important relationships exist among the components of soil in terms of both weight/mass and volume. These relationships define new parameters that are useful in working with soils.

In terms of volume, the following new parameters are important—*void ratio, porosity,* and *degree of saturation*. Void ratio (e) is the ratio (expressed as a decimal fraction) of volume of voids to volume of solids.

$$e = \frac{V_v}{V_s} \tag{2–7}$$

Porosity (n) is the ratio (expressed as a percentage) of volume of voids to total volume.

$$n = \frac{V_v}{V} \times 100\% \tag{2–8}$$

Degree of saturation (S) is the ratio (expressed as a percentage) of volume of water to volume of voids.

$$S = \frac{V_w}{V_v} \times 100\% \tag{2–9}$$

In terms of weight/mass, the new parameters are *water content, unit weight, dry unit weight, unit mass* (or *density*), *dry unit mass* (or *dry density*), and *specific gravity of solids.* (*Note:* The terms *unit weight* and *unit mass* imply *wet unit weight* and *wet unit mass.* If *dry unit weight* or *dry unit mass* is intended, the adjective *dry* is indicated explicitly.) Water content (w) is the ratio (expressed as a percentage) of weight of water to weight of solids or the ratio of mass of water to mass of solids.

$$w = \frac{W_w}{W_s} \times 100\% = \frac{M_w}{M_s} \times 100\% \tag{2–10}$$

Unit weight (γ) is total weight (weight of solid plus weight of water) divided by total volume (volume of solid plus volume of water plus volume of air).

$$\gamma = \frac{W}{V} \tag{2–11}$$

Dry unit weight (γ_d) is weight of solids divided by total volume.

$$\gamma_d = \frac{W_s}{V} \tag{2–12}$$

Unit mass (ρ) is total mass divided by total volume.

$$\rho = \frac{M}{V} \tag{2–13}$$

Dry unit mass (ρ_d) is mass of solids divided by total volume.

$$\rho_d = \frac{M_s}{V} \quad \textbf{(2–14)}$$

Specific gravity of solids (G_s) is the ratio of unit weight of solids (weight of solids divided by volume of solids) to unit weight of water or of unit mass of solids (mass of solids divided by volume of solids) to unit mass of water.

$$G_s = \frac{W_s/V_s}{\gamma_w} = \frac{W_s}{V_s\gamma_w} \quad \textbf{(2–15)}$$

$$G_s = \frac{M_s/V_s}{\rho_w} = \frac{M_s}{V_s\rho_w} \quad \textbf{(2–16)}$$

where γ_w and ρ_w are the unit weight and unit mass of water, respectively.

The unit weight of water varies slightly with temperature, but at normal temperatures, it has a value of around 62.4 pounds per cubic foot (lb/ft^3) or 9.81 kilonewtons per cubic meter (kN/m^3). The unit mass (density) of water is 1000 kilograms per cubic meter (kg/m^3) or 1 gram per cubic centimeter (g/cm^3). A useful conversion factor is as follows: 1 lb/ft^3 = 0.1571 kN/m^3, or 1 kN/m^3 = 6.366 lb/ft^3.

Geotechnical engineers must be proficient in determining these parameters based on laboratory evaluations of weight/mass and volume of the components of a soil. Use of a block diagram (as shown in Figure 2–10) is recommended to help obtain answers more quickly and accurately. Five example problems follow.

EXAMPLE 2–6

Given

1. The weight of a chunk of moist soil sample is 45.6 lb.
2. The volume of the soil chunk measured before drying is 0.40 ft^3.
3. After the sample is dried out in an oven, its weight is 37.8 lb.
4. The specific gravity of solids is 2.65.

Required

1. Water content.
2. Unit weight of moist soil.
3. Void ratio.
4. Porosity.
5. Degree of saturation.

Solution

See Figure 2–11. (Boldface data on the figure indicate given information. Other data are calculated in the solution of the problem.)

1. Water content(w) $= \dfrac{W_w}{W_s} \times 100\% = \dfrac{45.6\text{ lb} - 37.8\text{ lb}}{37.8\text{ lb}} \times 100\%$

$= 20.6\%$

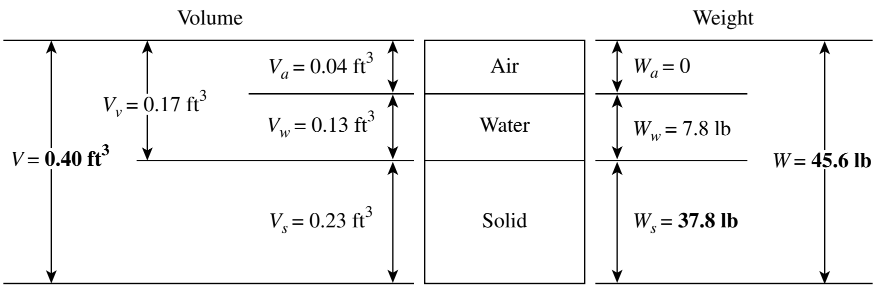


FIGURE 2–11 Block diagram showing components of soil for Example 2–6.

2. Unit weight of moist soil $(\gamma) = \dfrac{W}{V} = \dfrac{45.6 \text{ lb}}{0.40 \text{ ft}^3} = 114.0 \text{ lb/ft}^3$

3. $V_w = \dfrac{W_w}{\gamma_w} = \dfrac{45.6 \text{ lb} - 37.8 \text{ lb}}{62.4 \text{ lb/ft}^3} = 0.13 \text{ ft}^3$

$$V_s = \frac{W_s}{G_s\gamma_w} = \frac{37.8 \text{ lb}}{(2.65)(62.4 \text{ lb/ft}^3)} = 0.23 \text{ ft}^3$$

$$V_a = V - V_w - V_s = 0.40 \text{ ft}^3 - 0.13 \text{ ft}^3 - 0.23 \text{ ft}^3 = 0.04 \text{ ft}^3$$

$$V_v = V - V_s = 0.40 \text{ ft}^3 - 0.23 \text{ ft}^3 = 0.17 \text{ ft}^3$$

or

$$V_v = V_a + V_w = 0.04 \text{ ft}^3 + 0.13 \text{ ft}^3 = 0.17 \text{ ft}^3$$

Void ratio $(e) = \dfrac{V_v}{V_s} = \dfrac{0.17 \text{ ft}^3}{0.23 \text{ ft}^3} = 0.74$

4. Porosity $(n) = \dfrac{V_v}{V} \times 100\% = \dfrac{0.17 \text{ ft}^3}{0.40 \text{ ft}^3} \times 100\% = 42.5\%$

5. Degree of saturation $(S) = \dfrac{V_w}{V_v} \times 100\% = \dfrac{0.13 \text{ ft}^3}{0.17 \text{ ft}^3} \times 100\% = 76.5\%$

EXAMPLE 2–7

Given

1. The moist mass of a soil specimen is 20.7 kg.
2. The specimen's volume measured before drying is 0.011 m³.
3. The specimen's dried mass is 16.3 kg.
4. The specific gravity of solids is 2.68.

Required

1. Void ratio.
2. Degree of saturation.
3. Wet unit mass.
4. Dry unit mass.
5. Wet unit weight.
6. Dry unit weight.

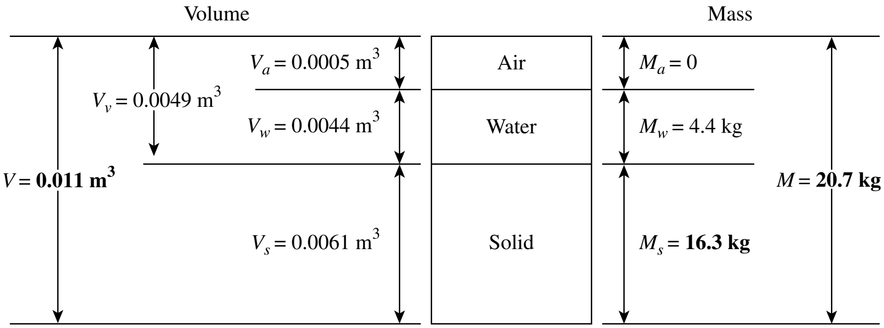


FIGURE 2–12 Block diagram showing components of soil for Example 2–7.

Solution

See Figure 2–12.

1. $V_s = \dfrac{M_s}{G_s \rho_w} = \dfrac{16.3 \text{ kg}}{(2.68)(1000 \text{ kg/m}^3)} = 0.0061 \text{ m}^3$

$V_w = \dfrac{M_w}{\rho_w} = \dfrac{20.7 \text{ kg} - 16.3 \text{ kg}}{1000 \text{ kg/m}^3} = 0.0044 \text{ m}^3$

$V_a = V - V_w - V_s = 0.011 \text{ m}^3 - 0.0044 \text{ m}^3 - 0.0061 \text{ m}^3$
$= 0.0005 \text{ m}^3$

$V_v = V - V_s = 0.011 \text{ m}^3 - 0.0061 \text{ m}^3 = 0.0049 \text{ m}^3$

or

$V_v = V_a + V_w = 0.0005 \text{ m}^3 + 0.0044 \text{ m}^3 = 0.0049 \text{ m}^3$

Void ratio $(e) = \dfrac{V_v}{V_s} = \dfrac{0.0049 \text{ m}^3}{0.0061 \text{ m}^3} = 0.80$

2. Degree of saturation $(S) = \dfrac{V_w}{V_v} \times 100\% = \dfrac{0.0044 \text{ m}^3}{0.0049 \text{ m}^3} \times 100\% = 89.8\%$

3. Wet unit mass $(\rho) = \dfrac{M}{V} = \dfrac{20.7 \text{ kg}}{0.011 \text{ m}^3} = 1882 \text{ kg/m}^3$

4. Dry unit mass $(\rho_d) = \dfrac{M_s}{V} = \dfrac{16.3 \text{ kg}}{0.011 \text{ m}^3} = 1482 \text{ kg/m}^3$

5. Wet unit weight $(\gamma) = \rho g = (1882 \text{ kg/m}^3)(9.81 \text{ m/s}^2)$

$= 18{,}460 \dfrac{\text{kg} \cdot \text{m}}{\text{s}^2} \Big/ \text{m}^3 = 18{,}460 \text{ N/m}^3$

$= 18.46 \text{ kN/m}^3$

6. Dry unit weight $(\gamma_d) = \rho_d g = (1482 \text{ kg/m}^3)(9.81 \text{ m/s}^2)$

$= 14{,}540 \dfrac{\text{kg} \cdot \text{m}}{\text{s}^2} \Big/ \text{m}^3 = 14{,}540 \text{ N/m}^3$

$= 14.54 \text{ kN/m}^3$

EXAMPLE 2–8

Given

An undisturbed soil sample has the following data:

1. Void ratio = 0.78.
2. Water content = 12%.
3. Specific gravity of solids = 2.68.

Required

1. Wet unit weight.
2. Dry unit weight.
3. Degree of saturation.
4. Porosity.

Solution

See Figure 2–13. Because the void ratio (e) = 0.78,

$$\frac{V_v}{V_s} = 0.78; \qquad V_v = 0.78V_s \tag{A}$$

$$V_v + V_s = V = 1\ \text{m}^3 \tag{B}$$

(A volume of 1 m^3 is assumed.)
Substitute Eq. (A) into Eq. (B).

$$0.78V_s + V_s = 1\ \text{m}^3$$

$$V_s = 0.56\ \text{m}^3$$

$$V_v = 1\ \text{m}^3 - 0.56\ \text{m}^3 = 0.44\ \text{m}^3$$

$$V_s = \frac{W_s}{G_s\gamma_w}; \qquad 0.56\ \text{m}^3 = \frac{W_s}{(2.68)(9.81\ \text{kN/m}^3)}$$

$$W_s = 14.72\ \text{kN}$$

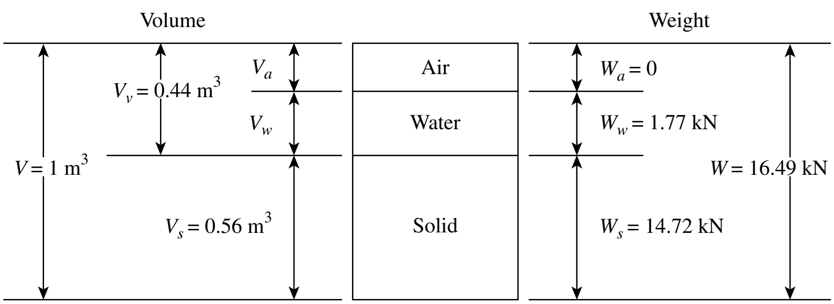


FIGURE 2–13 Block diagram showing components of soil for Example 2–8.

From the given water content, $W_w/W_s = 0.12$,

$$W_w = 0.12W_s = (0.12)(14.72\ \text{kN}) = 1.77\ \text{kN}$$

1. Wet unit weight(γ) $= \dfrac{W}{V} = \dfrac{W_w + W_s}{V} = \dfrac{1.77\ \text{kN} + 14.72\ \text{kN}}{1\ \text{m}^3}$
   $= 16.49\ \text{kN/m}^3$
2. Dry unit weight $(\gamma_d) = \dfrac{W_s}{V} = \dfrac{14.72\ \text{kN}}{1\ \text{m}^3} = 14.72\ \text{kN/m}^3$
3. $V_w = \dfrac{W_w}{\gamma_w} = \dfrac{1.77\ \text{kN}}{9.81\ \text{kN/m}^3} = 0.18\ \text{m}^3$

   Degree of saturation $(S) = \dfrac{V_w}{V_v} \times 100\% = \dfrac{0.18\ \text{m}^3}{0.44\ \text{m}^3} \times 100\% = 40.9\%$
4. Porosity $(n) = \dfrac{V_v}{V} \times 100\% = \dfrac{0.44\ \text{m}^3}{1\ \text{m}^3} \times 100\% = 44.0\%$

EXAMPLE 2–9

Given

1. A 100% saturated soil has a wet unit weight of 120 lb/ft^3.
2. The water content of this saturated soil was determined to be 36%.

Required

1. Void ratio.
2. Specific gravity of solids.

Solution

See Figure 2–14.

$$W_w + W_s = 120\ \text{lb} \quad \textbf{(A)}$$

$$\frac{W_w}{W_s} = 0.36 \quad \textbf{(B)}$$

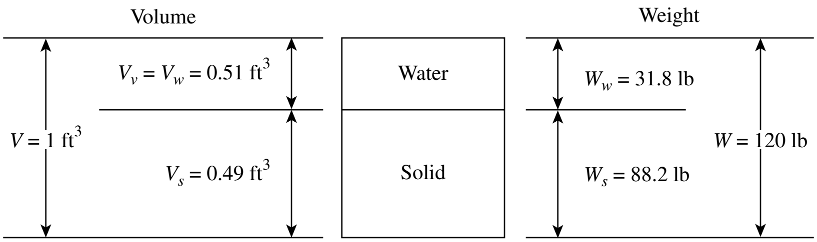


FIGURE 2–14 Block diagram showing components of soil for Example 2–9.

From Eq. (B), $W_w = 0.36W_s$; substitute into Eq. (A).

$$0.36W_s + W_s = 120 \text{ lb}$$
$$W_s = 88.2 \text{ lb}$$
$$W_w = 0.36W_s = (0.36)(88.2 \text{ lb}) = 31.8 \text{ lb}$$

1. $V_w = \dfrac{W_w}{\gamma_w} = \dfrac{31.8 \text{ lb}}{62.4 \text{ lb/ft}^3} = 0.51 \text{ ft}^3$

$V_s = V - V_w = 1 \text{ ft}^3 - 0.51 \text{ ft}^3 = 0.49 \text{ ft}^3$

$e = \dfrac{V_v}{V_s} = \dfrac{V_w}{V_s} = \dfrac{0.51 \text{ ft}^3}{0.49 \text{ ft}^3} = 1.04$

Note: In this problem, because the soil is 100% saturated, $V_v = V_w$.

2. $V_s = \dfrac{W_s}{G_s\gamma_w}$; $\quad 0.49 \text{ ft}^3 = \dfrac{88.2 \text{ lb}}{(G_s)(62.4 \text{ lb/ft}^3)}$

$G_s = 2.88$

EXAMPLE 2–10

Given

A soil sample has the following data:

1. Void ratio = 0.94.
2. Degree of saturation = 35%.
3. Specific gravity of solids = 2.71.

Required

1. Water content.
2. Unit weight.

Solution

See Figure 2–15. From the given void ratio, $e = V_v/V_s = 0.94$,

$$V_v = 0.94V_s \quad \textbf{(A)}$$
$$V_v + V_s = 1 \text{ m}^3 \quad \textbf{(B)}$$

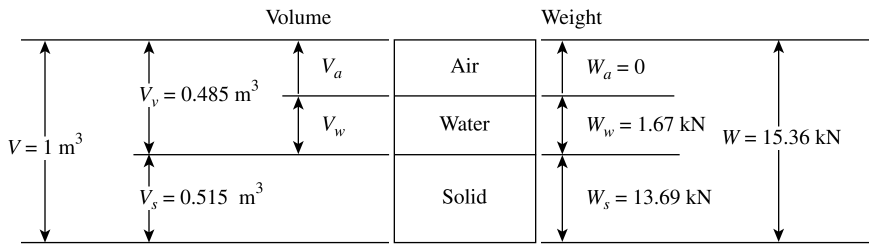


FIGURE 2–15 Block diagram showing components of soil for Example 2–10.

Substitute Eq. (A) into Eq. (B).

$$0.94V_s + V_s = 1 \text{ m}^3$$
$$V_s = 0.515 \text{ m}^3$$
$$V_v = 0.485 \text{ m}^3$$

From the given degree of saturation, $S = V_w/V_v = 0.35$,

$$V_w = 0.35V_v$$
$$V_w = (0.35)(0.485 \text{ m}^3) = 0.170 \text{ m}^3$$
$$W_w = (0.170 \text{ m}^3)(9.81 \text{ kN/m}^3) = 1.67 \text{ kN}$$
$$W_s = (V_s)(G_s)(\gamma_w) = (0.515 \text{ m}^3)(2.71)(9.81 \text{ kN/m}^3) = 13.69 \text{ kN}$$

1. Water content $(w) = \dfrac{W_w}{W_s} \times 100\% = \dfrac{1.67 \text{ kN}}{13.69 \text{ kN}} \times 100\% = 12.2\%$

2. Unit weight $(\gamma) = \dfrac{W}{V} = \dfrac{W_w + W_s}{V} = \dfrac{1.67 \text{ kN} + 13.69 \text{ kN}}{1 \text{ m}^3} = 15.36 \text{ kN/m}^3$

2–7 PERMEABILITY, CAPILLARITY, AND FROST HEAVE

As indicated in Section 2–5, water is a component of soil, and its presence in a given soil may range from virtually none to saturation—the latter case occurring when the soil's void space is completely filled with water. Soil properties and characteristics are influenced by changes in water content. This section introduces three phenomena that are directly related to water in soil: permeability, capillarity, and frost heave. These as well as other factors pertaining to water in soil are discussed in more detail in Chapter 5.

Permeability refers to the movement of water within soil. Actual water movement is through the voids, which might be thought of as small, interconnected, irregular conduits. Because the water moves through the voids, it follows that soils with large voids (such as sands) are generally more permeable than those with smaller voids (such as clays). Additionally, because soils with large voids generally have large void ratios, it may be generalized that permeability tends to increase as the void ratio increases. Because water movement can have profound effects on soil properties and characteristics, it is an important consideration in certain engineering applications. Construction procedures, as well as the behavior of completed structures, can be significantly influenced by water movement within soil. For example, the rate of consolidation of soil and related settlement of structures on soil are highly dependent on how permeable a given soil is. Also, the amount of leakage through and under dams and hydrostatic uplift on dams (and other structures) are influenced by soil permeability. Additional examples where permeability is a factor in geotechnical engineering are infiltration into excavations and dewatering therefrom, stability of slopes and embankments, and erosion. The type, manner, and practical effects of water movement are discussed in Chapter 5. The flow of water through soil is governed by Darcy's law, which is also covered in Chapter 5.

Capillarity refers to the rise of water (or other liquids) in a small-diameter tube inserted into the water, the rise being caused by both cohesion of the water's molecules (surface tension) and adhesion of the water to the tube's wall. The amount of rise of water in the tube above the water level surrounding the tube is inversely proportional to the tube's diameter. With soils, capillarity occurs at the water table (see Section 3–4) when water rises from saturated soil below into dry or partially saturated soil above the water table. The "capillary tubes" through which water rises in soils are actually the void spaces among soil particles. Because the voids interconnect in varying directions (not just vertically) and are irregular in size and shape, accurate calculation of the height of capillary rise is virtually impossible. It is known, however, that the height of capillary rise is associated with the mean diameter of a soil's voids, which is in turn related to average grain size. In general, the smaller the grain size, the smaller the void space, and consequently the greater will be the capillary rise. Thus, clayey soils, with the smallest grain size, should theoretically experience the greatest capillary rise, although the rate of rise may be very slow because of the characteristically low permeability of such soils. In fact, the largest capillary rise for any particular length of time generally occurs in soils of medium grain sizes (such as silts and very fine sands).

It is well known from physics that water expands when it is cooled and freezes. When the temperature in a soil mass drops below water's freezing point, water in the voids freezes and therefore expands, causing the soil mass to move upward. This vertical expansion of soil caused by freezing water within is known as *frost heave.* Serious damage may result from frost heave when structures such as pavements and building foundations supported by soil are lifted. Because the amount of frost heave (i.e., upward soil movement) is not necessarily uniform in a horizontal direction, cracking of pavements and building walls and/or floors may occur. When the temperature rises above the freezing point, frozen soil thaws from the top downward. Because resulting melted water near the surface cannot drain through underlying frozen soil, an increase in water content of the upper soil, a decrease in its strength, and subsequent settlement of structures occur. Clearly, alternate lifting and settling of pavements and structures as a result of frost heave are undesirable, may cause serious structural damage, and should be avoided or at least minimized.

2–8 COMPRESSIBILITY

When soil is compressed, its volume is decreased. This decrease in volume results from reduction in voids within the soil and consequently can be expressed as a reduction in void ratio (e). Soil compression, which results from loading and causes reduction in the volume of voids (or decrease in void ratio), is usually brought on by the extruding of water and/or air from the soil. If saturated soil is subjected to the weight of a building and water is subsequently squeezed out or otherwise lost, resulting soil compression can cause undue building settlement. If water is added to the soil, soil expansion may occur, causing building uplift.

Settlement resulting from the compressibility of soil varies depending on whether a soil is cohesionless or cohesive. Cohesionless soils (such as sands and gravels) generally compress relatively quickly. In most cases, most of the settlement a structure built on cohesionless soil will undergo takes place during the construction phase. Additionally, compression of cohesionless soils can be induced by vibration more easily and more quickly than of cohesive soils.

Compressibility is more pronounced in the case of cohesive soils (such as clays), where soil moisture plays a part in particle interaction. Because of lower permeabilities, cohesive soils compress much more slowly because the expulsion of water from the small soil pores is so slow. Hence, the ultimate volume decrease of a cohesive soil and associated settlement of a structure built on this soil may not occur until some time after the structure is loaded.

It is helpful to consider total settlement as a two-phase process—*immediate settlement* and *consolidation settlement.* Immediate settlement occurs very rapidly—within days or even hours after a structure is loaded. Consolidation settlement occurs over an extended period of time (months or years) and is characteristic of cohesive soils. Consolidation settlement can be further divided into *primary consolidation* and *secondary consolidation* (sometimes called *creep*). Primary consolidation occurs first; it occurs faster and is generally larger, easier to predict, and more important than secondary consolidation. Secondary consolidation occurs subsequent to primary consolidation. It is thought to occur less due to extrusion of water from the voids and more as a result of some type of plastic deformation of the soil.

The preceding discussion of compressibility of soil is presented here to give a brief introduction to this subject because the purpose of this chapter is to introduce various engineering properties of soils. A more comprehensive treatment of compressibility is given in Chapter 7.

2–9 SHEAR STRENGTH

Shear strength of soil refers to its ability to resist shear stresses. Shear stresses exist in a sloping hillside or result from filled land, weight of footings, and so on. If a given soil does not have sufficient shear strength to resist such shear stresses, failures in the forms of landslides and footing failures will occur.

Shear strength results from frictional resistance to sliding, interlocking between adjacent solid particles in the soil, and cohesion between adjacent soil particles. Because the ability of soil to support an imposed load is determined by its shear strength, the shear strength of soil is of great importance in foundation design (e.g., in determining a soil's bearing capacity), lateral earth pressure calculation (e.g., for retaining wall and sheet piling designs), slope stability analysis (for earth cuts, dams, embankments, etc.), pile design, and many other considerations. As a matter of fact, shear strength of soil is often a factor in soil problems.

The shear strength of a given soil may be approximately described by the Coulomb equation:

$$s = c + \bar{\sigma} \tan \phi \tag{2–17}$$

where
s = shear strength
c = cohesion
$\bar{\sigma}$ = effective intergranular normal (perpendicular to the shear plane) pressure
ϕ = angle of internal friction
$\tan \phi$ = coefficient of friction

The preceding discussion of shear strength of soil is presented here to give an introduction to this subject. A more comprehensive treatment of shear strength of both cohesionless and cohesive soils, including certain long-term effects on shear strength of cohesive soil, is given in Chapter 8.

The shear strength parameters, c and ϕ, in Eq. (2–17) can be determined directly or indirectly by standard field or laboratory tests (see Chapter 8).

2–10 COMPACTION—IMPROVING ENGINEERING PROPERTIES OF SOIL

In geotechnical engineering practice, the soils encountered at construction sites may be of poor quality for construction purposes. Specifically, they may be weak, highly compressible, or highly permeable. In such cases, the engineering properties of the soils may be improved by *mechanical stabilization* or *densification,* also called *compaction.*

Compaction is the densification of soils by pressing the soil particles more tightly together by expelling air from the void space. It generally results in a modification of the soil's water content and an increase in its density. Three important effects of compaction are (1) an increase in the soil's shear strength, (2) a decrease in future settlement of the soil, and (3) a decrease in its permeability. These three effects are beneficial for various types of earth construction, such as highways, airfields, and earthen dams, and, as a general rule, the greater the compaction, the greater these benefits will be.

Soil compaction is effected by the application of mechanical energy. The common types of field compaction equipment used for this purpose are (1) smooth wheel roller, (2) sheepsfoot roller, (3) pneumatic roller, and (4) vibratory roller.

The preceding discussion of compaction is presented here to give a brief introduction to this subject. A more comprehensive treatment of compaction is given in Chapter 4.

2–11 COMPACTNESS—RELATIVE DENSITY

In granular soils, compressibility and shear strength (covered in two previous sections) are related to the compactness of the soil grains. For a soil in its densest condition, its void ratio is the lowest, and it exhibits the highest shear strength and the greatest resistance to compression. Conversely, in its loosest condition, its void ratio is the highest, and its shear strength and resistance to compression are the lowest. Soils in a natural state generally exhibit characteristics somewhere between these two extremes. *Compactness* refers to the relative condition of a given soil between these two extremes.

To evaluate the relative condition of a given granular soil, the *in situ* void ratio can be determined and compared to the void ratio when the soil is in its densest condition and when it is in its loosest condition. Then, the *relative density* (D_r) can be evaluated by the equation

$$D_r = \frac{e_{max} - e_0}{e_{max} - e_{min}} \times 100\% \qquad \textbf{(2–18)}$$

where e_{max} = highest void ratio possible for a given soil (void ratio of the soil in its loosest condition)
e_0 = void ratio of the soil in-place
e_{min} = lowest void ratio possible for the soil (void ratio of the soil in its densest condition)

Relative density can also be evaluated in terms of maximum, minimum, and in-place dry unit weights (γ_{max}, γ_{min}, and γ, respectively) by the equation

$$D_r = \frac{\gamma_{max}(\gamma - \gamma_{min})}{\gamma(\gamma_{max} - \gamma_{min})} \times 100\% \qquad \textbf{(2–19)}$$

This equation is generally more convenient to use because it is easier to evaluate dry unit weights than void ratios.

Values of γ_{min} or e_{max} for a given soil can be determined by performing standard laboratory tests on a quantity of the soil that has been dried, pulverized, and poured slowly from a small height through a funnel into a container. Values of γ_{max} or e_{min} can be found in the laboratory by prolonged vibration of the soil under a vertical load.

Clearly, the relative density of any soil varies between 0 and 100%. Soils having relative densities less than 15% are considered to be "very loose," whereas those with values between 15 and 35% are "loose." "Medium dense" soils have relative densities between 35 and 65%, whereas "dense" soils have values between 65 and 85%. Soils with relative densities greater than 85% are considered to be "very dense."

Relative density may be used as an indicator of the degree of compactness of *in situ* soils and/or of compacted fills. In the latter case, a required relative density might be a specification requirement. Relative density may also be used as a rough indicator of soil stability. A low value of relative density indicates a "loose" soil, which would tend to be relatively unstable, whereas a soil with a high value of relative density would tend to be more stable.

EXAMPLE 2–11

Given

1. A fine, dry sand with an in-place unit weight of 18.28 kN/m^3.
2. The specific gravity of solids is 2.67.
3. The void ratio at its densest condition is 0.361.
4. The void ratio at its loosest condition is 0.940.

Required

Relative density of the sand.

Solution

From Eq. (2–15),

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{18.28 \text{ kN}}{(2.67)\,(9.81 \text{ kN/m}^3)} = 0.6979 \text{ m}^3$$

$$V_v = V - V_s = 1 \text{ m}^3 - 0.6979 \text{ m}^3 = 0.3021 \text{ m}^3$$

$$e_0 = \frac{V_v}{V_s} = \frac{0.3021 \text{ m}^3}{0.6979 \text{ m}^3} = 0.433$$

From Eq. (2–18),

$$D_r = \frac{e_{\max} - e_0}{e_{\max} - e_{\min}} \times 100\% \qquad \textbf{(2–18)}$$

$$D_r = \frac{0.940 - 0.433}{0.940 - 0.361} \times 100\% = 87.6\%$$

2–12 PROBLEMS

2–1. Draw a gradation curve and find the median size, effective size, and coefficients of uniformity and of curvature for a soil sample that has the following test data for mechanical grain-size analysis:

U.S. Sieve Size	Size Opening (mm)	Mass Retained (g)
⅜ in.	9.50	0
No. 4	4.75	42
No. 10	2.00	146
No. 40	0.425	458
No. 100	0.150	218
No. 200	0.075	73
Pan	—	63

2–2. A sample of soil was tested in the laboratory, and the test results were listed as follows. Classify the soil by both the AASHTO system and the Unified Soil Classification System.

1. Liquid limit = 29%.
2. Plastic limit = 19%.
3. Mechanical grain-size analysis:

U.S. Sieve Size	Percentage Passing
1 in.	100
¾ in.	90
⅜ in.	82
No. 4	70
No. 10	65
No. 40	54
No. 200	25

2–3. An undisturbed chunk of soil has a wet weight of 62 lb and a volume of 0.56 ft^3. When dried out in an oven, the soil weighs 50 lb. If the specific gravity of solids is found to be 2.64, determine the water content, wet unit weight of soil, dry unit weight of soil, void ratio, porosity, and degree of saturation.

2–4. A 72-cm^3 sample of moist soil weighs 141.5 g. When it is dried out in an oven, it weighs 122.7 g. The specific gravity of solids is found to be 2.66. Find the water content, void ratio, porosity, degree of saturation, and wet and dry unit weights.

2–5. A soil specimen has a water content of 18% and a wet unit weight of 118.5 lb/ft^3. The specific gravity of solids is found to be 2.72. Find the dry unit weight, void ratio, and degree of saturation.

2–6. An undisturbed soil sample has a void ratio of 0.56, water content of 15%, and specific gravity of solids of 2.64. Find the wet and dry unit weights in lb/ft^3, porosity, and degree of saturation.

2–7. A 100% saturated soil has a wet unit weight of 112.8 lb/ft^3, and its water content is 42%. Find the void ratio and specific gravity of solids.

2–8. A 100% saturated soil has a dry unit weight of 15.80 kN/m^3, and its water content is 26%. Find the saturated unit weight, void ratio, and specific gravity of solids.

2–9. A 100% saturated soil has a void ratio of 1.33 and a water content of 48%. Find the unit weight of soil in lb/ft^3 and specific gravity of solids.

2–10. The water content of a 100% saturated soil is 35%, and the specific gravity of solids is 2.70. Determine the void ratio and unit weight in lb/ft^3.

2–11. A soil sample has the following data:

1. Degree of saturation = 42%.
2. Void ratio = 0.85.
3. Specific gravity of solids = 2.74.

Find its water content and unit weight in lb/ft^3.

2–12. A 0.082-m^3 sample of soil weighs 1.445 kN. When it is dried out in an oven, it weighs 1.301 kN. The specific gravity of solids is found to be 2.65. Find the water content, void ratio, porosity, degree of saturation, and wet and dry unit weights.

2–13. The wet unit weight of a soil sample is 18.55 kN/m^3. Its specific gravity of solids and water content are 2.72 and 12.3%, respectively. Find the dry unit weight, void ratio, and degree of saturation.

2–14. A fine sand has an in-place unit weight of 18.85 kN/m^3 and a water content of 5.2%. The specific gravity of solids is 2.66. Void ratios at densest and loosest conditions are 0.38 and 0.92, respectively. Find the relative density.

2–15. Derive an expression for $e = f(n)$, where e is void ratio and n is porosity.

2–16. Derive an expression for $n = f(e)$, where n is porosity and e is void ratio.

2–17. A sand sample has a porosity of 38% and specific gravity of solids of 2.66. Find the void ratio and wet unit weight in lb/ft^3 if the degree of saturation is 35%.

2–18. A proposed earthen dam will contain 5,000,000 m^3 of earth. Soil to be taken from a borrow pit will be compacted to a void ratio of 0.78. The void ratio of soil in the borrow pit is 1.12. Estimate the volume of soil that must be excavated from the borrow pit.

2–19. A soil sample with a water content of 14.5% and unit weight of 128.2 lb/ft^3 was dried to a unit weight of 118.8 lb/ft^3 without changing its void ratio. What is its new water content?

2–20. The unit weight, relative density, water content, and specific gravity of solids of a given sand are 17.98 kN/m^3, 62%, 7.6%, and 2.65, respectively.

1. If the minimum void ratio for this soil is 0.35, what would be its maximum void ratio?
2. What is its unit weight in the loosest condition?

2–21. A soil sample has a degree of saturation of 30.4% and void ratio of 0.85. How much water must be added per cubic foot of soil to increase the degree of saturation to 100%?

2–22. A soil sample has the following properties:

1. $e_{max} = 0.95$.
2. $e_{min} = 0.38$.
3. $D_r = 47\%$.
4. $G_s = 2.65$.

Find dry and saturated unit weights in both lb/ft^3 and kN/m^3.