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CONVERSION FACTORS

English — S.I.

Length

1 ft. = .3048 m

Force

1 lb. = 4.4482 N

Mass

1 slug = 14.554 kg

Pressure

1 lb/ft\(^2\) = 47.85 \frac{N}{m^2} = 47.85 \text{ Pa}

1 lb/in\(^2\) = 6.90 \text{ kPa}

14.7 lb/in\(^2\) = 101.4 \text{ kPa}

1 ft. water = 2.99 \text{ kPa}

Acceleration of gravity

\[ g = 9.81 \frac{m}{\text{sec}^2} = 32.2 \frac{ft}{\text{sec}^2} \]

Unit Weight of Water

\[ 1 \frac{lb}{ft^3} = 0.157 \frac{kN}{m^3} \]

\[ 62.5 \frac{lb}{ft^3} = 9.81 \frac{kN}{m^3} = 1 \frac{gm}{cc} \]
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Geotechnical engineering developed from the science of soil mechanics, which grew largely through the work of Karl Terzaghi and his associates in the early part of the twentieth century. Both the science and the art of geotechnical engineering are changing rapidly, and many important advances have been made in the past decade.

This book is designed to provide an introduction to methods of analysis at an elementary level for undergraduate engineering students. In order to be successful, a practicing geotechnical engineer must combine a thorough knowledge of analysis with experience and common sense.

Generally, soil mechanics books contain a great deal of reference material and many subjects that are appropriately taught at the graduate level. This book attempts to separate the basic ideas that are needed for a good understanding of geotechnical analysis from those that might best be left for another text on advanced problems and to treat these subjects in a way designed for optimum understanding by a college student. Teaching is thus a primary objective of this book, and no attempt is made to provide a large amount of reference material for the practicing engineer.

The first nine chapters of the text are arranged to provide a suitable sequence for a two-quarter, three-credits-per-quarter course for junior or senior engineering students. In our class at Utah State University we cover all the topics in the first nine chapters in the undergraduate courses. The last two chapters contain supplementary material that, although still basic in nature and desirable for use by some instructors, is not necessary for an adequate elementary introduction to analysis.

The SI system of units is used exclusively in this text, except in a very few instances where values may be given in English units for the purpose of comparison. No attempt is made to change gradually from the English system
to the SI system. We believe that complete and sudden immersion in the SI system will result in the student's ability to think in terms of these units more clearly and more rapidly than with a gradual change.

We would like to acknowledge and express our appreciation to those who helped in preparing this text. Dr. Michael L. DeBloois and Dr. Steven J. Soulier of the Instructional Media Department at Utah State University provided support and consultation during the initial stages of our work. Mr. Thomas J. Allen worked all of the problems at the end of each chapter for the solutions manual and Mr. David W. Nyby helped prepare the computer programs. Dr. Joseph Olsen and Mr. Richard E. Riker used a draft form of the text in their classes and provided helpful comments. Dr. James K. Mitchell, Dr. W. L. Schroeder, Dr. Robert M. Koerner and Dr. Louis J. Thompson reviewed the manuscript and provided valuable comments. For the past four years we have used the manuscript as the text for our soil mechanics classes and many improvements were made from student comments and from this opportunity to field test the product. Genevieve Fonnesbeck, Suzanne Wilson and Jolene Kiefer typed the manuscript.

Irving S. Dunn
Loren R. Anderson
Fred W. Kiefer

Logan, Utah
January, 1980
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LIST OF SYMBOLS

\[ \begin{align*}
A &= \text{area (} L^2 \text{)} \text{ or constant} \\
A_v &= \text{area of voids (} L^2 \text{)} \\
a &= \text{area (} L^2 \text{)} \text{ or length (} L \text{)} \\
\alpha_c &= \text{ratio of contact area to total area} \\
B &= \text{width of footing (} L \text{)} \\
b &= \text{distance} \\
C, C_1, C_2 &= \text{constants} \\
C_c &= \text{compression index} \\
C_u &= \text{uniformity coefficient} \\
C_r &= \text{cohesive force (} F \text{)} \\
C_s &= \text{shape coefficient} \\
c_r &= \text{coefficient of consolidation} \\
C_w &= \text{wall adhesive force (} F \text{)} \text{ or water table correction factor} \\
C_z &= \text{coefficient of curvature} \\
c &= \text{cohesion} \\
c_1, c_2, c_3 &= \text{constants} \\
D &= \text{depth or particle diameter (} L \text{)} \\
D_m &= \text{representative particle diameter (} L \text{)} \\
D_f &= \text{depth of footing (} L \text{)} \\
D_{10} &= \text{particle diameter with 10\% finer than} \\
D_r &= \text{relative density (granular soils)} \\
d &= \text{distance (} L \text{)} \\
d_c, d_q &= \text{depth factors for bearing capacity} \\
E &= \text{modulus of elasticity (} F/L^2 \text{)} \text{ or energy (} FL \text{)} \\
E &= \text{horizontal soil force (} F \text{)} \\
e &= \text{void ratio} \\
e_o &= \text{initial void ratio} \\
e_{\text{max}}, e_{\text{min}} &= \text{maximum or minimum void ratio} \\
F &= \text{safety factor or percent passing the 0.075 mm sieve} \\
F &= \text{force} \\
F_g, F_s, F_T &= \text{gravitational force or particle surface force or surface tension force (} F \text{)} \\
f_s &= \text{soil-sleeve friction (} F/L^2 \text{)} \\
G &= \text{specific gravity} \\
H &= \text{height or depth (} L \text{)} \\
H_o &= \text{initial sample height} \\
H_s &= \text{height of soil solids} \\
h &= \text{hydraulic head (} L \text{)} \text{ or vertical distance (} L \text{)} \\
h_e &= \text{hydraulic elevation head (} L \text{)} \\
h_L &= \text{hydraulic head loss (} L \text{)} \\
h_p &= \text{hydraulic pressure head} \\
h_T &= \text{total hydraulic head (} L \text{)} \\
h_v &= \text{velocity head (} L \text{)} \\
\Delta h &= \text{change in hydraulic head (} L \text{)} \\
I_{\text{B}} &= \text{Boussinesq pressure influence coefficient, area load} \\
I_w &= \text{Westergaard pressure influence coefficient, area load} \\
i &= \text{hydraulic gradient or angle}
\end{align*} \]
\( i_c \) = critical hydraulic gradient
\( i_a \) = inclination factor for bearing capacity
\( K_a \) = active earth pressure coefficient
\( K_f \) = principal stress ratio at failure
\( K_o \) = lateral earth pressure coefficient
\( K_p \) = passive earth pressure coefficient
\( k \) = coefficient of permeability \((L/T)\)
        (hydraulic conductivity)
\( k' \) = coefficient of permeability of transformed section \((L/T)\)
\( L \) = length \((L)\)
\( LI \) = liquidity index
\( LL \) = liquid limit (%)
\( M \) = integer or constant
\( M_B \) = Boussinesq pressure coefficient (area load)
\( M_i \) = mass of soil retained \((M)\)
\( M_s \) = mass of dry soil \((M)\)
\( M_w \) = mass of water \((M)\)
\( M_W \) = Westergaard pressure coefficient (area load)
\( m \) = integer or shear strength by Bishop’s modified method of slices \((F)\)
\( m_v \) = coefficient of volume change \((L^2/F)\)
\( N \) = ratio, integer, or blow count for standard penetration test
\( N \) = normal force \((F)\)
\( N_B \) = Boussinesq pressure coefficient (point load)
\( N_W \) = Westergaard pressure coefficient (point load)
\( N_q, N_d, N_c \) = bearing-capacity factors
\( n \) = integer number or porosity
\( P \) = pile penetration resistance \((F)\)
\( P_a \) = active pressure force \((F)\)
\( P_n \) = normal force \((F)\)
\( P_p \) = passive pressure force \((F)\)
\( PI \) = plasticity index
\( PL \) = plastic limit
\( p = \frac{\sigma_1 + \sigma_3}{2} (F/L^2) \)
\( Q \) = flow volume \((L^3)\)
\( Q \) = force or load \((F)\)
\( q \) = volume flow rate \((L^3/T)\)
\( q \) = load per unit area \((F/L^2)\)
\( q = \frac{\sigma_1 - \sigma_3}{2} (F/L^2) \)
\( q_a \) = allowable bearing capacity \((F/L^2)\)
\( q_c \) = cone bearing capacity \((F/L^2)\)
\( q_u \) = unconfined compressive strength \((F/L^2)\)
\( R \) = radius \((L)\) or force \((F)\)
\( R_f \) = friction ratio \(f_s/q_c\%\)
\( r \) = radius \((L)\)
\( r_u \) = pore pressure coefficient
\( S \) = shear strength \((F/L^2)\)
\( S \) = distance or settlement \((L)\)
\( S_r \) = degree of saturation
\( S_u \) = undrained shear strength \((F/L^2)\)
\( SL \) = shrinkage limit (%)
\( s \) = pile tip movement, distance \((L)\)
        or shear resistance \((F/L^2)\)
\( s_d \) = shape factor for bearing
\( T \) = time factor or time
\( T = \) torque (FL), force (F), or time (T)

\( T_s = \) surface tension (F/L)

\( t = \) time (T)

\( U = \) water force (F) or degree of consolidation

\( u = \) water or pore pressure (F/L^2)

\( u_e = \) excess pore pressure (F/L^2)

\( V = \) total volume (L^3)

\( V_a = \) volume of air (L^3)

\( V_s = \) volume of solids (L^3)

\( V_v = \) volume of voids (L^3)

\( V_w = \) volume of water (L^3)

\( v = \) velocity (L/T)

\( \nu = \) actual or pore velocity (L/T)

\( W = \) total weight (F)

\( W_s = \) weight of dry soil solids (F)

\( W_w = \) weight of water (F)

\( \text{WCR} = \) weighted creep ratio

\( \text{WCD} = \) weighted creep distance (L)

\( w = \) water content (%)

\( x, y, z = \) coordinate distance (L)

\( \alpha = \) angular measure

\( \beta = \) constant or angular measure

\( \gamma = \) unit weight (F/L^3)

\( \gamma_b = \) buoyant unit weight (F/L^3)

\( \gamma_d = \) dry unit weight (F/L^3)

\( \gamma_{sat} = \) saturated unit weight (F/L^3)

\( \gamma_w = \) unit weight of water (F/L^3) (9.81 kN/m^3)

\( \gamma_{d_{max}}, \gamma_{d_{min}} = \) maximum and minimum dry unit weight (F/L^3)

\( \gamma_i = \) initial unit weight (F/L^3)

\( \Delta = \) displacement (L)

\( \delta = \) wall friction angle

\( \epsilon = \) strain

\( \eta = \) structural viscosity (FT/L^2)

\( \theta = \) angular measure

\( \theta_p = \) angle of slip lines for passive retaining wall

\( \lambda = \) decimal portion of cross-sectional area occupied by water

\( \mu = \) viscosity (FT/L^2)

\( \nu = \) Poisson’s ratio

\( \rho = \) mass density of matter (M/L^3) or radius of curvature (L)

\( \Sigma = \) summation

\( \sigma = \) total normal stress (F/L^2)

\( \overline{\sigma} = \) effective normal stress (F/L^2)

\( \sigma_o = \) initial effective stress (F/L^2)

\( \sigma_v = \) total vertical normal stress (F/L^2)

\( \sigma_x, \sigma_y, \sigma_\theta = \) normal stress on \( x, y, \) or \( \theta \) plane (F/L^2)

\( \sigma_1, \sigma_2, \sigma_3 = \) principal stresses (F/L^2)

\( \tau = \) shearing stress (F/L^2)

\( \tau_{xy} = \) shearing stress associated with the \( x-y \) plane (F/L^2)

\( \tau_\theta = \) shearing stress on \( \theta \) plane (F/L^2)

\( \phi = \) friction angle or velocity potential

\( \overline{\phi} = \) effective friction angle
SOIL PROPERTIES

Subsurface engineering is an art; soil mechanics is an engineering science. . . . We would do well to recall and examine the attributes necessary for the successful practice of subsurface engineering. There are at least three: knowledge of precedents, familiarity with soil mechanics, and a working knowledge of geology. . . . (Peck, 1962)

The purpose of this text is to familiarize the reader with the fundamental principles of soil mechanics. These principles involve applying an understanding of the physical properties of soil to the following.

- Analysis and design of earth structures such as dams and embankments.
- Evaluation of the stability of artificial and natural slopes.
- Evaluation of the ability of soil deposits to provide support for various structures.
- Evaluation of the magnitude and distribution of earth pressures against various structures.
- Prediction of water movement through soil.
- Improvement of soil properties by chemical or mechanical methods.

After the principles of soil mechanics have been mastered, the engineer will have the necessary analytical tools to predict theoretically the behavior of soil under various conditions. Familiarity with the principles of soil mechanics will provide the young engineer with the framework to interpret and evaluate experience and thus begin to develop a knowledge of precedents.

Illustrations of several typical projects that involve the application of the principles of soil mechanics are shown in Figs. 2-1, 3-1, 3-2, and 5-1. Circular steel sheet pile cells are used to support the wharf structure shown in Fig. 2-1. The sheet pile cells must be designed to resist rupture from soil pressure within
each cell as well as to resist overturning and sliding from the lateral pressures exerted by the soil behind each cell. Figure 3-1 illustrates the effect of the two failure criteria (shear failure, excessive settlement) used in the design of spread footings. The design of a zoned earth dam such as that shown in Figs. 3-2 and 4-1 must consider both the mass stability of the embankment and the amount of seepage that is likely to occur through the embankment and foundation. Figure 5-1 shows the test apparatus used to determine the design loads for a pile foundation.

Unlike many engineering disciplines, in geotechnical engineering the engineer must work with a material (soil) for which the physical properties are extremely variable and difficult to evaluate. The determination of the physical properties thus becomes part of the problem-solving process that is as difficult as, and equally important to, the other phases of the process.

The geotechnical engineer has a role in every project that involves earth structures, that requires a soil or rock foundation, or that is constructed below the ground surface. Although every project is different to some extent, there is a
Figure 3.1  A typical design chart used to proportion spread footings. The chart was developed from the results of a subsurface investigation at a particular site and using the principles of soil mechanics.

![Diagram of Allowable Bearing Pressure](image)

FOOTING WIDTH (B) IN FEET

NOTE: Allowable bearing pressure is limited by failure by shear with a safety factor of 3 or by a total settlement of 1 inch.

ALLOWABLE BEARING PRESSURE SPREAD FOOTINGS

Figure 3.2  Oroville Dam maximum section.
A general procedure that is usually applicable in carrying out the soil and foundation work for most projects. This procedure is discussed here in order to give the beginning geotechnical engineering student a general feeling for the processes involved in the applications of soil mechanics. As the student's understanding of the principles of soil mechanics develops, it would be well to review this procedure from time to time. The procedure can be broken down into the following major parts (Worth, 1972).

1. **Define the Project Concept.** This involves establishing the purpose of the project; the schedule for design and construction; the number, type, and location of proposed structures; plans for future expansion of the project; potential future use of the facilities; and special or unusual performance requirements such as vibrating foundations.
2. Project Site Reconnaissance. This involves a review of geologic literature, a review of existing subsurface information, and an inspection of the project site.

3. Develop a "Working Hypothesis" of the Subsurface Conditions. The "working hypothesis" (expected subsurface conditions) is developed from the definition of the project concept and on the information gained from the reconnaissance study.

4. Plan a Field Investigation to Test the "Working Hypothesis." A "working hypothesis" of subsurface conditions is necessary in order to plan intelligently the various features of a subsurface investigation such as the number and depth of test borings, the number and type of soil samples to be obtained, and the type of field tests to be performed.
It is generally necessary to modify certain features of the subsurface investigation as information from the field activities develops.

5. **Develop a Model for Analysis.** The information obtained from the field investigation will probably reveal soil conditions that vary between borings. These conditions are generally much too complicated to analyze, and it will be necessary to develop an idealized model of the subsurface conditions that can be analyzed using the principles of soil mechanics. It is generally necessary to perform laboratory tests on selected samples from the borings in order to establish the physical parameters required to analyze the model.

6. **Evaluate Alternative Schemes.** The model of the subsurface conditions is used to analyze and evaluate (using the principles of soil mechanics) various alternative design and construction schemes. The evaluation of alternatives may involve comparing the serviceability and cost of various foundation types for a multistory building, comparing various designs of a zoned earth dam, comparing methods of dewatering a large open excavation below the groundwater table, comparing various types of retaining structures, etc.

7. **Make Specific Recommendations.** In making specific recommendations, consideration is generally given to cost, reliability, construction time, and environmental impact.

8. **Prepare Plans and Specifications.** Plans and specifications are prepared that reflect the conditions that were assumed during the evaluation of alternatives.

9. **Construction Inspection and Consultation.** It is important that the project is constructed in accordance with plans and specifications; therefore, detailed inspection of the construction operations is essential. Additionally, construction activities, particularly during excavation for the foundation, often reveal information about the subsurface conditions that were not evident at the time the subsurface investigation was made. If these conditions are radically different, it may be necessary to make design modifications during the construction stage.

10. **Performance Feedback.** All too often a soil and foundation engineer's involvement with a project ends at the time of completion of the foundation unless, of course, problems develop that can be related to foundation conditions. If soil engineers are to develop a knowledge of precedence (as Peck (1962) suggests, this is the most important attribute for one practicing the art of subsurface engineering), then it is very important to observe the long-range performance of all projects, not just those involved in catastrophic failures.
This text, in providing the reader with an understanding of the fundamental principles of soil mechanics, will provide a theoretical basis on which to carry out the procedure outlined above and will specifically introduce the tools to make the required analysis of alternatives in step 6 and to develop the physical parameters necessary to describe the simplified subsurface model of step 5. A soil engineer’s ability to carry out a complete project will develop from first-hand experience and from reading the literature and thereby gaining from the experience of others.

A primary responsibility of the soil engineer is to design foundation systems, earth support systems, earth structures, etc., that will perform satisfactorily for the intended life of the structure. As in other areas of structural mechanics, the design must satisfy two specific criteria.

- There must be an adequate factor of safety against a failure by shear.
- Deformations must be within tolerable limits.

In a shear failure, stresses in a soil system exceed the strength of the soil, and this generally results in a collapse of the system. Landslides in natural and artificial slopes and overturned retaining walls are examples of this type of failure. These failures occur as a result of increasing the stresses along the failure plane or of decreasing the strength of the soil along the failure plane. Stresses can be increased from an external load or from a change in the stress distribution by some means, such as making a highway cut at the base of a natural slope. The strength of soil is often decreased during earthquakes, when the soil is subjected to a cyclic loading condition.

Excessive deformation of a foundation system can make a structure unusable. The amount of settlement that can be considered tolerable depends on the function of the structure. Undesirable deformations are caused by both expansion and compression of soil. Certain clay soils expand if the water content of the soil increases, and this can cause foundations and retaining walls to deform excessively. Many soils are very frost susceptible and expand during freezing temperatures, causing damage to highways, building foundations, retaining walls, and other structures. Sufficient measures must, therefore, be taken during design to prevent damage from expanding soils. If a foundation is not adequately designed, excessive settlement of the structure may occur as a result of compression of the underlying soil. The compression can be caused by the weight of the structure, by lowering the level of the groundwater table, or by vibrations from machines and earthquakes.

SOIL-FORMING PROCESS AND HISTORY

The definition of soil that is used by a civil engineer is rather arbitrary and is somewhat different from that used by a geologist, a soil scientist, or a lay person.
A civil engineer considers soil to include all the material, organic and inorganic, overlying bedrock. In interpreting and using the work of other disciplines, the engineer must keep in mind that there are many basic differences in the terminology and definitions used to classify and describe the physical and chemical behavior of soil. The terminology and definitions used in this text are those that are common to the engineering profession.

Based on origin, soils can be broadly classified as organic or inorganic. Organic soils are mixtures in which a significant part is derived from the growth and decay of plant life and in some instances from the accumulation of skeletons or shells of small organisms. Inorganic soils are derived from either chemical or mechanical weathering of rocks.

Inorganic soil that is still located at the place where it was formed is referred to as a residual soil. If the soil has been moved to another location by gravity, water, or wind, it is referred to as a transported soil.

A knowledge of the history of a given soil deposit can, in a general way, reveal much about its engineering properties. The engineering properties are essentially a function of the physical and chemical characteristics of the parent material, the type of weathering that formed the soil, whether the deposit is a residual or transported soil, the method of movement and deposition for a transported soil, the stress history of the soil deposit, the chemical history of the pore water, and the history of the position of the water table. Although detailed sampling and testing is required for a proper evaluation of the engineering properties of a soil, much information can be obtained from a knowledge of the soil type and its history. The following section gives examples of the general engineering characteristics of some soils.

GENERAL ENGINEERING CHARACTERISTICS OF SOME SOIL TYPES

The list of soil types and their general engineering characteristics as presented in this section is not an all-inclusive list. It is intended to show that, with some experience, a knowledge of the soil type and its history can provide preliminary information regarding the soil’s general engineering characteristics. A description of the soil types in a given area is often available for many projects and can be obtained from geologic maps, soil maps, and reports of subsurface investigation made at adjacent sites. This kind of information should be considered preliminary. It is used by soil and foundation engineers only for planning subsurface investigations and laboratory testing programs and during feasibility studies to anticipate foundation requirements.

- Loose sand is simply a sand deposit with a low density. Vibratory loads tend to densify these deposits. Therefore, special measures must be taken in
designing foundations for buildings that are to house machines, because the vibrations from operating the machine may induce intolerable settlements. Loose sands also present problems in high seismic risk areas, because seismic loading can induce liquefaction if the sand is saturated, as well as cause significant settlement.

- Loess is a deposit of relatively uniform, windblown silt. It has a relatively high vertical permeability but low horizontal permeability. Loess soils become very compressible when saturated. This often causes problems with hydraulic structures such as canals and earth dams that are constructed on loess foundations. Special design measures are necessary for these cases and may require prewetting the foundation to induce settlement before starting construction.

- Normally consolidated clays are clay soils that have never been subjected to a pressure greater than the existing pressure. These soils generally tend to be highly compressible, have a low ultimate bearing capacity, and, as with all clay soils, have a very low permeability. Because of the low ultimate bearing capacity and high compressibility, the soil is often not capable of supporting structures on shallow foundations. In these cases, other foundation types must be considered, such as pile foundations where the structural loads are transferred to a lower soil or rock strata that has a higher bearing capacity, or a floating foundation where the quantity of soil excavated for basement levels will equal the weight of the structure.

- Overconsolidated clays are clays that in the past have been subjected to a pressure greater than the existing pressure. Highly overconsolidated clays generally tend to have a rather high ultimate bearing capacity and are relatively incompressible.

- Bentonite is a highly plastic clay resulting from the decomposition of volcanic ash. It is an expansive soil that swells considerably when saturated. This can cause problems in the performance of foundations, sidewalks, concrete slabs, and other structural elements if the soil is subjected to seasonal changes in water content. On the other hand, bentonite is often used beneficially as an impermeable pond liner.

- Peat is fibrous, partly decomposed organic matter or a soil containing large amounts of fibrous organic matter. Peats have a very high void ratio and are extremely compressible. Su and Prysock (1972) reported that the settlement of an embankment 2.68 m high and underlain by 8.24 m of peat and 12.4 m of peaty clay was 2.13 m in 13 years. The ultimate settlement of the embankment was predicted to be 2.59 m after 25 years.

**PARTICLE SIZE**

The size of soil particles range from boulders more than 1 m in diameter to clay-size particles less than 0.001 mm in diameter. The table of Fig. 10-1 shows...
the common soil types (based on grain size) and their approximate particle-size range. In general, the principles of soil mechanics that are developed in this text apply to soils with particle sizes ranging from clay size to gravel.

PARTICLE FORCES AND BEHAVIOR

The behavior of individual soil particles and their interaction with other particles is influenced by the following forces:

- Weight of the particle \( F_g \).
- Particle surface forces \( F_s \).

Weight is the result of gravitational forces and is a function of the volume of the particle. For equidimensional particles such as spheres of diameter \( D \), the weight, \( F_g \), is directly proportional to \( D^3 \). Particle surface forces are of an electrical nature. They are caused by unsatisfied electrical charges in the particle’s crystalline structure. Surface forces, \( F_s \), are directly proportional to the surface area and, hence, for equidimensional particles, to \( D^2 \).

The ratio of the weight of a particle to the particle’s surface forces, \( F_g/F_s \), is directly proportional to \( D \). Thus, for large particle sizes, which include soil particles in the coarse fraction (> 0.075 mm), the weight of the particle is predominant over the surface forces. As the particle diameter decreases, the ratio, \( F_g/F_s \), decreases; thus, for very small values of \( D \), the surface forces predominate. This accounts for the cohesive nature of most fine-grained soils.
Characteristics of the Fine-Soil Fraction

Surface forces play a significant role in the behavior of clay soils and some silts. The crystalline structure of clays forms thin platy-shaped particles that carry a net negative charge on the flat surfaces of the particles. This charged surface attracts the cations that are in the pore water and even tends to orient water molecules into a somewhat structured arrangement. A basic understanding of clay particles and the aqueous solution that surrounds the particles helps in interpreting the engineering behavior of clays. The nature of clay particles can be illustrated by describing the three most common subgroups of the clay minerals: kaolinites, illites, and montmorillonites.

There are two basic crystalline units that form the clay minerals, and the manner in which they are combined differentiates between the three clay mineral subgroups mentioned above. One crystalline unit is the silicon-oxygen tetrahedron, as shown in Fig. 11-1a. The tetrahedrons combine to form the silica

Figure 11-1 Basic crystalline units that form the clay minerals. (a) Silicon-oxygen tetrahedron. (b) Silica Sheet. (c) Aluminium octahedron. (d) Gibbsite sheet. (After Lambie and Whitman.)
sheet, as shown in Fig. 11-1b. The second crystalline unit is the aluminium octahedron, Fig. 11-1c. Combining aluminium octahedrons forms the gibbsite sheet shown in Fig. 11-1d. The arrangement of the molecules in the silica and gibbsite sheets allows these sheets to fit together very closely. Relatively strong bonds hold the sheets in place. Two-layer and three-layer structures of silica and gibbsite sheets are shown in Figs. 12-1 and 13-1, respectively.

The clay mineral kaolinite is a basic two-layer unit (about 7.2Å thick and indefinite in the other direction) formed by stacking a gibbsite sheet on a silica sheet, as shown in Fig. 12-1a and as shown symbolically in Fig. 12-1b. The actual mineral is formed by a number of these two-layer units stacked one on top of the other. The linkage between the units consists of hydrogen bonding and secondary valence forces.

Montmorillonite is a basic three-layer unit (about 9.5Å thick and indefinite in the other directions) formed by placing one silica sheet on the top and one on the bottom of a gibbsite sheet, as shown symbolically in Fig. 13-1. Isomorphous

---

**Figure 12-1** The structure of kaolinite. (a) Atomic structure. (b) Symbolic structure. (After Lambe and Whitman.)
substitution (replacement) of magnesium or iron for the aluminum in the gibbsite sheet is common and somewhat changes the characteristics of montmorillonite. A small amount of isomorphous substitution of aluminum for silicon in the silica sheet may also occur. The bonding between successive three-layer montmorillonite units is by secondary valence forces and exchangeable ion linkage, but it is very weak and water may enter between the units, causing the mineral to swell. Therefore, montmorillonite clays are very expansive, and this function must be considered in designing foundations.

Illite is a very stable three-layer unit that consists of the basic montmorillonite units bonded by secondary valence forces and potassium ions, as shown symbolically in Fig. 13-2. There is always substantial (about 20%) isomorphous substitution of aluminum for silicon in the silica sheet of illite. The mineral does not swell by the introduction of water between sheets, as does montmorillonite.

The above description of the three common clay minerals is very brief and simplified. For a more in-depth study of clay mineralogy, refer to texts that are devoted entirely to this subject, such as Grim (1953).
Individual clay particles consist of many basic clay mineral units stacked on one another. Because these basic units have a sheeted structure, the particles end up with a platy shape. Their surface dimensions are many times the particle thickness. The general shape of particles and the associated surface charge are shown in Fig. 14-1.

Most of the undisturbed clay deposits and the earth embankments of clay soil that are of concern to engineers are partially or fully saturated. Furthermore, the pore water generally contains dissolved cations that can interact with the negatively charged surface of the clay particles. The behavior of clay soils is, therefore, greatly affected by the nature of the individual clay particles and pore water.

Since the surfaces of the clay particles are negatively charged, they tend to adsorb (attract) the positively charged cations that are present in the pore water. The water molecules adjacent to the negatively charged surface may also undergo alteration and become structured. This water is also considered

Figure 14-1  Negatively charged clay particle and surrounding aqueous solution.

[Diagram of negatively charged clay particle with surrounding aqueous solution, labeled: + Positively charged cation, ← Polar water molecule, — Net negative charge on the particle surface]
adsorbed to the surface of the clay particle. The relationship between a negatively charged mineral particle and the associated aqueous solution that surrounds the particle is shown as a schematic sketch of a colloidal micella in Fig. 14-1.

The cations distribute themselves around the negatively charged surface of the clay particles with the greatest density near the surface and a decreasing density with increasing distance from the surface. The cations form a positively charged zone or layer which, together with a negatively charged surface of the clay particle, makes up the electric double layer. The generally accepted concept of the double layer was presented by Stern (1924) and is shown in Fig. 15-1.

**Figure 15-1** Stern double layer.
The nature of the electric double layer affects the structure of the aggregates of clay particles and, hence, the physical properties of the soil.

The cations are not permanently attached to the surface of the clay particles and can be replaced by other cations. This process of replacement is known as cation exchange. There is a known hierarchy of replaceable cations. For example, potassium ions (K+) tend to replace sodium ions (Na+). Thus, if a sodium soil is leached with a solution of potassium chloride (KCl), most of the Na+ ions will be replaced by K+ ions. This process of leaching will change the characteristics of the double layer and, hence, the physical properties of the soil.

**Characteristics of the Coarse-Soil Fraction**

The ratio of the volume to surface area for coarse-grained soils is large enough that electrical surface forces are negligible. Additionally, the pressures encountered in most soil mechanics applications are low enough that individual particles do not fracture. Hence, the aggregate physical properties of the coarse fraction (sand and gravel) are essentially functions of relative density (closeness of the particles) and particle shape (angular, rounded, etc.) and are relatively unaffected by the mineral composition of the soil.

**WEIGHT-VOLUME RELATIONSHIPS**

Figure 17-1 is a schematic representation of an element of soil, showing it as a three-phase system. An aggregate of solid particles, which are assumed to be incompressible under the loads normally encountered in engineering practice, makes up the solid phase of the system. The void spaces between the particles are filled with either liquid or gas or both and make up the remaining two phases.

The relationships defined in Fig. 17-1 between the volumes and weights of each phase are very useful in describing and evaluating the physical properties of soil. It is important that students of soil mechanics become familiar with these relationships and make them part of their technical vocabulary.

The total volume of the element of soil schematically represented in Fig. 17-1 is the sum of the volume of the voids (V_v) and the volume of the solids (V_s). The volume of the voids is the sum of the volume of gas (usually air) (V_a) and the volume of the liquid (usually water) (V_w). There are three useful relationships between the volumes of each phase.

- Void ratio (ε) is the ratio between the volume of the voids and the volume of the solids; it is always expressed as a decimal. Void ratio is used extensively in soil mechanics to express various physical parameters as a function of the soil’s density.
Figure 17-1 Schematic representation of an element of soil showing symbols, definitions, and useful weight-volume relationships.

\[ V = \text{total volume} \]
\[ V_v = \text{volume of voids} \]
\[ V_a = \text{volume of air} \]
\[ V_w = \text{volume of water} \]
\[ V_s = \text{volume of solids} \]
\[ W = \text{total weight} \]
\[ W_w = \text{weight of water} \]
\[ W_s = \text{weight of solids} \]

**Volume Relationships**

**Void ratio**
\[ e = \frac{V_v}{V_s} \]

**Porosity**
\[ n = \frac{V_v}{V} \]

**Degree of saturation**
\[ S_r = \frac{V_w}{V_v} \]

**Supplementary Useful Relationships**
\[ n = \frac{e}{1 + e} \]
\[ e = \frac{n}{1 - n} \]
\[ G_w = S_r e, \quad W = W_s(1 + w) \]

**Weight Relationships**

**Moisture content**
\[ w = \frac{W_w}{W_s} \]

**Weight-Volume Relationships**

**Total unit weight (Moist Unit Weight)**
\[ \gamma = \frac{W}{V} \]

**Buoyant unit weight**
\[ \gamma_b = \gamma_{at} - \gamma_w \]
\[ \delta_j = \frac{W_s}{V_s} \]

**Dry unit weight**
\[ \gamma_d = \frac{W_s}{V} \]

**Specific Gravity**
\[ G = \frac{\gamma_s}{\gamma_w}, \quad \gamma_s = \frac{W_s}{V_s} \]

17
Figure 18-1  Porosity, void ratio, and unit weight of typical soils in natural state. (After Peck, Hanson, and Thornburn, 1974.)

<table>
<thead>
<tr>
<th>Description</th>
<th>Porosity (n)</th>
<th>Void Ratio (e)</th>
<th>Water Content (w)</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>kN/m³</td>
</tr>
<tr>
<td>1. Uniform sand, loose</td>
<td>0.46</td>
<td>0.85</td>
<td>32</td>
<td>14.1</td>
</tr>
<tr>
<td>2. Uniform sand, dense</td>
<td>0.34</td>
<td>0.51</td>
<td>19</td>
<td>17.1</td>
</tr>
<tr>
<td>3. Mixed-grained sand, loose</td>
<td>0.40</td>
<td>0.67</td>
<td>25</td>
<td>15.6</td>
</tr>
<tr>
<td>4. Mixed-grained sand, dense</td>
<td>0.30</td>
<td>0.43</td>
<td>16</td>
<td>18.2</td>
</tr>
<tr>
<td>5. Windblown silt (loess)</td>
<td>0.50</td>
<td>0.99</td>
<td>21</td>
<td>13.4</td>
</tr>
<tr>
<td>6. Glacial till, very mixed-grained</td>
<td>0.20</td>
<td>0.25</td>
<td>9</td>
<td>20.7</td>
</tr>
<tr>
<td>7. Soft glacial clay</td>
<td>0.55</td>
<td>1.2</td>
<td>45</td>
<td>11.9</td>
</tr>
<tr>
<td>8. Stiff glacial clay</td>
<td>0.37</td>
<td>0.6</td>
<td>22</td>
<td>16.7</td>
</tr>
<tr>
<td>9. Soft slightly organic clay</td>
<td>0.66</td>
<td>1.9</td>
<td>70</td>
<td>9.1</td>
</tr>
<tr>
<td>10. Soft very organic clay</td>
<td>0.75</td>
<td>3.0</td>
<td>110</td>
<td>6.8</td>
</tr>
<tr>
<td>11. Soft montmorillonitic clay</td>
<td>0.84</td>
<td>5.2</td>
<td>194</td>
<td>4.2</td>
</tr>
</tbody>
</table>

\( w = \) water content when saturated, in percent of dry weight.
- Porosity ($\eta$) is the ratio of the volume of the voids to the total volume; it is expressed either as a decimal or as a percentage. Although porosity is used widely by some disciplines as a means of expressing the void volume, void ratio is used more commonly in soil mechanics.
- Degree of saturation ($S_s$) is the ratio of the volume of the water to the volume of the voids; it is expressed either as a decimal or as a percentage.

The total weight ($W$) of the soil element is the sum of the dry weight of the solids ($W_s$) plus the weight of the liquid phase ($W_w$). The gaseous phase is assumed to be weightless. Water or moisture content ($w$) is defined as the ratio of the weight of the water to the dry weight of the solids ($W_s$); it is usually expressed as a percentage. Some typical values of natural moisture content (for saturated soils) are given in the table of Fig. 18-1.

The unit weight of any substance (weight per unit volume) is the weight of the substance divided by its volume. In soil mechanics the unit weight of a soil mass can be described in terms of total unit weight ($\gamma$), dry unit weight ($\gamma_d$), and buoyant or submerged unit weight ($\gamma_b$). Total unit weight is defined as the total weight divided by the total volume, and dry unit weight is the weight of the solids (dry weight) divided by the total volume. Some typical values for total unit weight and dry unit weight are given in Fig. 18-1. When the soil is below the water table, it is often convenient to use its buoyant unit weight ($\gamma_b$), which is equal to the saturated unit weight minus the unit weight of water.

Specific gravity is the ratio of the unit weight of a substance to the unit weight of water ($\gamma_w$) at 4°C. In soil mechanics, specific gravity generally refers to

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**Figure 19-1** Specific gravity of some important soil constituents. (After Peck, Hanson, and Thornburn, 1974.)

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum</td>
<td>2.32</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>2.65–2.80</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>2.56</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>2.6</td>
</tr>
<tr>
<td>Illite</td>
<td>2.8</td>
</tr>
<tr>
<td>Chlorite</td>
<td>2.6–3.0</td>
</tr>
<tr>
<td>Quartz</td>
<td>2.66</td>
</tr>
<tr>
<td>Talc</td>
<td>2.7</td>
</tr>
<tr>
<td>Calcite</td>
<td>2.72</td>
</tr>
<tr>
<td>Muscovite</td>
<td>2.8–2.9</td>
</tr>
<tr>
<td>Dolomite</td>
<td>2.87</td>
</tr>
<tr>
<td>Aragonite</td>
<td>2.94</td>
</tr>
<tr>
<td>Biotite</td>
<td>3.0–3.1</td>
</tr>
<tr>
<td>Augite</td>
<td>3.2–3.4</td>
</tr>
<tr>
<td>Hornblende</td>
<td>3.2–3.5</td>
</tr>
<tr>
<td>Limonite</td>
<td>3.8</td>
</tr>
<tr>
<td>Hematite, hydrous</td>
<td>4.3±</td>
</tr>
<tr>
<td>Magnetite</td>
<td>5.17</td>
</tr>
<tr>
<td>Hematite</td>
<td>5.2</td>
</tr>
</tbody>
</table>

---

Weight-Volume Relationships
the specific gravity of the solid particles \((G)\), and is defined as the ratio of the unit weight of the solid particles to the unit weight of water \(\gamma_w V_s\).

The value of the specific gravity can be determined from laboratory tests (ASTM D-854). Typical specific gravity values for some of the most important soil constituents are given in Fig. 19-1.

Unit weight can also be defined in terms of void ratio, specific gravity, the unit weight of water, and either moisture content or degree of saturation. These definitions are given as Eq. 17-1a to 17-1e.

The examples shown in Figs. 20-1 and 21-1 illustrate how these weight-volume relationships are often used.

**Figure 20-1**

**Example Problem** A sample of saturated clay from a consolidometer has a total mass of 1526 g and a dry mass of 1053 g. The specific gravity of the solid particles is 2.7. For this sample determine the water content, void ratio, porosity, and total unit weight.

**Solution** These quantities can be calculated using the relationships given on Fig. 17-1.

**Water content**

\[
\frac{w}{W_s} = \frac{M_w}{M_s} = \frac{1526 - 1053}{1053} = 0.449 \text{ or } 44.9\% 
\]

**Void ratio**

\[
e = \frac{wG}{S_r} = \frac{0.45(2.7)}{1} = 1.212 
\]

**Porosity**

\[
n = \frac{e}{1 + e} = \frac{1.212}{1 + 1.212} 
\]

\[
n = 0.548 \text{ or } 54.8\% 
\]

**Total unit weight**

\[
\gamma = \left(\frac{G + S_r e}{1 + e}\right)\gamma_w = \frac{2.7 + 1(1.212)}{1 + 1.212} = 9.81 
\]

\[
\gamma = 17.33 \text{ kN/m}^3 
\]
Figure 21-1

Example problem A laboratory sample of silty clay has a volume of 14.88 cm$^3$, a total mass of 28.81 g, a dry mass of 24.83 g, and a specific gravity of 2.7. For this sample determine the void ratio and the degree of saturation.

Solution These quantities can be calculated from the relationships given on Fig. 17-1.

**Void ratio**

$$e = \frac{V_v}{V_s}$$

$$V_s = \frac{W_s}{G\gamma_w} = 24.83$$

$$V_s = 9.2 \text{ cm}^3$$

$$V_v = V - V_s = 14.88 - 9.2$$

$$V_v = 5.68 \text{ cm}^3$$

$$e = \frac{5.68}{9.2} = 0.618$$

**Degree of saturation**

$$G_w = S_r e$$

$$w = \frac{W_w}{W_s} = \frac{28.81 - 24.83}{24.83} = 0.16$$

$$S_r = \frac{G_w}{e} = \frac{2.7 \times 0.16}{0.618} = 0.70$$

or  

$$S_r = 70\%$$

**Alternate method**

$$\gamma_d = \frac{G}{1 + e} \gamma_w = \frac{W_s}{V}$$

$$e = \frac{G\gamma_w V}{W_s} - 1$$

$$e = \frac{2.7 \times 1 \times (14.88)}{24.83} - 1$$

$$e = 0.618$$
Figure 22-1  Relative density (granular soil only).

\[
D_r = \frac{\varepsilon_{\text{max}} - \varepsilon}{\varepsilon_{\text{max}} - \varepsilon_{\text{min}}} \times 100\% \quad (22-1a)
\]

\[
D_r = \frac{\gamma_d}{\gamma_{d\text{max}}} \times \frac{\gamma_{d\text{max}} - \gamma_{d\text{min}}}{\gamma_{d\text{min}}} \times 100\% \quad (22-1b)
\]

\( \varepsilon_{\text{max}} \) = maximum void ratio  
\( \varepsilon_{\text{min}} \) = minimum void ratio  
\( \varepsilon \) = void ratio of soil deposit  
\( \gamma_d \) = dry unit weight of soil deposit  
\( \gamma_{d\text{min}} \) = minimum dry unit weight  
\( \gamma_{d\text{max}} \) = maximum dry unit weight

The unit weight of a given soil deposit can generally be related to certain engineering properties such as strength and compressibility. For granular soils, an especially useful relationship is relative density \( (D_r) \), which is defined by Eqs. 22-1a and 22-1b.

Since dry unit weight is much easier to measure than void ratio, the second relationship is generally used to calculate relative density. The maximum and minimum dry unit weights are determined in the laboratory using a standard procedure (ASTM D-2049). The minimum dry unit weight is determined by pouring dry soil (granular) into a container from a standard height without compaction. The maximum dry unit weight is determined by placing soil in a container and then densifying it on a vibrating table. A surcharge weight is placed on the soil while it is being vibrated. Samples are densified dry and fully saturated, and the greatest dry unit weight from the two methods is used in computing the relative density. The field value of \( \gamma_d \) for use in Eq. 22-1b is found by excavating a volume of soil (the dry weight is measured in the laboratory) and measuring the volume of the hole, using water in a rubber balloon or a premeasured volume of dry sand. Nuclear gauges are now being widely used to measure the dry unit weight in the field. The nuclear gauge has the advantage of providing an immediate evaluation of dry unit weight.

**METHODS OF MECHANICAL ANALYSIS**

The process by which soil particles are separated into the soil types as defined by various ranges of particle size (Fig. 10-1) is called mechanical analysis. Soil particles larger than approximately 0.075 mm (coarse fraction) are generally separated into particle-size ranges using a nest of sieves. A sample of soil is
placed on the top sieve (largest openings are on the top sieve with progressively smaller openings on lower sieves), and the nest of sieves is vibrated to allow the individual particles to fall through the sieve openings. The grain-size distribution (particle-size distribution) of the soil is then determined by measuring the dry weight of material retained on each sieve. The results of the analysis are then plotted on semilog paper, as shown in Fig. 23-1.

Standard sieves are made from a woven wire fabric. Specifications have been established to control the size of the wire and the maximum permissible deviation from the average opening. The sieve opening is the distance between parallel wires of the fabric. U.S. Standard sieve-size numbers and their corresponding openings are given in Fig. 24-1 for several common sieve sizes. A complete list of sieve sizes is given in the ASTM Standards (E-11).

The procedure of using sieves to determine the grain-size distribution is referred to as dry mechanical analysis (ASTM D-422). Most soil particles of silt size or larger are usually rounded or angular and nearly equidimensional in shape; therefore, a sieve analysis gives a reasonably accurate measure of particle size. An exception is silt-size particles of mica, which are very flat and platy shaped. The results of a sieve analysis on this material must be used with caution.

The size of particles less than 0.075 mm (fine fraction) is generally determined by using a wet mechanical analysis procedure. In this method, a suspension of soil and water is prepared, and the relative size of particles is determined by measuring the time required for the particles to fall out of
Figure 24-1  Selected sieve sizes.

<table>
<thead>
<tr>
<th>U.S. Standard Sieve Number</th>
<th>Opening (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4.75</td>
</tr>
<tr>
<td>10</td>
<td>2.00</td>
</tr>
<tr>
<td>20</td>
<td>0.850</td>
</tr>
<tr>
<td>40</td>
<td>0.425</td>
</tr>
<tr>
<td>60</td>
<td>0.250</td>
</tr>
<tr>
<td>80</td>
<td>0.180</td>
</tr>
<tr>
<td>100</td>
<td>0.150</td>
</tr>
<tr>
<td>140</td>
<td>0.106</td>
</tr>
<tr>
<td>200</td>
<td>0.075</td>
</tr>
</tbody>
</table>

suspension. Stokes’ law relates the terminal velocity of a sphere falling through a liquid to the square of its diameter and can be used to obtain the grain-size distribution. The time required for the particles to fall out of suspension can be measured by using a hydrometer to determine the specific gravity of the suspension at a given point. As all the particles of a certain size fall past the point, the specific gravity of that point decreases, and the value of the specific gravity of the suspension then becomes a measure of the grain-size distribution of the material. The development of this method is given in Fig. 24-2.

There are several inherent errors in using Stokes’ law to determine the grain-size distribution of fine-grained soils.

Figure 24-2

**Derivation**  Develop the relationships necessary to determine the grain-size distribution of a fine grained soil by the hydrometer method.

**Solution**  A sample of soil of dry weight $W_s$ and specific gravity $G$ is placed into a cylinder of water. The soil–water solution is thoroughly mixed so that at the beginning of the test the solution has the same particle-size distribution at all points. After the solution is mixed, the cylinder is placed down and the particles begin to fall out of suspension.

The initial unit weight of the suspension ($\gamma_i$) can be expressed in terms of the dry weight of the soil ($W_s$), the weight of water ($W_w$) and the total volume ($V$) as

$$\gamma_i = \frac{W_s + W_w}{V}$$
This expression can also be stated in terms of the specific gravity of the solid particles $G$ and the unit weight of water as

$$\gamma_i = \gamma_w + \left( \frac{G - 1}{G} \right) \frac{W_s}{V}$$

Now consider a point at depth $L$ in the soil–water solution and let $T$ designate the time elapsed since the start of sedimentation. The size of particle $D$ that would fall a distance $L$ in time $T$ can be computed from Stoke’s law.

$$v = \frac{L}{T} = CD^2$$

where

$v$ = terminal velocity

$T$ = elapsed time

$L$ = distance

$C$ = known constant

Solving Stoke’s law for $D$ yields

$$D = \sqrt{\frac{L}{TC}} \quad (25-1a)$$

At depth $L$ and time $T$ there will be no particles larger than $D$, and all particles smaller than $D$ will be at the initial concentration. Let $P$ equal the ratio of the dry weight of particles smaller than $D$ to the total dry weight of the soil sample. The weight of particles per unit volume at depth $L$ and time $T$ then becomes

$$\frac{PW_s}{V}$$

and the unit weight of the suspension at this depth and time can be expressed in a manner similar to the initial unit weight as

$$\gamma = \gamma_w + \left( \frac{G - 1}{G} \right) \left( \frac{PW_s}{V} \right)$$

Solving this expression for $P$ yields

$$P = \frac{G}{(G - 1)} \left( \frac{V}{W_s} \right) (\gamma - \gamma_w) \quad (25-1b)$$

The ratio $P$ represents the fraction of particles less than diameter $D$ as given in Eq. 25-1a. Therefore, by measuring the unit weight $\gamma$ of a soil water solution at a known depth $L$ and at time $T$, Eqs. 25-1a and 25-1b can be used to furnish a point on the grain size distribution curve. A common method to determine $\gamma$ is by using a hydrometer, and a specific test procedure is provided in ASTM D-422.
• The particles are never truly spherical. In fact, the shapes may bear little resemblance to spheres.
• The body of water is not indefinite in extent, and since many particles are present, the fall of any particle is influenced by the presence of other particles; similarly, particles near the side walls of the container are affected by the presence of the wall.
• The average value for specific gravity of grains is used; the values for some particles may differ appreciably from the average value. (Taylor, 1948)

The standard testing procedures that have been developed to determine the grain-size distribution by a sedimentation method, such as the hydrometer method (ASTM D-422), are such that the second and third errors listed above are minimized. The first item cannot be overcome, however, and the concept of an equivalent diameter must be accepted.

The grain-size distribution curves as determined from the wet and dry methods can be combined to produce the complete grain-size distribution for a soil that has both significant coarse and fine factions. However, in doing so, it must be realized that there is a difference in the definition of particle size as measured by the two methods. In a sieve analysis, a long cylindrical particle of diameter $D$ would fit through the same sieve opening as a spherical particle of diameter $D$. These two particle shapes are very different, but their particle size as determined by a sieve analysis would be the same. Particle size as measured in a sedimentation method is an equivalent diameter equal to the diameter of a perfect sphere that would fall out of suspension at the same rate as the soil particle.

**PARTICLE-SIZE GRADATION**

The effective particle size of a given soil is defined as a particle size for which 10% of the material by weight is smaller than that size. Curve B in Fig. 23-1 has an effective particle size ($D_{10}$) equal to 0.17 mm. Other particle sizes are also frequently used in describing or classifying soils. The $D_{50}$ of a soil is the median particle size. The $D_{85}$ and $D_{15}$ sizes are used to design filters for drainage systems in earth dams and other structures.

A soil that has a nearly vertical grain-size distribution curve (all particles of nearly the same size) is called a uniform soil. If the curve extends over a rather large range, the soil is called well graded. The distinction between a uniform and a well-graded soil can be defined numerically by the uniformity coefficient $C_u$ and the coefficient of curvature $C_s$. The uniformity coefficient is defined by the ratio

$$C_u = \frac{D_{60}}{D_{10}}$$
The coefficient of curvature is defined as

\[ C_z = \frac{D_{30}^2}{D_{16}D_{60}} \]

Soils with \( C_z \) less than 4 are said to be uniform, and soils with \( C_z \) greater than 4 (6 for sands) are well graded provided that the grain-size distribution curve is smooth and reasonably symmetrical. The coefficient of curvature \( C_z \) is a measure of symmetry and shape of the gradation curve. For a well-graded soil, \( C_z \) will be between 1 and 3.

The soil of curve \( B \) in Fig. 23-1 has a uniformity coefficient of 2.8 and is, therefore, a uniform soil.

The uniformity coefficient and the coefficient of curvature are used as part of the Unified soil classification system shown in Fig. 37-1.

CONSTANCY AND PLASTICITY
OF FINE-GRAINED SOILS

Remolded Soils

The consistency of clays and other cohesive soils is greatly influenced by the water content of the soil. If a clay slurry is slowly dried it will pass from a liquid state to a plastic state and finally into semisolid and solid states. The water content at which a soil passes from one state to another is different for different soils and can be used, in a qualitative way, to distinguish between, or classify, different fine-grained soil types.

Figure 28-1 shows a general relationship between volume and water content of a remolded soil. Superimposed in Fig. 28-1 are limits that divide the curve into four consistency states. The transition from one state to another occurs over a range of water contents, and the limits are, therefore, of an arbitrary nature. These limits were developed by Atterberg (1911) and are known as the Atterberg limits.

At very high water contents, the soil behaves as a viscous liquid in that it flows and will not hold a specific shape. The lowest water content at which the soil is in a liquid state is called the liquid limit (LL), and a specific test procedure (ASTM D-423) has been developed to determine this water content. The test is performed by placing a soil pat in a cup and making a standard-sized groove in the pat (Fig. 29-1). The cup is then dropped onto a hard surface from a height of 10 mm. The liquid limit is defined as the water content when the groove closes over 12.7 mm (½ in.) at 25 blows.

Soil is considered to be in a plastic state when it can be molded or worked into a new shape without crumbling. The lowest water content at which a soil is considered to be in a plastic state is the plastic limit (PL) of the soil. The plastic...
limit is determined by rolling a pat of soil into a thread. When the thread begins to crumble at a diameter of 3.18 mm (1/8 in.), the water content is the plastic limit (ASTM D-424).

Figure 28-1 shows that at some water content the soil will maintain a constant volume even if the water content decreases. This water content is defined as the shrinkage limit (SL), and a standard test procedure is used to determine its value (ASTM D-427).

The difference between the liquid limit and the plastic limit is the plasticity index (PI) and represents the range of water content over which the soil behaves in a plastic state.

Casagrande (1948) observed that many properties of clays and silts can be correlated with the Atterberg limits by means of the plasticity chart. The chart in Fig. 29-2 is divided into regions. By plotting the plasticity index and liquid limit of the soil, the soil may be classified. Inorganic clays lie above the A line, and inorganic silts and organic silts and clays lie below the A line. The Unified classification symbols in Fig. 29-2 are defined on page 37. The liquid limit of the soil can then be used to further classify the soil as having low plasticity or high plasticity. Organic soils are generally dark gray or black and have a distinctive odor.
Figure 29-1  Liquid limit device.

Figure 29-2  Plasticity chart.
Undisturbed Soils

Remolding changes the consistency of most undisturbed cohesive soils. The strength of a cylindrical soil sample in simple compression is referred to as the unconfined compressive strength and is often used to describe the consistency of undisturbed cohesive soils. Values of unconfined compressive strength are correlated with the consistency of the soil in Fig. 30-1.

The strength and other physical characteristics of a given fine-grained soil are functions of the soil’s structure and the nature of the negatively charged mineral particle and the associated aqueous solution that surrounds the particle. Structure refers to the orientation and distribution of the individual particles. There are two extremes of soil structure, as shown in Fig. 31-1: flocculated structure and dispersed structure. In a flocculated structure the particles are edge to face and tend to attract each other; in a dispersed structure the particles are face to face (parallel) and tend to repel each other. In an actual soil deposit the soil structure is likely to be somewhere intermediate between flocculated and dispersed. In general, an element of flocculated soil will have a higher shear strength, a lower compressibility, and a higher permeability than the same element of soil at the same void ratio, but with a dispersed structure.

The structure of a specific soil deposit depends on how the deposit was originally formed, the characteristics of the pore water during the deposit’s formation, and the history of the deposit since its formation. Lambe (1958) suggests that the soil structure of a compacted clay can tend to be essentially flocculated or dispersed, depending on the compactive effort and the remolding water content of the soil.

\[
\begin{array}{|c|c|c|}
\hline
\text{Consistency} & \text{Unconfined Compressive Strength, } q_u & \text{kN/m}^2 & \text{kip/ft}^2 \\
\hline
\text{Very soft} & <25 & <0.5 \\
\text{Soft} & 25-50 & 0.5-1 \\
\text{Medium} & 50-100 & 1-2 \\
\text{Stiff} & 100-200 & 2-4 \\
\text{Very stiff} & 200-400 & 4-8 \\
\text{Hard} & \text{Over 400} & >8 \\
\hline
\end{array}
\]

Figure 30-1  Consistency of soil in terms of unconfined compressive strength.
Remolding a cohesive soil generally causes a loss of strength. The loss of strength is probably caused by the following two conditions.

- Rearranging the particles toward a dispersed structure.
- Disturbing the chemical equilibrium of the particles and associated adsorbed ions and water molecules within the double layer.

The rearrangement of particles (change of the soil structure) is essentially an irreversible process, and the structure is not likely to improve with time. The soil may regain some strength with time, however, as a result of reestablishing a degree of chemical equilibrium. A regaining of strength with time is referred to as thixotropy.

The term sensitivity is used to describe the relative loss of strength of a soil after remolding, and it is defined as the ratio of the undisturbed unconfined compressive strength to the remolded unconfined compressive strength. Terzaghi and Peck (1967) group the sensitivity of clays as 2 to 4 for most clays, 4 to 8 for sensitive clays, and 8 to 16 for extrasensitive clays. The sensitivity of some clays, however, can be much higher. Bjerrum (1967) reported values for the sensitivity of a particular clay in the Drammen Valley, Norway, to range from 200 to 300. This very high sensitivity was caused by leaching the marine
sediment clay with fresh water. The leaching process changed the characteristics of the double layer and thus the physical properties of the clay. The liquid limit of the soil was lowered to the point that it was less than the natural water content of the soil. Remolding this clay, because of the action of an external force, causes the consistency of the soil to approach that of a viscous liquid. Clays with these extremely high sensitivities are referred to as quick clays.

The liquidity index (LI) of a soil is defined as

$$LI = \frac{w - PL}{LL - PL} = \frac{w - PL}{PI}$$

It can be used to evaluate the behavior of a soil deposit if it is disturbed. The liquidity index of a soil will be greater than 1 if the natural water content ($w$) is greater than the liquid limit of the soil. Remolding this soil would then transform it into viscous liquid (thick slurry). When the natural water content is less than the plastic limit, the liquidity index is negative. For a negative liquidity index, a soil would be in the solid or semisolid state.

**SOIL CLASSIFICATION SYSTEMS**

Classification systems are used to group soils in accordance with their general behavior under given physical conditions. Soils that are grouped in order of performance for one set of physical conditions will not necessarily have the same order for performance under some other set of physical conditions. Thus, a number of classification systems have been developed depending on the intended purpose of the system.

Soil classification has proved to be very useful to the soil engineer. It can give general guidance through making available in an empirical manner the results of the field experience of others. However, classification systems must be used with caution. Blindly determining physical properties, such as compressibility, from empirical relationships and then using them in detailed calculations can lead to disastrous results.

**Soil Classification Based on Grain Size**

Grain size seems to be an obvious method by which to classify soils, and most of the earliest efforts to establish classification systems were based on grain size. Figure 33-1 shows several of these classification systems. The MIT system has probably been the most widely used. Since natural soil deposits generally contain a range of particle sizes, it is necessary to determine the grain-size distribution curve and then indicate the percentage of soil in each size range. The U.S. Department of Agriculture developed a grain-size classification system that specifically names a soil depending on the percentages of sand, silt, and clay.
Figure 33-1  Soil classification based on grain size.

<table>
<thead>
<tr>
<th>Classification System</th>
<th>Grain Size, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>MIT, 1931</td>
<td>Gravel</td>
</tr>
<tr>
<td>AASHO, 1970</td>
<td>Gravel</td>
</tr>
<tr>
<td>Unified 1953</td>
<td>Gravel</td>
</tr>
</tbody>
</table>

Figure 33-2 shows the triangular chart used to classify soil by this system. The percentages of sand, silt, and clay are plotted on the chart and the region where the point falls classifies the soil. The example in Fig. 34-1 classifies a soil based on its grain-size distribution.

Figure 33-2  Triangular soil classification chart. (U.S. Department of Agriculture.)
Figure 34-1

Example Problem  Use the triangular soil classification chart (Fig. 33-2) to classify the soil with a grain-size distribution as shown on curve C of Fig. 23-1.

Solution  The triangular chart is only applicable for material less than 2.0 mm, and curve C shows that 2% of the material is coarser than 2.00 mm. However, since this does not represent a significant quantity of the material, the soil will be classified on the basis of the fraction less than 2.00 mm.

Based on the MIT particle-size classification and adjusting the percentages of sand, silt, and clay from the distribution curve to include only the −2.0 mm material, the percentages are:

- Sand 43%
- Silt 44%
- Clay 13%

Plotting these values on the triangular chart of Fig. 33-2 classifies the soil as loam.

If a significant portion of the material had been in the gravel-size range, the material might have been classified as a gravelly loam.

Although grain size seems to provide a very convenient means of classifying soil, it has a major shortcoming. There is very little relationship between grain size and physical properties for fine-grained soil. Other soil classification systems that include consistency and plasticity characteristics of the fine fraction have, therefore, been developed.

AASHTO Classification System

The AASHTO soil classification system was originally developed in the late 1920s by the U.S. Bureau of Public Roads for the classification of soils for highway subgrade use. The system originally classified soils into eight groups, A-1 to A-8. Group A-1 was considered to be the best suited for use as a highway subgrade. Soils of decreasing suitability are given higher group numbers. After some revision, this system was adopted by the American Association of State Highway Officials in 1945. The chart shown in Fig. 35-1 represents AASHTO Designation M145-73, adopted in 1973.

A soil is classified by proceeding from left to right on the chart to find the first group into which the soil test data will fit.

Soils containing fine-grained material are further identified by their group index. The group index is defined by the following equation.

\[
\text{Group index} = (F - 35)[0.2 + 0.005(\text{LL} - 40)] \\
+ 0.01(F - 15)(\text{PI} - 10)
\]
<table>
<thead>
<tr>
<th>General Classification</th>
<th>Granular Materials (35% or less passing 0.075 mm)</th>
<th>Silt-Clay Materials (More than 35% passing 0.075 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A-1</td>
<td>A-2</td>
</tr>
<tr>
<td></td>
<td>A-1-a</td>
<td>A-1-b</td>
</tr>
<tr>
<td></td>
<td>A-2-4</td>
<td>A-2-5</td>
</tr>
<tr>
<td></td>
<td>A-2-6</td>
<td>A-2-7</td>
</tr>
<tr>
<td></td>
<td>A-4</td>
<td>A-5</td>
</tr>
<tr>
<td></td>
<td>A-6</td>
<td>A-7</td>
</tr>
<tr>
<td></td>
<td>A-7-5</td>
<td>A-7-6</td>
</tr>
<tr>
<td></td>
<td>A-7-6</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sieve Analysis, Percent Passing:</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00 mm (No. 10)</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Characteristics of Fraction Passing 0.425 mm (No. 40)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit</td>
</tr>
<tr>
<td>Plasticity index</td>
</tr>
<tr>
<td>6 max N.P.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Usual Types of Significant Constituent Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone Fragments</td>
</tr>
<tr>
<td>Gravel and Sand</td>
</tr>
<tr>
<td>Fine Sand</td>
</tr>
<tr>
<td>Silty or Clayey Gravel Sand</td>
</tr>
<tr>
<td>Silty Soils</td>
</tr>
<tr>
<td>Clayey Soils</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>General Rating as Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent to Good</td>
</tr>
<tr>
<td>Fair to Poor</td>
</tr>
</tbody>
</table>

aPlasticity 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.
Figure 36-1

Example Problem  Use the AASHTO soil classification system to classify the soil with a grain-size distribution as shown on curve A of Fig. 23-1. The liquid limit (LL) and plastic index (PI) of the material passing the 0.425-mm (No. 40) sieve for this soil are 39% and 19%, respectively.

Solution  The AASHTO soil classification system is shown in Fig. 35-1. From curve A, the percentages of material passing the 2.0-mm, 0.425-mm, and 0.075-mm sieves are:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0 mm (No. 10)</td>
<td>41%</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>29.5%</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>21%</td>
</tr>
</tbody>
</table>

Based on the preceding characteristics, the appropriate subgroup from Fig. 35-1 is A-2-6. Since the subgroup is A-2-6, only use the PI portion of the formula to calculate the group index.

The group index for the soil is:

Group index = \( (F - 35)[(0.2 + 0.005(LL-40)) + 0.01(F - 15)(PI - 10)] \)

Group index = 0.01(21-15)(19-10)

Group index = 0.54 or Group index = 1

The final classification for this soil is A-2-6 (1).

where

\( F \) = percent passing 0.075 mm (No. 200) sieve, expressed as a whole number

LL = liquid limit

PI = plasticity index

The group index is always reported as a whole number unless it is negative, for which it is reported as zero. When calculating the group index of A-2-6 and A-2-7 subgroups, only the PI portion of the formula shall be used. The group index is reported as part of the AASHTO classification. If the group index of an A-7-6 soil is 15, the classification should be reported as A-7-6 (15). The higher the value of the group index, the less suitable the material for use as a highway subgrade. A group index of 0 indicates a "good" subgrade material, and a group index 20 or greater indicates a "very poor" subgrade material. The example shown in Fig. 36-1 classifies a soil according to the AASHTO classification system.

Unified Soil Classification System

The most popular soil classification system among soil and foundation engineers is the Unified soil classification system. This system was first developed by
<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>GROUP SYMBOLS</th>
<th>TYPICAL NAMES</th>
<th>CLASSIFICATION CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE GRAINED SOILS</td>
<td>GW</td>
<td>Well-graded gravels and gravel-sand mixtures, little or no fines</td>
<td>$C_p = \frac{D_{10}}{D_{60}}$ Greater than 4</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Poorly graded gravels and gravel-sand mixtures, little or no fines</td>
<td>$C_p = \frac{1}{\frac{D_{30}}{D_{60}}}$ Between 1 and 3</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td>Not meeting both criteria for GW</td>
</tr>
<tr>
<td>SANDS</td>
<td>SW</td>
<td>Well-graded sands and gravelly sands, little or no fines</td>
<td>Not meeting both criteria for GW</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly graded sands and gravelly sands, little or no fines</td>
<td>Atterberg limits plotting below &quot;A&quot; line or plasticity index less than 4</td>
</tr>
<tr>
<td>SANDS WITH SILTS</td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
<td>Atterberg limits plotting above &quot;A&quot; line and plasticity index greater than 7</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, sand-clay mixture</td>
<td>Atterberg limits plotting in hatched area requiring use of dual symbols</td>
</tr>
<tr>
<td>FINE GRAINED SOILS</td>
<td>ML</td>
<td>Inorganic silts, very fine sands, rock flour, silt or clayey fine sands</td>
<td>Less than 5% Pas 0.075 mm sieve</td>
</tr>
<tr>
<td>SILTS AND CLAYS</td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
<td>$C_p = \frac{D_{10}}{D_{60}}$ Greater than 6</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic silts and organic silty clays of low plasticity</td>
<td>$C_p = \frac{1}{\frac{D_{30}}{D_{60}}}$ Between 1 and 3</td>
</tr>
<tr>
<td></td>
<td>MH</td>
<td>Inorganic silts, micaceous or distomaceous fine sands or silts, elastic silts</td>
<td>Atterberg limits plotting below &quot;A&quot; line or plasticity index less than 7</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td>Atterberg limits plotting above &quot;A&quot; line and plasticity index greater than 7</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity</td>
<td>Atterberg limits plotting in hatched area requiring use of dual symbols</td>
</tr>
</tbody>
</table>
Figure 38-1 Engineering-use chart based on the Unified soil classification. (USBR, 1963.)

<table>
<thead>
<tr>
<th>Typical Names of Soil Groups</th>
<th>Group Symbols</th>
<th>Important Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Permeability When</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compacted</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Well-Graded Gravels, Gravel Sand Mixtures, Little or No Fines</td>
<td>GW</td>
<td>Pervious</td>
</tr>
<tr>
<td>Poorly Graded Gravels, Gravel-Sand Mixtures, Little or No Fines</td>
<td>GP</td>
<td>Very Pervious</td>
</tr>
<tr>
<td>Silty Gravels, Poorly Graded Gravel-Sand Silt Mixtures</td>
<td>GM</td>
<td>Semi Pervious To Impervious</td>
</tr>
<tr>
<td>Clayey Gravels, Poorly Graded Gravel-Sand Clay Mixtures</td>
<td>GC</td>
<td>Impervious</td>
</tr>
<tr>
<td>Well Graded Sands, Gravelly Sands, Little or No Fines</td>
<td>SW</td>
<td>Pervious</td>
</tr>
<tr>
<td>Poorly Graded Sands, Gravelly Sands, Little or No Fines</td>
<td>SP</td>
<td>Pervious</td>
</tr>
<tr>
<td>Silty Sands, Poorly Graded Sand-Silt Mixtures</td>
<td>SM</td>
<td>Semi Pervious To Impervious</td>
</tr>
<tr>
<td>Clayey Sands, Poorly Graded Sand-Clay Mixtures</td>
<td>SC</td>
<td>Impervious</td>
</tr>
<tr>
<td>Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands with Slight Plasticity</td>
<td>ML</td>
<td>Semi Pervious To Impervious</td>
</tr>
<tr>
<td>Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays</td>
<td>CL</td>
<td>Impervious</td>
</tr>
<tr>
<td>Organic Silts and Organic Silt-Clays of Low Plasticity</td>
<td>OL</td>
<td>Semi Pervious To Impervious</td>
</tr>
<tr>
<td>Inorganic Silts, Micaceous Or Diatomaceous Fine Sandy Or Silty Soils, Elastic Silts</td>
<td>MH</td>
<td>Semi Pervious To Impervious</td>
</tr>
<tr>
<td>Inorganic Clays of High Plasticity, Fat Clays</td>
<td>CH</td>
<td>Impervious</td>
</tr>
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### Figure 39-1

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<td>EROSION RESISTANCE</td>
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</table>

* Low number indicates preferred soil.
Casagrande (1948) and was known as the Airfield classification system. It was adopted with minor modifications by the U.S. Bureau of Reclamation and the U.S. Corps of Engineers in 1952. In 1969 the American Society for Testing and Materials (ASTM) adopted the Unified system as a standard method for classification of soils for engineering purposes (ASTM D-2487).

As shown in Fig. 37-1, the Unified system divides soils into three main groups: coarse grained, fine grained, and highly organic. Coarse-grained soils are those with more than 50 percent of the material retained on the No. 200 sieve (0.075 mm). Coarse-grained soils are divided into gravels (G) and sands (S). The gravels and sands are grouped according to their gradation and silt or clay content, as well graded (W), poorly graded (P), containing silt material (M), or containing clay material (C). A typical classification thus might be GP for a poorly graded gravel.

Fine-grained soils are those for which more than 50% of the material passes the No. 200 sieve. They are divided into silts (M), clays (C), and organic silts and clays (O), depending on how they plot on the plasticity chart (liquid-limit, plasticity-index relationship). The designation L and H are added to the fine-grained symbols to indicate low plasticity and high plasticity respectively (liquid limits below and above 50%). Highly organic soils (peats) are visually identified. The example shown in Fig. 40-1 classifies a soil in accordance with the Unified soil classification system. Figure 38-1 and Fig. 39-1 show a use chart, based on the Unified soil classification system, that can be used in a general way to characterize the important properties and the relative desirability of a soil for various uses.

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**Figure 40-1**

**Example Problem** Use the Unified soil classification system to classify the soil with a grain-size distribution as shown on curve A of Fig. 23-1. The liquid limit (LL) and plastic index (PI) of the material passing the 0.425-mm (No. 40) sieve are 39% and 19% respectively.

**Solution** The Unified soil classification system is shown in Fig. 37-1. More than 50% of the material is retained on the 0.075-mm sieve, which initially groups the soil as a coarse-grained soil. Furthermore, since more than 50% of the coarse fraction is retained on the 4.75-mm sieve it is a gravel soil. More than 12% of the material passes the 0.075-mm sieve and, therefore, the soil is a gravel with fines. The nature of the fines can be determined from plotting the liquid limit and plastic limit on the plasticity chart that accompanies the Unified system. The plastic index is greater than 7 and the limits plot above the A-line; therefore, this soil is classified as GC (clayey gravel).
Soil engineers should commit to memory the details of the classification system that is most appropriate to their specific area of specialization. A continuing effort should then be made to field-classify soils according to that system and to check the field classification against laboratory results.

PROBLEMS

1-1 Briefly describe two national or international events that involved soil mechanics.

1-2 Comment on the responsibilities of a soil and foundation engineer with regard to the design and construction of a municipal sewage treatment plant.

1-3 Describe the type of foundation system that was used on a building you are familiar with. Describe any unusual subsurface conditions. Speculate on why that type of foundation was used.

1-4 Comment on the importance of “defining the project concept” before starting a subsurface investigation.

1-5 Comment on the importance of “performance feedback.”

1-6 Describe a well-graded gravel and a uniform gravel.

1-7 Distinguish between the clay minerals kaolinite and illite.

1-8 Why do the flat surfaces of a clay mineral have a negative charge?

1-9 What is isomorphous substitution?

1-10 Explain why montmorillonite is an expansive clay.

1-11 Laboratory tests on a certain soil sample provided the following results: liquid limit = 60%, plastic limit = 30%, shrinkage limit = 25%. What is the plasticity index of the soil? Calculate the liquidity index for a natural water content of 36%.

1-12 The Atterberg limits of a particular soil are reported as liquid limit = 60%, shrinkage limit = 40%, and plastic limit = 35%. Are these values reasonable? Explain.

1-13 Define sensitivity and thixotropy.

1-14 Describe a clay particle and the environment that surrounds the particle.

1-15 What is the electric double layer?

1-16 Explain why electrical surface forces are important for fine-grained soils but have little effect on coarse-grained soils.
1-17 How many cubic meters of fill can be constructed at a void ratio of 0.7 from 191,000 m$^3$ of borrow material that has a void ratio of 1.2?

1-18 The average void ratio of a given material is 0.67, and the specific gravity of the solid particles is 2.65. If the water content is 8%, calculate the total unit weight and the degree of saturation.

1-19 The wet unit weight of a soil is 18.80 kN/m$^3$, the specific gravity of the solid particles of the soil is 2.67, and the moisture content of the soil is 12% by dry weight. Calculate dry unit weight, porosity, void ratio, and degree of saturation.

1-20 An undisturbed sample of clay soil is trimmed to fit into a consolidometer. The diameter of the consolidometer is 63.50 mm (2.5 in.) and the height is 25.4 mm (1.0 in.). The wet mass of the consolidation sample is 142.2 g, and the specific gravity of the solid particles is 2.7. At the completion of the test the soil sample is removed from the consolidometer and dried, and the oven-dried mass is determined to be 98.72 g. For this sample, determine the initial water content, void ratio, porosity, and degree of saturation.

1-21 Describe what is meant by dispersed and flocculated structure. What effect does structure have on the physical properties of clays?

1-22 Explain why clay soils lose strength as a result of remolding.

1-23 Explain why a soil classification system based on grain size alone is a poor method of classifying soils for engineering purposes.

1-24 Use the AASHTO soil classification system to classify the soils with grain-size distributions as shown on curves $B$ and $C$ of Fig. 23-1. The fines for the soil of curve $B$ are nonplastic. The liquid limit (LL) and plasticity index (PI) of the material passing the 0.425 mm (No. 40) sieve for the soil of curve $C$ are 32% and 12%, respectively.

1-25 Use the Unified soil classification system to classify the soils in problem 1-24.

1-26 A certain soil has 98% of the particles (by weight) finer than 1 mm, 59% finer than 0.1 mm, 24% finer than 0.01 mm, and 11% finer than 0.001 mm. Draw the grain-size distribution curve and determine the approximate percentage of the total weight in each of the various size ranges according to the MIT size classification. Determine the effective size ($D_{10}$) and the uniformity coefficient for this soil. Classify the soil using the U.S. Department of Agriculture triangular chart.
1-27 Tests run on a given soil produced the following results.

**LL = 27%**  **PI = 15%**

<table>
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<th>Grain Size (mm)</th>
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<td>61</td>
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<tr>
<td>0.150</td>
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<td>25</td>
</tr>
<tr>
<td>0.075</td>
<td>200</td>
<td>15</td>
</tr>
<tr>
<td>0.0065</td>
<td>___</td>
<td>3</td>
</tr>
<tr>
<td>0.001</td>
<td>___</td>
<td>0.5</td>
</tr>
</tbody>
</table>

For the given soil:

(a) Plot the grain-size distribution curve.
(b) Determine the effective particle size \((D_{10})\).
(c) Classify the soil by the U.S. Department of Agriculture's triangular chart.
(d) Classify the soil by the AASHTO classification system.
(e) Classify the soil by the Unified classification system.