17

Strength and Deformation

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17.1 Introduction

The shear strength of soil is generally characterized by the Mohr–Coulomb failure criterion. This criterion states that there is a linear relationship between the shear strength on the failure plane at failure (τ_{ff}) and the normal stress on the failure plane at failure (σ_{ff}) as given in the following equation:

$$\tau_{ff} = \sigma_{ff} \tan \phi + c \tag{17.1}$$

where ϕ is the friction angle and c is the intrinsic cohesion. The strength parameters (ϕ, c) are used directly in many stability calculations, including bearing capacity of shallow footings, slope stability, and stability of retaining walls. The line defined by Eq. (17.1) is called the **failure envelope.** A **Mohr's circle** tangent to a point on the failure envelope (σ_{ff}, τ_{ff}) intersects the x-axis at the **major** and **minor principal stresses** at failure (σ_{1f}, σ_{3f}) as shown in Fig. 17.1. For many soils, the failure envelope is actually slightly concave down rather than a straight line. However, for most situations Eq. (17.1) can be used with a reasonable degree of accuracy provided the strength parameters are determined over the range of stresses that will be encountered in the field problem. For a comprehensive review of Mohr's circles and the Mohr–Coulomb failure criterion, see Lambe and Whitman [1969] and Holtz and Kovacs [1981].

17.2 Strength Parameters Based on Effective Stresses and Total Stresses

The shear strength of soils is governed by **effective stress** ($\boldsymbol{\sigma'}$), which is given by

$$\sigma' = \sigma - u \tag{17.2}$$

where σ is the **total stress** and u is the pore water pressure. Equation (17.1) written in terms of effective stresses is

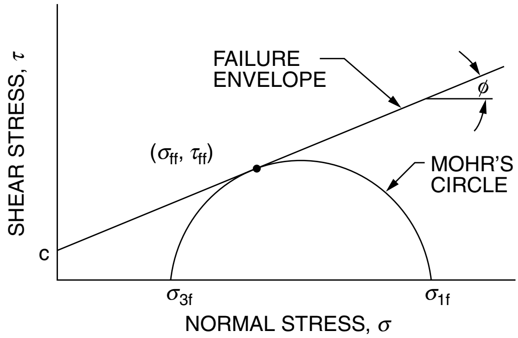

FIGURE 17.1 Mohr–Coulomb failure criteria.

$$\tau_{ff} = (\sigma_{ff} - u)\tan\phi' + c' = \sigma'_{ff}\ \tan\phi' + c' \tag{17.3}$$

where σ'_{ff} is the effective normal stress on the failure plane at failure, and ϕ' and c' are the friction angle and cohesion based on effective stresses. Use of Eq. (17.3) requires knowledge of the pore pressure, but this can be difficult to predict for fine-grained soils. In this case, it is often convenient to assess stability of a structure based on total applied stresses and strength parameters based on total stresses. Determination of strength parameters based on effective and total stresses is discussed further in the following sections.

17.3 Laboratory Tests for Shear Strength

The choice of appropriate shear strength tests for a particular project depends on the soil type, whether the parameters will be used in a total or effective stress analysis, and the relative importance of the structure. Laboratory tests are discussed in this chapter and field tests were discussed in Chapter 15. Common laboratory tests include direct shear, triaxial, direct simple shear, unconfined compression, and laboratory vane. The applicability, advantages, disadvantages, and sources of additional information for each test are summarized in Table 17.1.

Of the available tests, the triaxial test is often used for important projects because of the advantages listed in Table 17.1. The types of triaxial tests are classified according to their drainage conditions during the consolidation and shearing phases of the tests. In a consolidated-drained (CD) test the sample is fully drained during both the consolidation and shear phases of the test. This test can be used to determine the strength parameters based on effective stresses for both coarse- and fine-grained soils. However, the requirement that the sample be sheared slowly enough to allow for complete drainage makes this test impractical for fine-grained soils. In a consolidated-undrained (CU) test the sample is drained during consolidation but is sheared with no drainage. This test can be used for fine-grained soils to determine strength parameters based on total stresses or, if pore pressures are measured during shear, strength parameters based on effective stresses. For the latter use, a CU test is preferred over a CD test because a CU test can be sheared much more quickly than a CD test. In an unconsolidated-undrained (UU) test the sample is undrained during both the consolidation and shear phases. The test can be used to determine the undrained shear strength of fine-grained soils. Further discussion of triaxial tests is given in Holtz and Kovacs [1981] and Head [1982, 1986].

17.4 Shear Strength of Granular Soils

Granular soil is a frictional material. The friction angle (ϕ') is affected by the grain size distribution and dry density. In general, ϕ' increases as the dry density increases and as the soil becomes more well graded, as illustrated in Figs. 17.2 and 17.3. Other typical values of ϕ' for granular soils are given in Holtz and

TABLE 17.1 Summary of Common Shear Strength Tests

Test Type	Applicability	Advantages	Disadvantages	Additional Information
Direct shear	a. Effective strength parameters for coarse-grained and fine-grained soils	a. Simple and inexpensive b. Thin sample allows for rapid drainage of fine-grained soils	a. Only for drained conditions b. Failure plane forced to occur at joint in box c. Nonuniform distribution of stress and strain d. No stress-strain data	a. ASTM D3080* b. U.S. Army, 1970 c. Saada and Townsend, 1981 d. Head, 1982
Triaxial	a. Effective and total strength parameters for coarse-grained and fine-grained soils b. Compared to direct shear tests, triaxial tests are preferred for fine-grained soils	a. Easy to control drainage b. Useful stress-strain data c. Can consolidate sample hydrostatically or to *in situ* K_o state of stress d. Can simulate various loading conditions	a. Apparatus more complicated than other types of tests b. Drained tests on fine-grained soils must be sheared very slowly	a. ASTM D2850* b. U.S. Army, 1970 c. Donaghe et al., 1988 d. Head, 1982 e. Head, 1986
Direct simple shear	a. Most common application is undrained shear strength of fine-grained soils	a. K_o consolidation b. Gives reasonable values of undrained shear strength for design use	a. Nonuniform distribution of stress and strain	a. Bjerrum and Landva, 1966 b. Saada and Townsend, 1981
Unconfined	a. Undrained shear strength of 100% saturated samples of homogenous, unfissured clay b. Not suitable as the only basis for design on critical projects	a. Very rapid and inexpensive	a. Not applicable to soils with fissures, silt seams, varves, other defects, or less than 100% saturation b. Sample disturbance not systematically accounted for	a. ASTM D2166* b. U.S. Army, 1970 c. Head, 1982
Lab vane	Same as for unconfined test	Same as for unconfined test	Same as for unconfined test	Head, 1982

* Designation for American Society of Testing and Materials test procedure.

Kovacs [1981] and Carter and Bentley [1991]. The friction angle also increases as the angularity of the soil grains increases and as the surface roughness of the particles increases. Wet sands tend to have a ϕ' that is 1° or 2° lower than for dry sands [Holtz and Kovacs, 1981]. The **intermediate principal stress** ($\boldsymbol{\sigma_2}$) also affects ϕ'. In triaxial tests σ_2 is equal to either the major principal stress or minor principal stress (σ_1 or σ_3, respectively); however, most field problems occur under **plane strain conditions** where $\sigma_3 \le \sigma_2 \le \sigma_1$. It has been found that ϕ' for plane strain conditions (ϕ'_{ps}) is higher than for triaxial conditions (ϕ'_{tx}) [Ladd et al., 1977]. Lade and Lee [1976] recommend the following equation for estimation of ϕ'_{ps}:

$$\phi'_{ps} = 1.5\ \phi'_{tx} - 17^\circ \qquad (\phi'_{tx} > 34^\circ) \tag{17.4a}$$

$$\phi'_{ps} = \phi'_{tx} \qquad (\phi'_{tx} \le 34^\circ) \tag{17.4b}$$

In practice ϕ' for granular soils is determined using correlations with results from SPT, CPT, and other *in situ* tests, as discussed in Chapter 15, or laboratory tests on samples compacted to the same density as the *in situ* soil. Appropriate laboratory tests are drained direct shear and CD triaxial tests. CU triaxial tests with pore pressure measurements are sometimes used for granular soils with appreciable fines. The method used to prepare the remolded sample and the direction of shearing relative to the direction of deposition has been found to affect ϕ' by up to 2.5° [Oda, 1977; Mahmood and Mitchell, 1974; Ladd

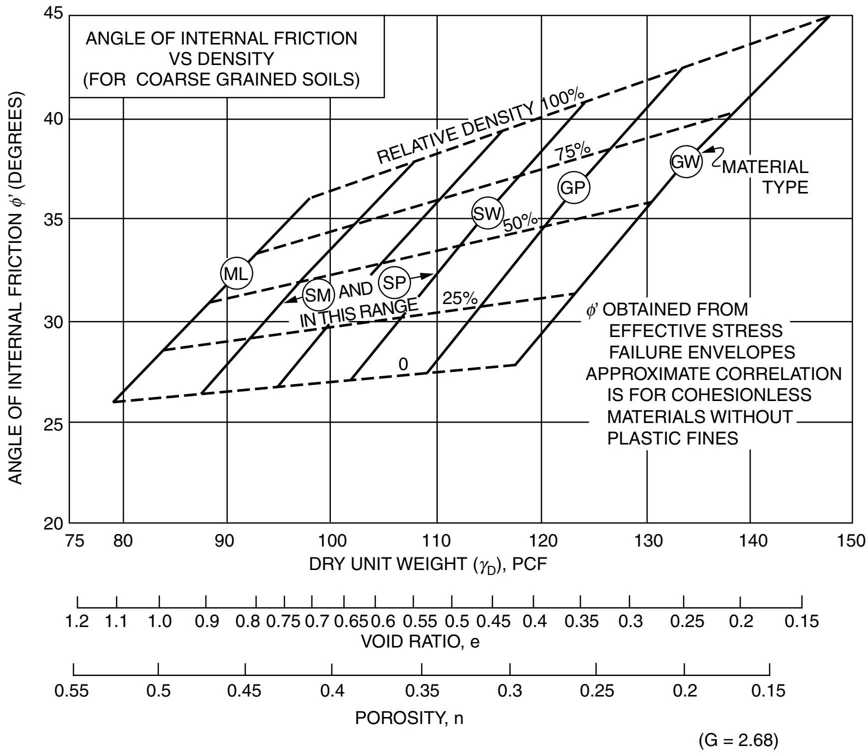

FIGURE 17.2 Correlation of friction angle of granular soils with soil classification and relative density. (*Source:* U.S. Navy. 1986. *Soil Mechanics,* Design Manual 7.1, p. 7.1-149. Naval Facilities Engineering Command, Alexandria, VA.)

et al., 1977]. The c' of granular soils is zero except for lightly cemented soils which can have an appreciable c' [Clough et al., 1981; Head, 1982].

17.5 Shear Strength of Cohesive Soils

The friction angle of cohesive soil based on effective stresses generally decreases as the plasticity increases. This is shown for normally consolidated clays in Fig. 17.4. The c' of normally consolidated, noncemented clays with a preconsolidation stress (defined in Chapter 19) of less than 10,000 to 20,000 psf (500 to 1000 kPa) is generally less than 100 to 200 psf (5 to 10 kPa) [Ladd, 1971]. Overconsolidated clays generally have a lower ϕ' and a higher c' than normally consolidated clays. Compacted clays at low stresses also have a much higher c' [Holtz and Kovacs, 1981].

The shear strength of cohesive soils based on effective stresses is generally determined using a CU triaxial test with pore pressure measurements. To obtain accurate pore pressure measurements it is necessary to fully saturate the sample using the techniques described in U.S. Army [1970], Black and Lee [1973], and Holtz and Kovacs [1981]. This test can be run much more quickly than a CD triaxial test and it has been shown that the ϕ' from both tests are similar [Bjerrum and Simons, 1960].

For clays and some sedimentary rocks that are deformed slowly to large strains under drained conditions, it may be necessary to use the **residual friction angle ϕ'_r**, which can be significantly lower than ϕ'. The ϕ'_r for the clay minerals kaolinite, illite, and montmorillonite range from 4° to 12° [Mitchell, 1993]. ϕ'_r generally decreases as the clay fraction (percent of particle sizes smaller than 0.002 mm) increases [Mitchell, 1993]. Test procedures for ϕ'_r are discussed in Saada and Townsend [1981].

The shear strength of cohesive soils based on total stresses is described in terms of the undrained shear strength (c_u). If the soil is saturated, the undrained friction angle ϕ_u is always zero. For partly saturated

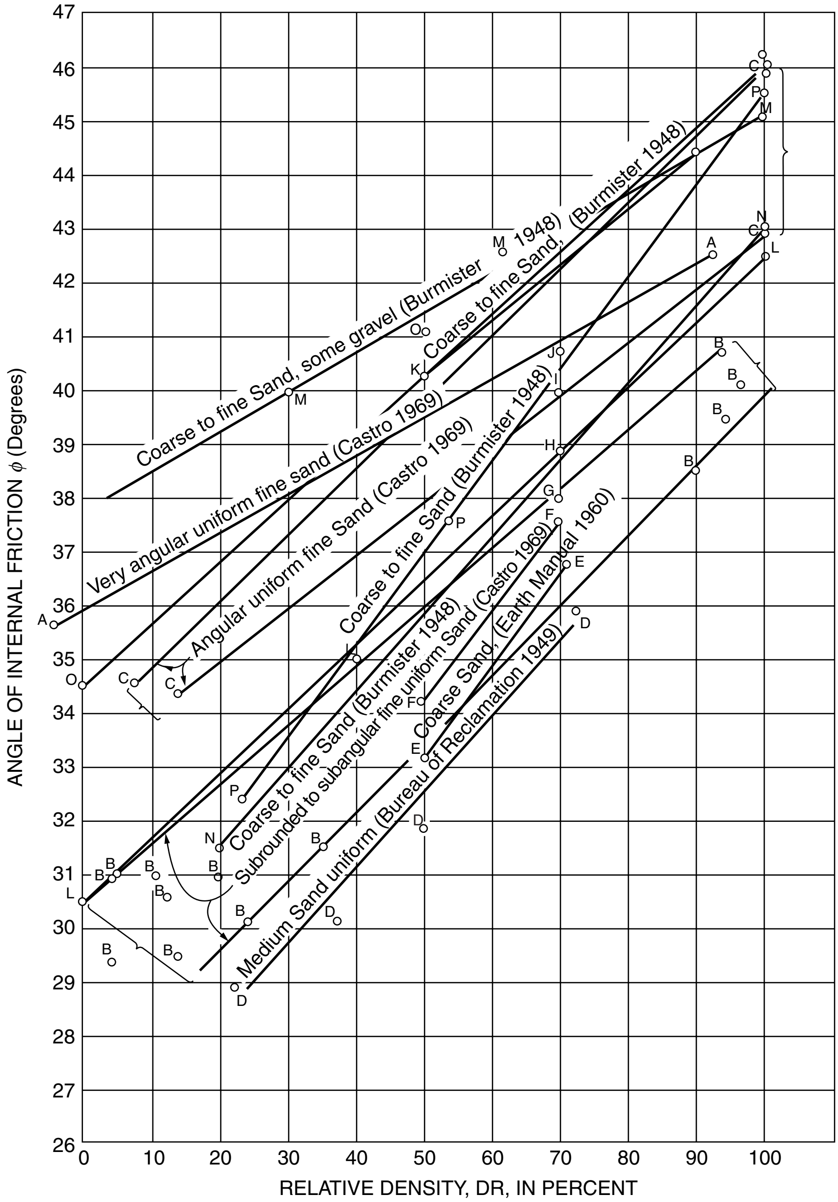

FIGURE 17.3 Friction angle versus relative density for cohesionless soils. (*Source:* Hilf, J. W. 1991. Compacted fill. In *Foundation Engineering Handbook,* ed. H. -Y. Fang, p. 268. Van Nostrand Reinhold, New York. With permission.)

soils, such as compacted soils, it is possible to have $\phi_u > 0$. Normally consolidated and lightly overconsolidated clays generally exhibit the stress strain behavior shown in Fig. 17.5. Overconsolidated and cemented clays generally reach a peak at small strains and then lose strength with further straining, which is also shown in Fig. 17.5. Similar behavior occurs for sensitive clays, that is, clays that lose strength when they are remolded.

The undrained shear strength of clay is a function of its stress history. This is often expressed in dimensionless form as the ratio of c_u / σ'_{vc} where σ'_{vc} is the effective vertical consolidation stress. This ratio is empirically related to the overconsolidation ratio (OCR) by [Ladd, 1991]:

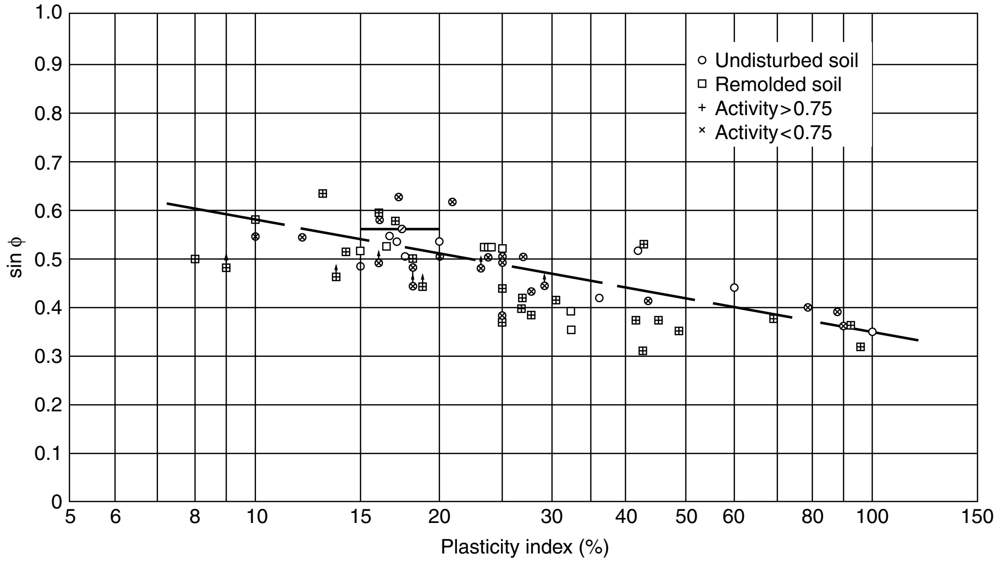

FIGURE 17.4 Friction angle of fine grained soil based on effective stresses versus plasticity index [Kenney, 1959]. (*Source:* Lambe, T. C., and Whitman, R. V. 1969. *Soil Mechanics*, p. 307. John Wiley & Sons, Inc. New York. Copyright © 1969.)

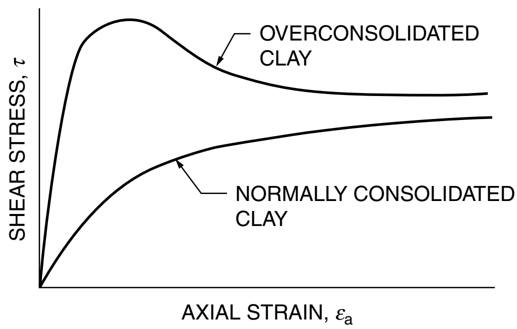

FIGURE 17.5 Stress-strain behavior of normally consolidated and heavily overconsolidated clay.

$$c_u / \sigma'_{vc} = S(\mathrm{OCR})^m \tag{17.5}$$

where

S = 0.22 ± 0.03 for sedimentary clay plotting above A-line on plasticity chart
S = 0.25 ± 0.05 for silts and organic clays plotting below A-line
OCR = overconsolidation ratio = σ'_p/σ'_{vc} (see Chapter 19)
σ'_p = preconsolidation pressure (see Chapter 19)
m = $0.88(1 - C_s/C_c)$
C_s = swelling index from consolidation test (see Chapter 19)
C_c = compression index from consolidation test (see Chapter 19)

Alternately, the undrained shear strength can be expressed as the ratio of c_u/σ'_p. This is shown for the results from K_0 consolidated triaxial compression (TC), triaxial extension (TE), and direct simple shear (DSS) tests in Fig. 17.6. In a TC test the vertical stress is increased to failure while in a TE test the vertical stress is decreased to failure. It is seen that TC tests give higher strengths than TE tests while DSS tests

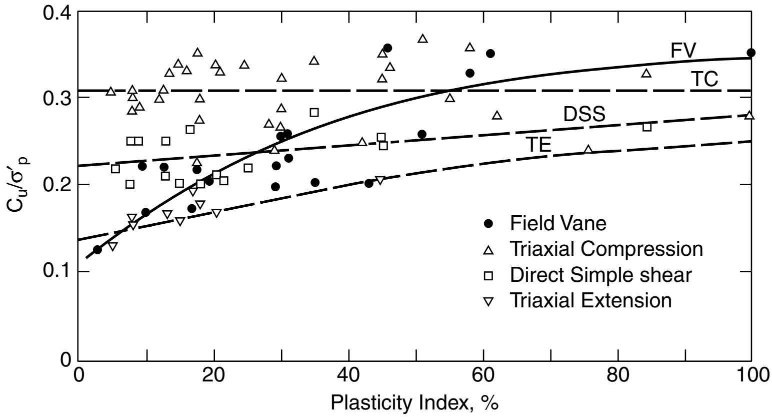

FIGURE 17.6 Undrained shear strength from K_o consolidated CU triaxial compression, triaxial extension, and direct simple shear tests as well as field vane tests. (*Source:* Mesri, G. 1989. A reevaluation of $s_{u(\text{mob})} = 0.22\,\sigma'_p$. *Canadian Geotechnical J.* 26(1): 163. With permission.)

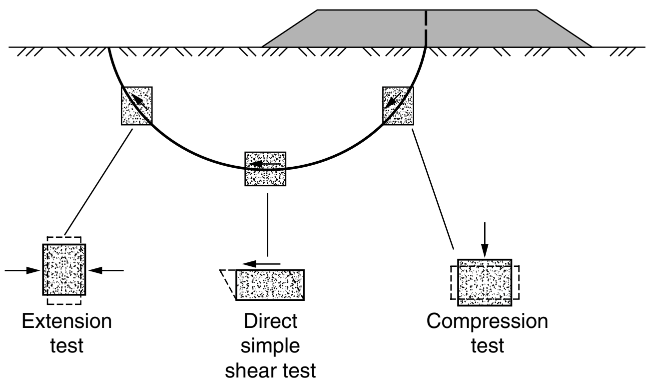

FIGURE 17.7 Relevance of laboratory shear tests to shear strength in the field. (*Source:* Bjerrum, L. 1972. Embankments on soft ground. In *Performance of Earth and Earth-Supported Structures,* Vol. II, p. 16. ASCE, New York. With permission of ASCE.)

give results that are intermediate between the two. Results from field vane (FV) tests are also shown in Fig. 17.6. The applicability of undrained shear strengths from TC, TE, and DSS tests to a typical stability problem is shown in Fig. 17.7. Thus, Mesri [1989] concluded that an average of the results from these three tests would be reasonable for use in design. When this is applied to the data in Fig. 17.6, the following relationship results:

$$c_u = 0.22\sigma'_p \tag{17.6}$$

Mesri [1975] found an identical relationship using results from the FV test and a similar relationship was obtained by Larsson [1980] from a back analysis of 15 embankment failures. It is significant that Eq. (17.6) is independent of the plasticity index of the soil and that the same relationship was obtained using results from laboratory and field tests. This tends to confirm Bjerrum's [1973] conclusion that the "field vane test is the best possible approach for determining the strength for undrained strength stability analysis" [Mesri, 1989, p. 164]. Furthermore, Eq. (17.6) provides a valuable technique for estimating the undrained shear strength of soft clays using σ'_p profiles from consolidation test results.

In practice, the undrained shear strength is often determined *in situ* using field vane tests. For routine projects, c_u may be determined from the results of unconfined or lab vane tests; however, the resulting strength will generally be less than the *in situ* value because of sample disturbance. Undrained direct simple shear tests also can give reasonable estimates of c_u [Ladd, 1981]. For important projects, CU triaxial tests are often performed on undisturbed samples. For highly structured clays with high sensitivities and water contents in excess of the liquid limit and for cemented clays, the sample should first be recompressed to its *in situ* K_0 state of stress to minimize the effects of sample disturbance [Bjerrum, 1973; Jamiolkowski et al., 1985]. For unstructured, uncemented clays, the SHANSEP technique can be used to develop the relationship between c_u/σ'_{vc} and OCR [Ladd and Foote, 1974; Ladd et al., 1977; Jamiolkowski et al., 1985]. For both cases it is necessary to perform both TC and TE tests as Fig 17.6 shows that TC results greatly overestimate the shear strength on a failure surface while the average of TC and TE results yields a more realistic shear strength for use in design. UU triaxial tests do not give meaningful stress-strain data and often give scattered c_u results because of the inability of this test to account for varying degrees of sample disturbance [Jamiolkowski et al., 1985] making the use of this test undesirable for important projects.

17.6 Elastic Modulus of Granular Soils

The **elastic modulus** (***E_s***) of granular soils based on effective stresses is a function of grain size, gradation, mineral composition of the soil grains, grain shape, soil type, relative density, soil particle arrangement, stress level, and prestress [Lambe and Whitman, 1969; Ladd et al., 1977; Lambrechts and Leonards, 1978]. A granular soil is prestressed if, at some point in its history, it has experienced a stress level that is greater than is currently acting on the soil. This is analogous to overconsolidation of a fine-grained soil, which is discussed in Chapter 19. Of the several factors controlling E_s, the ones having the largest influence are prestress, which can increase E_s by more than a factor of six, and extreme differences in relative density, which can make a fivefold difference in E_s [Lambrechts and Leonards, 1978]. The effect of stress level on modulus is often represented by [Janbu, 1963]

$$E_i = Kp_a\left(\frac{\sigma_3}{p_a}\right)^2 \tag{17.7}$$

where E_i is the initial slope of a stress–strain curve, σ_3 is the minor principal stress, K is a dimensionless modulus number that varies from 300 to 2000, n is an exponent number typically between 0.3 and 0.6, and p_a is atmospheric pressure in the same units as σ_3 and E_i [Mitchell, 1993]. Typical values of K and n are given in Wong and Duncan [1974].

Measuring E_s is very difficult since it is nearly impossible to measure the prestress of an *in situ* deposit of granular soil or to obtain undisturbed samples for laboratory testing. While CD triaxial tests can be used to measure E_s [Head, 1986], they are restricted to reconstituted samples that cannot duplicate the *in situ* prestress. For these reasons, E_s is often estimated using *in situ* tests (Chapter 15). Typical values of E_s and Poisson's ratio (μ) for normally consolidated granular soils are given in Table 17.2.

TABLE 17.2 Typical Values of Elastic Modulus and Poisson's Ratio for Granular Soils

Type of Soil	Elastic Modulus, E_s MPa	Elastic Modulus, E_s lb/in.^{2}	Poisson's ratio, μ
Loose sand	10–24	1,500–3,500	0.20–0.40
Medium dense sand	17–28	2,500–4,000	0.25–0.40
Dense sand	35–55	5,000–8,000	0.30–0.45
Silty sand	10–17	1,500–2,500	0.20–0.40
Sand and gravel	69–170	10,000–25,000	0.15–0.35

Source: Das, B. M. 1990. *Principles of Foundation Engineering*, 2nd ed., p. 161. PWS-Kent Publishing Co., Boston. With permission.

TABLE 17.3 Approximate Relationship between Undrained Young's Modulus and Undrained Shear Strength

OCR*	E_s/c_u PI** < 30	30 < PI < 50	PI > 50
<3	600	300	125
3 to 5	400	200	75
>5	150	75	50

* OCR = overconsolidation ratio (defined in Chapter 19).
** PI = plasticity index (defined in Chapter 15).

Source: U.S. Navy, 1986. *Soil Mechanics,* Design Manual 7.1, p. 7.1-215. Naval Facilities Engineering Command. Alexandria, VA.

17.7 Undrained Elastic Modulus of Cohesive Soils

The undrained elastic modulus (E_u) of cohesive soils is a function primarily of soil plasticity and overconsolidation (defined in Chapter 19). It can be determined from the slope of a stress-strain curve obtained from an undrained triaxial test [Holtz and Kovacs, 1981]. However, E_u is very sensitive to sample disturbance, which results in values measured in laboratory tests that are too low [Lambe and Whitman, 1969; Jamiolkowski et al., 1985]. Alternatively, E_u can be measured using *in situ* tests (Chapter 15) or a crude estimate of E_u can be made from the undrained shear strength using the empirical relations shown in Table 17.3. However, there is significant variability in the ratio E_s/c_u, which has been reported to vary from 40 to more than 3000 [Holtz and Kovacs, 1981].

Defining Terms

Effective stress (σ') — Intergranular stress that exists between soil particles.
Elastic modulus (E_s) — Ratio of the change in stress divided by the corresponding change in strain for an axially loaded sample. Also called Young's modulus.
Failure envelope — A line tangent to a series of Mohr's circles at failure.
Intermediate principal stress (σ_2) — In a set of three principal stresses acting at a point in a soil mass, the intermediate principal stress is the one that is less than or equal to the major principal stress but greater than or equal to the minor principal stress.
Major principal stress (σ_1) — The largest of a set of three principal stresses acting at a point in a soil mass.
Minor principal stress (σ_3) — The smallest of a set of three principal stresses acting at a point in a soil mass.
Mohr's circle — A graphical representation of the state of stress at a point in a soil mass.
Plane strain conditions — A loading condition where the normal strain on one plane is zero as would occur for a long retaining wall or embankment.
Principal planes — A set of three orthogonal (mutually perpendicular) planes that exist at any point in a soil mass on which the shear stresses are zero.
Principal stresses — The normal stresses acting on a set of three principal planes.
Residual friction angle (ϕ'_r) — For clays and some sedimentary rocks it is the friction angle that is reached after very large strains.
Total stress (σ) — The sum of the effective stress and the pore water pressure.

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Further Information

Holtz and Kovacs [1981], Lambe and Whitman [1969], and Mitchell [1993] are recommended for a review of the fundamentals of the shear strength of soils.

Laboratory testing procedures are discussed in Head [1982, 1986], U.S. Army [1970], Lambe [1951], and Bowles [1992] as well as the ASTM procedures referenced in Table 17.1.

Major conferences on the shear strength and deformation properties of soil include *Research Conference on Shear Strength of Cohesive Soils,* ASCE, 1960; *Laboratory Shear Testing of Soils,* ASTM STP 361, 1964; *Laboratory Shear Strength of Soil,* STP 740, 1980; and *Advanced Triaxial Testing of Soil and Rock,* ASTM STP 977, 1986.

Relevant state-of-the-art papers include Bjerrum [1973]; Ladd et al. [1977]; and Jamiolkowski et al. [1985].