CHAPTER 2

GEOTECHNICAL AND INDEX PROPERTIES: LABORATORY TESTING; SETTLEMENT AND STRENGTH CORRELATIONS

2-1 INTRODUCTION

This chapter reviews those physical and engineering properties of soils of principal interest for the analysis and design of foundation elements considered in this text. These primarily include the following:

1. Strength parameters[1]
   Stress-strain modulus (or modulus of elasticity), E_s; shear modulus, G', and Poisson's ratio, μ; angle of internal friction, ϕ; soil cohesion, c
2. Compressibility indexes for amount and rate of settlement
   Compression: index, C_c, and ratio, C_c'; recompression: index, C_r, and ratio, C_r'; coefficient of consolidation, c_v; coefficient of secondary compression, C_α
3. Gravimetric-volumetric data
   Unit weight, γ; specific gravity, G_s; void ratio, e, or porosity, n; water content, w_i (where $i = N$ for natural, L for liquid limit, or P for plastic limit; e.g., w_P = plastic limit)

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[1] Symbols and definitions generally follow those of ASTM D 653 except E_s, G', and μ (refer also to "List of primary symbols" following the Preface). It is common to subscript E for soil as E_s, for concrete E_c, etc. G' will be used for shear modulus, as G_s is generally used for specific gravity. The symbol μ is commonly used for Poisson's ratio; however, ASTM D 653 suggests ν, which is difficult to write by hand.

4. Permeability, also called *hydraulic conductivity* (sometimes required)
   k = coefficient of permeability (or hydraulic conductivity)

The symbols shown here will be consistently used throughout the text and will not be subsequently identified/defined.

The more common laboratory tests also will be briefly commented on. For all laboratory tests we can immediately identify several problems:

1. Recovery of good quality samples. It is not possible to recover samples with zero disturbance, but if the disturbance is a minimum—a relative term—the sample quality may be adequate for the project.
2. Necessity of extrapolating the results from the laboratory tests on a few small samples, which may involve a volume of $\pm 0.03\ \text{m}^3$, to the site, which involves several thousands of cubic meters.
3. Laboratory equipment limitations. The triaxial compression text is considered one of the better test procedures available. It is easy to obtain a sample, put it into the cell, apply some cell pressure, and load the sample to failure in compression. The problem is that the cell pressure, as usually used, applies an even, all around (isotropic) compression. In situ the confining pressure prior to the foundation load application is usually anisotropic (vertical pressure is different from the lateral value). It is not very easy to apply anisotropic confining pressure to soil samples in a triaxial cell—even if we know what to use for vertical and lateral values.
4. Ability and motivation of the laboratory personnel.

The effect of these several items is to produce test results that may not be much refined over values estimated from experience. Items 1 through 3 make field testing a particularly attractive alternative. Field tests will be considered in the next chapter since they tend to be closely associated with the site exploration program.

Index settlement and strength correlations are alternatives that have value in preliminary design studies on project feasibility. Because of both test limitations and costs, it is useful to have relationships between easily determined index properties such as the liquid limit and plasticity index and the design parameters. Several of the more common correlations are presented later in this chapter. Correlations are usually based on a collection of data from an extensive literature survey and used to plot a best-fit curve or to perform a numerical regression analysis.

2-2 FOUNDATION SUBSOILS

We are concerned with placing the foundation on either *soil* or *rock*. This material may be under water as for certain bridge and marine structures, but more commonly we will place the foundation on soil or rock near the ground surface.

Soil is an aggregation of particles that may range very widely in size. It is the by-product of mechanical and chemical weathering of rock. Some of these particles are given specific names according to their sizes, such as gravel, sand, silt, clay, etc., and are more completely described in Sec. 2-7.

Soil, being a mass of irregular-shaped particles of varying sizes, will consist of the particles (or solids), voids (pores or spaces) between particles, water in some of the voids, and air taking up the remaining void space. At temperatures below freezing the pore water may freeze, with resulting particle separation (volume increase). When the ice melts particles close up (volume decrease). If the ice is permanent, the ice-soil mixture is termed *permafrost.* It is evident that the pore water is a variable state quantity that may be in the form of water vapor, water, or ice; the amount depends on climatic conditions, recency of rainfall, or soil location with respect to the GWT of Fig. 1-1.

Soil may be described as *residual* or *transported.* Residual soil is formed from weathering of parent rock at the present location. It usually contains angular rock fragments of varying sizes in the soil-rock interface zone. Transported soils are those formed from rock weathered at one location and transported by wind, water, ice, or gravity to the present site. The terms *residual* and *transported* must be taken in the proper context, for many current residual soils are formed (or are being formed) from transported soil deposits of earlier geological periods, which indurated into rocks. Later uplifts have exposed these rocks to a new onset of weathering. Exposed limestone, sandstone, and shale are typical of indurated transported soil deposits of earlier geological eras that have been uplifted to undergo current weathering and decomposition back to soil to repeat the geological cycle.

Residual soils are usually preferred to support foundations as they tend to have better engineering properties. Soils that have been transported—particularly by wind or water—are often of poor quality. These are typified by small grain size, large amounts of pore space, potential for the presence of large amounts of pore water, and they often are highly compressible. Note, however, exceptions that produce poor-quality residual soils and good-quality transported soil deposits commonly exist. In general, each site must be examined on its own merits.

2-3 SOIL VOLUME AND DENSITY RELATIONSHIPS

The more common soil definitions and gravimetric-volumetric relationships are presented in this section. Figure 2-1 illustrates and defines a number of terms used in these relationships.

Void ratio e. The ratio of the volume of voids V_v to the volume of soils V_s in a given volume of material, usually expressed as a decimal.

$$e = \frac{V_v}{V_s} \qquad 0 < e \ll \infty \tag{2-1}$$

For soils, e ranges from about 0.35 in the most dense state to seldom over 2 in the loosest state.

Porosity n. The ratio of the volume of voids to the total volume V_t, expressed as either a decimal or a percentage.

$$n = \frac{V_v}{V_t} \tag{2-2}$$

Water content w. The ratio of the weight of water W_w to the weight of soil solids W_s, expressed as a percentage but usually used in decimal form.

$$w = \frac{W_w}{W_s} \times 100 \qquad (\%) \tag{2-3}$$

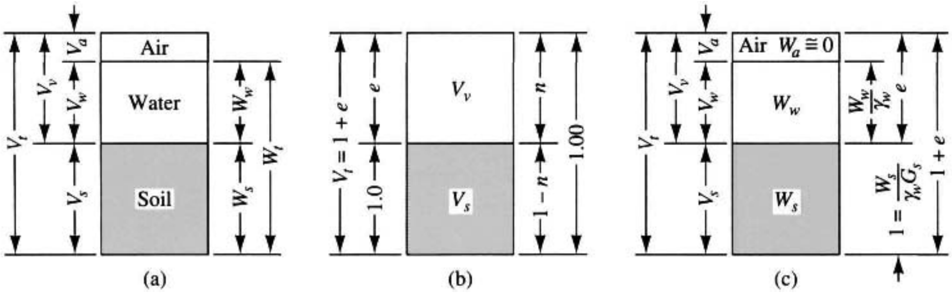

Figure 2-1 Block diagrams showing: (*a*) Weight/volume relationships for a soil mass; (*b*) volume/void relationships; (*c*) volumes expressed in terms of weights and specific gravity.

Unit density (or **mass***)* ρ. The ratio of mass per unit of volume. In the Fps system the values are the same as unit weight following. The SI system gives units of kg/m^3 but a preferred usage unit is g/cm^3. Note that 1 g/cm^3 = 1 Mg/m^3 = 1 tonne/m^3. Often unit density is called "density."

Unit weight γ. The weight of a unit volume of soil (or other material) in force units. The general expression is

$$\gamma = \frac{W_t}{V_t} \tag{2-4}$$

Commonly used units are kN/m^3 or pcf, kcf. The symbol may be subscripted to identify particular state values as $\gamma_{\text{dry}} = W_s/V_t$, etc. The unit weight can vary from a minimum at the dry state to a maximum at the saturated (voids full of water) state for a given particle arrangement.

Degree of saturation S. The ratio of the volume of water to the total volume of soil voids, expressed as a percentage but used as a decimal.

$$S = \frac{V_w}{V_v} \times 100 \qquad (\%) \tag{2-5}$$

A "saturated" soil as obtained from beneath the groundwater table may have a computed S of between 95 and 100 percent.

Specific gravity G. The usual definition for soil is the same as found in most elementary physics textbooks. The unit weight of distilled water is standard at 4°C, but the usual laboratory temperatures in the range of 15 to 25°C do not introduce serious errors. G is usually subscripted to identify the quantity; for soil grains, obtain G_s as

$$G_s = \frac{W_s/V_s}{\gamma_w} = \frac{\gamma_s}{\gamma_w} \tag{2-6}$$

The unit weight of water may be taken as 9.807 kN/m^3, 62.4 pcf, or more commonly as 1 g/cm^3 so that the factor γ_w drops out of the calculations—as long as γ_s is also in units of g/cm^3.

These six basic definitions in equation form are sufficient to develop any needed relationships for geotechnical engineering problems. For example, a useful relationship between void ratio e and porosity n can be obtained from the block diagram of Fig. 2-1*b* as follows:

Let the volume of solids $V_s = 1.00$ (since values are symbolic anyway). This relation gives directly that $e = V_v$ from Eq. (2-1). Placing these values on the left side of the block diagram (as shown) gives the total volume directly as $V_t = 1 + e$. Now using Eq. (2-2), we have

$$n = \frac{V_v}{V_t} = \frac{e}{1+e} \tag{2-7}$$

and, solving Eq. (2-7) for e, we obtain

$$e = \frac{n}{1-n} \tag{2-8}$$

A useful expression for dry unit weight can be obtained similarly by making reference to the block diagram of Fig. 2-1a (right side). By inspection we have $W_t = W_s + W_w$ (the air has negligible weight). From Eq. (2-3) we have $W_w = wW_s$ (where w is in decimal form). Also, dividing W_s and W_t by V_t gives the dry and wet unit weights so

$$\gamma_{\text{dry}} + w\gamma_{\text{dry}} = \gamma_{\text{wet}}$$

which gives

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1+w} \tag{2-9}$$

A useful relation for the void ratio in terms of S, w, and G_s is obtained by using $\gamma_w = 1\text{ g/cm}^3$ as follows:

1. From Eq. (2-6) and referring to the block diagram of Fig. 2-1c, obtain

   $$G_w = \frac{W_w}{V_w\gamma_w}$$

   and because $G_w = \gamma_w = 1$, the weight of water W_w (in grams) $= V_w\gamma_w G_w = V_w$ (in cubic centimeters, cm^3).
2. Let $V_s = 1.0\text{ cm}^3$, and from Eq. (2-1) obtain $V_v = eV_s = e$.
3. From Eq. (2-5) and using S as a decimal, obtain directly

   $$V_w = SV_v$$

   Substitution of W_w for V_w from step 1 and V_v from step 2 gives

   $$W_w = Se$$

4. From Eq. (2-6) obtain the weight of soil solids as

   $$W_s = V_s\gamma_w G_s.$$

   which for $V_s = 1\text{ cm}^3$ gives $W_s = G_s$.
5. From Eq. (2-3) for water content and using above step 3 for W_w and step 4 for W_s, obtain

   $$w = \frac{W_w}{W_s} = \frac{Se}{G_s}$$

6. Solving step 5 for the void ratio e, we obtain

$$e = \frac{w}{S}G_s \tag{2-10}$$

and when $S = 1$ (a saturated soil), we have $e = wG_s$.

The dry unit is often of particular interest. Let us obtain a relationship for it in terms of water content and specific gravity of the soil solids G_s. From Fig. 2-1c the volume of a given mass $V_t = 1 + e$, and with e obtained from Eq. (2-10) we have

$$V_t = 1 + \frac{w}{S}G_s$$

Also, in any system of units the weight of the soil solids is

$$W_s = V_s\gamma_w G_s = \gamma_w G_s \quad \text{when } V_s = 1 \quad \text{as used here}$$

The dry unit weight is

$$\gamma_{\text{dry}} = \frac{W_s}{V_t} = \frac{\gamma_w G_s}{1 + (w/S)G_s} \tag{2-11}$$

and for $S = 100$ percent,

$$\gamma_{\text{dry}} = \frac{\gamma_w G_s}{1 + wG_s} \tag{2-11a}$$

From Eq. (2-9) the wet unit weight is

$$\gamma_{\text{wet}} = \gamma_{\text{dry}}(1 + w)$$

$$\gamma_{\text{wet}} = \frac{\gamma_w G_s(1 + w)}{1 + (w/S)G_s} \tag{2-12}$$

These derivations have been presented to illustrate the use of the basic definitions, together with a basic block diagram on which is placed known (or assumed) values. It is recommended that a derivation of the needed relationship is preferable to making a literature search to find an equation that can be used.

Example 2-1. A cohesive soil specimen (from a split spoon; see Chap. 3 for method) was subjected to laboratory tests to obtain the following data: The moisture content $w = 22.5$ percent; $G_s = 2.60$. To determine the approximate unit weight, a sample weighing 224.0 g was placed in a 500-cm^3 container with 382 cm^3 of water required to fill the container. The reader should note the use of standard laboratory units.

Required.

1. The wet unit weight, γ_{wet}
2. The dry unit weight, γ_{dry}
3. Void ratio e and porosity n

4. Degree of saturation S
5. Dry bulk specific gravity

Solution.

Step 1. The wet unit weight is obtained from total sample weight as

$$\rho_{\text{wet}} = \frac{W_t}{V_t} = \frac{224.0 \text{ g}}{(500 - 382) \text{ cm}^3} = \mathbf{1.898} \text{ g/cm}^3 \qquad \text{(wet density)}$$

and from Sec. 1-7 we have

$$\gamma_{\text{wet}} = 1.898 \times 9.807 = \mathbf{18.61} \text{ kN/m}^3$$

Step 2. The dry unit weight is obtained using Eq. (2-9):

$$\gamma_{\text{dry}} = \frac{18.61}{1.225} = \mathbf{15.19} \text{ kN/m}^3$$

Step 3. The void ratio e and porosity n require some volume computations as follows:

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{1.898/1.225}{2.60(1.0)} = 0.596 \text{ cm}^3 \text{ (or m}^3\text{)}$$

$$V_v = V_t - V_s = 1.000 - 0.596 = 0.404 \text{ cm}^3 \text{ (using cm)}$$

$$e = \frac{V_v}{V_s} = \frac{0.404}{0.596} = \mathbf{0.678}$$

$$n = \frac{V_v}{V_t} = \frac{0.404}{1.00} = \mathbf{0.404} \text{ (or 40.4\%)}$$

Step 4. To find the degree of saturation S it will be necessary to find the volume of water in the voids. The weight of water W_w is the difference between the dry and wet weights; therefore,

$$W_w = 1.898 - \frac{1.898}{1.225} = 0.349 \text{ g (in 1 cm}^3 \text{ of soil)}$$

From Eq. (2-6) for G_w obtain $V_w = W_w$ when using g and cm^3; therefore,

$$S = \frac{V_w}{V_v} \times 100 = \frac{0.349}{0.404} \times 100 = \mathbf{86.4\%}$$

Step 5. The dry bulk specific gravity is obtained as (dimensionless)

$$G_b = \frac{\gamma_{\text{dry}}}{\gamma_w} = \frac{15.19}{9.807} = \mathbf{1.549}$$

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2-4 MAJOR FACTORS THAT AFFECT THE ENGINEERING PROPERTIES OF SOILS

Most factors that affect the engineering properties of soils involve geological processes acting over long time periods. Among the most important are the following.

Natural Cementation and Aging

All soils undergo a natural cementation at the particle contact points. The process of aging seems to increase the cementing effect by a variable amount. This effect was recognized very early in cohesive soils but is now deemed of considerable importance in cohesionless deposits as well. The effect of cementation and aging in sand is not nearly so pronounced as for clay but still the effect as a statistical accumulation from a very large number of grain contacts can be of significance for designing a foundation. Care must be taken to ascertain the quantitative effects properly since sample disturbance and the small relative quantity of grains in a laboratory sample versus site amounts may provide difficulties in making a value measurement that is more than just an estimate. Field observations have well validated the concept of the cementation and aging process. Loess deposits, in particular, illustrate the beneficial effects of the cementation process where vertical banks are readily excavated.

Overconsolidation

A soil is said to be *normally consolidated* (nc) if the current overburden pressure (column of soil overlying the plane of consideration) is the largest to which the mass has ever been subjected. It has been found by experience that prior stresses on a soil element produce an imprint or stress history that is retained by the soil structure until a new stress state exceeds the maximum previous one. The soil is said to be *overconsolidated* (or *preconsolidated*) if the stress history involves a stress state larger than the present overburden pressure.

Overconsolidated cohesive soils have received considerable attention. Only more recently has it been recognized that overconsolidation may be of some importance in cohesionless soils. A part of the problem, of course, is that it is relatively easy to ascertain overconsolidation in cohesive soils but very difficult in cohesionless deposits. The behavior of overconsolidated soils under new loads is different from that of normally consolidated soils, so it is important—particularly for cohesive soils—to be able to recognize the occurrence.

The *overconsolidation ratio* (OCR) is defined as the ratio of the past effective pressure p'_c to the present overburden pressure p'_o:

$$\mathrm{OCR} = \frac{p'_c}{p'_o} \tag{2-13}$$

A **normally consolidated** soil has OCR $= 1$ and an **overconsolidated** soil has OCR > 1. OCR values of 1–3 are obtained for lightly overconsolidated soils. Heavily overconsolidated soils might have OCRs > 6 to 8.

An **underconsolidated** soil will have OCR < 1. In this case the soil is still consolidating. Over- or preconsolidation may be caused by a geologically deposited depth of overburden that has since partially eroded away. Of at least equally common occurrence are preconsolidation effects that result from shrinkage stresses produced by alternating wet and dry cycles. These readily occur in arid and semiarid regions but can occur in more moderate climates as well. Chemical actions from naturally occurring compounds may aid in producing an overconsolidated soil deposit. Where overconsolidation occurs from shrinkage, it is common for only the top 1 to 3 meters to be overconsolidated and the underlying material to be normally consolidated. The OCR grades from a high value at or near the ground surface to 1 at the normally consolidated interface.

Mode of Deposit Formation

Soil deposits that have been transported, particularly via water, tend to be made up of small grain sizes and initially to be somewhat loose with large void ratios. They tend to be fairly uniform in composition but may be stratified with alternating very fine material and thin sand seams, the sand being transported and deposited during high-water periods when stream velocity can support larger grain sizes. These deposits tend to stabilize and may become very compact (dense) over geological periods from subsequent overburden pressure as well as cementing and aging processes.

Soil deposits developed where the transporting agent is a glacier tend to be more varied in composition. These deposits may contain large sand or clay lenses. It is not unusual for glacial deposits to contain considerable amounts of gravel and even suspended boulders. Glacial deposits may have specific names as found in geology textbooks such as moraines, eskers, etc.; however, for foundation work our principal interest is in the uniformity and quality of the deposit. Dense, uniform deposits are usually not troublesome. Deposits with an erratic composition may be satisfactory for use, but soil properties may be very difficult to obtain. Boulders and lenses of widely varying characteristics may cause construction difficulties.

The principal consideration for residual soil deposits is the amount of rainfall that has occurred. Large amounts of surface water tend to leach materials from the upper zones to greater depths. A resulting stratum of fine particles at some depth can affect the strength and settlement characteristics of the site.

Quality of the Clay

The term *clay* is commonly used to describe any cohesive soil deposit with sufficient clay minerals present that drying produces shrinkage with the formation of cracks or fissures such that block slippage can occur. Where drying has produced shrinkage cracks in the deposit we have a *fissured clay.* This material can be troublesome for field sampling because the material may be very hard, and fissures make sample recovery difficult. In laboratory strength tests the fissures can define failure planes and produce fictitiously low strength predictions (alternatively, testing intact pieces produces too high a prediction) compared to in situ tests where size effects may either bridge or confine the discontinuity. A great potential for strength reduction exists during construction where opening an excavation reduces the overburden pressure so that expansion takes place along any fissures. Subsequent rainwater or even local humidity can enter the fissure so that interior as well as surface softening occurs.

A clay without fissures is an *intact clay* and is usually normally consolidated or at least has not been overconsolidated from shrinkage stresses. Although these clays may expand from excavation of overburden, the subsequent access to free water is not so potentially disastrous as for fissured clay because the water effect is more nearly confined to the surface.

Soil Water

Soil water may be a geological phenomenon; however, it can also be as recent as the latest rainfall or broken water pipe. An increase in water content tends to decrease the shear strength of cohesive soils. An increase in the pore pressure in any soil will reduce the shear strength. A sufficient increase can reduce the shear strength to zero—for cohesionless soils the end result is a viscous fluid. A saturated sand in a loose state can, from a sudden shock, also become a viscous fluid. This phenomenon is termed *liquefaction* and is of considerable importance when considering major structures (such as power plants) in earthquake-prone areas.

When soil water just dampens sand, the surface tension produced will allow shallow excavations with vertical sides. If the water evaporates, the sides will collapse; however, construction vibrations can initiate a cave-in prior to complete drying. The sides of a vertical excavation in a cohesive soil may collapse from a combination of rainfall softening the clay together with excess water entering surface tension cracks to create hydrostatic water pressure.

In any case, the shear strength of a cohesive soil can be markedly influenced by water. Even without laboratory equipment, one has probably seen how cohesive soil strength can range from a fluid to a brick-like material as a mudhole alongside a road fills during a rain and subsequently dries. Ground cracks in the hole bottom after drying are shrinkage (or tension) cracks.

Changes in the groundwater table (GWT) may produce undesirable effects—particularly from its lowering. Since water has a buoyant effect on soil as for other materials, lowering the GWT removes this effect and effectively increases the soil weight by that amount. This can produce settlements, for all the underlying soil "sees" is a stress increase from this weight increase. Very large settlements can be produced if the underlying soil has a large void ratio. Pumping water from wells in Mexico City has produced areal settlements of several meters. Pumping water (and oil) in the vicinity of Houston, Texas, has produced areal settlements of more than 2 meters in places. Pumping to dewater a construction site can produce settlements of 30 to 50 mm within short periods of time. If adjacent buildings cannot tolerate this additional settlement, legal problems are certain to follow.

2-5 ROUTINE LABORATORY INDEX SOIL TESTS

Some or all of the following laboratory tests are routinely performed as part of the foundation design process. They are listed in the descending order of likelihood of being performed for a given project.

Water Content w

Water content determinations are made on the recovered soil samples to obtain the natural water content w_N. Liquid (w_L) and plastic (w_P) tests are commonly made on cohesive soils both for classification and for correlation studies. Water content determinations are also commonly made in soil improvement studies (compaction, using admixtures, etc.).

Atterberg Limits

The *liquid* and *plastic limits* are routinely determined for cohesive soils. From these two limits the *plasticity index* is computed as shown on Fig. 2-2*a*. The significance of these three terms is indicated in Fig. 2-2*a* along with the qualitative effect on certain cohesive soil properties of increasing either I_P or w_L. The plasticity index is commonly used in strength correlations; the liquid limit is also used, primarily for consolidation estimates.

The liquid and plastic limit values, together with w_N, are useful in predicting whether a cohesive soil mass is preconsolidated. Since an overconsolidated soil is more dense, the void ratio is smaller than in the soil remolded for the Atterberg limit tests. If the soil is located below the groundwater table (GWT) where it is saturated, one would therefore expect that smaller void ratios would have less water space and the w_N value would be smaller. From this we might deduce the following:

If w_N is close to w_L,	soil is normally consolidated.
If w_N is close to w_P,	soil is some- to heavily overconsolidated.
If w_N is intermediate,	soil is somewhat overconsolidated.
If w_N is greater than w_L,	soil is on verge of being a viscous liquid.

Although the foregoing gives a qualitative indication of overconsolidation, other methods must be used if a quantitative value of OCR is required.

We note that w_N can be larger than w_L, which simply indicates the in situ water content is above the liquid limit. Since the soil is existing in this state, it would seem that overburden pressure and interparticle cementation are providing stability (unless visual inspection indicates a liquid mass). It should be evident, however, that the slightest remolding disturbance has the potential to convert this type of deposit into a viscous fluid. Conversion may be localized, as for pile driving, or involve a large area. The larger w_N is with respect to w_L, the greater the potential for problems. The *liquidity index* has been proposed as a means of quantifying this problem and is defined as

$$I_L = \frac{w_N - w_P}{w_L - w_P} = \frac{w_N - w_P}{I_P} \tag{2-14}$$

where, by inspection, values of $I_L \geq 1$ are indicative of a liquefaction or "quick" potential. Another computed index that is sometimes used is the *relative consistency*,[2] defined as

$$I_C = \frac{w_L - w_N}{I_P} \tag{2-14a}$$

Here it is evident that if the natural water content $w_N \leq w_L$, the relative consistency is $I_C \geq 0$; and if $w_N > w_L$, the relative consistency or consistency index $I_C < 0$.

Where site evidence indicates that the soil may be stable even where $w_N \geq w_L$, other testing may be necessary. For example (and typical of highly conflicting site results reported in geotechnical literature) Ladd and Foott (1974) and Koutsoftas (1980) both noted near-surface marine deposits underlying marsh areas that exhibited large OCRs in the upper zones with w_N near or even exceeding w_L. This is, of course, contradictory to the previously given general statements that if w_N is close to w_L the soil is "normally consolidated" or is about to become a "viscous liquid."

Grain Size

The grain size distribution test is used for soil classification and has value in designing soil filters. A soil filter is used to allow drainage of pore water under a hydraulic gradient with

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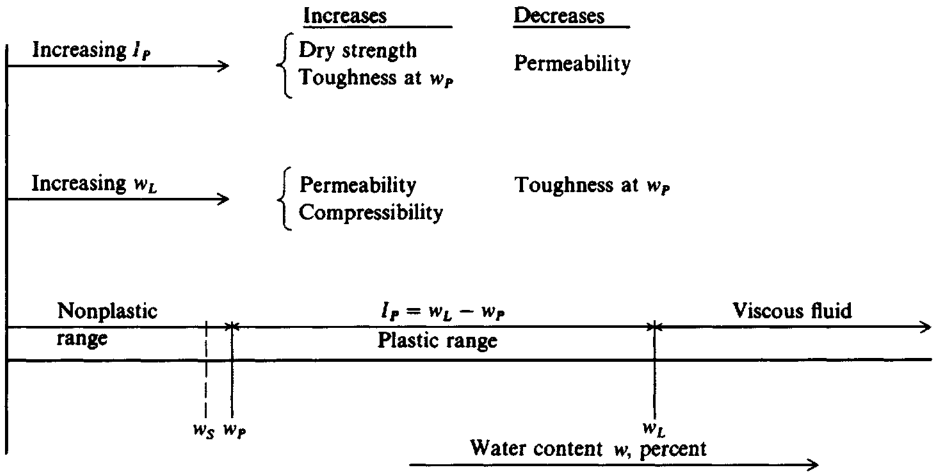
[2] This is the definition given by ASTM D 653, but it is more commonly termed the *consistency index,* particularly outside the United States.



(*a*) Relative location of Atterberg limits on a water content scale. Note that w_S may be to the right of w_P for some soils.

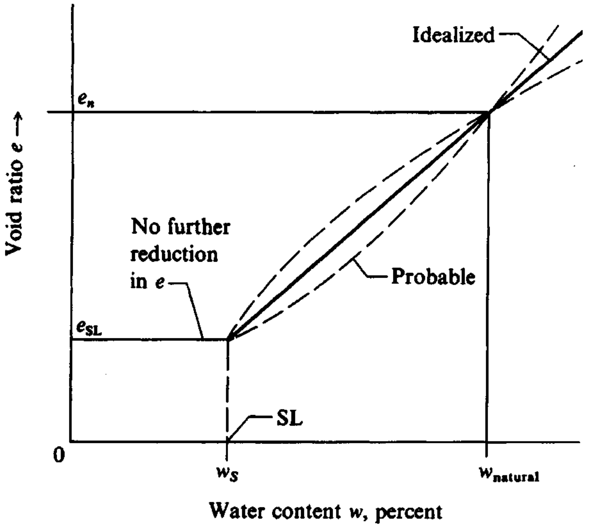


Figure 2-2 The Atterberg limits and some relationships to soil mass properties.

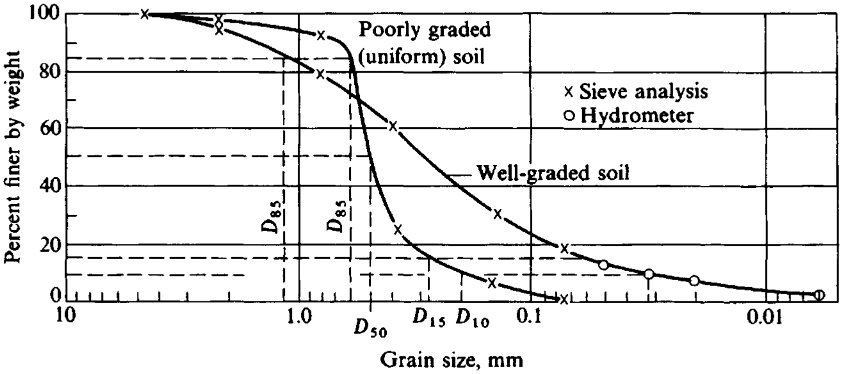
(*b*) Qualitative definition of the shrinkage limit.

erosion of soil fines minimized. Frequently, the grain size test is used to determine the D_{85}, D_{60}, D_{10} fractions (or percents). For example, on Fig. 2-3*a*, *b* the D_{85} (size for which 85 percent of sample is smaller) is about 1.1 mm for the "well-graded" soil. The D_{10} size is about 0.032 mm and was determined from the hydrometer test branch of the curve. The percent clay (particles smaller than 0.002 mm) can be determined from a grain size curve

U.S. as of 1994		British (B.S.)		German DIN		French	
Sieve no.	mm	Sieve no.	mm	Sieve no.	mm	Sieve no.	mm
4	4.76	—		—		—	
10*	2.00	8*	2.057	—		34*	2.000
20	0.841	16	1.003	—		31	1.000
30	0.595	30	0.500	500	0.500	28	0.500
		36†	0.422	400†	0.400	27†	0.400
40†	0.420	—		—		—	
50	0.297	52	0.295	—		—	
60	0.250	60	0.251	250	0.250	25	0.250
80	0.177	85	0.178	160	0.160	23	0.160
100	0.149	100	0.152	125	0.125	22	0.125
200	0.074	200	0.076	80	0.080	20	0.080
270	0.053	300	0.053	50	0.050	18	0.050

* Breakpoint between sand and gravel.
† Use for Atterberg limits.

(*a*)



(*b*)

Figure 2-3 (*a*) Various standard sieve numbers and screen openings; (*b*) grain size distribution curves.

such as this, which uses a combination of sieves and a hydrometer test. Typical sieve sizes as used for sands and silts are shown in Fig. 2-3*a*.

Unit Weight γ

Unit weight γ is fairly easy to estimate for a cohesive soil by trimming a block (or length of a recovered tube sample) to convenient size, weighing it, and then placing it in a volumetric jar and measuring the quantity of water required to fill the container. The unit weight is simply

$$\gamma_{\text{wet}} = \frac{\text{Weight of sample}}{\text{Volume of jar} - \text{volume of water to fill jar}}$$

If the work is done rapidly so that the sample does not have time to absorb any of the added water a very reliable value can be obtained. The average of several trials should be used if possible.

The unit weight of cohesionless samples is very difficult (and costly) to determine. Estimated values as outlined in Chap. 3 are often used. Where more accurate values are necessary, freezing and injection methods are sometimes used; that is, a zone is frozen or injected with a hardening agent so that a somewhat undisturbed block can be removed to be treated similarly as for the cohesive sample above. Where only the unit weight is required, good results can be obtained by recovering a sample with a piston sampler (described in Chap. 3). With a known volume initially recovered, later disturbance is of no consequence, and we have

$$\gamma_{\text{wet}} = \frac{\text{Weight of sample recovered}}{\text{Initial volume of piston sample}}$$

The unit weight is necessary to compute the in situ overburden pressure p_o used to estimate OCR and is necessary in the computation of consolidation settlements of Chap. 5. It is also used to compute lateral pressures against soil-retaining structures and to estimate skin resistance for pile foundations. In cohesionless materials the angle of internal friction ϕ depends on the unit weight and a variation of only 1 or 2 kN/m^3 may have a substantial influence on this parameter.

Relative Density D_r

Relative density is sometimes used to describe the state condition in cohesionless soils. Relative density is defined in terms of natural, maximum, and minimum void ratios e as

$$D_r = \frac{e_{\max} - e_n}{e_{\max} - e_{\min}} \tag{2-15}$$

It can also be defined in terms of natural (in situ), maximum, and minimum unit weight γ as

$$D_r = \left(\frac{\gamma_n - \gamma_{\min}}{\gamma_{\max} - \gamma_{\min}}\right)\left(\frac{\gamma_{\max}}{\gamma_n}\right) \tag{2-16}$$

The relative density test can be made on gravelly soils if the (−) No. 200 sieve (0.074 mm) material is less than 8 percent and for sandy soils if the fines are not more than about 12 percent according to Holtz (1973).

The relative density D_r is commonly used to identify potential liquefaction under earthquake or other shock-type loadings [Seed and Idriss (1971)]; however, at present a somewhat more direct procedure is used [Seed et al. (1985)]. It may also be used to estimate strength (Fig. 2-30).

It is the author's opinion that the D_r test is not of much value since it is difficult to obtain maximum and minimum unit weight values within a range of about ± 0.5 kN/m^3. The average maximum value is about this amount under (say 20.0 kN/m^3 − 0.5) and the minimum about this over (say, 15.0 kN/m^3 + 0.5). The definition is for the maximum and minimum values, but average values are usually used. This value range together with the uncertainty in obtaining the in situ value can give a potential range in computed D_r of up to 30 to 40 percent (0.3 to 0.4). Chapter 3 gives the common methods of estimating the in situ value of D_r. A simple laboratory procedure is given in Bowles (1992) (experiment 18) either to compute D_r or to obtain a unit weight for quality control.

Specific Gravity G_s

The specific gravity of the soil grains is of some value in computing the void ratio when the unit weight and water content are known. The test is of moderate difficulty with the major source of error deriving from the presence of entrapped air in the soil sample. Since G_s does not vary widely for most soils, the values indicated here are commonly estimated without performing a test.

Soil	G_s
Gravel	2.65–2.68
Sand	2.65–2.68
Silt, inorganic	2.62–2.68
Clay, organic	2.58–2.65
Clay, inorganic	2.68–2.75

A value of $G_s = 2.67$ is commonly used for cohesionless soils and a value of 2.70 for inorganic clay. Where any uncertainty exists of a reliable value of G_s, one should perform a test on a minimum of three small representative samples and average the results. Values of G_s as high as 3.0 and as low as 2.3 to 2.4 are not uncommon.

Shrinkage Limit w_S

This is one of the Atterberg limit tests that is sometimes done. The shrinkage limit is qualitatively illustrated in Fig. 2-2*b*. It has some value in estimating the probability of expansive soil problems. Whereas a low value of w_S indicates that only a little increase in water content can start a volume change, the test does not quantify the amount of ΔV. The problem of making some kind of estimate of the amount of soil expansion is considered in Sec. 7-9.4.

2-6 SOIL CLASSIFICATION METHODS IN FOUNDATION DESIGN

It is necessary for the foundation engineer to classify the site soils for use as a foundation for several reasons:

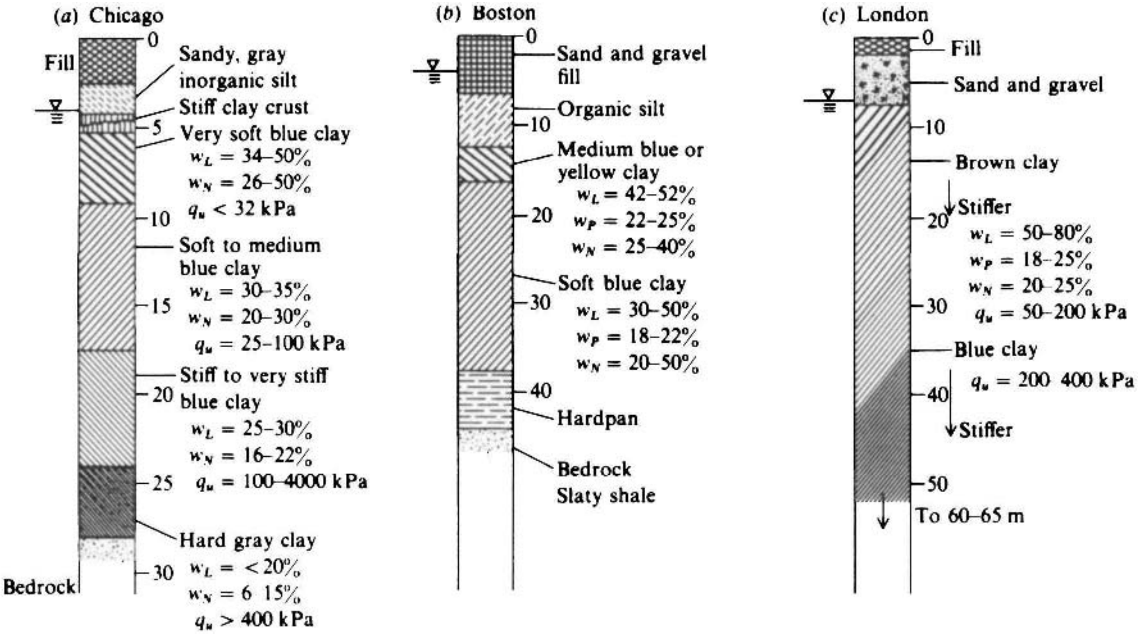
1. To be able to use the database of others in predicting foundation performance.
2. To build one's own local database of successes (or any failures).
3. To maintain a permanent record that can be understood by others should problems later develop and outside parties be required to investigate the original design.
4. To be able to contribute to the general body of knowledge in common terminology via journal papers or conference presentations. After all, if one is to partake in the contributions of others, one should be making contributions to the general knowledge base and not be just a "taker."

The Unified Soil Classification System (USCS) of Table 2-1 is much used in foundation work. A version of this system has been standardized by ASTM as D 2487 (in Volume 04.08: *Soil and Rock; Dimension Stone; Geosynthetics*). The standardized version is similar to the original USCS as given by Casagrande (1948) but with specified percentages of sand or gravel passing specific sieves being used to give the "visual description" of the soil. The original Casagrande USCS only classified the soil using the symbols shown in Table 2-1 (GP, GW, SM, SP, CL, CH, etc.), based on the indicated percentages passing the No. 4 and No. 200 sieves and the plasticity data. The author has always suggested a visual description supplement such as the following:

Soil data available	Soil description (using Table 2-1)
Sand, $C_u = 7$; $C_c = 1.3$, 95% passing No. 4 sieve, brown color	Well-graded, brown sand with a trace of gravel, SW
Gravel, 45% passes No. 4, 25% passes No. 200; $w_L = 42$, $w_P = 22$, tan color	Tan clayey gravel with sand, GC
70% passes No. 4 and 18% passes No. 200 sieve; $w_L = 56$; $w_P = 24$. Sample is firm and dark in color with a distinct odor	Organic gravelly, clayey sand, SC

It is evident in this table that terms "trace" and "with" are somewhat subjective. The soil color, such as "blue clay," "gray clay," etc., is particularly useful in soil classification. In many areas the color—particularly of cohesive soils—is an indication of the presence of the

Figure 2-4 Typical soil profiles at locations indicated. Values for soil properties indicate order of magnitude; they should not be used for design. Depths shown are in meters.



same soil stratum as found elsewhere. For example the "soft blue clay" on the soil profile of Fig. 2-4 for Chicago has about the same properties at any site in the Chicago area.

In foundation work the terms *loose, medium,* and *dense,* as shown in Table 3-4, and consistency descriptions such as *soft, stiff, very stiff,* etc., as shown in Table 3-5, are also commonly used in foundation soil classification. Clearly, all of these descriptive terms are of great use to the local geotechnical engineer but are somewhat subjective. That is, there could easily be some debate over what is a "medium" versus a "dense" sand, for example.

The D 2487 standard removed some of the subjectiveness of the classification and requires the following terminology:

$< 15\%$ is sand or gravel	use name (organic clay, silt, etc.)
$15\% < x < 30\%$ is sand or gravel	describe as clay or silt with sand, or clay or silt with gravel
$> 30\%$ is sand or gravel	describe as sandy clay, silty clay, or gravelly clay, gravelly silt

The gravel or sand classification is based on the percentage retained on the No. 4 (gravel) sieve or passing the No. 4 and retained on the No. 200 (sand) sieves. This explanation is only partial, as the new standard is too lengthy for the purpose of this textbook to be presented in detail.

Although not stated in D 2487, the standard is devised for using a computer program[3] to classify the soil. Further, not all geotechnical engineers directly use the ASTM standard, particularly if their practice has a history of success using the original USC system.

General Comments on Using Table 2-1

1. When the w_L-I_P intersection is very close to the "A" or $w_L = 50\%$ line, use dual symbols such as SC-SM, CL-ML, organic OL-OH, etc. to indicate the soil is borderline.
2. If the w_L-I_P intersection is above the "U" line one should carefully check that the tests and data reduction are correctly done. It may require redoing the limits tests as a check. The reason for this caution is that this line represents the upper limit of real soils so far analyzed.

Peat and Organic Soils

Strictly, *peat* is not a soil but rather an organic deposit of rotting wood from trees, plants, and mosses. If the deposit is primarily composed of moss, it may be termed a sphagnum peat. If the deposit has been somewhat contaminated with soil particles (silt, clay, sand) it may be named for the soil particles present as peaty silt, peaty sand, peaty clay, and so on. If the soil contamination is substantial (in a relative sense) the soil is more likely to be termed an

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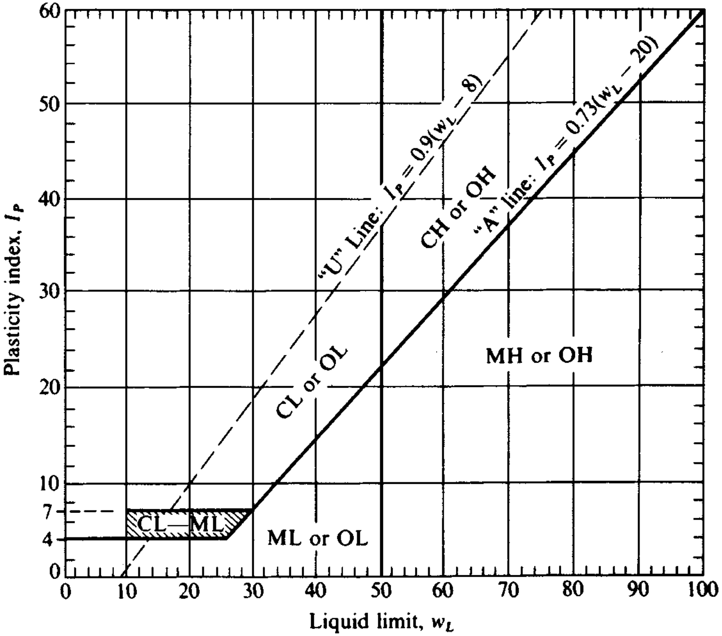
[3] A compiled computer program for use with D 2487 (along with several others) is available with the laboratory text *Engineering Properties of Soils and Their Measurement,* 4th ed., (1992), McGraw-Hill Book Company, New York, NY 10020; Tel.: (212) 512-2012.

TABLE 2-1

Unified soil classification [Casagrande (1948)]

Major divisions			Group symbols		Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentages of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC 5 to 12 % *Borderline* cases requiring dual symbols†	$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 4; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{10}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting C_u or C_c requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM*	*d*	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or I_P less than 4	Limits plotting in hatched zone with I_P between 4 and 7 are *borderline* cases requiring use of dual symbols.
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with I_P greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 6; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting C_u or C_c requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM*	*d*	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or I_P less than 4	Limits plotting in hatched zone with I_P between 4 and 7 are *borderline* cases requiring use of dual symbols.
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with I_P greater than 7	

Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50%)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	For all soils plotting nearly on "A" line use dual symbols, i.e., $I_P = 29.5$, $w_L = 60$ gives CH-OH or CH-MH. When w_L is near 50 use CL/CH, ML/MH. Take "nearly on" as ±2 percent.  **A-Chart**
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
	Silts and clays (Liquid limit greater than 50%)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		CH	Inorganic clays of high plasticity, fat clays	
		OH	Organic clays of medium to high plasticity, organic silts	
	Highly organic soils	Pt	Peat and other highly organic soils	

---

*Division of GM and SM groups into subdivisions of *d* and *u* are for roads and airfields only. Subdivision is based on Atterberg limits; suffix *d* used when w_L is 28 or less and the I_P is 6 or less; suffix *u* used when w_L is greater than 28.

†Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

organic soil. Generally a "peat" deposit is classified as such from visual inspection of the recovered samples.

There have been a number of attempts to quantify various engineering properties of peat (or peaty) deposits; however, it is usually necessary to consider the properties of each site. Several engineering properties such as unit weight, compressibility, and permeability will be heavily dependent on the type, relative quantity, and degree of decomposition (state) of the organic material present. Several recent references have attempted to address some of these problems:

Landva and Pheeney (1980)
Berry and Vickers (1975)
Edil and Dhowian (1981)
Lo et al. (1990)
Fox et al. (1992)
Stinnette (1992)

Organic soils are defined as soil deposits that contain a mixture of soil particles and organic (peat) matter. They may be identified by observation of peat-type materials, a dark color, and/or a woody odor. ASTM (D 2487 Section 11.3.2) currently suggests that the organic classification (OL, OH shown on the "A" chart of Table 2-1) be obtained by performing the liquid limit on the natural soil, then oven-drying the sample overnight and performing a second liquid limit test on the oven-dry material. If the liquid limit test after oven drying is less than 75 percent of that obtained from the undried soil, the soil is "organic." Oven drying of organic soils requires special procedures as given in ASTM D 2974.

After performing the liquid and plastic limits, one classifies an organic soil using the "A" chart of Table 2-1. The soil may be either an organic silt OL, OH, or an organic clay OL, OH depending on the liquid limit w_P and plasticity index I_P and where these values plot on the "A" chart. It is necessary to use both the qualifier "organic silt" or "organic clay" **and** the symbol OL or OH.

Approximate Field Procedures for Soil Identification

It is sometimes useful to be able to make a rapid field identification of the site soil for some purpose. This can be done approximately as follows:

1. Differentiate gravel and sand by visual inspection.
2. Differentiate fine sand and silt by placing a spoonful of the soil in a deep jar (or test tube) and shaking it to make a suspension. Sand settles out in $1\frac{1}{2}$ minutes or less whereas silt may take 5 or more minutes. This test may also be used for clay, which takes usually more than 10 minutes. The relative quantities of materials can be obtained by observing the depths of the several materials in the bottom sediment.
3. Differentiate between silt and clay as follows:
   a. Clay lumps are more difficult to crush using the fingers than silt.
   b. Moisten a spot on the soil lump and rub your finger across it. If it is smooth it is clay; if marginally streaked it is clay with silt; if rough it is silt.

c. Form a plastic ball of the soil material and shake it horizontally by jarring your hand. If the material becomes shiny from water coming to the surface it is silt.

4. Differentiate between organic and inorganic soils by visual inspection for organic material or a smell test for wood or plant decay odor.

2-7 SOIL MATERIAL CLASSIFICATION TERMS

The soil classification terms shown in Table 2-1 are widely used in classification. A number of other terms are used both by engineers and construction personnel, or tend to be localized. A few of these terms will be defined here as a reader convenience.

Bedrock

This is a common name for the parent rock, but generally implies a rock formation at a depth in the ground on which a structure may be founded. All other rocks and soils are derived from the original bedrock formed from cooling of molten magma and subsequent weathering. Bedrock extends substantially downward to molten magma and laterally in substantial dimensions. The lowermost part is igneous rock formed by cooling of the molten magma. This may, or may not, be overlain by one or more layers of more recently formed sedimentary rocks such as sandstone, limestone, shale, etc. formed from indurated soil deposits. The interface layers between igneous and sedimentary rocks may be metamorphic rocks formed from intense heat and pressure acting on the sedimentary rocks. In some cases a bedding rock layer—usually sedimentary in origin—may overlie a soil deposit. In earthquake areas the parent rock may be much fractured. Past areal uplifts may have produced zones of highly fragmented parent rock at the bedrock level.

Considering these factors, one might say that generally, bedrock makes a satisfactory foundation, but good engineering practice requires that one check the geological history of the site. In this context it is fairly common to refer to the bedrock with respect to the geological age of estimated formation as Cambrian, pre-Cambrian, etc.

Boulders

Boulders are large pieces of rock fractured from the parent material or blown out of volcanos (called bombs in this case). They may have volumes ranging from about $\frac{1}{2}$ to 8 or 10 m^3 and weigh from about one-half to several hundred tonnes. They may create disposal or excavation problems on or near the ground surface and problems in soil exploration or pile driving at greater depths when suspended in the soil matrix, as in glacial till. Large ones may be suitable to found pile or caissons on; however, size determination may be difficult, and placing a large load on a small suspended boulder may be disastrous.

Gravels and Smaller

Rock fragments smaller than boulders grade into cobbles, pebbles, gravel, sand, silt, and colloids in order of size as shown on Table 2-2. *Crushed stone* is gravel manufactured by crushing rock fragments from boulders or obtained from suitable rock formations by mining. *Bank-run gravel* is a common term for naturally occurring gravel lenses deposited along

TABLE 2-2
Usual size range for general soil classification terminology

Material	Upper, mm	Lower, mm	Comments
Boulders, cobbles	1000 $^+$	75 $^-$	
Gravel, pebbles	75	2 – 5	No. 4 or larger sieve
Sand	2 – 5	0.074	No. 4 to No. 200 sieve
Silt	0.074 – 0.05	0.006	Inert
Rock flour	0.006	?	Inert
Clay	0.002	0.001	Particle attraction, water absorption
Colloids	0.001	?	

rivers or from glaciers. *Pea gravel* is gravel screened to contain only sizes in a certain range (usually about 6 down to 3 mm) and is poorly graded because the > 6 mm and < 3 mm sizes are missing.

Gravels, sands, and silts are cohesionless materials that exist in deposits ranging from a state of loose to dense and coarse to fine. Most deposits, however, are in a medium to fairly dense state. These materials can have cohesion from clay minerals in the fine sand and silt filler that may be present.

Silt

Silts and clays are of particular interest in foundation engineering because they tend to be most troublesome in terms of strength and settlements. Silts and rock flour in the particle range of 0.074 mm down to about 0.001 mm are inert by-products of rock weathering. They may be *organic silts* (OL, OH) if contaminated with organic materials or inorganic (ML, MH) otherwise. Damp silt has an apparent cohesion from the cumulative effect of surface tension on the many small particles, but on drying minimal shrinkage (unless organic) takes place and the resulting dry lumps are easily broken by finger pressure.

Most silt deposits, however, are contaminated with clay minerals so that they have cohesion (dry lumps are not so easily broken). As little as 5 to 8 percent clay can give a silt deposit considerable cohesion, depending on the silt grain sizes and the type of clay mineral. At higher percentages of clay, or depending on its visual effects, a silt deposit may be loosely termed "clay," particularly by construction personnel. From an engineering viewpoint, however, we can see from the "A" chart of Table 2-1 that it is quite possible for a "clay" to have lower plasticity characteristics than a silt, i.e., a CL of say $w_L = 35$ and $I_P = 15$ versus a MH of $w_L = 60$ and $I_P = 25$.

Clay

The clay size (particles 0.002 mm and smaller) overlaps the silt sizes somewhat. The essential difference between the two, however, is that a clay mineral is not inert. It is a complex hydro-aluminum silicate,

$$Al_2O_3 \cdot nSiO_2 \cdot kH_2O$$

where n and k are numerical values of attached molecules and vary for the same mass. The clay mineral has a high affinity for water, and individual particles may absorb 100^+ times the particle volume. The presence or absence (during drying) of water can produce very large volume and strength changes. Clay particles also have very strong interparticle attractive forces, which account in part for the very high strength of a dry lump (or a clay brick). Water absorption and interparticle attraction collectively give the *activity* and cohesion to clay (and to soils containing clay minerals).

The three principal identified clay minerals can be characterized in terms of activity and plasticity:

Montmorillonite (or *smectite*)—Most active of the identified minerals. The activity, in terms of affinity for water and swell, makes this material ideal for use as a drilling mud in soil exploration and in drilling oil wells. It is also commonly injected into the ground around basement walls as a water barrier (swells to close off water flow paths) to stop basement leaks. It is also blended with local site material to produce water barriers to protect the GWT from sanitary landfill drainage. The I_P of an uncontaminated montmorillonite is 150^+.

Illite—A clay mineral that is intermediate in terms of activity. The I_P of a pure illite ranges from about 30 to 50.

Kaolinite—The clay mineral with the least activity. This material is commonly used in the ceramic industry and for brick manufacture. It is sometimes used as an absorbent for stomach medicine. The I_P of a pure kaolinite ranges from about 15 to 20.

Montmorillonite deposits are found mostly in arid and semiarid regions. All clay minerals weather into less active materials, e.g., to illite and then to kaolinite. As a consequence most "clay" deposits contain several different clay minerals. Only deposits of relatively pure clay have commercial value. Most of the remainder represent engineering problems. For example, in temperate regions it is not unusual for deposits to contain substantial amounts of montmorillonite or even lenses of nearly pure material.

Clay deposits with certain characteristics are common to certain areas and have been named for the location. For example the "Chicago blue clay," "Boston blue clay," "London clay" shown in Fig. 2-4 are common for those areas. Leda clay is found in large areas of Ottawa Province in Canada and has been extensively studied and reported in the *Canadian Geotechnical Journal.*

Local Terminology

The following are terms describing soil deposits that the geotechnical engineer may encounter. Familiarity with their meaning is useful.

a. *Adobe.* A clayey material found notably in the Southwest.
b. *Caliche.* A conglomeration of sand, gravel, silt, and clay bonded by carbonates and usually found in arid areas.
c. *Glacial till* or *glacial drift.* A mixture of material that may include sand, gravel, silt, and clay deposited by glacial action. Large areas of central North America, much of Canada, northern Europe, the Scandinavian countries, and the British Isles are overlain with glacial

till or drift. The term *drift* is usually used to describe any materials laid down by the glacier. The term *till* is usually used to describe materials precipitated out of the ice, but the user must check the context of usage, as the terms are used interchangeably. Moraines are glacial deposits scraped or pushed ahead (terminal), or alongside the glacier (lateral). These deposits may also be called ground moraines if formed by seasonal advances and retreats of a glacier. The Chicago, Illinois, area, for example, is underlain by three identifiable ground moraines.

d. *Gumbo.* A clayey or loamy material that is very sticky when wet.

e. *Hardpan.* This term may be used to describe caliche or any other dense, firm deposits that are excavated with difficulty.

f. *Loam.* A mixture of sand, clay, silt; an organic material; also called *topsoil.*

g. *Loess.* A uniform deposit of silt-sized material formed by wind action. Often found along the Mississippi River, where rising damp air affects the density of the air transporting the material, causing it to deposit out. Such deposits are not, however, confined to the Mississippi Valley. Large areas of Nebraska, Iowa, Illinois, and Indiana are covered by loess. Large areas of China, Siberia, and southeastern Europe (southern Russia and Ukraine) and some areas of western Europe are covered with loess. Loess is considered to be a *transported* soil.

h. *Muck.* A thin watery mixture of soil and organic material.

i. *Alluvial deposits.* Soil deposits formed by sedimentation of soil particles from flowing water; may be lake deposits if found in lake beds; deltas at the mouths of rivers; marine deposits if deposited through saltwater along and on the continental shelf. Alluvial deposits are found worldwide. For example, New Orleans, Louisiana, is located on a delta deposit. The low countries of The Netherlands and Belgium are founded on alluvial deposits from the Rhine River exiting into the North Sea. Lake deposits are found around and beneath the Great Lakes area of the United States and Canada. Large areas of the Atlantic coastal plain, including the eastern parts of Maryland, Virginia, the Carolinas, the eastern part and most of south Georgia, Florida, south Alabama, Mississippi, Louisiana, and Texas consist of alluvial deposits. These deposits formed when much of this land was covered with the seas. Later upheavals such as that forming the Appalachian mountains have exposed this material. Alluvial deposits are fine-grained materials, generally silt-clay mixtures, silts, or clays and fine to medium sands. If the sand and clay layers alternate, the deposit is a *varved* clay. Alluvial deposits are usually soft and highly compressible.

j. *Black cotton soils.* Semitropical soils found in areas where the annual rainfall is 500 to 750 mm. They range from black to dark gray. They tend to become hard with very large cracks (large-volume-change soils) when dry and very soft and spongy when wet. These soils are found in large areas of Australia, India, and southeast Asia.

k. *Laterites.* Another name for residual soils found in tropical areas with heavy rainfalls. These soils are typically bright red to reddish brown in color. They are formed initially by weathering of igneous rocks, with the subsequent leaching and chemical erosion due to the high temperature and rainfall. Collodial silica is leached downward, leaving behind aluminum and iron. The latter becomes highly oxidized, and both are relatively insoluble in the high-pH environment (greater than 7). Well-developed laterite soils are generally porous and relatively incompressible. Lateritic soils are found in Alabama, Georgia, South

Carolina, many of the Caribbean islands, large areas of Central and South America, and parts of India, southeast Asia, and Africa.

l. Saprolite. Still another name for residual soils formed from weathered rock. These deposits are often characterized by a particle range from dust to large angular stones. Check the context of use to see if the term is being used to describe laterite soils or residual soils.

m. Shale. A fine-grained, sedimentary rock composed essentially of compressed and/or cemented clay particles. It is usually laminated, owing to the general parallel orientation of the clay particles, as distinct from claystone or siltstone, which are indurated deposits of random particle orientation. According to Underwood (1967), shale is the predominant sedimentary rock in the Earth's crust. It is often misclassified; layered sedimentary rocks of quartz or argillaceous materials such as argillite are not shale. Shale may be grouped as (1) compaction shale and (2) cemented (rock) shale. The compaction shale is a transition material from soil to rock and can be excavated with modern earth excavation equipment. Cemented shale can sometimes be excavated with excavation equipment but more generally requires blasting. Compaction shales have been formed by consolidation pressure and very little cementing action. Cemented shales are formed by a combination of cementing and consolidation pressure. They tend to ring when struck by a hammer, do not slake in water, and have the general characteristics of good rock. Compaction shales, being of an intermediate quality, will generally soften and expand upon exposure to weathering when excavations are opened. Shales may be clayey, silty, or sandy if the composition is predominantly clay, silt, or sand, respectively. Dry unit weight of shale may range from about 12.5 kN/m^3 for poor-quality compaction shale to 25.1 kN/m^3 for high-quality cemented shale.

2-8 IN SITU STRESSES AND K_o CONDITIONS

Any new foundation load—either an increase (+) from a foundation or a decrease (−) from an excavation—imposes new stresses on the existing state of "locked in" stresses in the foundation soil mass. The mass response is heavily dependent on the previous stress history, so one of the most important considerations in foundation engineering is to ascertain this stress imprint. The term *imprint* is used since any previously applied stresses that are larger than those currently existing have been locked into the soil structure and will affect subsequent stress-response behavior until a new set of larger stresses are applied to produce a new imprint. Of course, the stress history is lost in varying degrees (or completely) when the soil is excavated/remolded or otherwise disturbed as in sample recovery. Factors contributing to loss of stress history during sampling are outlined in Sec. 3-5.

In situ, the vertical stresses act on a horizontal plane at some depth z. These can be computed in any general case as the sum of contributions from n strata of unit weight γ_i and thickness z_i as

$$p_o = \sum_{i=1}^{n} \gamma_i z_i \tag{a}$$

The unit weight for a homogeneous stratum is of the general form

$$\gamma = A_1 + A_2 z^m \tag{b}$$

with the constants A_1, A_2, and m determined by obtaining weight values at several depths z and plotting a best-fit curve. In practice, at least for reasonable depths on the order of 5 to 10 meters, a constant value is often (incorrectly) used. An alternative is to divide the deposit into several "layers" and use a constant unit weight γ_i for each as in Eq. (*a*).

In most cases involving geotechnical work, the *effective stress* p'_o is required so that below the GWT one uses the effective soil unit weight computed as

$$\gamma' = \gamma_{\text{sat}} - \gamma_w \tag{c}$$

For any soil deposit formation the plan area is usually rather large and the depth continually increases until either deposition or interior weathering stops. This change produces a gradual vertical compression of the soil at any given depth; similarly, γ increases under compression so that in nearly all cases unit weight $\gamma = f$ (depth). Since the lateral dimension is large there is little reason for significant lateral compression to occur. For this reason it is logical to expect that vertical locked-in *effective stresses* p'_o would be larger than the effective lateral stresses σ'_h at the same point. We may define the ratio of the horizontal to vertical stresses as

$$K = \frac{\sigma_h}{p_o} \tag{d}$$

which is valid for any depth z at any time.

Over geological time the stresses in a soil mass at a particular level stabilize into a steady state and strains become zero. When this occurs the vertical and lateral stresses become principal stresses acting on principal planes.[4] This effective stress state is termed the *at-rest* or K_o condition with K_o defined as

$$K_o = \frac{\sigma'_h}{p'_o} \tag{2-17}$$

Figure 2-5 qualitatively illustrates the range of K_o and the relationship of p_o and σ_h in any homogeneous soil. Note the qualitative curves for preconsolidation in the upper zone of some soil from shrinkage/chemical effects. This figure (see also Fig. 2-45) clearly illustrates the anisotropic ($\sigma_v \neq \sigma_h$) stress state in a soil mass.

Because of the sampling limitations given in Sec. 3-5 it is an extremely difficult task to measure K_o either in the laboratory or in situ. A number of laboratory and field methods are cited by Abdelhamid and Krizek (1976); however, from practical limitations the direct simple shear device (Fig. 2-26*b*) is the simplest for direct laboratory measurements. Field methods will be considered in the next chapter, but note that they are very costly for the slight improvement—in most cases—over using one of the simple estimates following. In these equations use the effective angle of internal friction ϕ' and not the total stress value.

Jaky (1948) presented a derived equation for K_o that is applicable to both soil and agricultural grains (such as corn, wheat, oats, etc.) as

$$K_o = \frac{1 - \sin\phi'}{1 + \sin\phi'}\left(1 + \frac{2}{3}\sin\phi'\right) \tag{2-18}$$

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[4] Stresses acting on planes on which no strains or shearing stresses exist are defined as principal stresses, and the planes are principal planes.

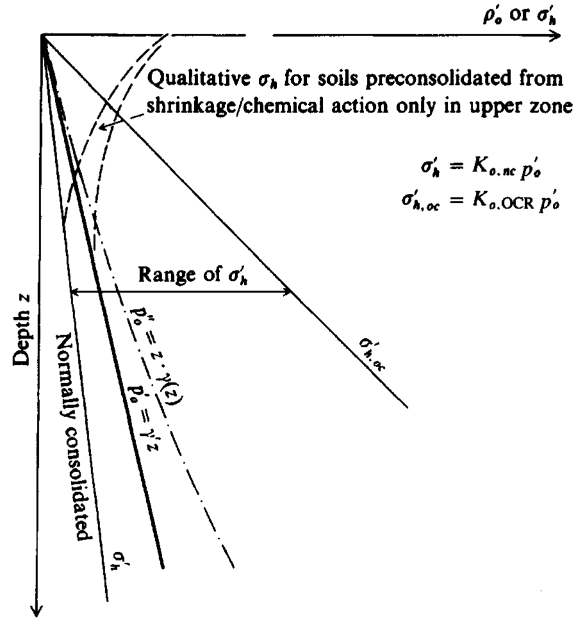

Figure 2-5 Qualitative vertical and lateral pressures in a soil. Although the linear vertical (also called geostatic) pressure profile is commonly used, the p''_o effective pressure profile is more realistic of real soils since γ usually increases with depth. The lateral pressure profile range is for the geostatic pressure profile and would be curved similarly to the p''_o curve for real soils.

which has been simplified—and erroneously called “Jaky’s equation”—to the following:

$$K_o = 1 - \sin\phi' \tag{2-18a}$$

This equation is very widely used and has proved reasonably reliable [see extensive regression analysis by Mayne and Kulhawy (1982)] in comparing initial to back-computed K_o values in a number of cases and for normally consolidated materials. Kezdi (1972) suggests that for sloping ground Jaky’s equation can be used as follows:

$$K_o = \frac{1 - \sin\phi'}{1 + \sin\beta} \tag{2-19}$$

where β is the angle with the horizontal with sign so that K_o is either increased or reduced as site conditions dictate. This reference also gives a partial derivation of the Jaky equation for any interested user.

Brooker and Ireland (1965) (for normally consolidated clay) suggest

$$K_o = 0.95 - \sin\phi' \tag{2-20}$$

Alpan (1967) (for normally consolidated clay) suggests

$$K_o = 0.19 + 0.233 \log_{10} I_P \tag{2-21}$$

An equation similar to Eq. (2-21) is given by Holtz and Kovacs (1981, on Fig. 11.69) as

$$K_o = 0.44 + 0.0042 I_P \tag{2-21a}$$

where I_P is in percent for both Eqs. (2-21).

We can readily derive a value for K_o in terms of Poisson’s ratio based on the definition of K_o being an effective stress state at zero strain. From Hooke’s law [Eq. (2-64)] the lateral strain in terms of the effective horizontal (x, z) and vertical (y) stresses is

$$\epsilon_x = 0 = \frac{1}{E_s}(\sigma_x - \mu\sigma_y - \mu\sigma_z) = \epsilon_z$$

With $\sigma_x = \sigma_z = K_o\sigma_y$ we obtain, on substitution into the preceding and canceling,

$$K_o = \frac{\mu}{1-\mu} \tag{2-22}$$

For a cohesionless soil μ is often assumed as 0.3 to 0.4, which gives $K_o = 0.43$ to 0.67, with a value of 0.5 often used.

It is extremely difficult to obtain a reliable estimate of K_o in a normally consolidated soil, and even more so in overconsolidated soils (OCR > 1). A number of empirical equations based on various correlations have been given in the literature [see the large number with cited references given by Mesri and Hayat (1993)]. Several of the more promising ones are:

Alpan (1967) and others have suggested that the overconsolidated consolidation ratio $K_{o,\text{OCR}}$ is related to the normally consolidated value $K_{o,\text{nc}}$ in the following form

$$K_{o,\text{OCR}} = K_{o,\text{nc}} \times \text{OCR}^n \tag{2-23}$$

where $n = f(\text{test, soil, locale})$ with a value range from about 0.25 to 1.25. For *overconsolidated sand*, n can be estimated from Fig. 2-6. For *cohesive* soil, Wroth and Houlsby (1985, p. 12) suggest n as follows:

$$n = 0.42 \text{ (low plasticity—}I_P < 40\%)$$
$$n = 0.32 \text{ (high plasticity—}I_P > 40\%)$$

However, $n \approx 0.95$ to 0.98 was obtained from in situ tests on several clays in eastern Canada [Hamouche et al. (1995)].

Mayne and Kulhawy (1982) suggest that a *mean* value of $n = 0.5$ is applicable for both sands and clays and that $n = \sin\phi'$ is also a good representation for sand. Their sugges-

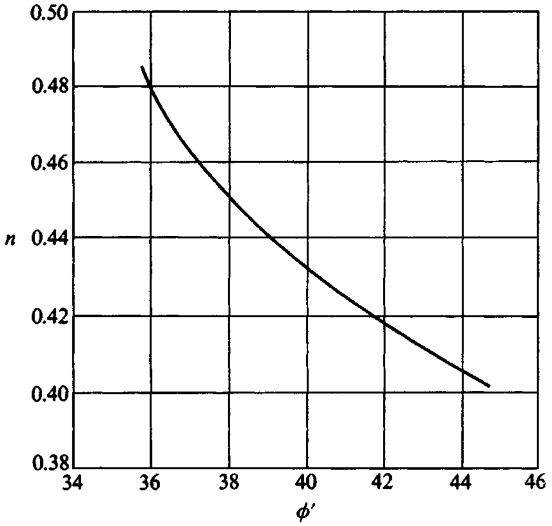

Figure 2-6 Exponent n for sands. [*After Alpan (1967).*]

tions are based on a semi-statistical analysis of a very large number of soils reported in the literature.

The exponent n for *clays* was also given by Alpan (1967) in graph format and uses the plasticity index I_P (in percent). The author modified the equation shown on that graph to obtain

$$n = 0.54 \times 10^{-I_P/281} \tag{2-24}$$

And, as previously suggested (for sands), we can use

$$n = \sin \phi' \tag{2-24a}$$

The n-values previously given by Wroth and Houlsby (1985) can be obtained from Eq. (2-24) using an average "low plasticity" I_P of about 30 ($n = 0.42$) and a "high plasticity" I_P of about 65 percent ($n = 0.32$).

Mayne (1984) suggests that the range of valid values for the overconsolidated $K_{o,\mathrm{OCR}}$ using Eq. (2-23) for cohesive soils depends on the normalized strength ratio s_u/p'_c being less than 4—at least for noncemented and intact clays. Therefore, this ratio is indirectly used for Eq. (2-23), but it will be directly used in the following section.

2-8.1 Overconsolidated K_o Conditions

The equation for the overconsolidation ratio (OCR) was given in Sec. 2-4, and it is repeated here for convenience:

$$\mathrm{OCR} = \frac{p'_c}{p_o} \tag{2-13}$$

In this equation the current overburden pressure p'_o can be computed reasonably well, but the value of the preconsolidation pressure p'_c is at best an estimate, making a reliable computation for OCR difficult. The only method at present that is reasonably reliable is to use the consolidation test described in Sec. 2-10 to obtain p'_c. The alternative, which is likely to be less precise, is to use some kind of in situ testing to obtain the s_u/p'_i ratio (where $i = o$ or c) and use a chart such as Fig. 2-36 given later in Sec. 2-11.9.

There are a number of empirical correlations for OCR based on the s_u/p'_o ratio (the undrained shear strength, s_u, divided by the current in situ effective overburden pressure p'_o) and on in situ tests that are defined later in Chap. 3. The following were taken from Chang (1991):

For the field vane test:

$$\mathrm{OCR} = 22(s_u/p'_o)_{\mathrm{fv}}(I_P)^{-0.48}$$

$$\mathrm{OCR} = \frac{(s_u/p'_o)_{\mathrm{fv}}}{0.08 + 0.55 I_P}$$

For the cone penetrometer test:

$$S = \frac{s_u}{p'_o} = \frac{(q_c - p_o)}{p'_o} \cdot \frac{1}{N_k}$$

$$S_1 = 0.11 + 0.0037 I_P \quad \text{[see Eq. (2-60)]}$$

These two S-values are then used to compute the OCR as

$$\text{OCR} = (S/S_1)^{1.13+0.04(S/S_1)}$$

Section 3-11.1 gives an alternative method to compute the OCR from a cone penetration test using Eqs. (3-17).

For the flat dilatometer test:

$$\text{OCR} = 0.24K_D^{1.32}$$

In these equations I_P = plasticity index in percentage; q_c = cone resistance; p_o = total (not effective) overburden pressure; N_k = cone factor that is nearly constant at $N_k = 12$ for OCR ≤ 8; K_D = horizontal stress index for the dilatometer. All of these terms (see Symbol list) are either used later in this chapter or in Chap. 3. There are a number of other equations given by Chang but these tend to summarize his discussion best.

With the value of OCR and the current in situ effective pressure p'_o one can use Eq. (2-13) to back-compute the preconsolidation pressure p'_c.

An estimate for $K_{o,\text{OCR}}$ is given by Mayne (1984) based on the analysis of a number of clay soils reported in the literature. The equation is as follows:

$$K_{o,\text{OCR}} = K_{o,\text{nc}}(A + s_u/p'_o) \tag{2-25}$$

In this equation note that the ratio s_u/p'_o uses the effective current overburden pressure p'_o. The variable A depends on the type of laboratory test used to obtain the s_u/p'_c ratio as follows:

Test	A	Comments
CK_oUC	0.7	K_o-consolidated—undrained compression
CIUC	0.8	Isotropically consolidated
CK_oDSS	1.0	Direct simple shear test

The upper limit of $K_{o,\text{OCR}}$ appears to be the passive earth pressure coefficient K_p (defined in Chap. 11), and a number of values reported in the literature range from 1.5 to 1.7. It would appear that the upper limit of any normally consolidated soil would be $K_{o,\text{nc}} \leq 1.0$ since a fluid such as water has $K_o = 1.0$ and no normally consolidated soil would have a value this large.

Example 2-2. Compare K_o by the several approximate methods given in this section for both a normally consolidated (nc) clay and for a clay with a known value of OCR = 5.0.

Other data:

$\phi' = 20^\circ$ $I_P = 35\%$ (nc)
$\phi' = 25^\circ$ $I_P = 32\%$ (OCR = 5)

Solution. For the normally consolidated case, we may write the following:

1. Use Brooker and Ireland's Eq. (2-20):

$$K_{o,\text{nc}} = 0.95 - \sin\phi' = 0.95 - \sin 20 = 0.61$$

2. Use Eqs. (2-21):

$$\begin{aligned}
(2\text{-}21):\qquad K_{o,\mathrm{nc}} &= 0.19 + 0.233\log I_P\\
&= 0.19 + 0.233\log 35 = 0.55\\
(2\text{-}21a):\qquad K_{o,\mathrm{nc}} &= 0.44 + 0.0042 I_P = 0.59
\end{aligned}$$

In the absence of better data, use the average of these as

$$K_{o,\mathrm{nc}} = \frac{0.61 + 0.55 + 0.59}{3} = \mathbf{0.58}$$

For the overconsolidated case, we calculate as follows:

1. Use Eq. (2-23), but first use Eq. (2-24) to find exponent n:

$$n = 0.54 \times 10^{-I_P/281} \rightarrow n = 0.42 \quad \text{for } I_P = 32\%$$

Now, $K_{o,\mathrm{OCR}} = K_{o,\mathrm{nc}} \times \mathrm{OCR}^n \rightarrow$ use $K_{o,\mathrm{nc}} = 0.58$ just found

$$K_{o,\mathrm{OCR}} = 0.58 \times \mathrm{OCR}^{0.42} = 0.58 \times 5^{0.42} = 1.14$$

2. Use Eq. (2-24*a*) for an alternative n:

$$n = \sin\phi' = \sin 25 = 0.423$$

$$K_{o,\mathrm{OCR}} = 0.58 \times 5^{0.423} = 1.15 \text{ (vs. 1.14 just computed)}$$

3. Use Eq. (2-25) and assume CIUC testing so $A = 0.8$. Also estimate a value for s_u/p'_c. For this use Eq. (2-59) following:

$$\begin{aligned}
\frac{s_u}{p'_o} &= 0.45\sqrt{I_P} \quad \text{[Eq. (2-59)]}\\
&= 0.45(0.35)^{0.5} = 0.27 \quad \text{(using nc value for } I_P)
\end{aligned}$$

Substitution into Eq. (2-25) gives

$$K_{o,\mathrm{OCR}} = 0.58(0.8 + 0.27) = 0.62$$

We can obtain a best estimate using all three values to obtain

$$K_{o,\mathrm{OCR}} = \frac{1.14 + 1.15 + 0.62}{3} = \mathbf{0.97}$$

or, since 0.62 is little different from the average nc value of 0.58, we might only use the two values of 1.14 and 1.15 to obtain

$$K_{o,\mathrm{OCR}} = \frac{1.14 + 1.15}{2} = \mathbf{1.14}$$

One should use a value of about **1.1**, as 1.14 implies more precision than is justified by these procedures.

Conventional usage is to call all values K_o. For computations such as in this example it is necessary to distinguish between the normally consolidated value $K_{o,\mathrm{nc}}$ and the overconsolidated value $K_{o,\mathrm{OCR}}$ as a compact means of identification in equations such as Eqs. (2-23) and (2-25).

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