CHAPTER 3

ENGINEERING PROPERTIES OF SOIL AND ROCK

3-1. Scope. This chapter considers engineering properties of soil and rock useful in designing foundations under static loading. Dynamic properties are discussed in chapter 17.

a. Correlations. Tables and charts based on easily determined index properties are useful for rough estimating or confirming design parameters. Testing procedures employed by different soil laboratories have influenced correlations presented to an unknown degree, and the scatter of data is usually substantial; caution should, therefore, be exercised in using correlation values. Undisturbed soil testing, either laboratory or field, or both, should be used for final design of major foundations. On smaller projects, an economic analysis should determine if a complete soil exploration/laboratory testing program is justified in lieu of a conservative design based on correlation data. Complex subsurface conditions may not permit a decision on solely economic grounds.

b. Engineering properties. Properties of particular interest to the foundation engineer include-

(1) Compaction.
(2) Permeability.
(3) Consolidation-swell.
(4) Shear strength.
(5) Stress-strain modulus (modulus of elasticity) and Poisson's ratio.

3-2. Compaction characteristics of soils. The density at which a soil can be placed as fill or backfill depends on the placement water content and the compaction effort. The Modified Compaction Test (CE 55) or comparable commercial standards will be used as a basis for control. The CE 55 test is described in TM 5-824-2/AFM 88-6, Chapter 2. (See app A for references.) Other compaction efforts that may be occasionally used for special projects include-

a. Standard compaction test: Three layers at 25 blows per layer Hammer = 5.5 pounds with 12-inch drop

b. Fifteen-blow compaction test:
Three layers at 15 blows per layer
Hammer = 5.5 pounds with 12-inch drop

The results of the CE 55 test are represented by compaction curves, as shown in figure 3-1, in which the water content is plotted versus compacted dry density. The ordinate of the peak of the curve is the maximum dry density, and the abscissa is the optimum water content W_{opt}. Table 3-1 presents typical engineering properties of compacted soils; see footnote for compacted effort that applies.

3-3. Density of cohesionless soils.

a. Relative density of cohesionless soils has a considerable influence on the angle of internal friction, allowable bearing capacity, and settlement of footings. An example of the relationship between relative density and in situ dry densities may be conveniently determined from figure 3-2. Methods for determining in situ densities or relative densities of sands in the field are discussed in chapter 4.

b. The approximate relationship among the angle of friction, +, DR, and unit weight is shown in figure 3-3; and between the coefficient of uniformity, Cu, and void ratio, in figure 3-4.

c. The relative compaction of a soil is defined as

$$RC = \frac{Y_{field}}{Y_{max\ (lab)}} \times 100(\text{percent}) \quad (3\text{-}1)$$

where y_{field} = dry density in field and $y_{max\ (lab)}$ = maximum dry density obtained in the laboratory. For soils where 100 percent relative density is approximately the same as 100 percent relative compaction based on CE 55, the relative compaction and the relative density are related by the following empirical equation:

$$RC = 80 + 0.2D_R (D_R > 40 \text{ percent}) \quad (3\text{-}2)$$

3-4. Permeability.

a. Darcy's law. The laminar flow of water through soils is governed by Darcy's law:

$$q = kiA \quad (3\text{-}3)$$

where

q = seepage quantity (in any time unit consistent with k)
k = coefficient of permeability (units of velocity)
i = h/L = hydraulic gradient or head loss, h, across the flow path of length, L
A = cross-sectional area of flow

b. Permeability of soil. The permeability - depends primarily on the size and shape of the soil grains, void ratio, shape and arrangement of voids, degree of saturation, and temperature. Permeability is determined in the laboratory by measuring the rate of flow of water through a specimen under known hydraulic gradient, i. Typical permeability values, empirical relationships, and methods for obtaining the coefficient of permeability are shown in figure 3-5. Field pumping tests are the most reliable means of determining the permeability of natural soil deposits (para 4-5). Permeability obtained in this manner is the permeability in a horizontal direction. The vertical permeability of natural soil deposits is affected by stratification and is usually much lower than the horizontal permeability.

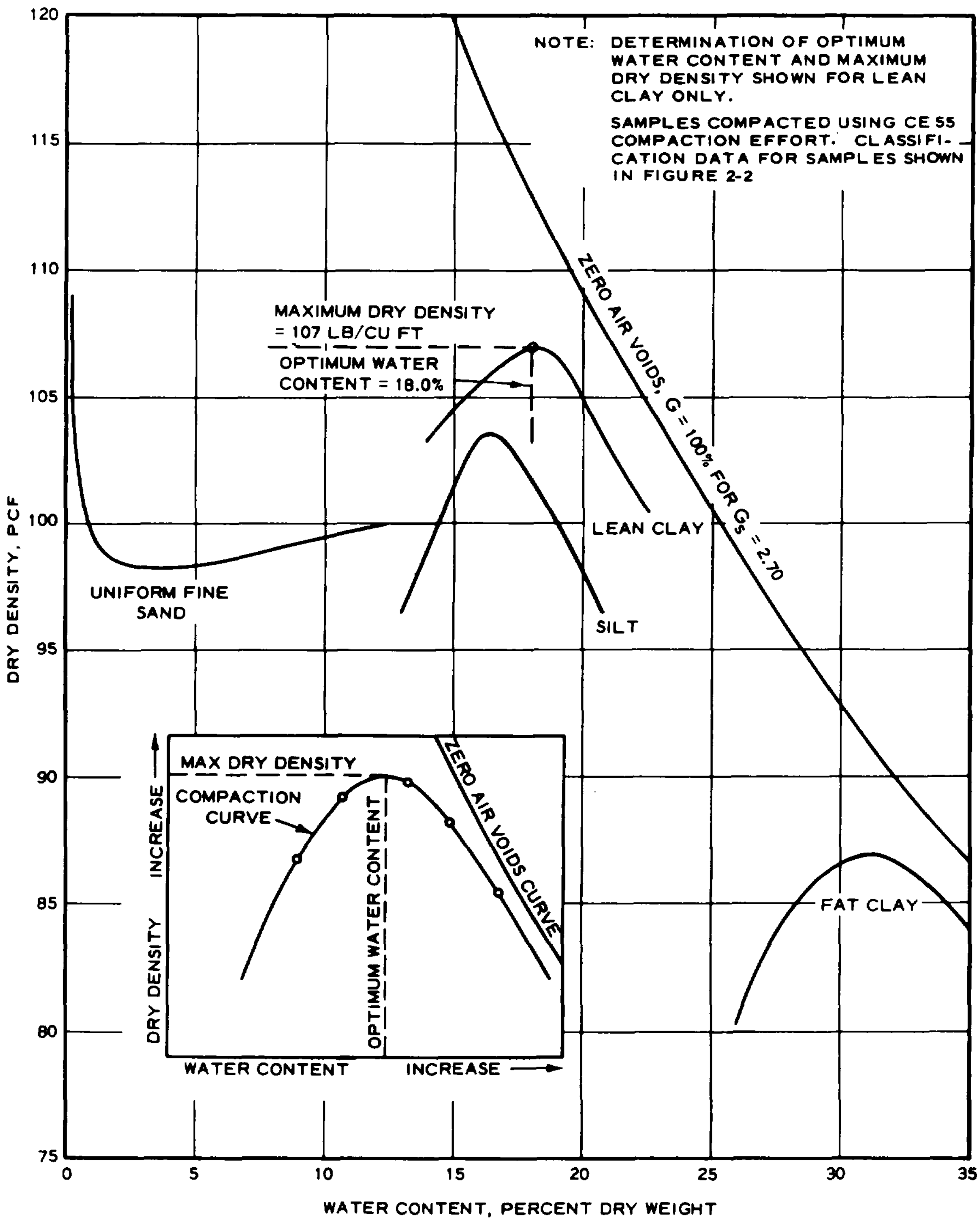

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Figure 3-1. Typical CE 55 compaction test data.

Table 3-1. Typical Engineering Properties of Compacted Materials

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Water, Content Percent	Typical Value of Compression: At 2.5 ksf (20 psi) Percent of Original Height	Typical Value of Compression: At 7.2 ksf (50 psi) Percent of Original Height	Typical Strength Characteristics: Cohesion (As Compacted) psf	Typical Strength Characteristics: Cohesion (Saturated) psf	Typical Strength Characteristics: ϕ (Effective Stress Envelope) deg	Typical Coefficient of Permeability ft/min	Range of CBR Values	Range of Subgrade Modulus k lb/cu in.
GW	Well graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	5×10^{-2}	40-80	300-500
GP	Poorly graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	10^{-1}	30-60	250-400
GM	Silty gravels, poorly graded gravel-sand-silt	120-135	12-8	0.5	1.1	....	....	>34	$>10^{-6}$	20-60	100-400
GC	Clayey gravels, poorly graded gravel-sand-clay	115-130	14-9	0.7	1.6	....	....	>31	$>10^{-7}$	20-40	100-300
SW	Well graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	$>10^{-3}$	20-40	200-300
SP	Poorly graded clean sands, sand-gravel mix	100-120	21-12	0.8	1.4	0	0	37	$>10^{-3}$	10-40	200-300
SM	Silty sands, poorly graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	5×10^{-5}	10-40	100-300
SM-SC	Sand-silt clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	2×10^{-6}	....	
SC	Clayey sands, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	5×10^{-7}	5-20	100-300
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	10^{-5}	15 or less	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	5×10^{-7}	....	100-200
CL	Inorganic clays of low to med. plasticity	95-120	24-12	1.3	2.5	1800	270	28	10^{-7}	15 or less	50-200
OL	Organic silts and silt-clays, low plasticity	80-100	33-21	....	....	....	....	....	....	5 or less	50-100
MH	Inorganic clayey silts, elastic silts	75-95	40-24	2.0	3.8	1500	420	25	5×10^{-7}	10 or less	50-100
CH	Inorganic clays of high plasticity	80-105	36-19	2.6	3.9	2150	230	19	10^{-7}	15 or less	50-150
OH	Organic clays and silty clays	75-100	45-21	....	....	....	....	....	....	5 or less	25-100

Notes

1. All properties are for condition of "standard Proctor" maximum density. except values of k and CBH which are for CE55 ,maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from ISBR data.
3. Compression values are for vertical loading with complete lateral confinement.

*. 4. (>) indicates that typical property is greater than the value shown. ( ) indicates insufficient data available for an estimate.

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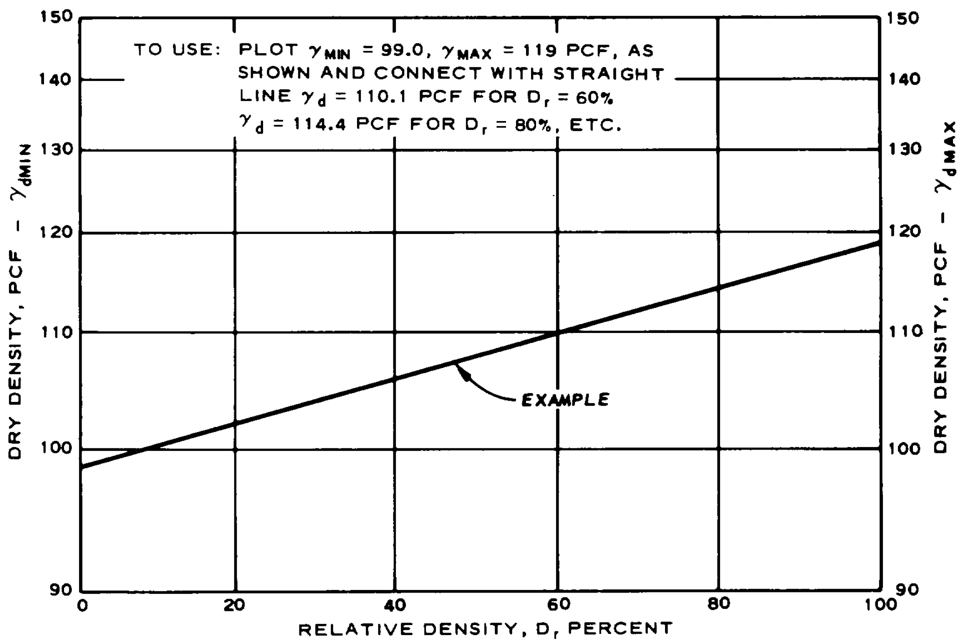

Figure 3-2. Relation between relative density and dry density (scaled to plot as a straight line).

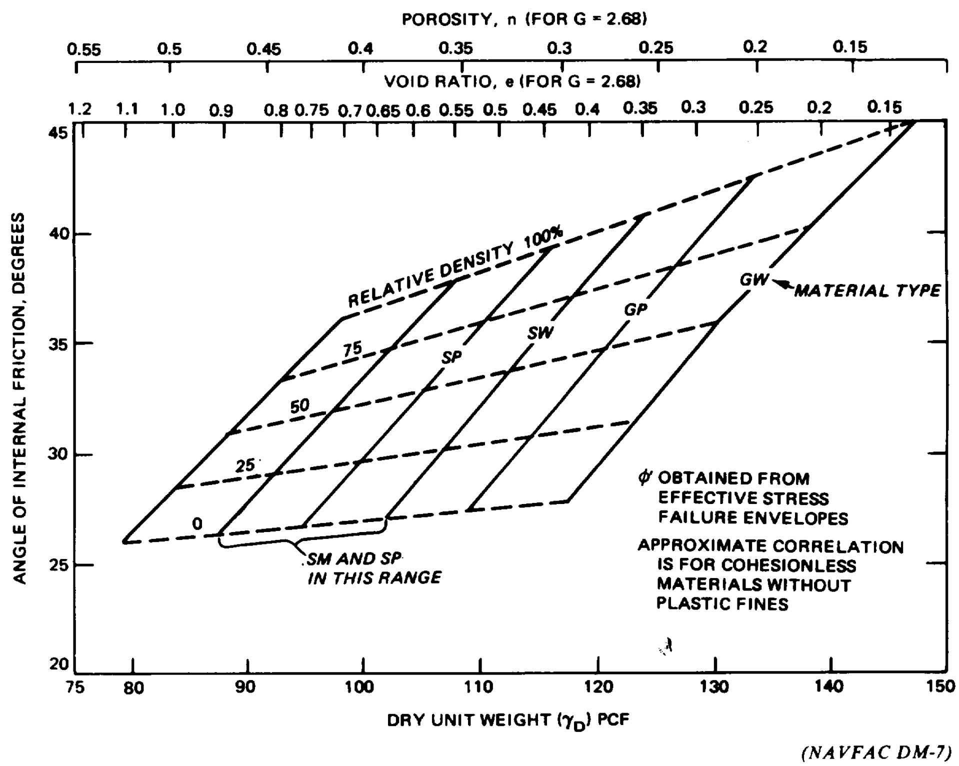

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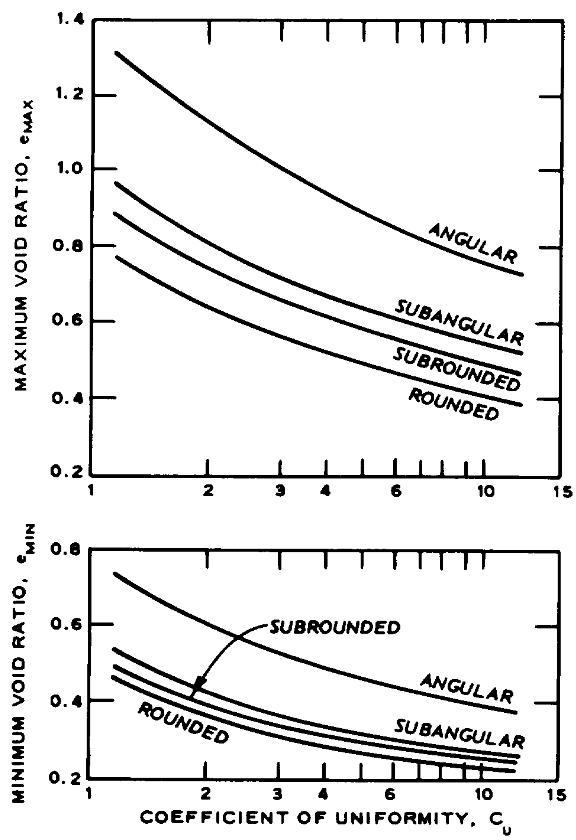
Figure 3-3. Angle of friction versus dry density for coarse-grained soils.

c. Permeability of rock. Intact rock is generally impermeable, but completely intact rock masses rarely occur. The permeability of rock masses is controlled by discontinuities (fissures, joints, cracks, etc.), and flow may be either laminar (Darcy's law applies) or turbulent, depending on the hydraulic gradient, size of flow path, channel roughness, and other factors. Methods for determining the in situ permeability of rock are presented in chapter 4.

3-5. Consolidation. Consolidation is a time-dependent phenomenon, which relates change that occurs in the soil mass to the applied load.

a. Consolidation test data. Consolidation or one-dimensional compression tests are made in accordance with accepted standards. Results of tests (fig 3-6) are presented in terms of time-consolidation curves and pressure-void ratio curves. The relationship between void ratio and effective vertical stress, p, is shown on a semilogarithmic diagram in figure 3-6. The test results may also be plotted as change in volume versus effective vertical stress. Typical examples of pressure - void ratio curves for insensitive and sensitive, normally loaded clays, and preconsolidated clays are shown in figure 3-7.

b. Preconsolidation pressure. The preconsolidation stress, pc, is the maximum effective stress to which the soil has been exposed and may result from loading or drying. Geological evidence of past loadings should be used to estimate the order of magnitude of preconsolidation stresses before laboratory tests are performed. The Casagrande method of obtaining the preconsolidation pressure from consolidation tests is shown in figure 3-7. Determining the point of greatest curvature



NOTE: THE MINIMUM VOID RATIOS WERE OBTAINED FROM SIMPLE SHEAR TESTS. CURVES ARE ONLY VALID FOR CLEAN SANDS WITH NORMAL TO MODERATELY SKEWED GRAIN-SIZED DISTRIBUTIONS.

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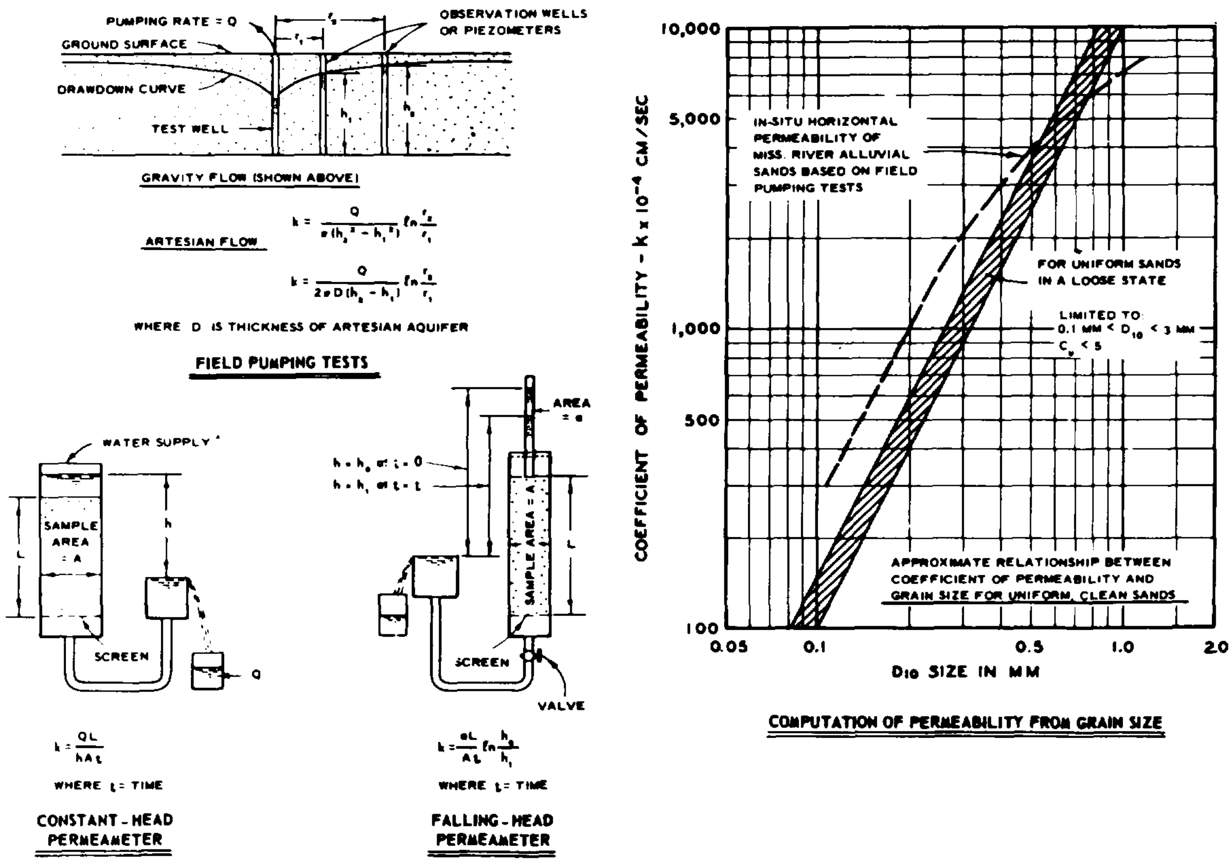
Figure 3-4. Generalized curves for estimating e_{max}, and e_{min} from gradational and particle shape characteristics.



PERMEABILITY CHARACTERISTICS OF SOILS

Coefficient of Permeability, k: cm/sec	ft/min	ft/yr	Relative Permeability	Soil Type	Method of Determination
10 1	20 2	10.5×10^5 1.05×10^5	High	Clean gravels Coarse sands	Field pumping tests, reliable if properly executed (High to Medium) Constant-head permeameter, reliable (High to Medium) Computation from grain size (High to Medium)
1000×10^{-4} 100×10^{-4} 10×10^{-4}	0.2 0.02 0.002	10,500 1,050 105	Medium	Medium sands Fine sands and sand and gravel mixtures Very fine sand	Falling-head permeameter (Medium to Practically impervious): Reliable
1×10^{-4} 0.1×10^{-4} 0.01×10^{-4}	2×10^{-4} 0.2×10^{-4} 0.02×10^{-4}	10.5 1.05 0.105	Low	Silty sands, organic silts Silts, glacial till Silty clay	Unreliable
100×10^{-9} 10×10^{-9} 1×10^{-9}	200×10^{-9} 20×10^{-9} 1×10^{-9}	105×10^{-4} 10.5×10^{-4} 1.05×10^{-4}	Practically impervious	"Impervious" soils, e.g., homogeneous clays below zone of weathering	Fairly reliable Computation from consolidation test data (reliable)

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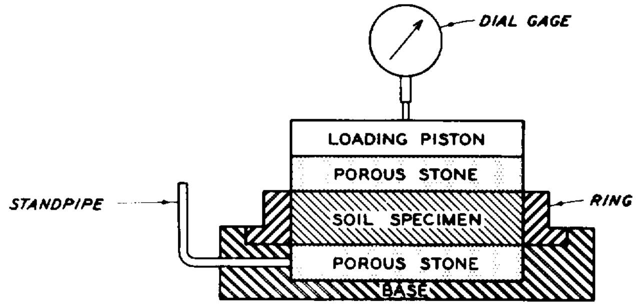
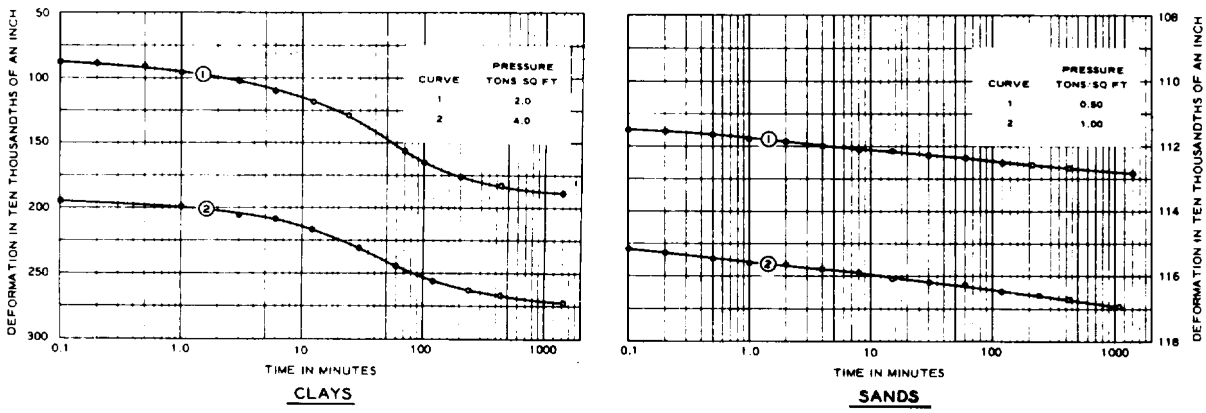
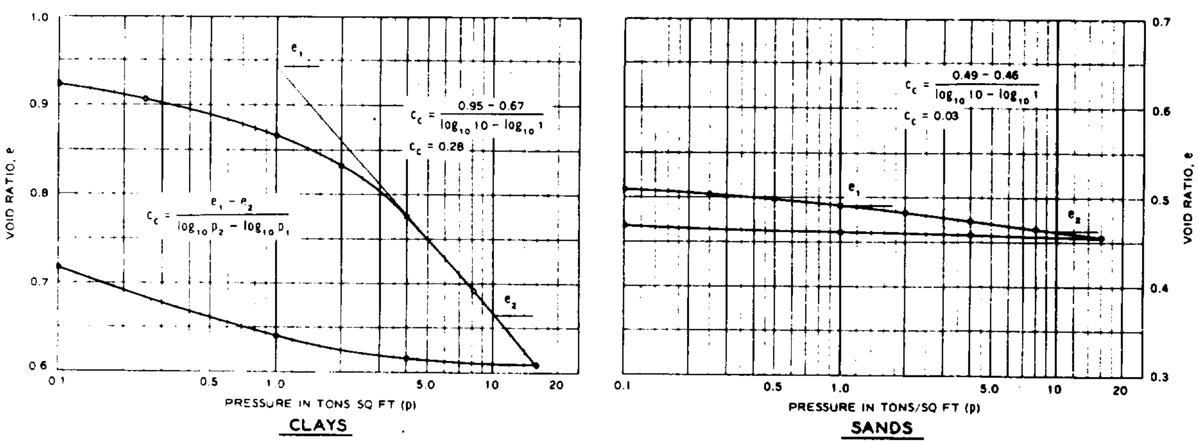
Figure 3-5. A summary of soil permeabilities and methods of determination.



CONSOLIDATION APPARATUS



EXAMPLES OF LABORATORY CONSOLIDATION CURVES



EXAMPLES OF LABORATORY PRESSURE - VOID RATIO CURVES

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Figure 3-6. Examples of laboratory consolidation test data.

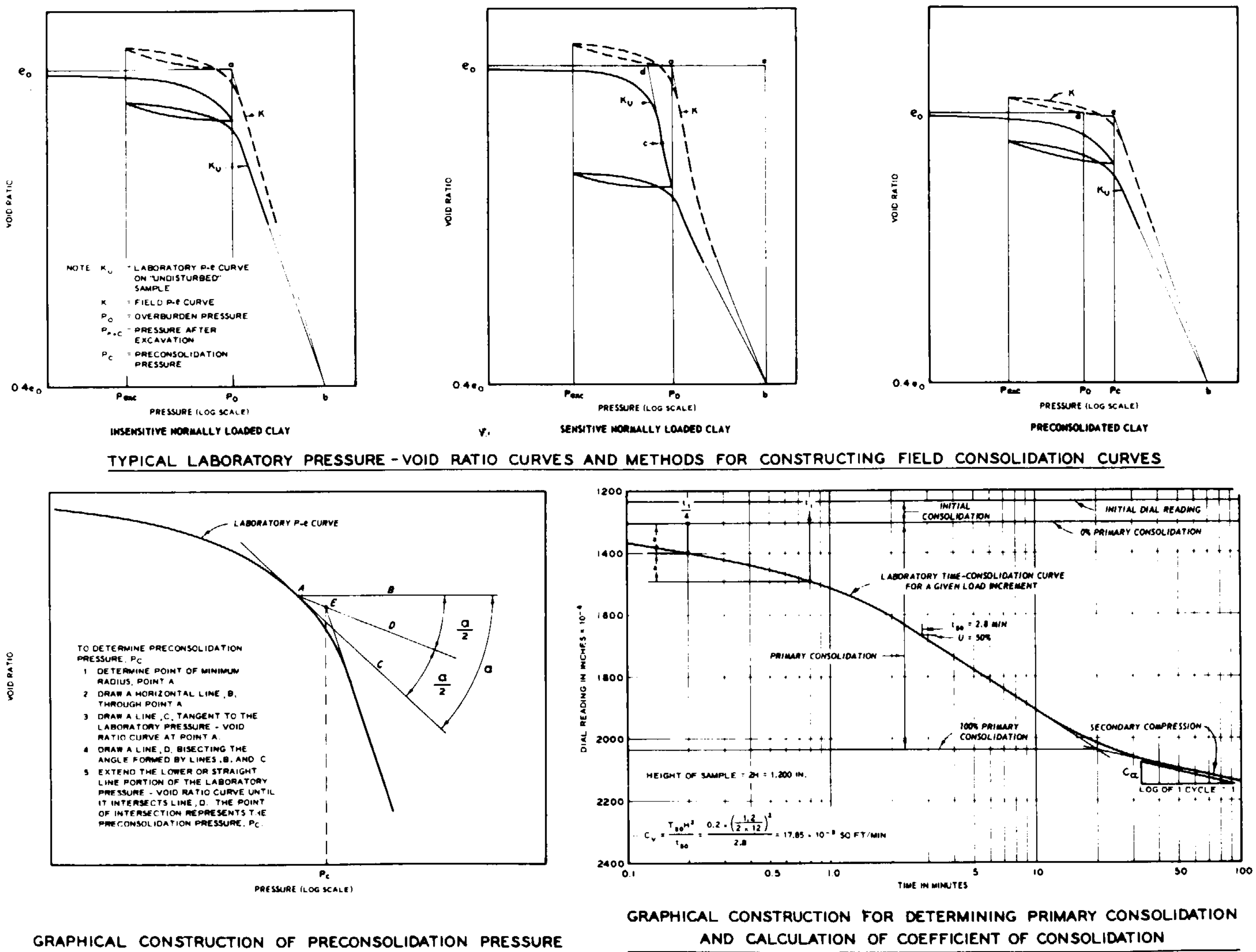


Figure 3- 7. Analyses of consolidation test data.

requires care and judgment. Sometimes it is better to estimate two positions of this point-one as small as likely, and the other as large as plausible, consistent with the data-and to repeat the construction for both cases. The result will be a range of preconsolidation stresses. Because the determination of pc involves some inevitable inaccuracy, the range of possible values may be more useful than a single estimate which falls somewhere in the possible range. The higher the quality of the test specimen, the smaller is the range of possible pc values. Approximate values of preconsolidation pressure may be estimated from figure 3-8 or 3-9. Table 3-2 can be used to obtain gross estimates of site preconsolidation. This table and figures 3-8 and 3-9 should be applied before consolidation tests are performed to assure test loads sufficiently high to define the virgin compression portion of e-log p plots.

c. Compression index. The slope of the virgin compression curve is the compression index C_c, defined in figure 3-6. Compression index correlations for approximations are given in table 3-3. When volume change is expressed as vertical strain instead of change in void ratio, the slope of the virgin compression part of the £ versus log p curve is the compression ratio, CR, defined as

$$CR = \frac{\Delta\varepsilon}{\log \frac{p_2'}{p_1'}} = \frac{C_c}{1 + e_0} \qquad (3\text{-}4)$$

where & is the change in vertical strain corresponding to a change in effective stress from p_1' to p_2', and e_0 is the initial (or in situ) void ratio. An approximate correlation between CR and natural water content in clays is given by the following:

$$CR = 0.006\ (w - 12) \qquad (3\text{-}5)$$

d. Coefficient of volume compressibility. The relationship between deformation (or strain) and stress for one-dimensional compression is expressed by the coefficient of volume compressibility, m , which is defined as

$$m_v = \frac{\Delta\varepsilon}{\Delta p} \quad \frac{\Delta\varepsilon}{\Delta p\,(1 + e_0)} = \frac{a_v}{1 + e_0} \qquad (3\text{-}6)$$

$$m_v = \frac{0.434\,C_c}{(1 + e_0)p'}$$

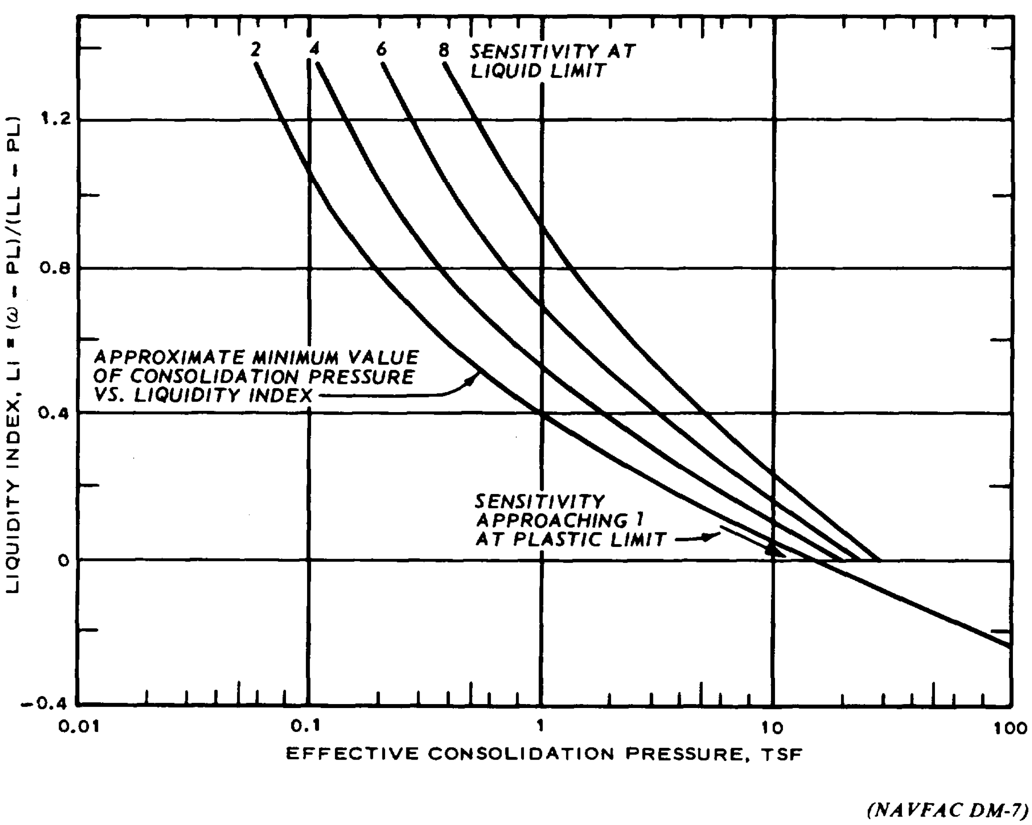


Figure 3-8. Approximate relation between liquidity index and effective overburden pressure, as a function of the sensitivity of the soil

where

$\Delta\varepsilon$ = change in vertical strain

Δp = P2 - p; = corresponding change in effective vertical stress

a_v = Ae/Ap = coefficient of compressibility

p' = average of initial and final effective vertical stress

The units of mv are the reciprocal of constrained modulus. Table 3-4 gives typical values of **mv** for several granular soils during virgin loading.

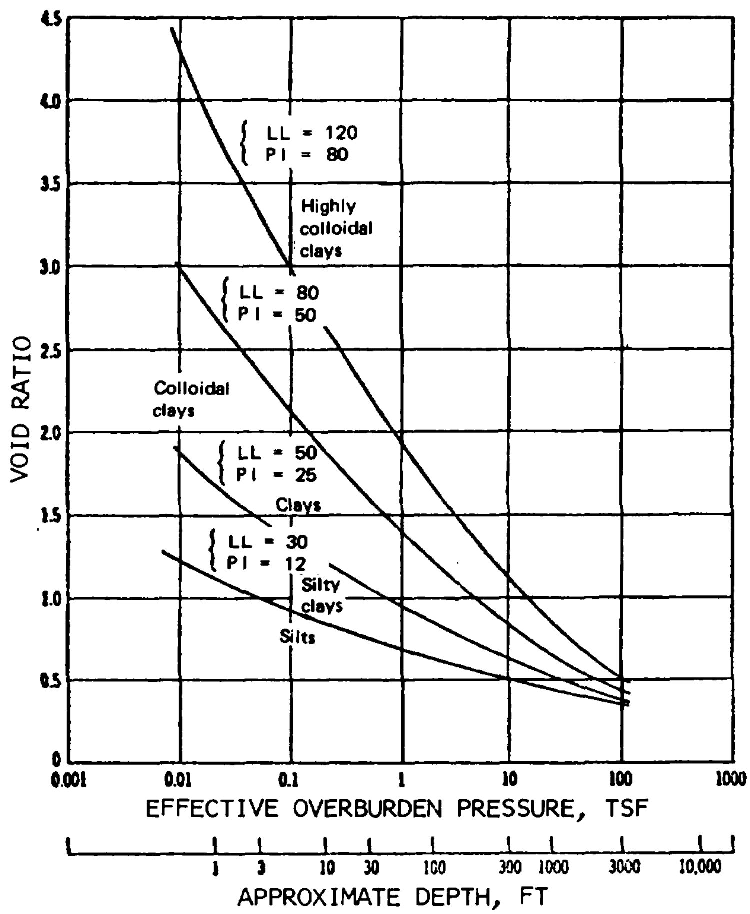
e. Expansion and recompression. If overburden pressure is decreased, soil undergoes volumetric expansion (swell), as shown in figure 3-7. The semilogarithmic, straight-line (this may have to be approximated) slope of the swelling curve is expressed by the swelling index, C_s, as

$$C_s = \frac{\Delta e}{\log \frac{p_2'}{p_1'}} \tag{3-8}$$

where Ae is the change in void ratio (strictly a sign applies to C_c, C_s., C_r, and m_v; however, judgment is usually used in lieu of signs). The swelling index is generally from one-fifth to one-tenth the compression index. Approximate values of C_s may be obtained from figure 3-10. The slope of the recompression curve is expressed by the recompression index, C_r, as follows:

$$C_r = \frac{\Delta e}{\log \frac{p_2'}{p_1'}} \tag{3-9}$$

The value of C_r is equal to or slightly smaller than C_s. High values of C_r/C_s are associated with overconsolidated clays containing swelling clay minerals.



(Courtesy of T. W. Lambe and R. V. Whitman, Soils Mechanics, 1969, p 320. Reprinted by permission of John Wiley & Sons, Inc., New York.)

Figure 3-9. Approximate relation between void ratio and effective overburden pressure for clay sediments, as a function of the Atterberg limits.

Table 3-2. Estimating Degree of Preconsolidation

Method	Remarks
Surface topography	Soil below alluvial valley filling should generally have a preconsolidation stress at least corresponding to elevation of abutments. In wide river valleys with terraces at several elevations, an elevation corresponding to previous surface elevation in the river valley may be several miles distant
Geological evidence	Ask geologist for estimate of maximum preconsolidation stress. Erosion may have removed hundreds of feet of material even in abutment area
Water content	If natural water content is near PL or below it, anticipate high preconsolidation stress. A high natural water content is not, itself, a suitable indicator of absence of overconsolidation
Standard penetration resistance	If blow counts are high, anticipate high preconsolidation stress. From blow counts, estimate undrained shear strength, s_u , in tons per square foot as approximately 1/15 of blow count. If estimated value is substantially more than corresponds to a s_u/p_o ratio of about 0.25, anticipate high preconsolidation stresses
Undisturbed sampling	If soil was too hard to sample with piston sampler, and a Denison or similar sampler was required, suspect high preconsolidation stress
Laboratory shear strengths	If higher than those corresponding to a s_u/p_o ratio of about 0.3, anticipate high preconsolidation stress
Compression index from consolidations tests	If compression index appears low for Atterberg limits of soil, suspect that test loads were not carried high enough to determine virgin compression curve and correct preconsolidation stress. Expected values for compression index can be estimated from correlations with water content and Atterberg limits (table 3-3)
Liquidity index and sensitivity	Estimate preconsolidation stress from figures 3-8 and 3-9 (p_c values may be low)

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f. Coefficient of consolidation. The soil properties that control the drainage rate of pore water are combined into the *coefficient of consolidation,* C_v, defined as follows:

$$C_v = \frac{k(1 + e_0)}{y_w a_v'} = \frac{k}{y_w m_v} \qquad (3\text{-}10)$$

alternatively,

$$C_v = \frac{TH^2}{t} \qquad (3\text{-}11)$$

where

k = coefficient of permeability in a vertical direction

e_0 = initial void ratio

y_y = unit weight of water

a_v = $\Delta e/\Delta p$ = coefficient of compressibility, vertical deformation

m_v = coefficient of volume compressibility

T = Time factor (para 5-5) that depends on percent consolidation and assumed pore pres-

Table 3-3. Compression Index Correlations

Clays

C_c = 0.012 w , wn in percent

C_c = 0.01 (LL-13)

Sand, uniform

C_c = 0.03 , loose to Cc = 0.06, dense

Silt, uniform

C_c = 0.20

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Table 3-4. Value of Coefficient of Compressibility (m_v) for Several Granular Soils During Virgin Loading

Soil	Relative Density, D_R	m_v x $^{10-4}$ per psi Effective Pressure: 9 to 14 psi	28 to 74 psi
Uniform gravel	0	2.3	1.1
1 < D < 5 mm	100	0.6	0.4
Well-graded sand	0	5.0	2.7
0.02 < D < 1 mm	100	1.3	0.6
Uniform fine sand	0	4.8	2.0
0.07 < D < 0.3 mm	100	1.4	0.6
Uniform silt	0	25.0	4.0
0.02 < D < 0.07 mm	100	2.0	0.9

(Courtesy of T W. Lambe and R. V. Whitman, Soils Mechanics, 1969, p 155. Reprinted by permission of John Wiley & Sons, Inc., New York.)

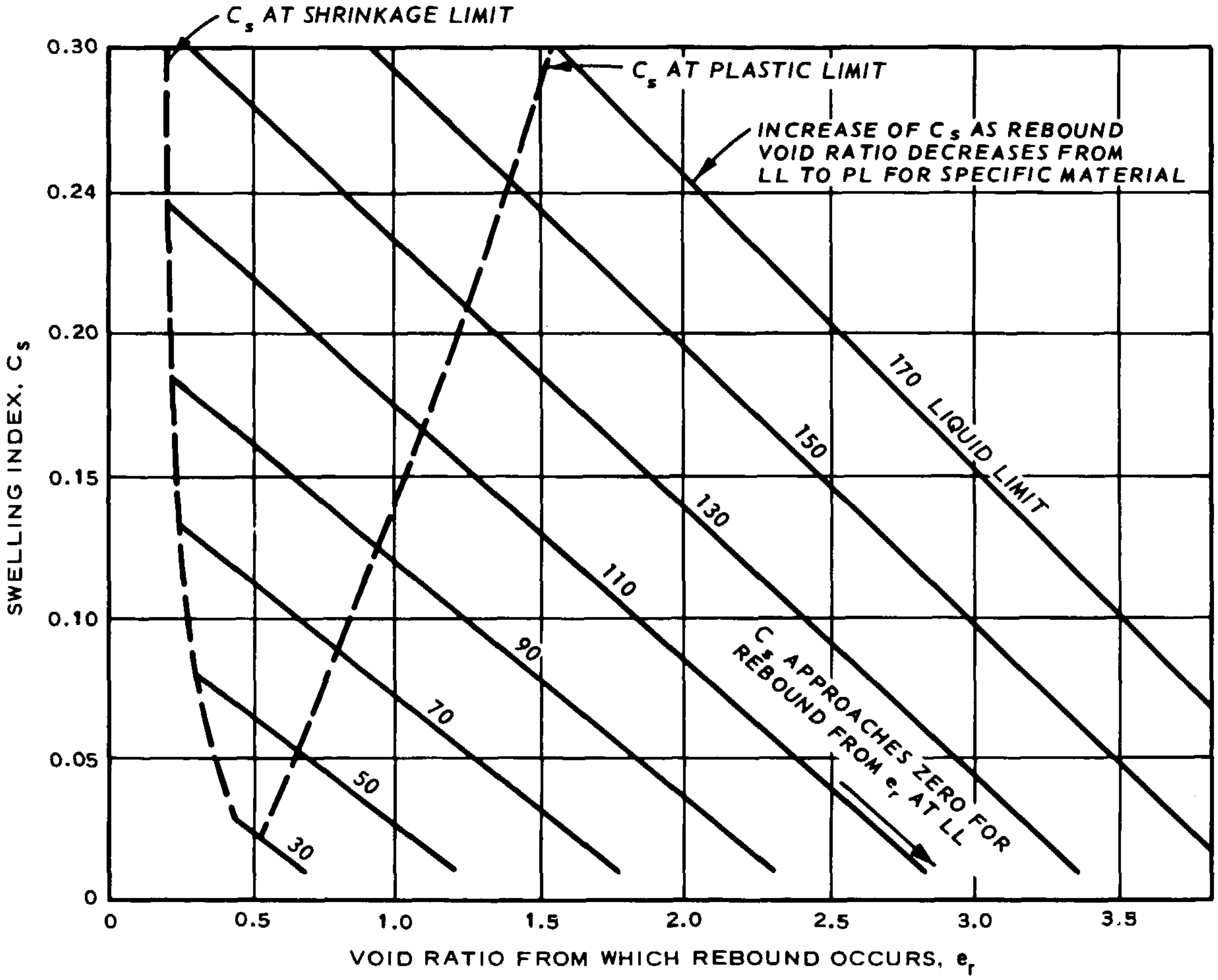

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Figure 3-10. Approximate correlations for swelling index of silts and clays.

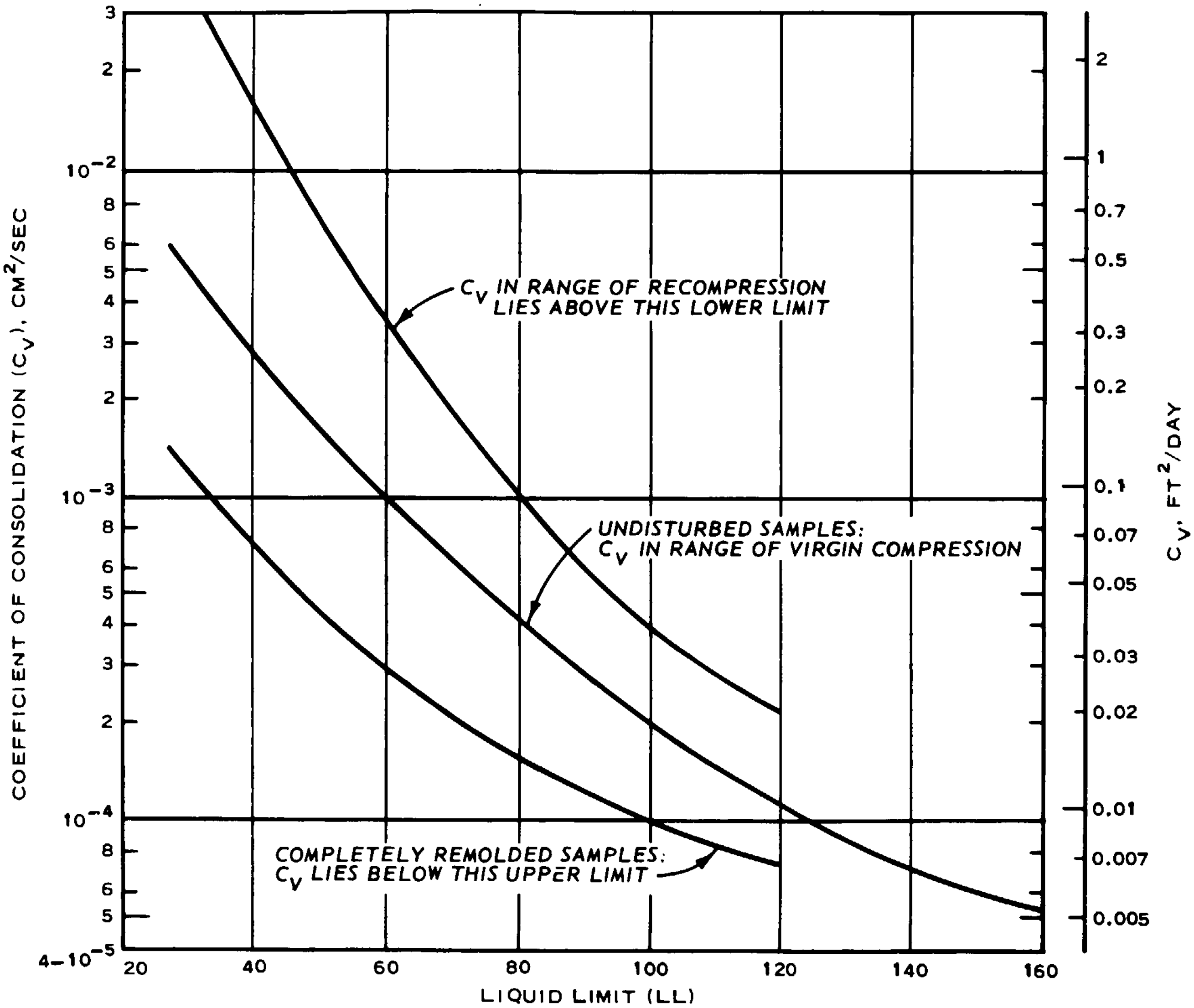


Figure 3-11. Correlations between coefficient of consolidation and liquid limit.

sure distribution in soil caused by load

H = length of longest drainage path (lab or field)

t = time at which the time factor is T for the degree of consolidation that has occurred (generally, use t_{50} for T = 0.197 and 50 percent consolidation)

Correlation between C_v and LL are shown in figure 3-11 for undisturbed and remolded soil.

g. Coefficient of secondary compression. The coefficient of secondary compression, C_α, is strain $\varepsilon_z = \Delta H/H_0$, which occurs during one log cycle of time following completion of primary consolidation (fig 3-7). The coefficient of secondary compression is computed as

$$C_\alpha = \frac{\Delta\varepsilon_z}{\log \frac{t}{t_p}} = \frac{\frac{\Delta H}{H_f}}{\log \frac{t}{t_p}} \qquad (3\text{-}12)$$

where t_p is time to complete primary consolidation, and H_f is total thickness of compressible soil at time t_p. Soils with high compressibilities as determined by the compression index of virgin compression ratio will generally also have high values of C_α. Highly sensitive clays and soils with high organic contents usually exhibit high rates of secondary compression. Overconsolidation can markedly decrease secondary compression. Depending on the degree of overconsolidation, the value of C_α is typically about *one-half* to *one-third* as large for pressures below the preconsolidation pressure as it is for the pressures above the preconsolidation pressures. For many soils, the value of C_α approximately equals 0.00015w, with w in percent.

h. Effects of remolding. Remolding or disturbance has the following effects relative to undisturbed soil:

(1) e-log p curve. Disturbance lowers the void ratio reached under applied stresses in the vicinity of the preconsolidation stress and reduces the distinct break in the curve at the preconsolidation pressure (fig 3-7). At stresses well above the preconsolidation stress, the e-log p curve approaches closely that for good undisturbed samples.

(2) *Preconsolidation stress.* Disturbance lowers the apparent preconsolidation stress.

(3) *Virgin compression.* Disturbance lowers the value of the compression index, but the effect may not be severe.

(4) Swelling and recompression. Disturbance increases the swelling and recompression indices.

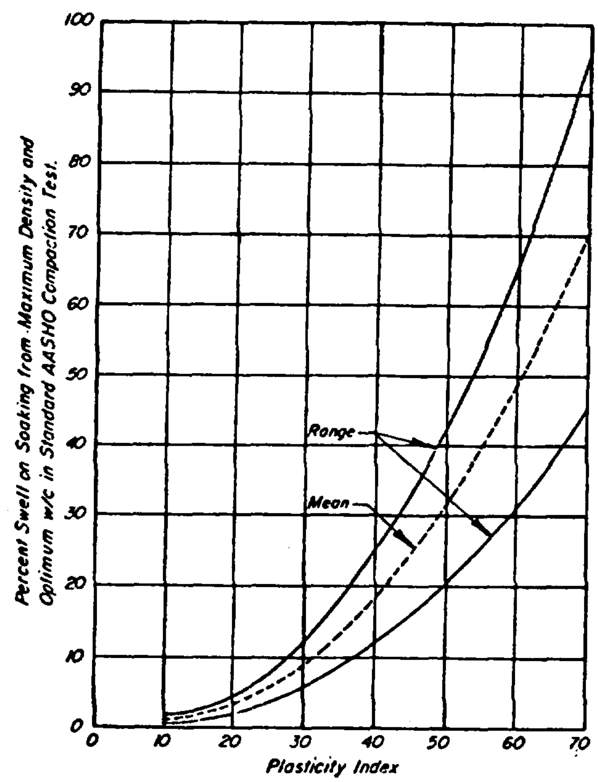
(5) *Coefficient of consolidation.* Disturbance decreases the coefficient of consolidation for both virgin compression and recompression (fig 3-11) in the vicinity of initial overburden and preconsolidation stresses. For good undisturbed samples, the value of C_v decreases abruptly at the preconsolidation pressure.

(6) Coefficient of secondary compression. Disturbance decreases the coefficient of secondary compression in the range of virgin compression.

3-6. Swelling, shrinkage, and collapsibility.

a. The swelling potential is an index property and equals the percent swell of a laterally confined soil sample that has soaked under a surcharge of 1 pound per square inch after being compacted to the maximum density at optimum water content according to the standard compaction test method. Correlation between swelling potential and PI for natural soils compacted at optimum water content to standard maximum density is shown in figure 3-12.

b. The amount of swelling and shrinkage depends on the initial water content. If the soil is wetter than the shrinkage limit (SL), the maximum possible shrinkage will be related to the difference between the actual water content and the SL. Similarly, little swell will occur after-the water content has reached some value above the plastic limit.



(Courtesy of H. B. Seed, J. Woodward. J and Lundgren, R., "Predication of Swelling Potential for Swelling Clay, " Journal, Soil Mechanics and Foundations Division, Vol 88, No. SM3, Part I. 1962, pp 53-87. Reprinted by permission of American Society of Civil Engineers, New York.)

Figure 3-12. Predicted relationship between swelling potential and plasticity index for compacted soils.

c. Collapsible soils are unsaturated soils that undergo large decreases in volume upon wetting with or without additional loading. An estimate of collapsibility (decrease in volume from change in moisture available) and expansion of a soil may be made from figure 3-13 based on in situ dry density and LL.

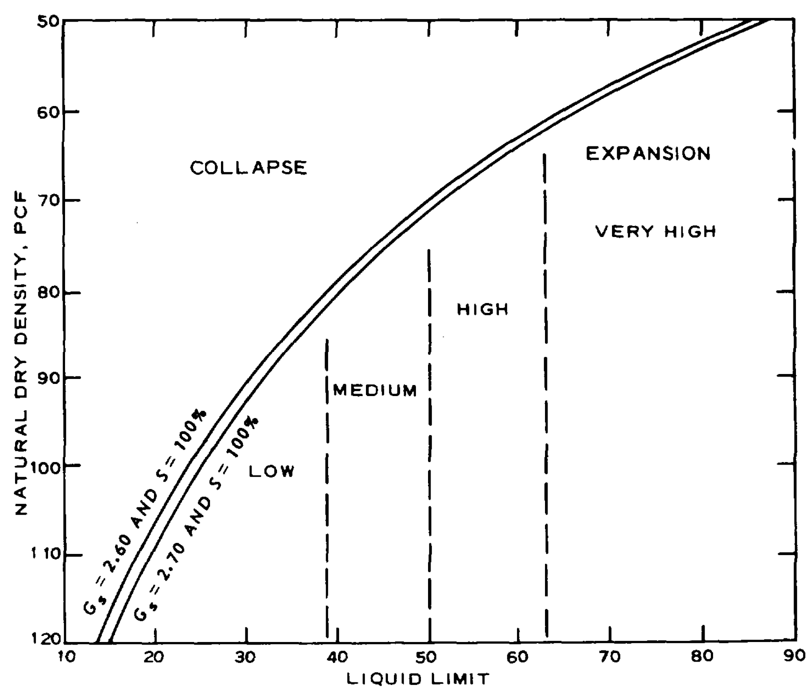
3-7. Shear strength of soils.

a. Undrained and effective strengths. The shear strength of soils is largely a function of the effective normal stress on the shear plane, which equals the total normal force less the pore water pressure. The shear strength, s, can be expressed in terms of the total normal pressure, σ, or the effective normal pressure, σ', by parameters determined from laboratory tests or, occasionally, estimated from correlations with index properties. The shear test apparatus is shown in figure 3-14. The equations for shear strength are as follows:

$s = c + \sigma \tan \phi$ (total shear strength parameters)

$s' = c' + (\sigma - u) \tan \phi'$ (effective shear strength parameters)

The total (undrained) shear strength parameters, c and ϕ, are designated as cohesion and angle of internal friction, respectively. Undrained shear strengths apply where there is no change in the volume of pore water (i.e., no consolidation) and are measured in the laboratory by shearing without permitting drainage. For saturated soils, $\phi = 0$, and the undrained shear strength, c, is designated as s_u. The effective stress parameters, c' and ϕ', are used for determining the shear strength provided pore pressures, u, are known. Pore pressure changes are caused by a change in either normal or shear stress and may be either positive or negative. Pore pressures are determined from piezometer observations during and after construction or, for design purposes, estimated on the basis of experience and behavior of samples subjected to shear tests. Effective stress parameters are computed from laboratory tests in which pore pressures induced during shear are measured or by applying the shearing load sufficiently slow to result in fully drained conditions within the test specimen.



(Courtesy of J. K. Mitchell and W. S. Gardner, "In Situ Measurement of Volume Change Characteristics, " Geotechnical Engineering Division Specialty Conference on In Situ Measurement of Soil Properties, 1975. North Carolina State University, Raleigh, N. C. Reprinted by permission of American Society of Civil Engineers, New York.)

Figure 3-13. Guide to collapsibility, compressibility, and expansion based on in situ dry density and liquid limit.

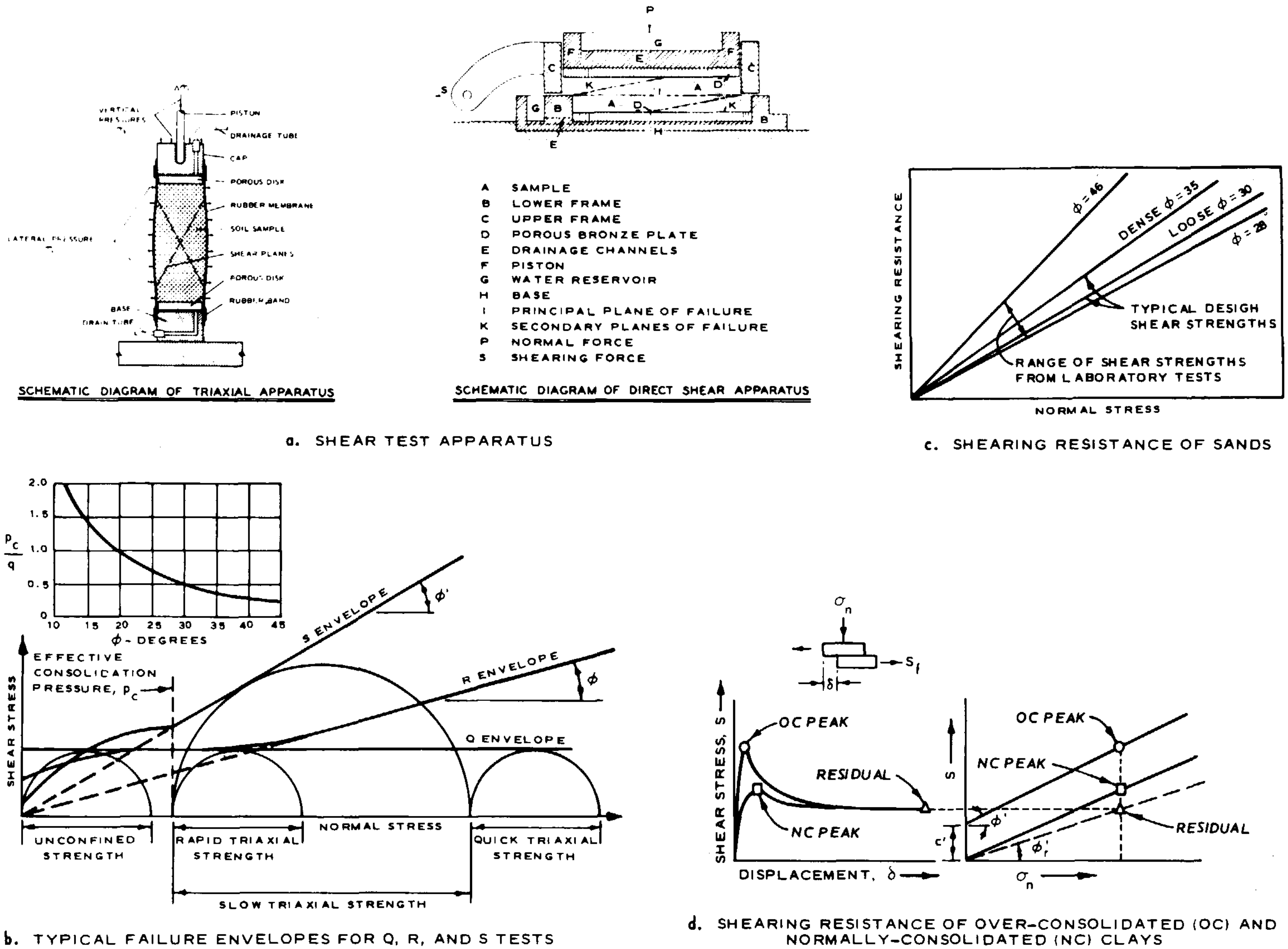
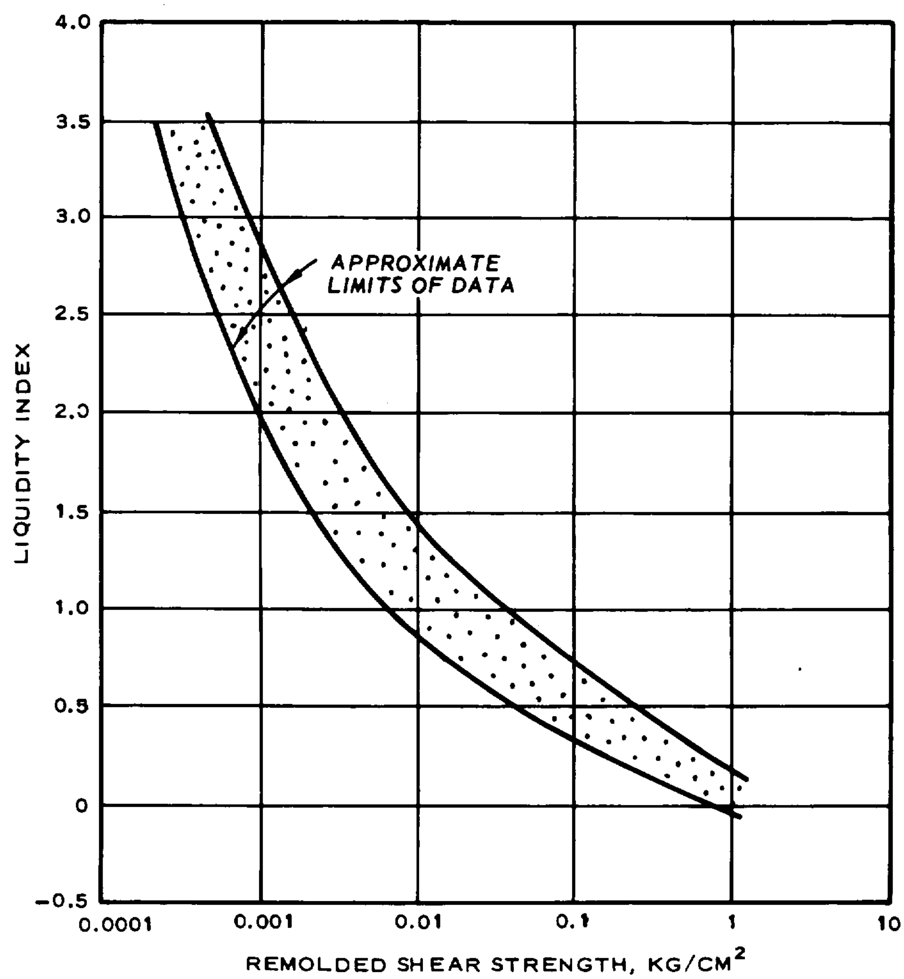


Figure 3-14. Shear test apparatus and shearing resistance of soils.

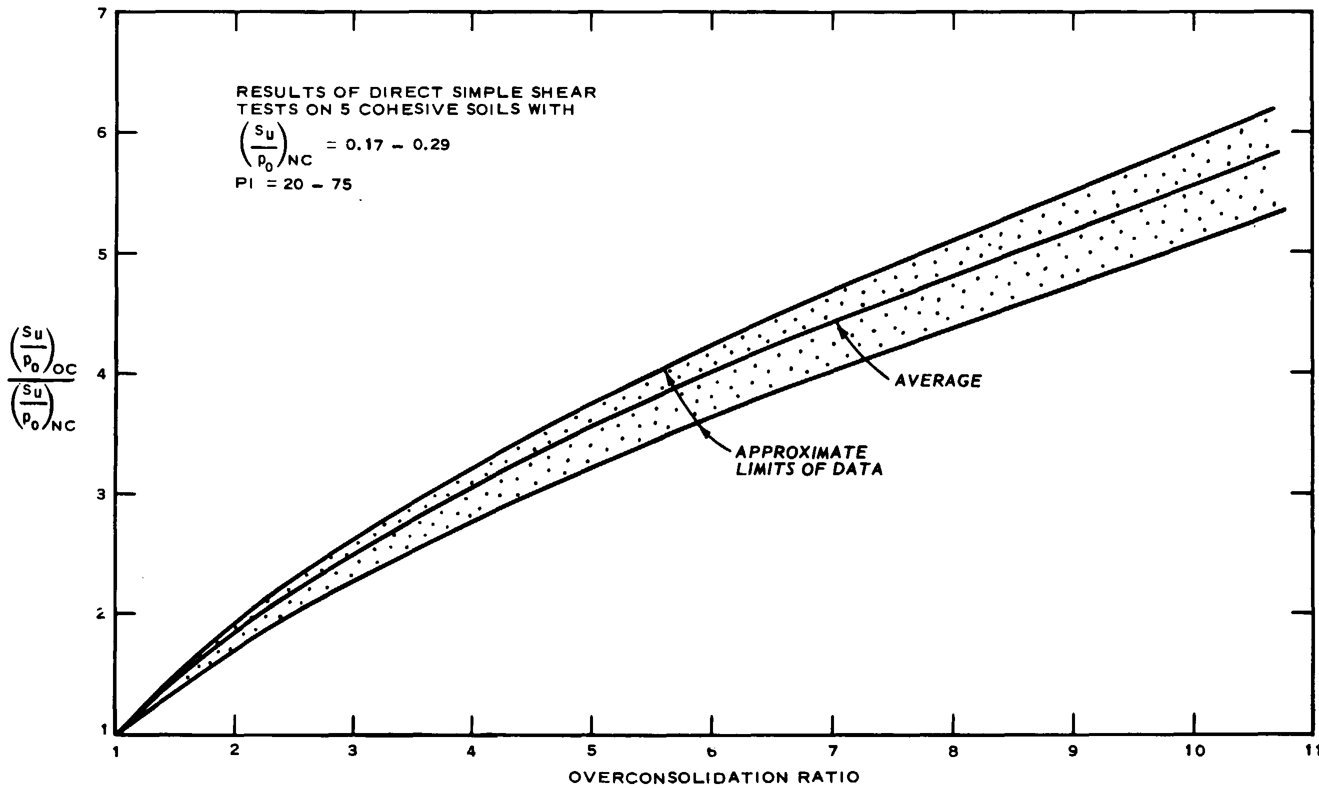
b. Undrained shear strength-cohesive soils. Approximate undrained shear strengths of fine-grained cohesive soils can be rapidly determined on undisturbed samples and occasionally on reasonably intact samples from drive sampling, using simple devices such as the pocket penetrometer, laboratory vane shear device, or the miniature vane shear device (Torvane). To establish the reliability of these tests, it is desirable to correlate them with unconfined compression tests. Unconfined compression tests are widely used because they are somewhat simpler than Q triaxial compression tests, but test results may scatter broadly. A more desirable test is a single Q triaxial compression test with the chamber pressure equal to the total in situ stress. Unconfined compression tests are appropriate primarily for testing saturated clays that are not jointed or slickensided. The Q triaxial compression test is commonly performed on foundation clays since the in situ undrained shear strength generally controls the allowable bearing capacity. Sufficient unconfined compression and/or Q tests should be performed to establish a detailed profile of undrained shear strength with depth. Undrained strengths may also be estimated from the standard penetration test, cone penetrometer soundings, and field vane tests, as discussed in chapter 4. For important structures, the effects of loading or unloading on the undrained shear strength should be determined by R (consolidated-undrained) triaxial compression tests on representative samples of each stratum.

c. Strength parameters, cohesive soils. The undrained shear strength of saturated clays can be expressed as



(Courtesy of W. N. Houston and J. K. Mitchell, "Property Interrelationships in Sensitive Clays," Journal, Soil Mechanics and Foundations Division, Vol 95, No. SM4. 1969, pp 1037-1062. Reprinted by permission of American Society of Civil Engineers. New York.)

Figure 3-15. Remoltded shear strength versus liquidity index relationships for different clays.



(Courtesy of C C. Ladd and R. Foott, "New Design Procedure for Stability of Soft Clays, " Journal, Geotechnical Engineering Division, Vol 100, No. GT7, 1974, pp 763-786. Reprinted by permission of American Society of Civil Engineers, New York.)

Figure 3-16. Normalized variation of s_u/p_0 ratio for overconsolidated clay.

$$s_u = c_u \quad \phi_u = 0 \qquad (3\text{-}11)$$

$$s_u = \frac{1}{2} (\sigma_1 - \sigma_3) = \frac{q_u}{2} \qquad (3\text{-}12)$$

and is essentially independent of total normal stress. The undrained cohesion intercept of the Mohr-Cou-lomb failure envelope is c_u.

(1) The undrained shear strength, s, , of normally consolidated cohesive soils is proportional to the effective overburden pressure, po. An approximate correlation is as follows:

$$\frac{s_u}{p_0} = 0.11 + 0.0037 \text{ PI} \qquad (3\text{-}13)$$

(2) A correlation between the remolded, undrained shear strength of clays and the liquidity index is shown in figure 3-15.

(3) A correlation between the normalized s_u/p_0 ratio of overconsolidated soils and the overconsolidation ratio (OCR) is presented in figure 3-16. The value of p_0 in (s_u/p_0)OC is the effective *present* overburden pressure. Values of (s_u/p_0) may be estimated from this figure when (s_u/p_o)NC and the OCR are known (NC signifies normally consolidated soils).

d. Sensitivity, cohesive soils. The sensitivity of a clay soil, St, is defined as follows:

$$S_t = \frac{\text{Undisturbed compressive strength}}{\text{Remolded compressive strength}}$$

(at same water content)

Terms descriptive of sensitivity are listed in table 3-5. Generalized relationships among sensitivity, liquidity index, and effective overburden pressure are shown in figure 3-17. The preconsolidation pressure, rather than the effective overburden pressure, should be used for overconsolidated soils when entering this figure. Cementation and aging cause higher values of sensitivity than given in figure 3-17.

e. Effective strength parameters, cohesive soils. As indicated in figure 3-14, the peak and residual strengths may be shown as failure and postfailure envelopes. Values of the peak drained friction angle for normally consolidated clays may be estimated from figure 3-18. After reaching the peak shear strength, overconsolidated clays strain-soften to a residual value of strength corresponding to the resistance to sliding on an established shear plane. Large displacements are necessary to achieve this minimum ultimate strength requiring an annular shear apparatus or multiple reversals in the direct shear box. Typical values of residual angles of friction are shown in figure 3-19.

f. Shear strength, cohesionless soils.

(1) In sandy soils, the cohesion is negligible. Because of the relatively high permeability of sands, the angle of internal friction is usually based solely on drained tests. The angle of internal friction of sand is primarily affected by the density of the sand and normally varies within the limits of about 28 to 46 de-

Table 3-5. Sensitivity of Clays

Sensitivity	Descriptive Term
0-1	Insensitive
1-2	Low sensitivity
2-4	Medium sensitivity
4-8	Sensitive
8-16	Extra sensitive
>16	Quick

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grees (fig 3-3). Approximate values of ϕ are given as follows:

ϕ = 30 + 0.15 DR for soils with less than 5 percent fines(3-14)

ϕ = 25 + 0.15 DR for soils with more than 5 percent fines(3-15)

Values of ϕ = 25 degrees for loose sands and ϕ = 35 degrees for dense sands are conservative for most cases of static loading. If higher values are used, they should be justified by results from consolidated- drained triaxial tests.

(2) Silt tends to be dilative or contractive depending upon the consolidation stresses applied. Thus, the back-pressure saturated, consolidated-undrained triaxial test with pore pressure measurements is used. If the silt is dilative, the strength is determined from the consolidated-drained shear test. The strength determined from the consolidated-undrained test is used if the silt is contractive. Typical values of the angle of internal friction from consolidated-drained tests commonly range from 27 to 30 degrees for silt and silty sands and from 30 to 35 degrees for loose and dense conditions. The consolidated-undrained tests yield 15 to 25 degrees. The shear strength used for design should be that obtained from the consolidated-drained tests.

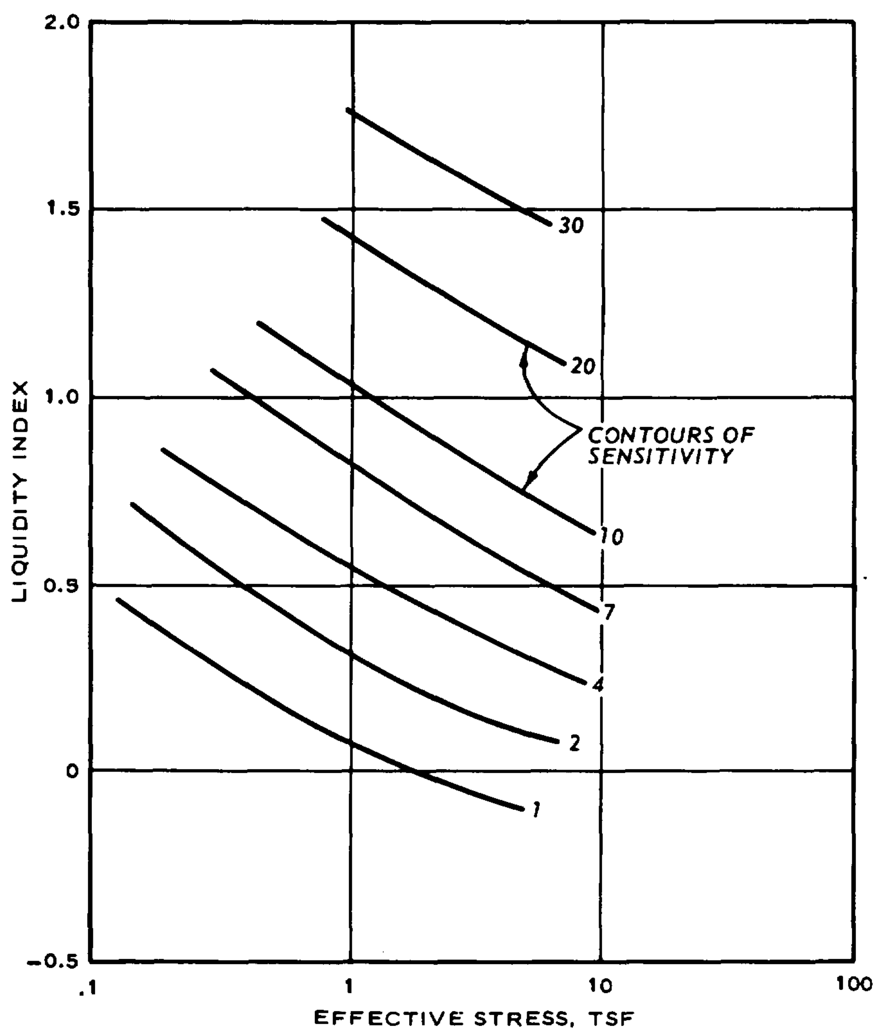

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Figure 3-17. General relationship between sensitivity, liquidity index, and effective overburden pressure.

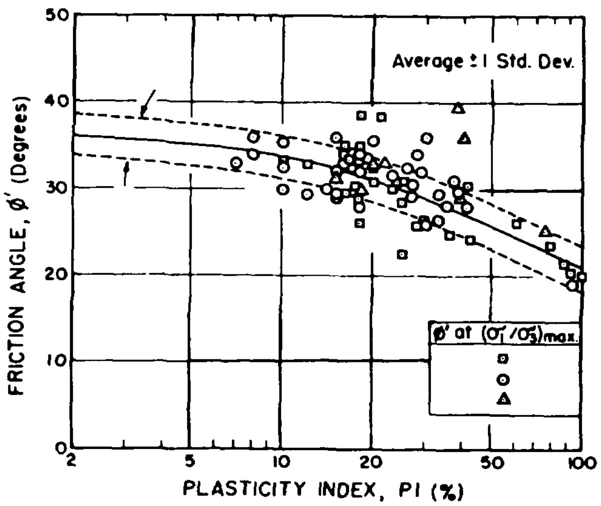

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Figure 3-18. Empirical correlation between friction angle and plasticity index from triaxial compression tests on normally consolidated undisturbed clays.

3-8. Elastic properties (E, μ). The elastic modulus and Poisson's ratio are often used in connection with the elasticity theory for estimating subsoil deformations. Both of these elastic properties vary nonlinearly with confining pressure and shear stress. Typical values given below refer to moderate confining pressures and shear stresses corresponding to a factor of safety of 2 or more.

a. In practical problems, stresses before loading are generally anisotropic. It is generally considered that the modulus of elasticity is proportional to the square root of the average initial principal stress, which may usually be taken as

$$\left(\sigma_v' \; \frac{1 + 2K_0}{3} \right)^{1/2} \qquad (3\text{-}16)$$

where K_0 is the coefficient of at-rest earth pressure (para 3-10) and δ_v' is the effective vertical stress. This proportionality holds for $0.5 < K_0 < 2$, when working stresses are less than one-half the peak strength.

b. The undrained modulus for normally consolidated clays may be related to the undrained shear strength, su, by the expression

$$\frac{E}{s_u} = 250 \text{ to } 500 \qquad (3\text{-}17)$$

where s_u is determined from Q tests or field vane shear tests. The undrained modulus may also be estimated from figure 3-20. Field moduli may be double these values.

c. Poisson's ratio varies with strain and may be as low as 0.1 to 0.2 at small strains, or more than 0.5.

3-9. Modulus of subgrade reaction.

a. The modulus of subgrade reaction, k_s, is the ratio of load intensity to subgrade deformation, or:

$$k_s = \frac{q}{\Delta} \qquad (3\text{-}18)$$

where

q = intensity of soil pressure, pounds or kips per square foot

Δ = corresponding average settlement, feet

b. Values of k. may be obtained from general order of decreasing accuracy:

(1) Plate or pile load test (chaps 4 and 12).

(2) Empirical equations (additional discussion in chap 10).

(3) Tabulated values (table 3-6).

3-10. Coefficient of at-rest earth pressure. The state of effective lateral stress in situ under at-rest conditions can be expressed through the coefficient of earth pressure at rest and the existing vertical overburden pressure. This ratio is termed Ko and given by the following:

$$K_o = \frac{\sigma_h'}{\sigma_v'} \qquad (3\text{-}19)$$

The coefficient of at-rest earth pressure applies for a condition of *no lateral strain.* Estimate values of K_0 as follows:

Normally consolidated soil

Sand:

$$K_0 = 1 - \sin \phi' \qquad (3\text{-}20)$$

Clay:

$$K_0 = 0.95 - \sin \phi' \qquad (3\text{-}21)$$

Figure 3-21 may be used for estimates of K_0 for both normally consolidated and overconsolidated soils in terms of PI. For overconsolidated soils, this figure applies mainly for unloading conditions, and reloading may cause a large drop in K_0 values. For soils that display high overconsolidation ratios as a result of desiccation, K_0 will be overestimated by the relationship shown in figure 3-21.

3-11. Properties of intact rock. The modulus ratio and uniaxial compressive strength of various intact rocks are shown in table 2 -7.

3-12. Properties of typical shales. Behavioral characteristics of shales are summarized in table 3-7; and physical properties of various shales, in table 3-8. Analyses of observed in situ behavior provide the most reliable means for assessing and predicting the behavior of shales.

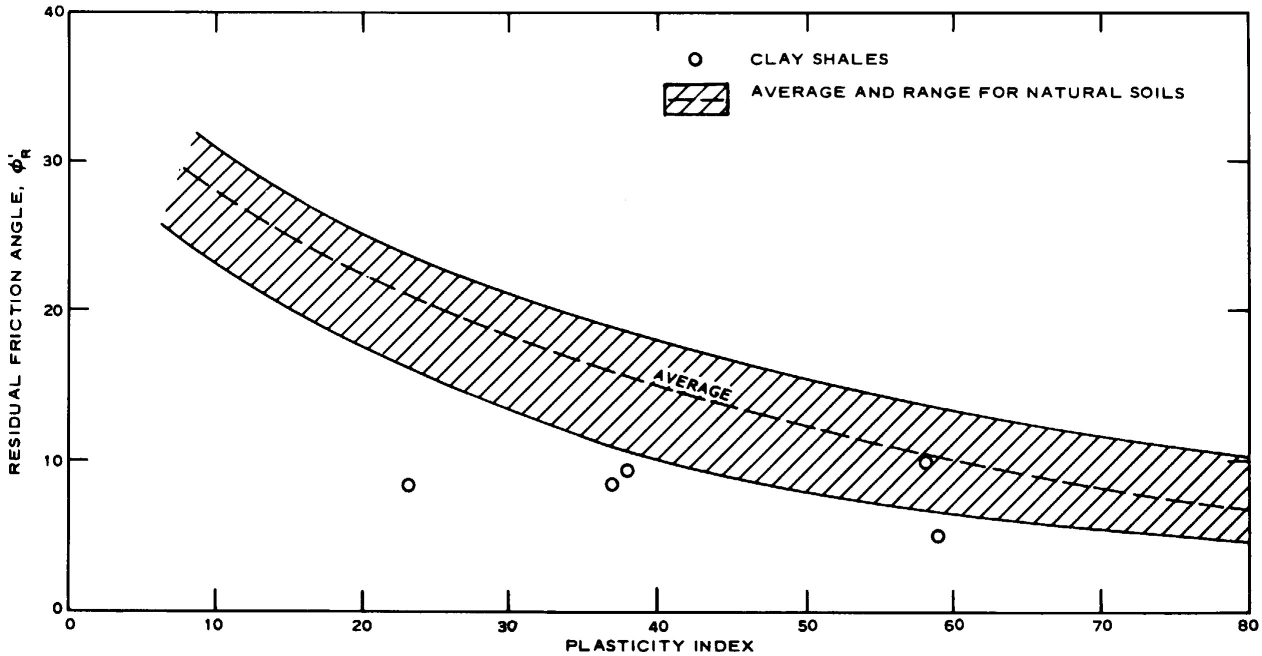

Figure 3-19. Relation between residual friction angle and plasticity index.

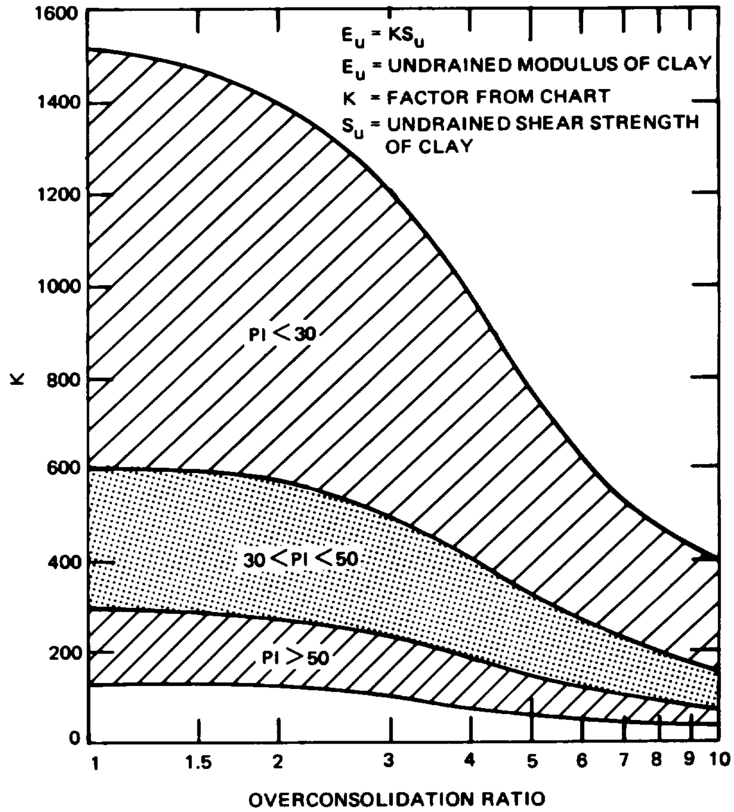

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Figure 3-20. Chart for estimating undrained modulus of clay.

Table 3-6. Values of Modulus of Subgrade Reaction (k_s) for Footings as a Guide to Order of Magnitude

Soil Type	Range of k_s, kcf[a]
Loose sand	30-100
Medium sand	60-500
Dense sand	400-800
Clayey sand (medium)	200-500
Silty sand (medium)	150-300
Clayey soil	
$q_u \leq 4$ ksf	75-150
$4 < q_u \leq 8$	150-300
$8 < q_u$	>300

[a] Local values may be higher or lower.

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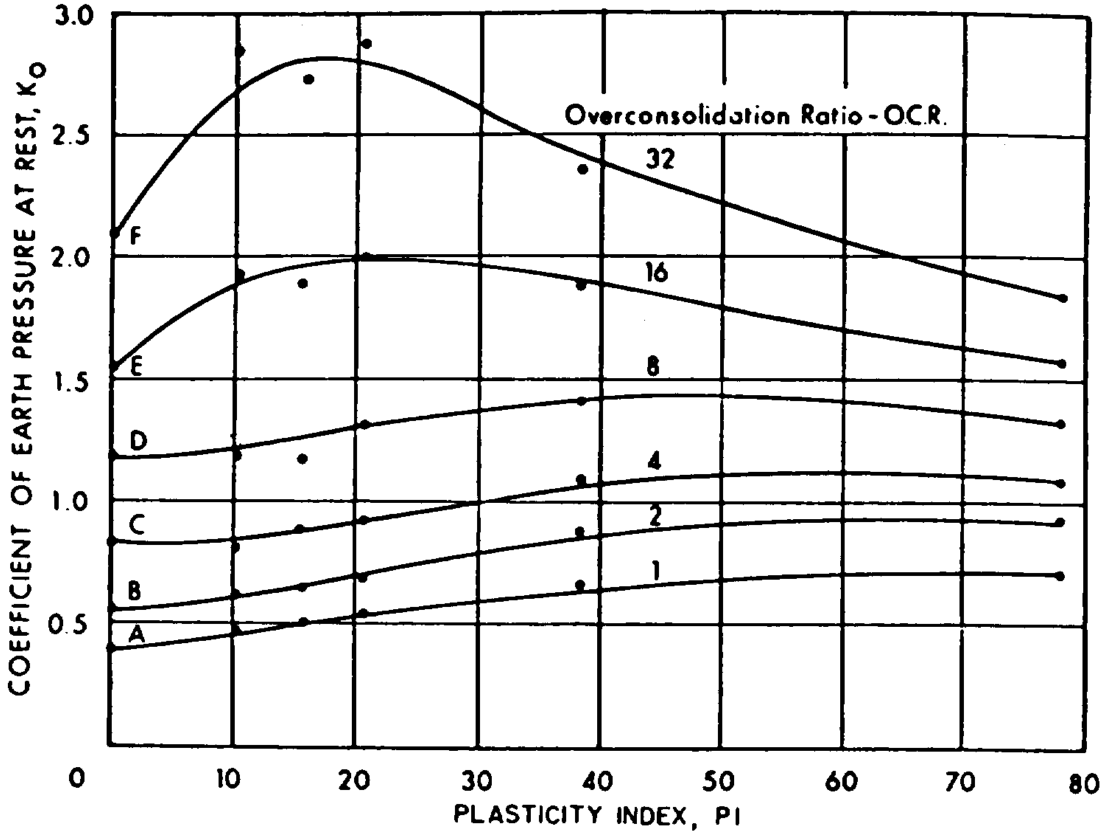

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Figure 3-21. Coefficient of earthpressure at rest (K_0) as a function of overconsolidated ratio and pasticity index.

Table 3- 7. An Engineering Evaluation of Shales

Physical Properties			Probable *In-situ* Behavior						
Laboratory tests and *in-situ* observations	Average range of values: Unfavorable	Average range of values: Favorable	High pore Pres-sure	Low bearing capacity	Tendency to rebound	Slope stability problems	Rapid slaking	Rapid erosion	Tunnel support prob-lems
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Compressive strength, in pounds per square inch	50 to 300		√	√					
		300-5000							
Modulus of elasticity, in pounds per square inch	20,000 to 200,000			√					√
		200,000 to 2×10^6							
Cohesive strength, in pounds per square inch	5 to 100				√	√			√
		100 to >1500							
Angle of internal friction, in degrees	10 to 20				√	√			√
		20 to 65							
Dry density, in pounds per cubic foot	70 to 110		√					√(?)	
		110 to 160							
Potential swell, in percentage	3 to 15				√	√		√	√
		1 to 3							
Natural Moisture content, in percentage	20 to 35		√			√			
		5-15							
Coefficient of permeability, in centimeters per second	10^{-5} to 10^{-10}		√			√	√		
		$>10^{-5}$							
Predominant clay minerals	Montmorillonite or Illite		√			√			
		Kaolinite & Chlorite							
Activity ratio $= \dfrac{\text{Plasticity index}}{\text{Clay content}}$	0.75 to >2.0					√			
		0.35 to 0.75							
Wetting and drying cycles	Reduces to grain sizes						√	√	
		Reduces to Flakes							
Spacing of rock defects	Closely Spaced			√		√		√(?)	√
		Widely Spaced							
• Orientation of rock defects	Adversely Oriented			√		√			√
		Favorably Oriented							
State of stress	>Existing Over-burden Load				√	√			√
		≅ Over-burden Load							

(Courtesy of L. B. Underwood, "Classification and Identification of Shales," Journal, Soil Mechanics and Foundations Division, Vol 93, No. SM6, 1967, pp 97-116. Reprinted by permission of American Society of Civil Engineers, New York.)

Table 3-8. Physical Properties of Various Shales

Name of Shale Formation, Age, Locality (1)	Compressive strength, in pounds per square inch (2)	Modulus of Elasticity, in pounds per square inch (3)	Cohesion, in pounds per square inch (4)	Angle of Internal Friction, in degrees (5)	Dry Density, in pounds per cubic foot (6)	Potential Swell in percentage (7)	Natural Moisture in percentage (8)	Predominant Clay Minerals (9)	Activity Ratio (10)
Bearpaw, Cret., Canada									
Weathered-	7 to 84	7500	3 to 6	6 to 20	85-95	0.5%-2%	29 to 36	Illite, Montmorillonite	0.30 to
Unweathered-	154-406	18,000	22	30	95-108	5%-20%	19 to 27	Mixed-layer	>1.5
Pierre, Cret., So. Dakota	70-1400	20,000-140,000	2 to 30	8-25	95-110	3% to 5%	18 to 27	do	0.3 to >2
Ft. Union, Tert., No. Dakota	70-1050	11,200-56,000	10	20	95-115	2% (?)	16-24	Illite	—
Pepper, Cret., Texas	28-70	—	2-6	7-14	110	—	20%	Illite Mont. Kaolinite	1.2
Del Rio, Cret., Texas	56-154	—	1-8	19-28	119	—	17%	Illite, Kaolinite, Mont.	1.0
Trinity, Cret., Texas	30-170	2,400-33,000	0-7	26	115-133		11-17%		
Taylor, Cret., Texas	250-550	6,000-20,000	1.5-25	8-30	112-118	—	15%-18%	—	—
Composite Cyclothem of Pennsylvanian Shales (Eastern Ohio and Western Penn.) — Silty Clayey	210	1,000,000	56	23	138		9.1		
Composite Cyclothem … — Carbonaceous	4165	—	1562	16	—	—	—	—	—
Composite Cyclothem … — Clay Bonded	2084	—	931	7	—	—	—	—	—
Composite Cyclothem … — Clayey Ferruginous	1661	—	488	29	—	—	—	—	—
Composite Cyclothem … — Sandy	3674	486,500	1600	9	—	—	—	—	—
Niobrara, Calc. Sh., Cret., Colo.								Illite, Beid.	
Mowry, Cret., Colo.								Kaolinite, Chlorite Illite	
Graneros, Cret., Colo.								Kaol., Ill., Mix-layer	
Morrison, Jura, Colo.								Mont., Illite	
Laramie, Cret., Colo.								Illite, Beid.	
Mauv, Calc. Shale, Cam., Utah	5220	2.3×10^6	1,160	64	164	—	2%	—	—
Quartzose Sh., Cam., Utah	17,770	2.3×10^6	3,390	45	165	—	4%	—	—

1 Ton/sq. ft. = approx. 14 lbs. per $inch^2$

(Courtesy of the American Society of Civil Engineers)