



PDHonline Course C272 (8 PDH)

Soil Properties and the Unified Soil Classification System (USCS)

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A. Identification and Classification

1. General. Most soils are a heterogeneous accumulation of mineral grains that are not cemented together. However, the term "soil" or "earth" as used in engineering includes virtually every type of uncemented or partially cemented inorganic and organic material in the exposure, is wholly excluded. In the design and construction of foundations and earthwork, the physical and engineering properties of soils, such as their density, permeability, shear strength, compressibility, and interaction with water, are of primary importance. A standard method is used for identifying and classifying soils into categories or groups that have distinct engineering characteristics. This enables a common understanding of soil behavior just by knowing the classification. Written exploration records (logs) as shown on figure 1 contain soil classifications and descriptions; they can be used to:

- Make preliminary estimates.
- Determine the extent of additional field investigations needed for final design.
- Plan an economical testing program.
- Extend test results to additional explorations.

For final design of important structures, visual soil classification must be supplemented by laboratory tests to determine soil engineering properties such as permeability, shear strength, and compressibility under expected field conditions. Knowledge of soil classification, including typical engineering properties of various soil groups, is especially valuable when prospecting for earth materials or investigating foundations for structures.

2. Unified Soil Classification System (USCS). In 1952, the Bureau of Reclamation (Reclamation) and the U.S. Army Corps of Engineers, with Professor Arthur Casagrande of Harvard University, reached agreement on modifications to his "Airfield Classification System" and named it: *Unified Soil Classification System* (USCS). During World War II, Professor Casagrande developed the "Airfield Classification System" for categorizing soils based on engineering properties related to airfield construction. After the war, Reclamation began using the system, which led to the modifications agreed upon in 1952. From 1952 to 1986, the system required minor changes particularly in presenting information in written logs. In 1984, American Society for Testing and Materials (ASTM), with encouragement and participation of several U.S. Government agencies, reevaluated the Unified Soil Classification System. New data and philosophies resulted in revised ASTM standards, which modified the 1952 system. While the soil classification system has evolved over many years, existing soil classifications and descriptions recorded remain valid. Therefore, these earlier logs and past descriptions must not be changed to conform to current standards.

3. Current Classification System. The Unified Soil Classification System has been through several transitions since it was developed. The current version of the USCS went into effect January 1, 1986. These procedures are found in ASTM D 2487 and D 2488; Standard formats for written logs of test pits and auger holes have been established. Uniformity of data presentation results in consistency and completeness, which enables efficient review of large numbers of logs.

7-1336-A (1-86) Bureau of Reclamation		LOG OF TEST PIT OR AUGER HOLE		HOLE NO. <u>EXAMPLE</u>	
FEATURE _____		PROJECT _____		GROUND ELEVATION <u>1234.7 ft</u>	
AREA DESIGNATION <u>Spillway Foundation</u>		METHOD OF EXPLORATION <u>backhoe</u>		LOGGED BY _____	
COORDINATES N <u>1111</u> E <u>2222</u>		APPROXIMATE DIMENSIONS <u>8 by 12 ft</u>		DATE(S) LOGGED <u>1-25 to 1-26, 1986</u>	
DEPTH WATER ENCOUNTERED <u>1/ NONE</u> DATE _____					
CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME)			
		3-5 in	5-12 in	PLUS 12 in	
CL in-place unit weight three sack samples 4.3 ft	0.0 to 4.3 ft LEAN CLAY: About 90% fines with medium plasticity, high dry strength, medium toughness; about 10% predominantly fine sand; maximum size, coarse sand; no reaction with HCl. IN-PLACE CONDITION: Firm, homogeneous, moist, reddish-brown In-place dry unit weight and moisture from test at 3.0 to 3.7 ft: 112.0 lbf/ft ³ , 11.7%. Three 50-lbm sack samples taken for testing from 18-inch-wide sampling trench for entire depth interval on east side of trench. Samples were mixed and quartered.				
SC 8.2 ft	4.3 to 8.2 ft CLAYEY SAND WITH COBBLES: About 55% coarse to fine, hard, subrounded sand; about 35% fines with medium plasticity, medium toughness; about 10% coarse to fine, hard, subrounded gravel; weak reaction with HCl. TOTAL SAMPLE (BY VOLUME): About 5% 3- to 5-inch hard, subrounded cobbles; remainder minus 3 inch; maximum dimension, 100 mm. IN-PLACE CONDITION: homogeneous, moist to 7.4 ft, wet 7.4 to 8.2 ft, gray; some mica present.	5			
REMARKS.		Date	Depth to water		
* Test pit backfilled before stable water level could be determined		01-31-86 8:30	7.4 ft		
		01-31-86 1:30	5.2 ft		
		01-31-86 5:00	4.7 ft*		

Figure-1.—Typical soil exploration log.

When preparing a log, the object is to present as complete and as clear a description as possible.

4. Basis of Unified Soil Classification System. The USCS is based on engineering properties of a soil; it is most appropriate for earthwork construction. The classification and description requirements are easily associated with actual soils, and the system is flexible enough to be adaptable for both field and laboratory use.

The USCS is a method for describing and categorizing a soil within a group that has distinct engineering properties. Upon recognizing a USCS symbol of a classification group or understanding the description, one can immediately deduce the approximate permeability, shear strength, and volume change potential of a soil and how it may be affected by water, frost, and other physical conditions. Also, using soil classification symbols can assist an engineer in estimating excavation and compaction characteristics, potential dewatering situations, and workability.

The USCS is based on: (a) the distribution (gradation) of various particle sizes, and (b) the plasticity characteristics of very fine particles. Particle-size distribution is determined from a gradation analysis test. In this test, soil is dried and then shaken through a series of sieves having progressively smaller openings. The soil mass retained on each sieve is measured, and the percentage that passes the different openings can be calculated.

The gradation analysis test is discussed in part B of this course. Plasticity characteristics of the fines (particles that pass a U.S.A. Standard 75- μm [No. 200] sieve and smaller) are determined by performing the Atterberg limits tests. These tests are performed to determine the moisture content of a soil when in a liquid, plastic, semisolid, or solid state. The tests are discussed in part B of this course. Gradation analysis data are used to determine percentages of gravel, sand, and fines in a soil; Atterberg limits are used to determine whether the fines are silt or clay. The four basic soil types are: **gravel, sand, silt, or clay**. Because soil can contain all of these components, the classification system is used to describe the major components; for example, clayey sand with gravel, sandy silt, or silty clay with sand. The percentages and behavior of each component can be determined from laboratory tests or they can be estimated by visual and manual tests in the field. However, in the USCS system, the method of classification must be stated, whether it is based on visual observations or on laboratory tests.

Unique factors must be realized when comparing the USCS system with other classification methods. First, the USCS is based only on the particles passing a 75-mm (3-in) sieve opening. While particles larger than 75 mm are not used to classify soil, they are of great importance in engineering use of a soil in earthwork. Therefore, cobbles and boulders are included in USCS names, and they are fully described in the narrative portion of field logs.

Second, silt and clay are differentiated by their physical behavior and not by their particle size. Other classification systems define silts and clays as having specific particle sizes with silts being the larger particle size. As a system based on engineering properties, the USCS is concerned only with plasticity characteristics of the fines and not the particle sizes.

5. Gravel, Sand, Silt, and Clay. The relative amounts of gravel, sand, and fines; the plasticity characteristics of the fines (silt, clay, or both); and the presence of cobbles and boulders all affect the soil's use as a construction material. These characteristics also affect selection of laboratory and field tests to be performed on the soil.

Gravel particles are those passing a 75-mm (3-in) sieve but retained on a 4.75-mm (No. 4) sieve. Sand particles pass a 425- μm sieve and are retained on a 75- μm (No. 200) sieve. Fines are soil particles that pass a 75-mm sieve; they are further characterized as silt or clay, based on their plasticity.

a. **Gravel and Sand.** Gravel and sand have essentially the same basic engineering properties, differing mainly in degree. The division of gravel and sand sizes by the 4.75-mm sieve is arbitrary and does not correspond to an abrupt change in properties. When devoid of fines, coarse-grained soils are pervious, easy to compact, and have little affect by moisture or frost action. Although particle shape, angularity, gradation, and size affect the engineering properties of coarse-grained soils, gravels are generally more pervious, more stable, and less affected by water or frost than are sands containing the same percentage of fines.

As a sand becomes finer and more uniform, it approaches the characteristics of silt, with corresponding decreased permeability and reduced stability in the presence of water. Visually, very fine sands are difficult to distinguish from silts. However, dry sand exhibits no cohesion (does not hold together) and feels gritty in contrast to the very slight cohesion and smooth feel of dried silt.

b. Silt and Clay (Fines). In soils, small amounts of fines may have important effects on engineering properties. As little as 10 percent of particles smaller than the 75-mm sieve size in sand and gravel may make the soil virtually impervious, especially when the coarse grains are well graded. Also, in well-graded sands and gravels, serious frost heaving may be caused by less than 10-percent fines. The utility of coarse-grained materials for surfacing roads, however, can be improved by adding a small amount of clay. This acts as a binder for the sand and gravel particles.

Soils containing large quantities of silt and clay are the most troublesome in engineering. These materials exhibit marked changes in physical properties with change of moisture content. For example, a hard, dry clay may be suitable as a foundation for heavy loads as long as it remains dry, but may turn into a quagmire when wet. Many fine-grained soils shrink on drying and expand on wetting; this may adversely affect structures founded upon them or constructed of them. Even when moisture content does not change, the properties of fine-grained soils may vary considerably between their natural condition in the ground and their state after being disturbed. Fine-grained soils, having been subjected to loading in geologic time, frequently exhibit a structure that gives the material unique properties in the undisturbed state. When soil is excavated for use as a construction material or when a natural deposit is disturbed, for example, by driving piles, soil structure is destroyed, and soil properties are radically changed.

Silts differ from clays in many respects, but because of similarity in appearance, often they are mistaken for each other. Dry, powdered silt and clay are indistinguishable from each other visually but are easily identified by their characteristics when wet or moist. Recognition of fines as having either silt or clay behavior is an essential part of the Unified Soil Classification System.

Silts are slightly plastic or nonplastic fines. They are inherently unstable when wet and have a tendency to become "quick" when saturated and unconfined, assuming the character of a viscous fluid, and commonly flow. Silts are fairly impervious, difficult to compact, and highly susceptible to frost heaving. Silt masses undergo change of volume with change of shape (the property of dilatancy), in contrast to clays, which retain their volume with change of shape (the property of plasticity). Silts vary in size and shape of particles; this is reflected mainly in compressibility. For similar conditions of previous geologic loading, the higher the liquid limit of a silt, the more compressible it becomes.

Clays are plastic fines. They have low resistance to deformation when wet but dry to hard, cohesive masses. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristic of clays. The small size, flat shape, and mineral composition of clay particles combine to produce a material both compressible and plastic. The higher the liquid limit of a clay, the more compressible it will be when compacted. Hence, in the Unified Soil Classification System, the liquid limit is used to distinguish between clays of high compressibility and those of low compressibility. Differences in plasticity of clays are reflected by the plasticity index. At the same liquid limit, the higher the plasticity index, the more cohesive the clay.

c. Organic Matter. The effect of organic matter upon engineering properties is recognized in the USCS, and organic soils are part of the classification system. Organic matter in the form of fully or partly decomposed vegetation can be found in many soils. Varying amounts of finely divided vegetable matter are found in sediments and often affect sediment properties sufficiently to influence their classification. Thus, the USCS recognizes organic silts having no or low plasticity and organic clays having medium to high plasticity. Even small amounts of organic matter in colloidal form in a clay may result in an appreciable increase in liquid limit (and plastic limit) of the material without increasing its plasticity index.

Generally, organic soils are dark gray or black in color and usually have a characteristic decaying odor. Organic clays feel spongy in the plastic range when compared to inorganic clays. The tendency for soils high in organic content to form voids through decay or to change the physical characteristics of a soil mass through chemical alteration makes them undesirable for engineering use. Soils containing even moderate amounts of organic matter are significantly more compressible and less stable than inorganic soils; hence, they are less desirable for engineering use.

6. Cobbles and Boulders. While particles of rock retained on a 75-mm (3-in) sieve are not considered when classifying a soil according to the USCS, they can have a significant effect on the construction use of a soil and may need to be removed. The presence of cobbles and boulders must *always* be noted in the group name of a soil and in any description of the soil. Cobbles pass a 300-mm (12-in) square opening but are retained on a 75-mm sieve. Boulders are too large to pass a 300-mm square opening. Note that for engineering use, the definitions of cobbles and boulders refer only to size. Some other definitions consider particle angularity.

For soils that may be used in compacted embankments or compacted backfill, cobbles must be identified as being larger or smaller than 125 mm (5 in) in diameter, as a typical compacted layer of cohesive soil is restricted to about 150 mm (6 in) in thickness. The maximum particle size allowed in the compacted layer is 125 mm. Thus, particles larger than 125 mm must be removed or separated. Logs and soil descriptions must state the percentage of cobbles smaller than and the percentage larger than 125 mm.

7. Shale, Crushed Rock, and Slag. The USCS can also be used for describing materials such as shale, siltstone, claystone, mudstone, sandstone, crushed rock, slag, cinders, and shells. These materials are not considered natural soils but they can be used as construction materials.

In some cases, materials variously described as shale, claystone, mudstone, or siltstone can be excavated with construction equipment and then broken down using diskings or other methods into a material that can be used to construct embankments or other earthworks. In the laboratory, these materials can be processed into a "soil" by grinding or slaking. They can then be classified according to the USCS to evaluate their properties as a construction material. However, these materials should *not* be primarily classified or described as soil because that is not their natural state.

To avoid misinformation, any log, report of laboratory test results, or description(s) must identify the original material as shale, claystone, etc. However, in the description of material properties, a USCS soil classification symbol and name may be used. The classification is secondary, and the symbol and name are to be enclosed in quotation marks to distinguish them from a typical soil classification as shown in the following example:

SHALE CHUNKS: Retrieved as 50 to 100-mm pieces of shale from power auger hole; dry, brown; no reaction with HCl. After a sample was laboratory processed by slaking in water for 24 hours, the sample was classified as "SANDY LEAN CLAY (CL)", 61-percent clayey fines, LL = 37, PI = 16; 33-percent fine to medium sand; 6-percent gravel-size pieces of shale.

The same format applies to other materials not considered to be natural soils, but which are used in construction such as sandstone, crushed rock, slag, cinders, and shells.

For some materials, the laboratory processing required to test the material as a soil may be significantly different than the processing at the project site using construction equipment. The differences must be considered when evaluating the material for construction. Test sections are often used as a final evaluation of whether to accept or reject these materials.

8. Soil Composition. In addition to soil classification, knowledge of the mineralogy and origin of soils can aid in evaluating the behavior of a soil. While silt particles and sand-size particles are generally equidimensional, clay particles are very small and are flaky, or platelike, as shown on figure 2. As clay components of soils become more predominant, the clay mineral characteristics assume great importance. Soil properties such as consistency, shear strength, and moisture content are directly influenced by the mineral constituents of the clay. When soils are fairly moist, clay particles are surrounded by water films. As dehydration takes place, the films become thinner and thinner until adjacent particles are held together by strong cohesive (capillary) forces. As soils are wetted, the films become weaker. Film strength also is related to the fineness and specific surface of the material and to compactness (density) which influences size of void spaces.

The mineralogic clays in soils affect their physical behavior. Two groups of clay minerals are of particular interest, the kaolin group and the montmorillonite group. The kaolin minerals have fixed crystal lattices or layered structure and exhibit only a small degree of hydration and absorptive properties. In contrast, montmorillonite minerals have expanding lattices and exhibit a higher order of hydration and cation absorption. Montmorillonite soils, with their expanding lattice structure and resulting capacity for wide ranges of water contents, can be particularly troublesome. Settlement from shrinkage, heave from swelling, and loss of strength and stability caused by shrinkage or swelling can create major structural problems. These problems are greatly magnified in the case of hydraulic structures.

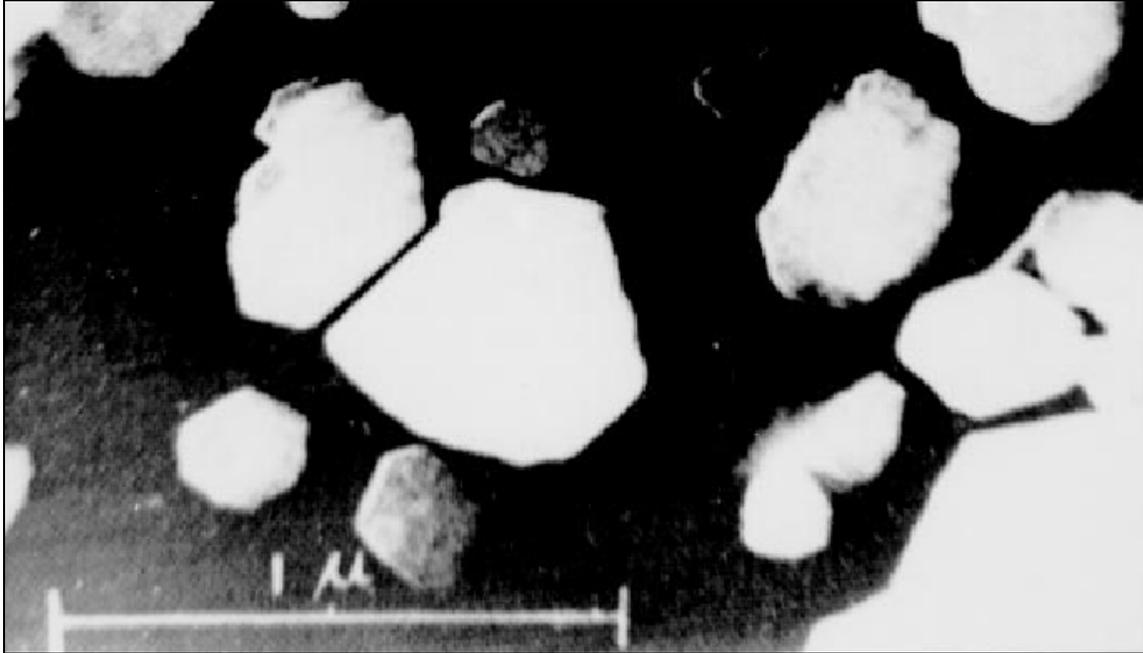


Figure 2.—Clay kaolinite particles (1) = 0.001 mm).

The clay mineral halloysite, illustrated on figure 3, can cause construction problems. Typically, it is a hollow, rodshaped particle and decreases soil density. The soil changes engineering behavior as its structure changes because of particle breakdown during drying or construction and the irreversible process of water being removed from inside the tubal particles.

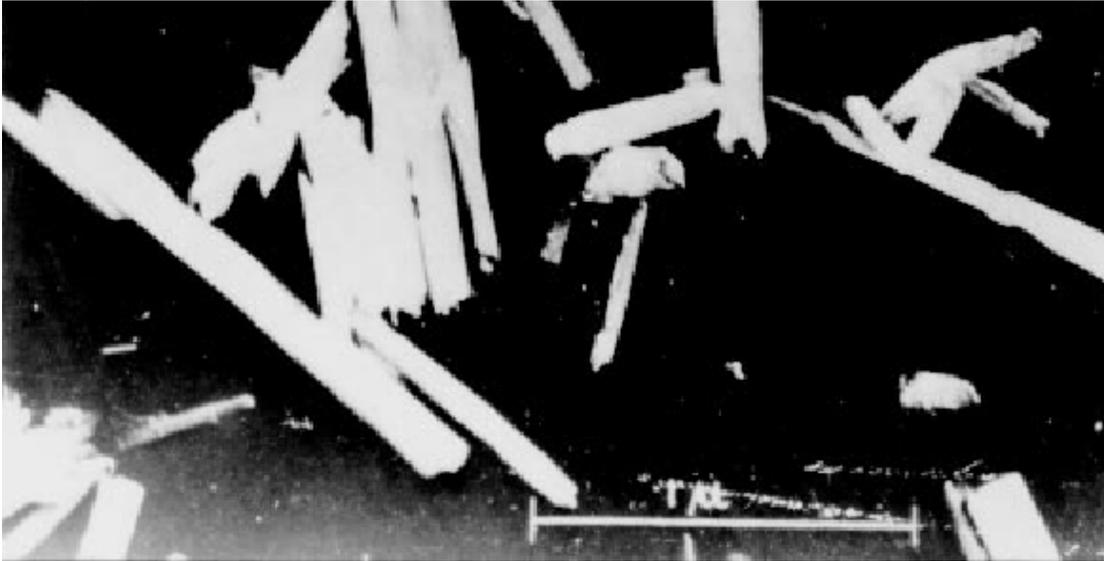


Figure 3.—Halloysite particles (1) = 0.001 mm).

The presence of mica in soils causes a highly compressible material. The thin, flaky particles act like springs that separate other soil particles, thus creating low densities and also deforming under load. This occurrence is especially important in sands containing mica.

Many low-density deposits are found in the arid and semiarid parts of the United States. These soils are unsaturated deposits of loose, wind-deposited loess and loess-like soils or colluvial and alluvial soils deposited by flash runoffs, often in the form of mudslides. In none of these situations have the soils been completely wetted or worked to allow breakdown and consolidation of the loose structure. Generally, these soils have high dry strengths created by a well dispersed clay binder. Major wind-blown loess deposits are found in the Plains States of Kansas and Nebraska and can be found in other areas. Loessial soils form high vertical faces that are stable as long as the water content is low. However, upon wetting, strength is largely lost and slope failures occur. Similarly, loessial soils support heavy structural loads on footings or pilings when dry; however, they lose their bearing capacity and resistance to compression when their loose structure collapses upon becoming wet.

Certain alluvial fan soil deposits such as those adjacent to the southwestern foothills of the Central Valley of California have similar characteristics. When dealing with hydraulic engineering works, where the subsoils will eventually become saturated, it is imperative to recognize these soils' characteristics and to take precautionary measures to improve them before constructing structures on them. Many engineering problems associated with the effects of mineralogy and origin are discussed later in more detail in this course.

B. Index Properties of Soils

9. General. Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail, and precision required for engineering properties tests. These simplified *index tests* or *physical properties tests* provide indirect information about engineering properties of soils; the most widely used are gradation analyses and Atterberg limits tests. Both tests define the limits of the various groups of soils when using the USCS. Moisture content and density relationships are commonly used as index properties in evaluating foundations and for construction control. Other index tests include:

- Laboratory penetration resistance needle tests on compacted specimens
- Field penetration resistance tests with split-tube drive samplers
- Unconfined compression tests on undisturbed or compacted samples of fine-grained cohesive soil

Data secured from index tests, together with descriptions of visual observations, are often sufficient for design purposes for minor structures. This information is used also in making preliminary designs for determining probable cost of a major structure and to limit the amount of detailed testing. An assumption is that construction materials within a limited area, having similar index properties, will exhibit similar engineering properties; however, correlations between index properties and engineering properties are not perfect. If index properties are used in design, a liberal factor of safety should be included.

10. Terms and Units of Measure. The terms and units of measure used here follow.

a. Mass. *Mass*, the amount of matter an object contains, remains constant even if temperature, shape, or other physical attributes of the object change. The object's mass does not depend on local gravitational attraction and is independent of the object's location in the universe. The output or reading from *any* type of balance or scale is *mass* not weight. Typical units of mass are gram (g) or kilogram (kg) and pound mass (lbm).

b. Density. *Density* is mass per unit volume. The symbol for density is the lowercase Greek letter rho, ρ . Units of density are megagram per cubic meter (Mg/m³) and pound mass per cubic foot (lbm/ft³).

c. Weight. *Weight* is the gravitational force that causes a downward acceleration of an object. Units of weight are the same as units of force: newton (N) and pound force (lbf).

d. Unit Weight. *Unit weight* is weight per unit volume. The symbol for unit weight is the lowercase Greek letter gamma, γ . Units of unit weight are kilonewton per cubic meter (kN/m³) and pound force per cubic foot (lbf/ft³).

e. Percentage and Decimal Use. Certain terms pertaining to soil properties are customarily expressed as a percentage, whereas others are usually expressed as decimals. For example, degree of saturation (*S*), moisture content (*w*), and porosity (*n*) are commonly written in percentage as:

$$S = 85.6\%, w = 16.2\%, \text{ and } n = 34.3\%$$

Conversely, void ratio (*e*) and specific gravity (*G_s*) are expressed in decimals as:

$$e = 3.52, \text{ and } G_s = 2.76$$

To avoid confusion in computations involving these quantities, a simple rule should be followed: Always express quantities as a decimal in all computations. The answers can be given as percentages provided the percent sign is used, as shown in the following example.

Given:

$$\gamma_d = 113.2 \text{ lbf/ft}^3, w = 16.2\%, G_s = 2.76, \text{ and } n = 34.3\%.$$

Find the degree of saturation, *S*

$$S = \frac{wG_s}{n} = \frac{0.162 \times 2.76}{0.343} = 0.856 \text{ or } 85\%$$

11. Gradation. Gradation is a descriptive term that refers to distribution and size of grains in a soil. It is determined by the gradation analysis of soils, and is presented in the form of a cumulative, grain-size curve in which particle

sizes are plotted logarithmically with respect to percentage (by dry mass) of the total specimen plotted to a linear scale (fig. 4).

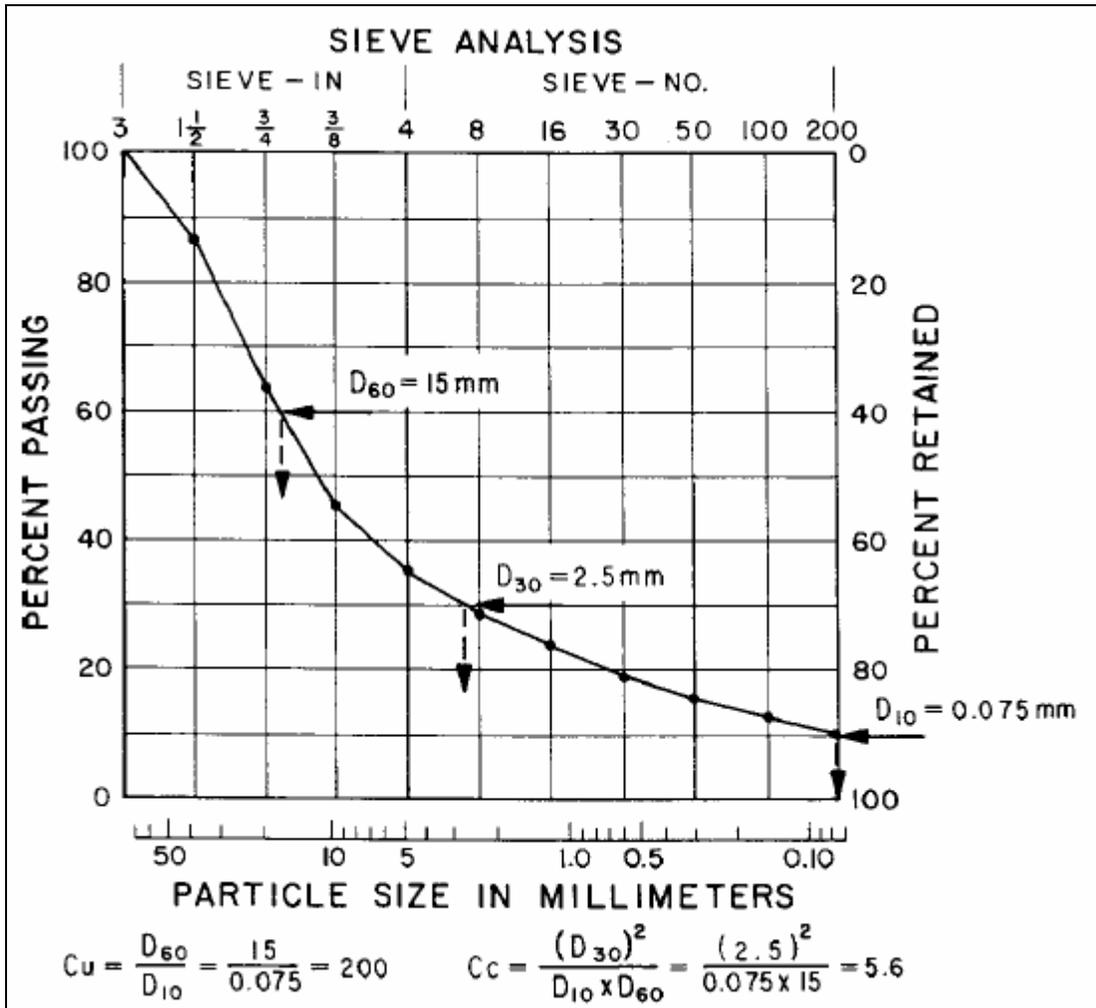


Figure- 4.—Gradation plot

Certain terms and expressions are used when referring to a gradation curve. A particular diameter of particle is indicated by *D* with a numeric subscript that corresponds to a point on the *curve* equivalent to the percentage of particles passing. On figure 4, *D*₁₀ equals 0.075 mm. This means 10 percent, by dry mass of soil, is composed of particles smaller than 0.075 mm. The 10-percent size (*D*₁₀ size) is also called the "effective" grain size. This term was introduced by Hazen in connection with his work on sanitary filters. Hazen found that sizes smaller than the effective grain size affected the functioning of filters as much or more than did the remaining 90 percent of sizes. Other sizes, such as *D*₁₅, *D*₅₀, and *D*₈₅, are also used in filter design. The sizes *D*₁₀, *D*₃₀, and *D*₆₀ are used in defining the gradation characteristics of a soil. The gradation curve is used to designate various soil components by grain size. A soil is said to be *well graded* when a good representation of all particle sizes exists from the largest to the smallest.

A soil is considered to be *poorly graded* if an excess or a deficiency of certain particle sizes occurs within the limits of the minimum and maximum sizes or if the range of predominant sizes falls within three or less consecutive sieve-size intervals on the gradation curve. A poorly graded soil is called *uniform* if all the particles are about the same size. When there is an absence of one or more intermediate sizes, the material is said to have a *gap* or *skip* gradation.

To determine whether a material is well graded or poorly graded, coefficients describing the extent and shape of the gradation curve have been defined as follows and illustrated on figure 4:

$$\text{Coefficient of uniformity, } C_u = \frac{D_{60}}{D_{10}}$$

$$\text{Coefficient of curvature, } C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

To be well graded, a material must have a coefficient of curvature between 1.0 and 3.0; in addition, the coefficient of uniformity must be greater than 4.0 for gravels and greater than 6.0 for sands. If one or both of these criteria are *not* satisfied, the soil is poorly graded. A poorly graded soil having a coefficient of uniformity of 2.0 or less is uniform

A soil sample may be divided into parts called *fractions based* on grain size. For example, in construction control of earthwork, soil is frequently divided into two fractions using the U.S.A. Standard 4.75-mm sieve (No. 4).

The fraction finer than the 4.75-mm sieve (the minus 4.75-mm fraction) is called the *control fraction*, and the fraction coarser than the 4.75-mm sieve (the plus 4.75- mm fraction) is called the *oversize*.

A number of different upper-size limits are frequently used in appraising soils. The workability of a soil is often affected appreciably by the largest soil particles present. Important upper-size limits for various applications are:

U.S.A. Standard Series Sieve Size		Application
300 mm	12 inch	Sand, gravel, and cobble fill; division between cobbles and boulders
125 mm	5 inch	Impervious rolled fill
75 mm	3 inch	Soil classification, large-scale laboratory testing, division between gravel and cobbles
4.75 mm	No. 4 sieve	Most laboratory testing, division between sand and gravel
425 μm	No. 40 sieve	Atterberg limits test
75 μm	No. 200 sieve	Division between fines and sand

Use of gradation in determining engineering properties is limited to coarse-grained materials. In poorly graded uniform materials, permeability increases approximately as the square of the effective grain size (D_{10}). For such materials, compressibility is normally small except for in very fine sands. Shear strength consists almost wholly of internal friction and is more or less independent of grain size. Uniform materials are generally workable; that is, they are easily excavated and compacted. As the range in sizes of coarse-grained soils increases:

- permeability decreases,
- compressibility decreases, and
- shear strength increases.

Workability of well-graded soil is good.

Amount and plasticity of fines influence the properties of coarse-grained materials. Permeability is reduced with increasing quantity of fines rapidly with a small quantity and at a slower rate with a large quantity. Compressibility and shear strength are affected only slightly by small percentages of fines in coarse-grained soils, but the effects increase with increase in fines.

12. Atterberg Limits. The physical properties of most fine-grained soils, and particularly clayey soils, are greatly affected by moisture content. The consistency of a clay may be very soft, that is, a viscous liquid; or it may be very hard, having the properties of a solid depending on its moisture content. In between these extremes, clay may be molded and formed without cracking or rupturing the soil mass. In this condition, it is referred to as being plastic.

Plasticity is an outstanding characteristic of clays and is used to identify and classify clayey soils.

In 1911, a Swedish soil scientist, A. Atterberg, developed a series of hand-performed tests for determining the clay activity or plasticity of soil. These tests are known as the *Atterberg limits* tests. The series of tests is common to engineering classification of soil.

The consistency of a soil can go through four stages:

1. liquid,
2. plastic,
3. semisolid, and
4. solid.

The stages are related to moisture content. Although the transition between stages is gradual, test conditions have been arbitrarily established to delineate the moisture content as a precise point in the transitions between the four stages.

These moisture contents, which are determined by oven-drying procedures, are called the:

- liquid limit,
- plastic limit, and
- shrinkage limit.

As the tests are performed only on the soil fraction that passes the U.S.A. Standard 425- μm (No. 40) sieve, the relation of this fraction to total material must be considered when determining the state of the total material from these tests.

The significance of the limits and their relation to the phases of a soil-water system can be explained by referring to figures 5 and 6. As a very wet, fine-grained soil dries, it passes progressively through different phases. In a very wet condition, the mass acts like a viscous liquid, which is referred to as the liquid state. As the soil dries, a reduction in volume of the mass takes place that is nearly proportional to the loss of water. When moisture in a soil is at a value equivalent to the liquid limit, the mass becomes plastic.

Liquid limit, LL, is that moisture content (expressed as a percentage of the dry mass of soil) when the soil first shows a small but definite shear strength as moisture content is reduced. Conversely, with increasing moisture, liquid limit is that moisture content when the soil mass just begins to become fluid under the influence of a series of standard blows.

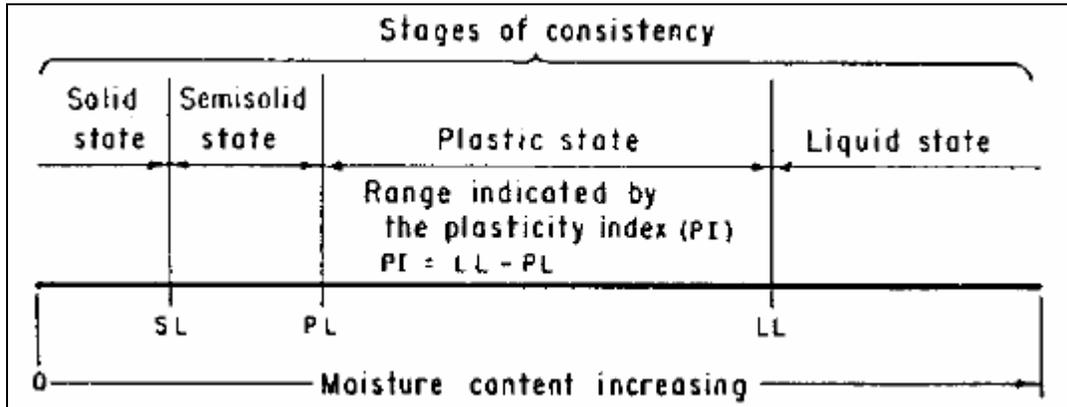


Figure-5.—Consistency limits.

PHASES OF A SOIL - WATER SYSTEM					
	Solid State	Semisolid State	Plastic State	Liquid State	Suspension
Moisture Content	Moisture content decreasing				
Atterberg Limits	Shrinkage Limit (SL)	Plastic Limit (PL)	Sticky Limit	Liquid Limit (LL)	
	Plasticity index (PI)				
Shrinkage	Volume Constant	Volume decreasing			
Consistency	Hard to very hard	Firm	Soft	Very soft	Slurry
					Water-held suspension
Shear Strength	Shear strength increasing			Very little to none	
Liquidity Index	$LI < 0$	$LI = 0$	$0 < LI < 1$	$LI = 1$	$LI > 1$

Figure-6.—Phases of a soil-water system

As moisture content is reduced below the liquid limit, the soil mass becomes stiffer and will no longer flow as a liquid. However, it will continue to be deformable, or plastic, without cracking until the plastic limit is reached.

Plastic limit, PL, is that moisture content (expressed as a percentage of the dry mass of soil) when the soil mass ceases to be plastic and becomes brittle, as determined by a procedure for rolling the soil mass into threads 3 mm (1/8 in) in diameter. Plastic limit is always determined by reducing the moisture content of the soil mass.

As the moisture content of soil is reduced below the plastic limit, the soil becomes a semisolid; that is, it can be deformed, but considerable force is required and the soil cracks. This condition is referred to as a *semisolid state*. As further drying takes place, the soil mass will eventually reach a *solid* state when further volume change (shrinkage) will not occur. Moisture content under this condition is called *shrinkage limit*, SL. This is the moisture content at which a reduction in moisture will not cause a decrease in the volume of the soil mass. Below the shrinkage limit, soil is considered to be a solid; that is, most particles are in very close contact and are very nearly in an arrangement which will result in the most dense condition.

In most fine-grained plastic soils, plastic limit will be appreciably greater than shrinkage limit. However, for coarser fine-grained soils (soils containing coarse silt and fine sand sizes), the shrinkage limit will be near the plastic limit. The shrinkage limit, together with other indexes, is useful in identifying expansive soils.

Plasticity index, PI, is the difference between the liquid and plastic limits, and represents the range of moisture content over which soil is plastic. (See figs. 5 and 6 and the plasticity chart on fig. 7.) Silts have low or no plasticity indexes, whereas clays have higher indexes. Plasticity index, in combination with liquid limit, indicates how sensitive a soil is to change in moisture content.

Liquidity index, LI, is the ratio of the difference between natural moisture content and plastic limit, w_n % PL, to the plasticity index, PI:

$$LI = \frac{w_n - PL}{PI}$$

Liquidity index is a useful indicator of the behavior of a fine-grained soil when sheared. If liquidity index is less than zero (negative), soil at natural moisture will exhibit brittle stress-strain behavior if sheared. If liquidity index is between zero and one, soil at natural moisture will behave like a plastic. If liquidity index is greater than one, soil at natural moisture will act like a viscous liquid when sheared.

Relative consistency, C_r , is the ratio of the difference between liquid limit and natural moisture content to the plasticity index:

$$C_r = \frac{LL - w_n}{PI}$$

If moisture content of a soil in its natural state, or in place, is greater than the liquid limit (relative consistency less than zero), any process of remolding will transform the soil into a thick, viscous slurry. If natural moisture content is less than the plastic limit (relative consistency greater than one), the soil cannot be remolded.

Soils may be grouped according to their liquid limits and plasticity indexes on a plasticity chart as shown on figure 7. Such plots can be useful in predicting properties of soils by comparing with similar plots for tested soils (see fig. 8).

As an index of engineering properties, soil plasticity applies only to fine-grained soils. The variation of engineering properties is generally related to the four zones on the plasticity chart (fig. 7), which determines the soil classification group. For soils plotting above the "A" line, permeability is very low. Compressibility increases with increasing liquid limit. For the same liquid limit, the greater the plasticity index, the greater will be the shear strength at the plastic limit. (see table 4 for soil names)

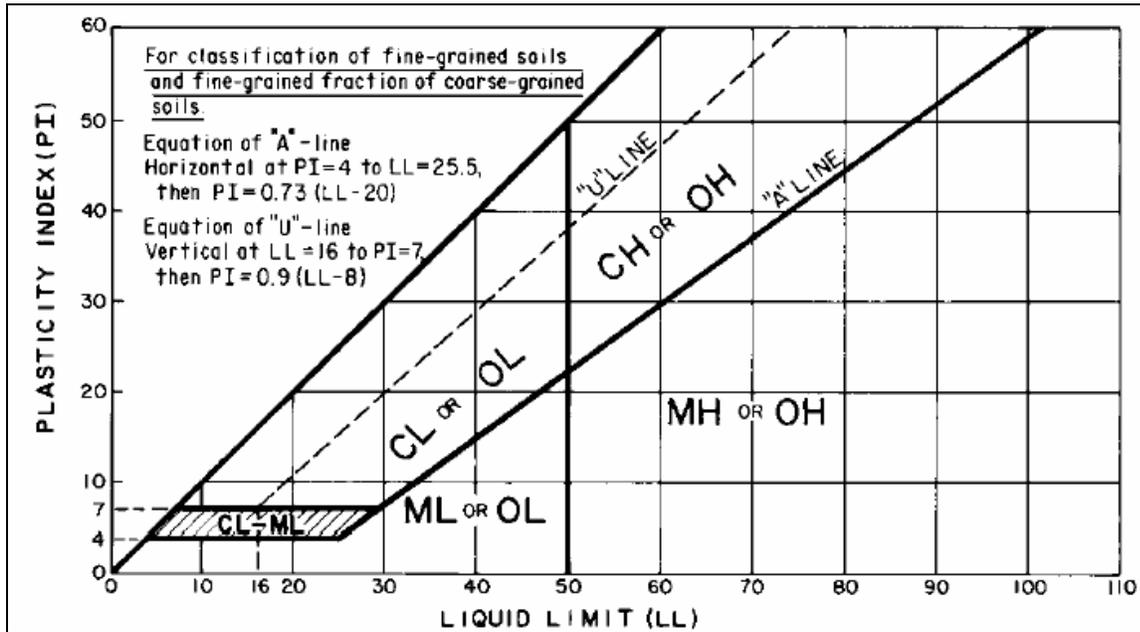


Figure-7.—Plasticity chart.

13. Porosity and Void Ratio. In evaluating a soil, either the amount of solids contained in a given volume or the remaining voids can be considered. Many of the computations in soil mechanics are simplified by considering the voids rather than the solids. Two expressions, *porosity* and *void ratio*, are used to define the void space. Porosity, *n*, is the ratio (expressed as a percentage) of space in the soil mass not occupied by solids (volume of voids) with respect to total volume. Void ratio, *e*, is the ratio of space not occupied by solid particles (volume of voids) to the volume of solid particles in a given soil mass. These relationships are expressed as follows:

$$n = \frac{V_v}{V_t} = \frac{e}{1 + e}$$

$$e = \frac{V_v}{V_s} = \frac{n}{1 - n}$$

where:

- n* = porosity expressed as a percentage
- e* = volume of voids to volume of solid particles
- V_t* = total volume
- V_v* = volume of voids
- V_s* = volume of solid particles

Porosity and void ratio are measures of the state or condition of a soil structure. As porosity and void ratio decrease, engineering properties of a given soil become more dependable with decreases in permeability and compressibility and an increase in strength. As porosity decreases, and, consequently, the void ratio decreases, excavating the material becomes more difficult. At a given moisture content, the compactive effort must be increased to obtain a decrease in porosity. However, similar properties may be obtained in different soils at widely different conditions of porosity. Engineering properties of a soil do not vary directly with its porosity; the relationship is complex.

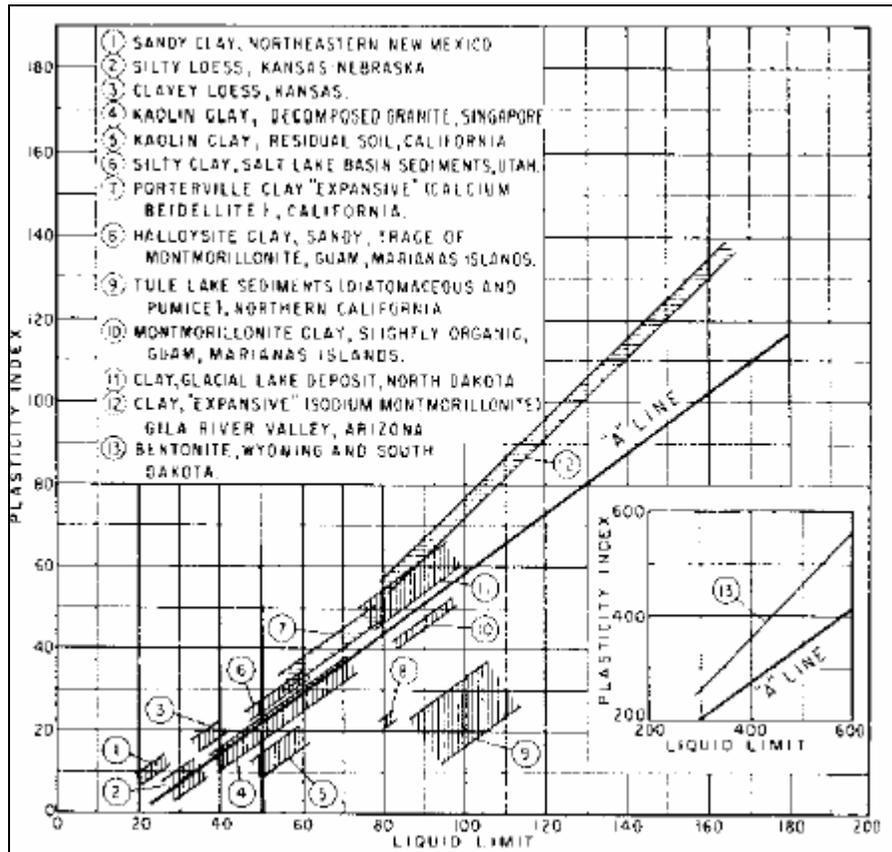


Figure- 8.—Typical relationships between the liquid limit and the plasticity index for various soils.

14. Specific Gravity. In investigating a soil, the most easily visualized condition involves the volume occupied by soil solids, V_s , the volume occupied by water, V_w , and the volume occupied by air, V_a , in the soil mass (fig. 9). However, most measurements are more readily obtained by mass. To correlate mass and volume, the *specific gravity* factor is required. Specific gravity is defined as the ratio between the density of a substance and the density of water at 4 deg. C. Several different types of specific gravity are in common use.

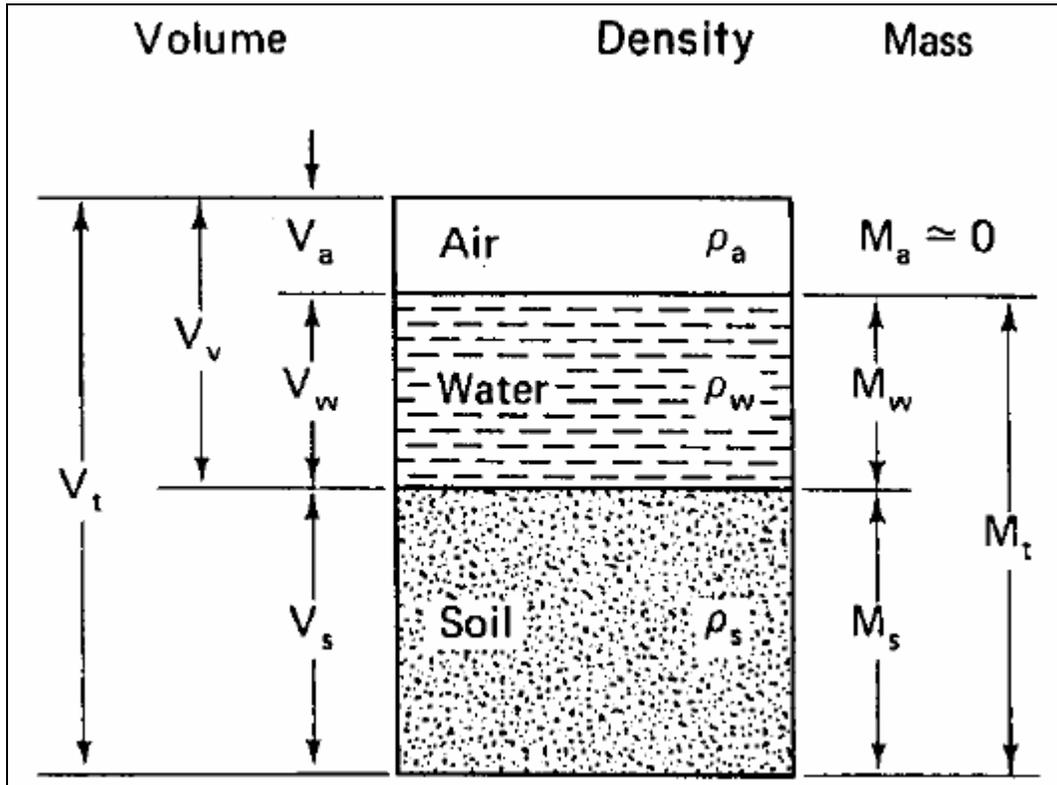


Figure-9.—Relationship between air, water, and solids in a soil mass.

Apparent specific gravity, G_a , is determined on soil particles as they occur naturally. Voids that exist within grains and cannot be filled with water are referred to as "impermeable voids." Apparent specific gravity ranges from 2.50 to 2.80 for most soils, with a majority of soils having an apparent specific gravity near 2.65. Unless specifically stated to the contrary, "specific gravity" (in this course) is assumed to mean the "apparent specific gravity." It is used to compute many important soil properties involving volume determinations such as porosity, void ratio, and degree of saturation as shown in the following equations:

$$n = 1 - \frac{V_s}{V_t} = 1 - \frac{\gamma_d}{\gamma_w G_a}$$

$$e = \frac{V_v}{V_s} - 1 = \frac{\gamma_w G_a}{\gamma_d} - 1$$

$$S = \frac{V_w}{V_v} = \frac{w G_a}{e}$$

where:

- e = volume of voids to volume of solid particles
- n = porosity expressed as a percentage
- s = degree of saturation
- w = moisture content
- V_v = volume of voids
- V_s = volume of solid particles

V_t = total volume

V_w = volume of water

γ_d = dry density

γ_w = density of water at 4 C

Bulk specific gravity (specific mass gravity), G_m , is the specific gravity having the permeable or surface voids of the particles filled with water. This value is smaller than the value determined for apparent specific gravity unless there are no permeable voids within the particles. The bulk specific gravity, saturated surface dry, of aggregate (sand and gravel) is used for concrete mix design and also for quality tests for riprap and rockfill materials. The bulk specific gravity of gravel particles may be determined on a wet, surface-dry basis, where permeable voids may not be entirely filled with water. Bulk specific gravity (wet surface dry) is used in all *in situ* density testing procedures. Bulk specific gravity (ovendry) is, in effect, the minimum dry density of all particles distributed throughout the entire or effective volume of particles; that is, both impermeable and permeable voids associated with individual particles in the ovendry condition are included in the volume determination. It is the smallest value of the different specific gravities.

As an index test, specific gravity is somewhat indicative of the durability of a material. Materials having low specific gravity are likely to break down and change properties with time whereas, high specific gravity materials normally do not deteriorate rapidly. The test is applicable primarily in evaluating coarse-grained materials and riprap or rockfill materials.

15. Moisture Content. Moisture content, w , is defined as the ratio (expressed as a percentage) of mass of water to mass of soil solids. Moisture (water) is the most influential factor affecting soil properties. Also, it is the principal factor subject to change either from natural causes or at the discretion of the engineer. In soil used as construction material or as encountered in foundations, the control of moisture often represents an important part of the structure's cost and may considerably influence the construction procedures used. Therefore, it is essential that the moisture content of a soil be determined, recorded, and reported in conjunction with all investigations, tests, and construction control work. Moisture content is an important factor in formulating designs, and the requirements for moisture are delineated in most specifications for construction. Regarding materials or foundation investigations, descriptive terms such as dry, moist, and wet may provide sufficient information to decide whether to use a given material. In all tests and in construction control procedures, a quantitative determination of moisture content is required. Moisture content is determined by a particular testing procedure.

Note that moisture content is expressed as a percentage of the dry mass of soil. Because moisture content is measured in terms of mass, it is independent of the volume occupied by the soil mass; that is, moisture content does not change whether material is in a loose state or in a dense state. Drying a soil specimen to a constant dry mass at a temperature of 110 C is accepted as a standard method for determining moisture content. Occasionally, however, soils are encountered for which this procedure is not valid; then, special methods are required. Soils containing organic matter, an appreciable amount of soluble solids, or unusual clay minerals such as halloysite or allophane require special treatment. Sometimes, ovendrying causes irreversible changes in soil properties; therefore, ovendried materials should not be used in laboratory tests unless specifically required.

a. **Optimum Moisture Content.** In 1933, R.R. Proctor showed that the dry density of a soil obtained by a given compactive effort depends on the amount of water the soil contains during compaction. He pointed out that, for a given soil and a given compactive effort, there is one moisture content that results in a maximum dry density of the soil, and that moisture contents both greater and smaller than this optimum value will result in dry densities less than the maximum (see fig. 1-10c). As early as 1934, moisture content at maximum dry density was termed *optimum moisture content*, w_o , by other investigators of soil compaction. Optimum moisture content is based on the compactive effort for the standard laboratory compaction test. Figure 11 shows the variation of optimum moisture content when compactive effort is varied from the standard.

b. **Absorbed Moisture.** Absorbed moisture is that on saturated surface-dry particles. Depending upon moisture conditions in a soil containing gravel-size particles, sufficient moisture may not always be available for the gravel to take up the maximum amount. The actual amount of absorbed moisture is determined by separating the gravel from

the other soil particles and determining the wet and dry mass of the gravel. If a soil has been separated into fractions by sieving, average moisture content of the total material, w , (in percent) may be calculated by:

$$w = P_1 \left(\frac{w_1}{100} \right) + P_2 \left(\frac{w_2}{100} \right) + \dots + P_n \left(\frac{w_n}{100} \right)$$

where:

P_1, P_2, P_n = percent of each size fraction in total material, %

w_1, w_2, w_n = moisture content of each size fraction, %

100 = constant representing the total materials, %

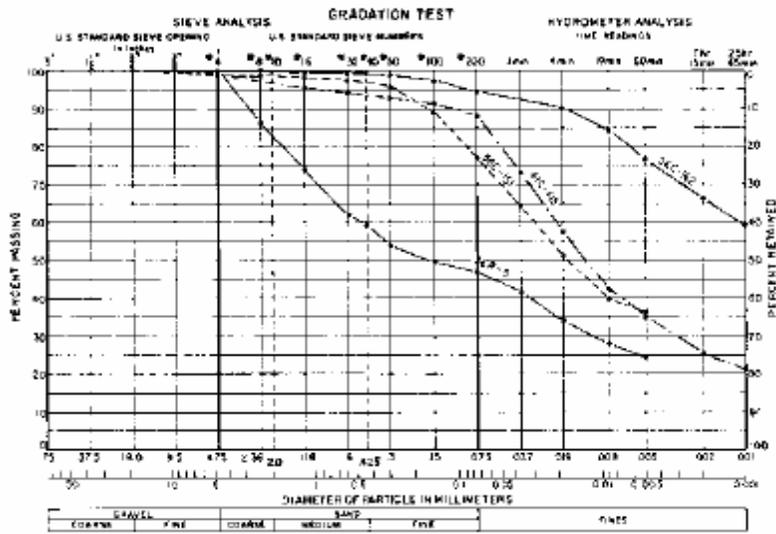
Engineering properties change so much with moisture content that it is used primarily to assist in interpreting other index properties. Primarily, moisture content is used to evaluate soils in their natural state, both as foundations and as construction materials sources.

16. Density and Unit Weight. Mass or weight of a unit volume of soil is an easily determined property. Consequently, density and unit weight are basic parameters to which all other performance characteristics are related. Relationships between density, unit weight, and other soil properties as a rule are complex, but in engineering practice, simple relationships are assumed to exist. A large number of qualified expressions are in common use for density or unit weight. To avoid confusion, the kind of density or unit weight reported must be clearly delineated. Generally, density is used in laboratory testing and construction control; unit weight is used in engineering analysis and design.

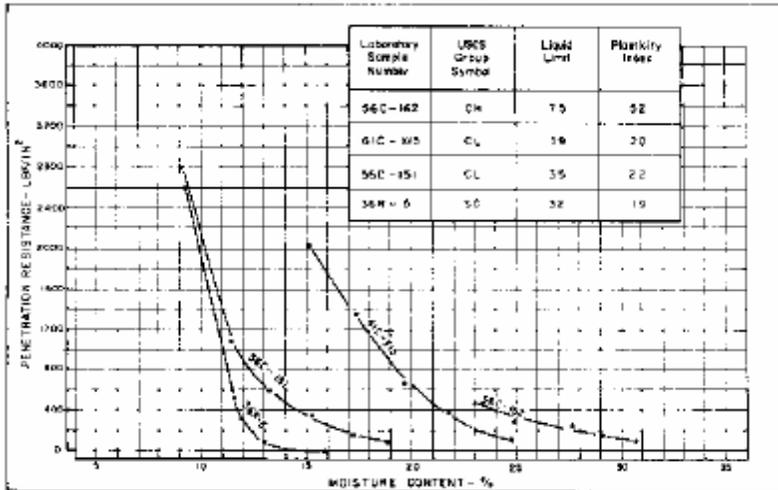
a. **In Situ Unit Weight.** The soil's unit weight as it exists in a natural deposit, at any particular time, is the in-place or natural unit weight. It can be determined either by measuring the mass and volume of an undisturbed sample of material or by measuring the mass of material removed from a hole of known volume. In-place density is calculated from mass and volume and then converted to unit weight. In-place unit weight is used in engineering computations such as:

- slope stability,
- bearing capacity,
- settlement analyses, and
- determining earth pressures.

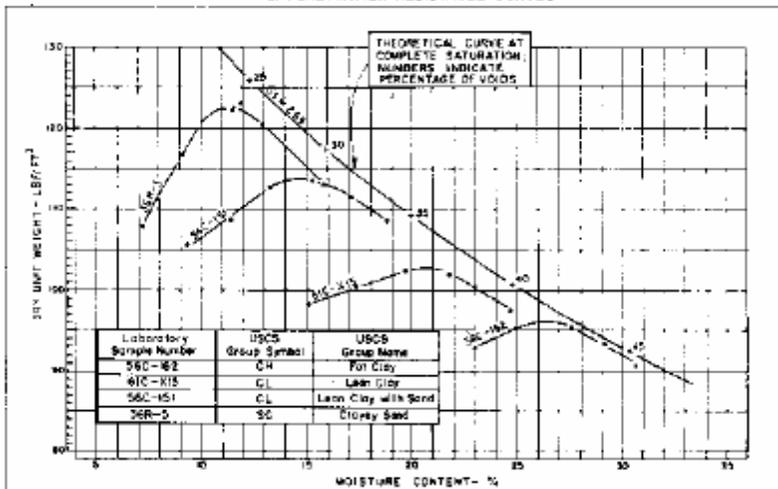
In some soil types, sufficient correlations exist between in-place unit weight and compression characteristics so that foundations for simple structures may be designed based only on those data and a knowledge of in-place unit weight in the foundation under consideration. The in-place or natural unit weight can be expressed as a wet or dry unit weight.



A. GRADATION CURVES



B. PENETRATION RESISTANCE CURVES



C. COMPACTION CURVES

Figure-10.—Gradation, penetration, and compaction curves for various soils.

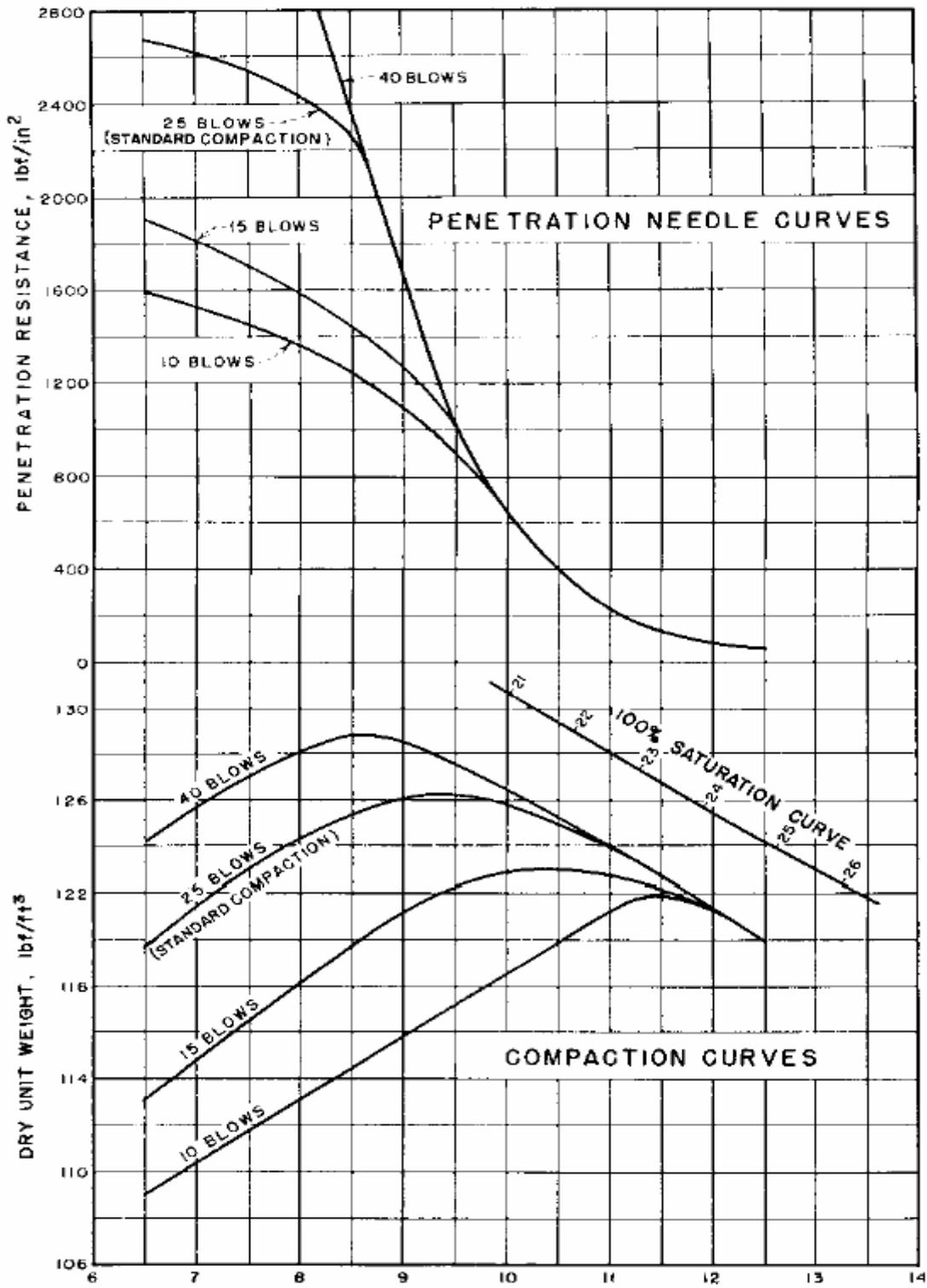


Figure-11.—Effect of compactive effort on the compaction and penetration-resistance curves.

b. Wet Unit Weight. The unit weight of the solid particles and the contained water is called wet unit weight, γ_{wet} . It includes but is not restricted to in-place unit weight. The method for its determination and its application is the same as that previously described in "In situ unit weight."

c. Dry Unit Weight. Dry unit weight is the normal expression for unit weight of soil. It is regarded as a fixed quantity independent of moisture content unless compactive effort is applied to the soil to change it. Dry unit weight, γ_d , is computed from wet unit weight, γ_{wet} , and moisture content using the following:

$$\gamma_d = \frac{\gamma_{wet}}{1 + \frac{w}{100}}$$

where:

w = moisture content

γ_d = dry unit weight

γ_{wet} = wet unit weight of solid particles plus contained water

d. Laboratory Maximum Dry Unit Weight. Dry unit weight produced with standard compactive effort at optimum moisture content is called the laboratory maximum dry unit weight. Test procedures are performed on the soil fraction that passes the U.S.A. Standard 4.75-mm (No. 4) sieve, which provides a compactive effort of 600 kN-m/m³ (12,375 ft-lbf/ft³). Figure 10c shows the variation in laboratory maximum dry unit weight for several soil types. Figure 11 shows typical compaction curves of various compactive efforts expressed in terms of number of standard blows per layer of soil. The maximum density and the optimum moisture content vary with the compactive effort. When the first laboratory compaction test procedure was developed, the compactive effort used was similar to the compactive effort of the construction equipment used to compact soils in the field. As construction equipment got heavier, additional procedures using various compactive efforts were developed. Therefore a single standard has been maintained. However, engineering documents may specify percentages of the maximum density and deviations from optimum moisture for various types of earthwork construction.

e. Relative Density. Soils consisting almost exclusively of coarse-grained particles (e.g., sands and gravels), when compacted by impact according to procedures outlined for determining laboratory maximum dry unit weight, have unit weight-moisture content relationships that correlate poorly with other properties. Furthermore, compaction curves are erratic; often they do not produce a definable maximum unit weight. To evaluate such soils, relative density (D_r) procedures consisting of three independent laboratory and field determinations have been developed.

When shear strength of coarse-grained soils is correlated with unit weight in the range between minimum and maximum index unit weights, a fairly reliable relationship exists. With a reasonable amount of construction control, a given type and amount of compactive effort can be expected to produce a related relative density. In practice, a relative density of 70 percent or greater is satisfactory for most conditions. However, special analyses of sands and gravels may be required to assess liquefaction potential in zones of high earthquake probability. While several different relationships for expressing the relative density exist, the original and the one now generally used is to express relative density (D_r) in terms of void ratio using the following:

$$D_r = \left(\frac{e_{max} - e}{e_{max} - e_{min}} \right) 100$$

where:

D_d = relative density expressed as a percentage

e = void ratio in place

e_{max} = void ratio in loosest state

e_{min} = void ratio in densest state

Relative density may also be expressed in terms of dry unit weight as follows:

$$D_d = \frac{\gamma_{dmax}}{\gamma_d} \left(\frac{\gamma_d - \gamma_{dmin}}{\gamma_{dmax} - \gamma_{dmin}} \right)$$

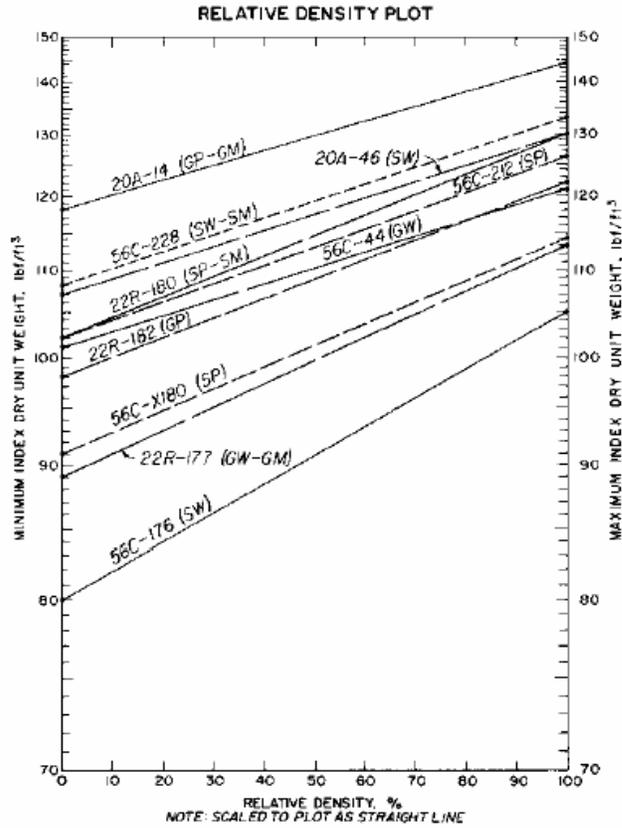
where:

γ_d = dry unit weight in place

γ_{dmax} = dry unit weight in densest state

γ_{dmin} = dry unit weight in loosest state

Figure 12 shows a chart from which relative density may be determined if the maximum index, minimum index, and in-place or compacted-fill dry unit weights of a material are known. Figure 12 also shows the maximum and minimum index dry unit weights for a variety of sands and gravels. Figure 13 shows the variation of maximum and minimum index unit weights as various percentages of gravel were added to a sand.



Laboratory Sample Number	USCS Classification		Index Dry Unit Weight		Gradation			Maximum Size Tested (in)
	Group Symbol	Group Name	Minimum (lb/ft ³)	Maximum (lb/ft ³)	Fines (%)	Sand (%)	Gravel (%)	
56C-44	GW	Well-graded gravel	101	121	5	6	89	3
22R-182	GP	Poorly graded gravel	98	122	2	6	92	3
22R-177	GW-GM	Well-graded gravel with silt	89	113	12	30	58	3
20A-14	GP-GM	Poorly graded gravel with silt	118	144	6	22	72	3
56C-176	SW	Well-graded sand	80	105	7	93	0	3/8
20A-46	SW	Well-graded sand	107	130	2	61	37	1 1/2
56C-212	SP	Poorly graded sand	102	126	4	95	1	3/4
56C-X180	SP	Poorly graded sand	91	114	4	95	1	3/8
56C-228	SW-SM	Well-graded sand with silt	108	133	10	90	0	3/4
22R-180	SP-SM	Poorly graded sand with silt	102	130	12	75	13	3/8

Figure-12.—Typical minimum and maximum dry unit weights for various coarse-grained soils.

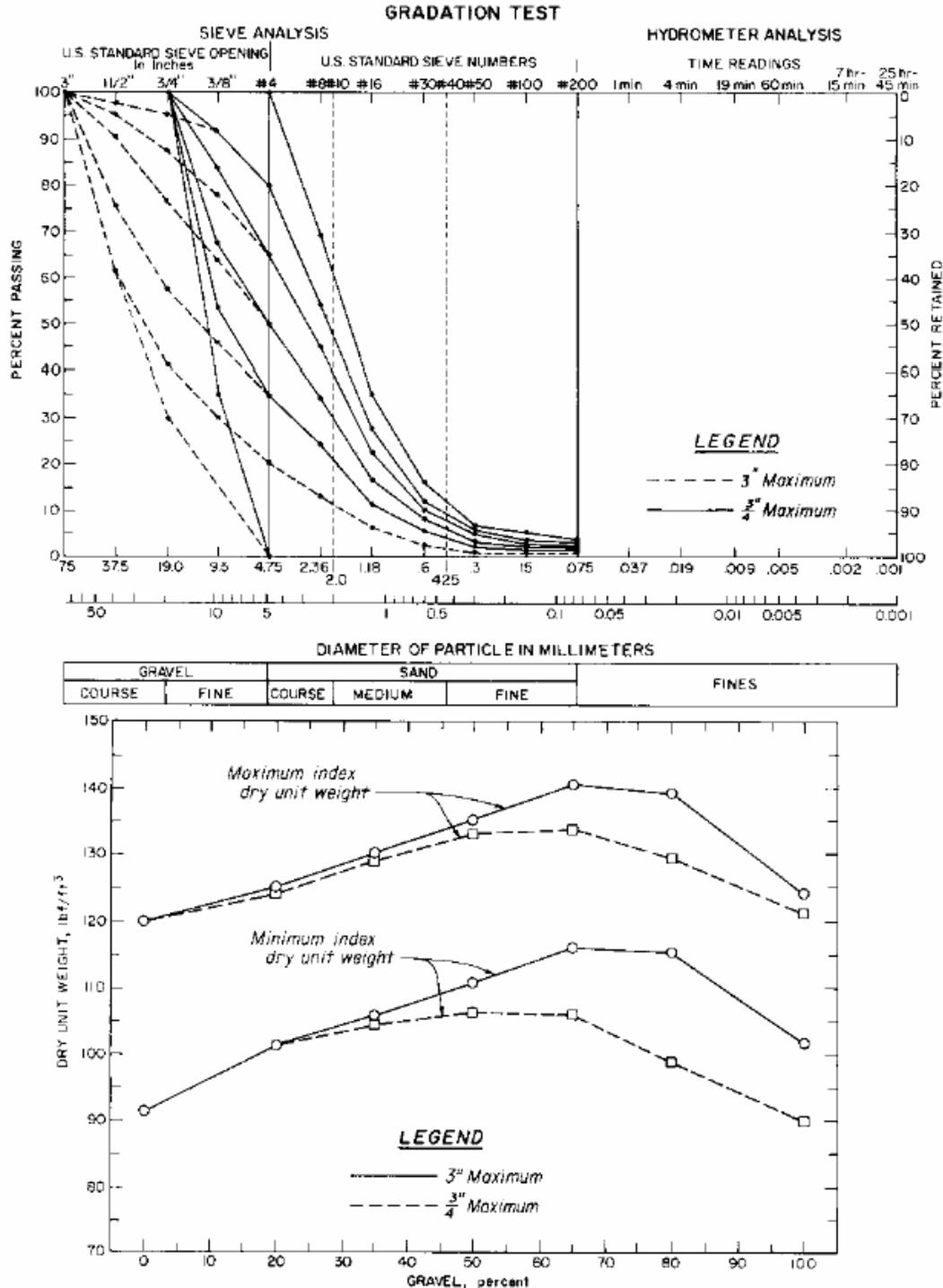


Figure-13.—Maximum and minimum index dry unit weights of typical sand and gravel soils].

f. **Compacted-Fill Unit Weight.** The soil's unit weight as it is compacted into a constructed fill is called the compacted-fill unit weight to distinguish it from in-place unit weight of a foundation or any of the various unit weights determined in the laboratory. Usually, it is expressed as dry unit weight, although wet unit weights are used to some extent. The compacted-fill unit weight may be determined by any one of a number of test procedures.

Compacted-fill unit weight is used primarily for construction control to ensure that compacted fill is at least as dense as assumed in preparing designs.

If a cohesive soil contains an appreciable quantity of coarse-grained particles retained on the U.S.A. Standard 4.75-mm (No. 4) sieve, an adjustment must be made in the compacted fill dry unit weight. The theoretical dry unit weight of total material, plus 4.75 mm and minus 4.75 mm, comparable to the laboratory maximum dry unit weight, is computed using the following:

$$\gamma_t = \frac{1}{\frac{P}{\gamma_w G_m} + \frac{1-P}{\gamma_{d_s}}}$$

where:

G_m = bulk specific gravity (oven dry) of plus 4.75-mm fraction

P = percentage of plus 4.75-mm material to total material (dry mass basis) expressed as a decimal

γ_t = theoretical dry unit weight of total material

γ_{d_s} = dry unit weight of minus 4.75-mm fraction

γ_w = unit weight of water at 4 deg. C

The Equation above gives theoretical dry unit weight of total material for different percentages of plus 4.75-mm (plus No. 4) particles of a given specific gravity assuming that each plus 4.75-mm particle is surrounded by minus 4.75-mm material. The equation is meaningless for percentages of plus 4.75-mm material for which the unit weight of plus 4.75-mm material corresponds to greater than 100-percent relative density of the plus 4.75-mm material when considered alone (fig. 14). This occurs at about 70 to 80 percent of plus 4.75-mm material, depending on gradation and on specific gravity. For plus 4.75-mm contents above about 30 percent, studies have shown compacted dry unit weight of soil in the field is less than the theoretical dry unit weight although, for many soils, compaction is not seriously affected until the percentage of plus 4.75-mm material rises to about 40 or 50 percent. For these soils, the usual assumptions of correlation between dry unit weight and other soil properties are not true. For such soils used in fill, special tests must be made. Figure 14 illustrates relationships of various dry unit weights of mixtures of plus 4.75-mm and minus 4.75-mm materials .

17. Penetration Resistance.

a. Soil Plasticity. In 1933, R.R. Proctor developed a simple instrument called a "soil-plasticity needle" that could be used to correlate moisture content-dry density relationships of soils used in construction . He noted that for a given compactive effort upon a given soil, a curve could be developed relating the penetration resistance of a small rod (needle) with the soil's moisture content.

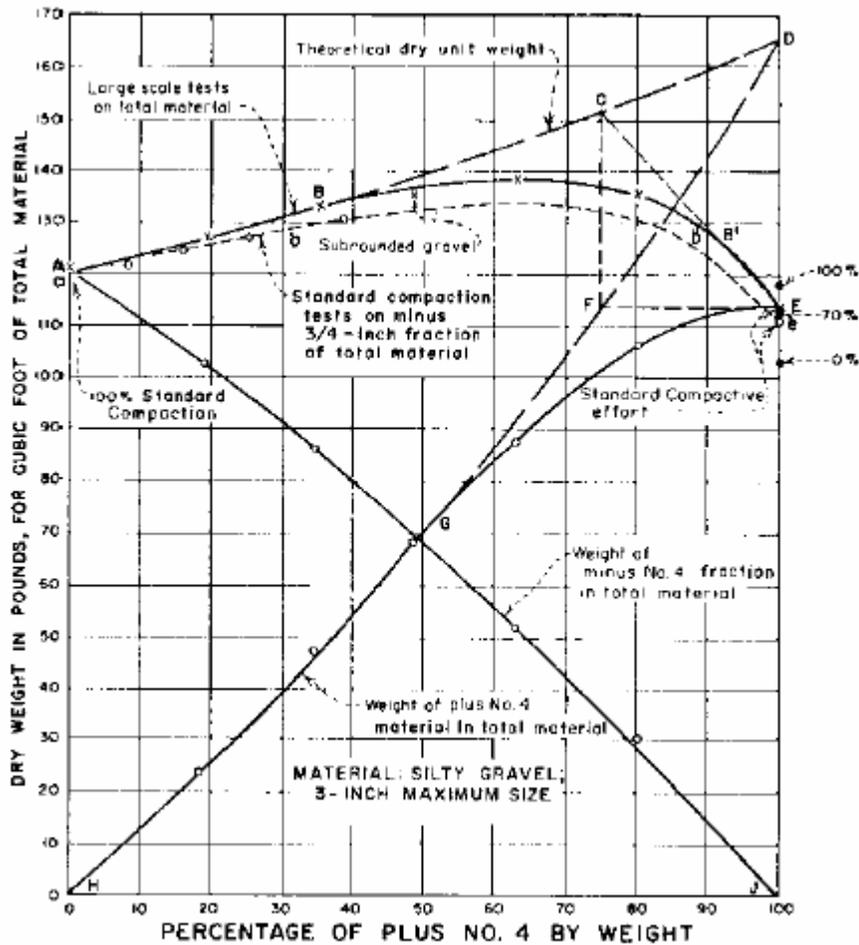


Figure-14.—Maximum dry unit weight versus gravel content for compacted soils.

Figures 10b and 11 show penetration resistance plotted as ordinate versus moisture content as abscissa. The curve begins with a rather high penetration resistance at a point on the compaction curve dry of optimum moisture content. Then, penetration resistance decreases rapidly and almost uniformly to a rather low value at a point wet of optimum moisture content. From there, the decrease in penetration resistance is gradual to zero near the liquid limit. The curve's straight line part, where penetration resistance decreases rapidly, is normally the useful part of the curve.

The penetration resistance test was formerly used to compare the resistance (of a specimen compacted according to the standard laboratory procedure) to the compaction achieved in the fill at the same moisture content. When fill material contained an appreciable amount of plus 4.75-mm particles, the test was not dependable. This approximate method of controlling compaction, called the "needle-density test," has now been replaced by the rapid method of construction control test.¹¹

b. Penetration Test. The standard penetration test¹² is used in foundation exploration to indicate relative density of *in situ* cohesionless soils and may be used to estimate unconfined compressive strength of cohesive soils *in situ*. The test is performed using a standard sampler and a standard sampling procedure. The sampler is at least 840 mm long, has a constant 50-mm outside diameter, and a constant 35-mm inside diameter. Penetration resistance is measured on the basis of the number of blows of a 64-kg rammer dropped 762 mm required for a sampler penetration of 305 mm (140 lbf at 30 in for 1 ft). The range in which these data are best interpreted extends from a lower limit of 5 to

10 blows per 305 mm to an upper limit of 30 to 50 blows per 305 mm. It is applicable only to nonlithified soils including fairly clean sands and gravels smaller than about 10 mm at a variety of moisture contents and in saturated or nearly saturated fine-grained soils. Soil deposits containing appreciable gravel content cannot be tested reliably because of the sampler bearing directly on gravel particles, damage to equipment, and possible plugging of the sampler barrel all of which result in high blow count values. Deposits containing coarse gravel, cobbles, or boulders typically result in penetration refusal and damage to equipment. Data from this test require interpretation by experienced practitioners and should take into account:

- soil type,
- moisture content,
- methods of drill hole advance,
- ground-water conditions, and
- purpose for performing the test.

Table-1.—Relationship between consistency, unconfined compressive strength, and penetration resistance, N
Consistency Field identification

Consistency	Field identification	Unconfined compressive strength, q_u , ton / ft ²	Penetration resistance, N
Very soft	Easily penetrated several inches by fist	Less than 0.25	Below 2
Soft	Easily penetrated several inches by thumb	0.25 to 0.5	2 to 4
Medium	Can be penetrated several inches by thumb with moderate effort	0.5 to 1.0	4 to 8
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 to 2.0	8 to 15
Very stiff	Readily indented by thumbnail	2.0 to 4.0	15 to 30
Hard	Indented by thumbnail with difficulty	Over 4.0	Over 30

Figure 15 shows the relationship of blow count to relative density. Figure 15a is a relationship determined from chamber testing of clean sands in the laboratory under various overburden pressures. The tests were performed on fine to coarse sands containing less than 5-percent fines; this relationship is not applicable to sands containing more than 10-percent fines or those containing gravel particles. The relationship is converted to a more convenient form, on figure 15b, for use in analyzing field data. The graph's vertical pressure scale can be converted to a depth scale by estimating the in-place wet density of soil. For tests below the water table, *in situ* submerged density must be used, taking into account the buoyant effect of water, that is, *in situ* wet density of the soil minus density of water. Using the graph in this manner, penetration resistance is plotted against depth; relative density is determined according to the relative density zone in which the point is plotted. More recently, additional laboratory chamber tests were performed by the Corps of Engineers, and their work confirms the relationships shown on figure 15.

The relative consistency of saturated fine-grained soils can be estimated by the standard penetration test. As the values obtained for these soils are only qualitative, table 1 and figure 16 can be used to provide an approximate evaluation of a soil.

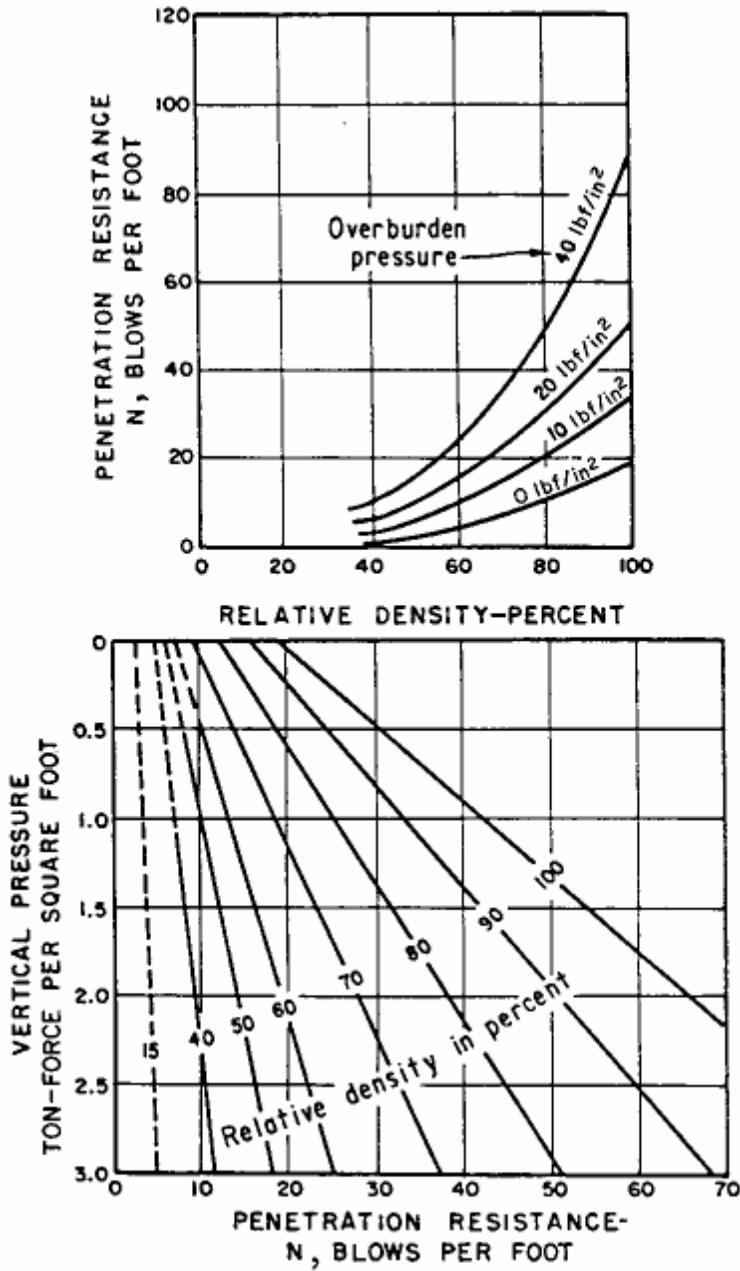


Figure-15.—Criterion for predicting relative density of sand from the penetration resistance test.

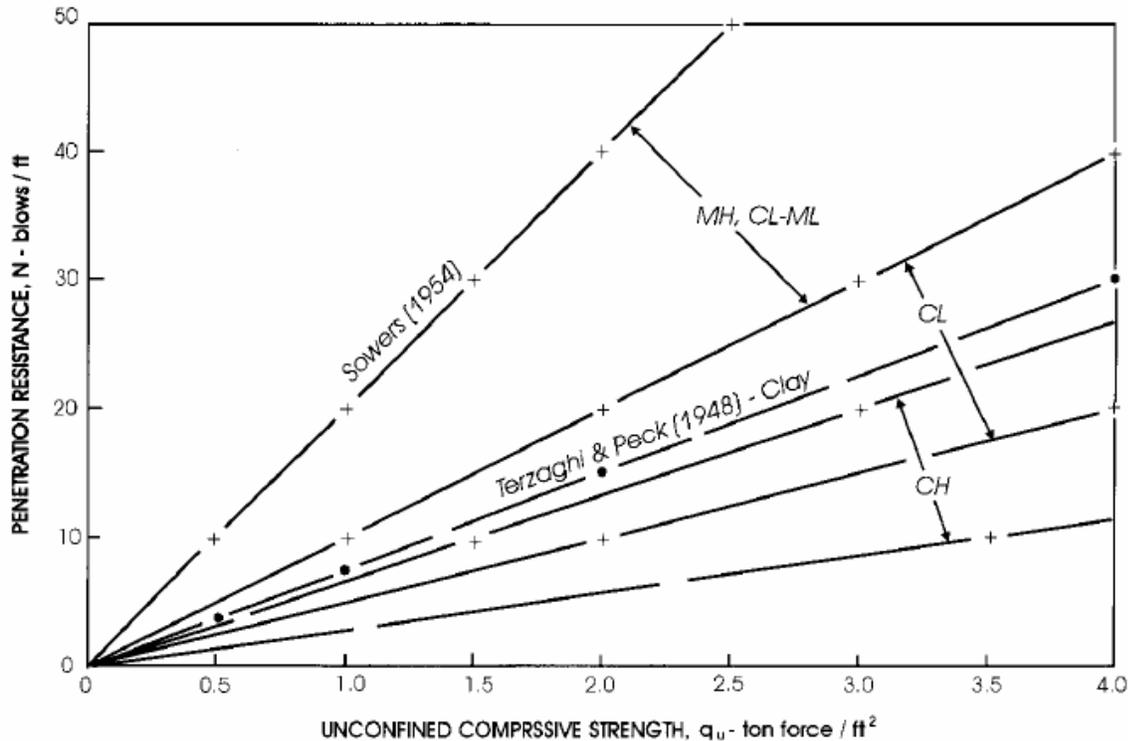


Figure-16.—Unconfined compressive strength versus penetration resistance, N, for various soils.

18. Unconfined Compressive Strength. The internal friction component of undrained shear strength for fine-grained soils in a saturated or nearly saturated state is usually small. A compression test made without benefit of confining pressure is assumed to provide an index of the available undrained shear strength of a soil that will be somewhat conservative. A variety of equipment has been developed for making the tests. The devices range from simplified equipment and procedures from which test results are of value only as an index test to rather complex equipment and test procedures that provide reliable data on engineering properties. The unconfined compressive strength of a soil is the strength of a cylindrical specimen (maximum axial load divided by cross-sectional area) tested in compression with the lateral pressure equal to atmospheric or zero gauge pressure.

19. Soluble Solids. The quantity of soluble solids present in a soil may be an important factor when considering the suitability of a soil for the foundation of a hydraulic structure or for constructing embankments. The effect of soluble solids on an earth structure depends on:

- Temperature,
- Minerals present in the soil and their solubility characteristics in water,
- Coefficient of permeability and, thus, the amount of water passing through the soil,
- Chemical characteristics of the water, and
- Other factors.

Therefore, the percentage of soluble solids is only an indication of possible effects. Soluble solids are more objectionable in materials having moderate to high permeability than in soils with low permeability.

The kinds of soluble minerals present in a soil or carried by groundwater are important when considering the type of cement to be used in constructing concrete structures in contact with soil. The aggressive substances which affect concrete structures are the sulfates of sodium, magnesium, and calcium. Soluble minerals present in soil and groundwater must be determined by chemical analyses. The presence of appreciable quantities of soluble minerals indicates that engineering properties may change in the presence of percolating water.

C. Engineering Properties

20. General. In evaluating a soil, the transition is gradual between those properties that serve only as broad guides to the character of material and quantitative properties which define specific performance characteristics. For example, moisture content and dry density are at times used as index properties and at other times used as engineering properties. The importance of the two properties in investigation and construction control, as indicators of the nature of material and of the quality of compaction, has frequently resulted in the belief that low moisture content and high dry density are the only desirable characteristics to attain in soils. This is not necessarily true. These properties are only indexes of the probable engineering behavior of a soil. Yet, both dry density and moisture content are so intimately a part of computations involved in design of structures of soil that they must frequently be evaluated as engineering properties.

The principal distinction between index and engineering properties is that the procedures for determining index properties are relatively simpler than those for determining engineering properties, which require considerable knowledge and skill to develop reliable information. This does not imply index properties tests are simple to perform. Skill and care are required when performing index tests. Considerably more special knowledge and skill are required to interpret and apply index information than are required to use information from tests for engineering properties. The most commonly listed soil engineering properties are strength, volume change, and permeability. In addition, but less clearly defined, are such characteristics as deterioration and workability for which quantitative evaluation is invalid.

Selecting samples and specimens for both index and engineering properties testing requires considerable thought to ensure materials selected for testing are representative of materials at the structure site. Also, careful sample and specimen selection are required to obtain maximum information from the testing program at minimum cost.

21. Shear Strength.

a. General. Soil has little strength compared to other materials used for building a structure. Moreover, compared to maximum soil strength, large variation can exist in strength, both from soil to soil and within a given soil type, depending on how it was deposited or placed.

Engineering computations using soil strength deal primarily with shear strength, the resistance to sliding of one mass of soil against another, and rarely with compressive or tensile strengths. In 1776, C.A. Coulomb, a French engineer and scientist, observed that the shear strength of a soil consisted of two parts: (1) one part dependent on the stress acting normal to the shear plane, (2) the other part independent of that stress. The parts were called internal friction and cohesion, respectively. The factor relating the normal component of stress to shear strength was designated by $\tan \phi$ and the unit of cohesion by c . The shear strength, s , may then be expressed as:

$$s = c + \sigma \tan \phi$$

where:

c = unit of cohesion

s = shear strength

σ = normal stress on the sliding surface

Φ = angle of internal friction

Many soils are either predominantly cohesive or noncohesive. For either type of soil, engineering computations can be simplified by dropping the smaller term in the shear strength equation, and the resulting solution is conservative. Much of early soil mechanics practice was based on this simplified assumption. The terms cohesive ($\Phi = 0$) and noncohesive or cohesionless ($c = 0$) soils are still in common use in referring to these soils.

Occasionally, a structure is so massive or the soil foundation is so weak that this simplifying assumption cannot be used. For a large earth embankment dam, this is almost always the case. In such a situation, determining the values of c and Φ with as much precision as possible is most important for both the proposed structure and foundation under a variety of probable loading conditions. It is extremely important that "undisturbed" samples from the foundation, secured during investigations, be truly representative of materials and conditions. Also, material placed in these earth structures (during construction) should comply with established design limitations, based on laboratory tests.

b. In Situ Shear Strength. *In situ* shear strength estimates can be derived from:

- standard penetration test
- vane shear test
- cone penetration tests
- borehole shear device test

Each field test has been used successfully. *In situ* shear strength has been estimated for a variety of soils using appropriate computations and empirical correlations. The Menard Pressuremeter and the flat blade dilatometer, as well as various other equipment and methods, have been used to determine modulus values of soils.

c. Direct Shear. In the laboratory, shear strength measurement is accomplished either by using the direct shear test or the triaxial shear test. The direct shear test, was developed first and is still used widely. It has the advantage of simplicity, but disadvantages include stress and strain concentrations within the specimen, and pore-water pressures cannot be monitored during the test.

Usually, the specimen is tested under consolidated-drained conditions. The specimen is allowed to fully consolidate under each applied normal pressure and is sheared at a rate slow enough to allow pore pressures to remain at equilibrium during shear. Therefore, test results are reported in terms of effective stress.

d. Triaxial Shear. The triaxial shear test apparatus was developed to permit control of other factors that influence shear strength, which cannot be evaluated with the direct shear test apparatus. Originally used as a research tool, the triaxial device was instrumental in better understanding the mechanics of shear in soils. Because of its capability to simulate a wide range of test conditions, the triaxial shear apparatus is now used for routine testing. The triaxial shear apparatus is used to perform four different test procedures.¹⁶ The procedure used to test a soil must be based on field loading and drainage conditions to which the soil will be subjected. Figure 17 indicates the relation between relative density and the angle of internal friction, Φ (also expressed as $\tan \Phi$), for compacted coarse-grained soils.

e. Pore-Water Pressure. Shear strength is primarily dependent on effective normal stress. Under an externally applied stress, soil grains are forced into more intimate contact, and the soil mass volume decreases. Because soil grain volume cannot be changed appreciably, this volume change must take place primarily in the soil voids or pores. If these pores are completely filled with water, their volume cannot be changed unless some water is drained from the soil mass, because water is considered incompressible. If drainage is prevented or impeded, stress will develop in the pore water opposing the externally applied stress. The developed stress is called excess pore-water pressure (i.e., pore-water pressure in excess of hydrostatic conditions). Even if the pores are filled only partially with water, pressures in the fluid combination of air and water will develop to a lesser degree, because volume change is possible (air is compressible), and additional stresses can be carried by the soil grains. However, a difference will exist

between pore-air pressure and pore-water pressure resulting from capillary tension or capillary stress of the water films. Therefore, when analyzing shear strength of unsaturated soils, consideration must be given to whether pore-air or pore-water pressure is used. Because pore pressures are opposed to the normal stress, total normal stress will be reduced whenever positive pore-water pressure is present.

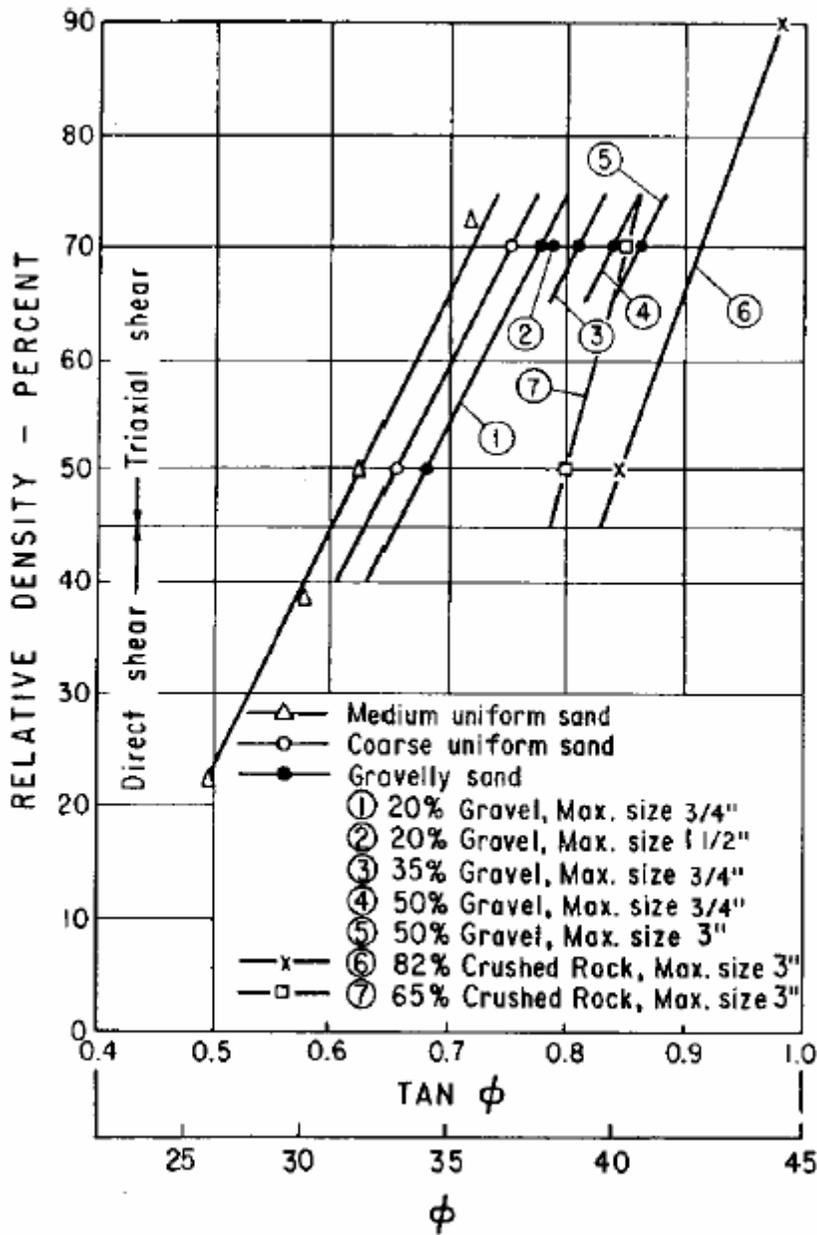


Figure-17.—Effect of relative density on the coefficient of friction, tan ϕ , for coarse-grained soils.

Based on this observation, the general case Coulomb equation must be rewritten:

$$s = c' + (\sigma - u) \tan \phi'$$

where:

- s = shear strength
- u = pore fluid pressure
- c' = effective cohesion
- σ = normal stress on the sliding surface
- ϕ' = effective angle of internal friction

The analysis can be made considering u as either pore-air pressure or pore-water pressure, depending on the application. Figure 18 shows the effect of pore fluid pressure on the shear characteristics of a lean clay. True strength characteristics of a soil are determined only when pore fluid pressures are accounted for during laboratory testing.

f. **Capillary Stress.** When the water in a soil does not completely fill the voids, a surface develops between the water and air. This surface is not flat, but is curved due to surface tension. The degree of curvature depends on:

- size of the void,
- nature of the material forming the sides of the void, and
- kind of liquid in the voids.

Because the surface is curved, it is stressed in tension; this tensile stress is imparted to the liquid in the pores. The effect of tensile stress is to pull the soil particles together. The action is contrary to pore-water pressure as described in the previous paragraphs, and it can influence the value of cohesion determined by laboratory tests.

Capillary stress is present when soil is not completely saturated and may have a significant influence on results of laboratory tests. The presence of capillary stress results in a higher shear strength for a soil than will exist when capillary stress is lessened or eliminated by wetting or saturation.

With information from the capillary test, a triaxial shear test can be analyzed either on the basis of pore-air pressure, which will indicate the shear strength when it is influenced by capillary stress, or on the basis of pore-water pressure, which does not include the influence of capillary stress and which can represent the condition of saturation.

g. **Sliding Resistance.** A special type of shear strength investigation involves the shear strength between dissimilar materials, commonly between soil and concrete or between soil and geosynthetic materials. Usually, this type of shear is identified as sliding resistance or interface friction. The nature of the problem makes it necessary to investigate this case by direct shear test methods.

h. **Residual Shear Strength.** Some soils exhibit brittle stress-strain behavior during shear. Materials such as shale, indurated clay, overconsolidated clay, or stiff fissured clay reach maximum shear stress after extremely small shear displacements. Studies indicate measured shear stress in these materials decreases rapidly with increasing shear strain beyond the point of maximum shear stress. Continued displacement beyond maximum shear stress will reduce the measured shear stress to a low constant value termed the "residual shear stress" (fig. 19).

22. Volume Change.

a. **General.** Volume change in a soil mass caused by both natural and artificial conditions introduces problems peculiar to soils and not encountered with other construction materials. Volume decrease is caused by pressure increase and is a function of time and permeability; it is associated with changes in moisture and air content, and it can occur as a result of compaction. Volume increase is a function of:

- pressure,
- density,
- moisture content, and
- soil type.

Terms that describe volume change phenomena are:

- *Compression*, Volume change as a result of elastic deformation or expulsion of air produced by application of a static external pressure.
- *Consolidation*, Volume change produced by application of static external pressure and achieved through pore-water movement with time.
- *Shrinkage*, Volume change produced by capillary stresses during drying of a soil.

- *Compaction*, Volume change produced by mechanical manipulation such as rolling, tamping, or vibrating.

The above terms apply to volume reduction. Corresponding terms apply to volume increase:

- *Rebound* as opposed to compression.
- *Expansion* as opposed to consolidation.
- *Swell* as opposed to shrinkage.
- *Loosening* or *scarifying* describes the operation opposite of compaction.

Ground surface movement is often the result of volume change in the underlying soil. Terms describing ground surface movement are:

- *Settlement*, Ground surface movement associated with volume decrease.
- *Heave*, Ground surface movement associated with volume increase.

Most often, volume change is associated with changes in volume of the voids and only to a limited extent with changes in volume of solid particles. If voids are, to a large extent, filled with air, an increase in pressure on the soil mass will result in compression without appreciable subsequent consolidation. Conversely, if voids are nearly or completely filled with water, little or no compression will take place immediately upon application of a pressure increment; and only when water drains from the soil mass can consolidation occur. If water can drain readily from the soil mass, consolidation may take place within a short period of time; but, if the soil has a low permeability, if the soil mass is extremely large, or if drainage is otherwise impeded, complete consolidation may require many years.

Volume change also is caused by particle rearrangement, particle breakdown, and physical or chemical absorption of moisture. Usually, particle rearrangement is associated with clay soils deposited underwater that have been stressed only with the weight of soil above them. Particle breakdown is most commonly found where residual soil is derived from rock that has been weathered and altered in place. Most clay soils have affinity for moisture which can be removed only with considerable effort. Fortunately, many of the clay minerals attain a state of saturation without great volume change; but a few (e.g., montmorillonite) will absorb or release large volumes of water and experience large shrinkage and swell.

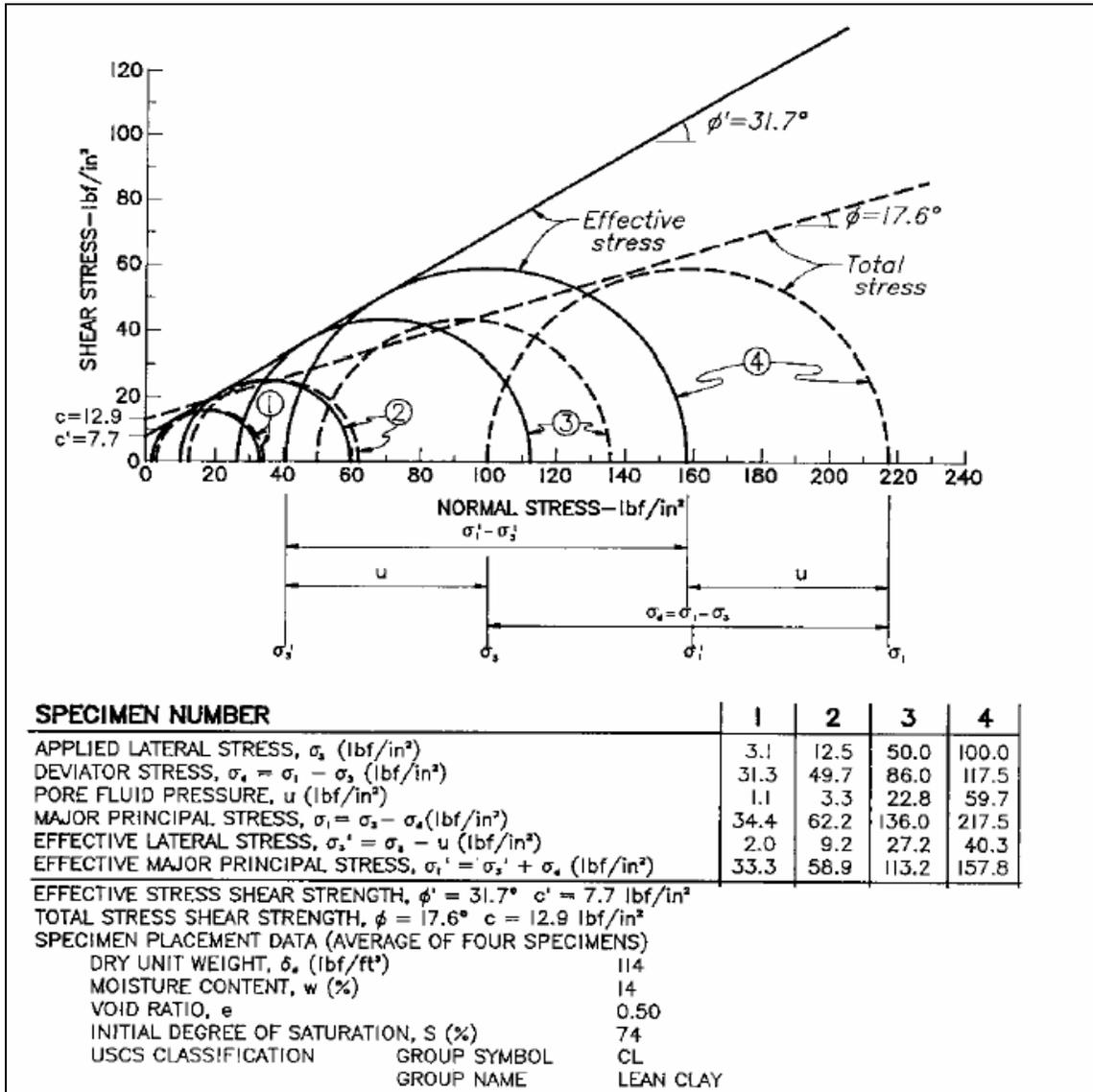


Figure-18.—Results of a consolidated-undrained (CIU) triaxial shear test on lean clay with and without considering pore pressure.

sufficient compaction can be applied to limit compression to a few percent. Where it can be applied, moisture control tends to reduce future consolidation by facilitating compaction. For some soils compacted dry of optimum moisture content, the soil grain structure will not assume its densest state as discernible from compaction curves shown on figure 10c. For such a condition, subsequent wetting may result in particle rearrangement and an accompanying volume strain called *saturation collapse*. This action can occur rapidly in some silty type soils.

Soils subject to shrinkage and swell can be used in earth structures if compacted under good moisture control and loaded sufficiently with other materials to prevent swelling. When swelling-type soils are used for the majority of an embankment, flatter slopes and greater soil volumes are required than for nonswelling soils. This may require more stringent zoning of an embankment, and consideration must be given to acquiring better materials from more distant sources as opposed to using swelling-type soils found locally.

Compressibility in a soil foundation is difficult to control. Removing compressible soil, using piles or piers, and use of spread footings or mats has been applied in specific cases with success. Variation in compressibility within the construction area can produce greater difficulty than the amount of compressibility encountered. Under earth dams, removal of questionable materials, drainage of wet soils, wetting of dry soils, and spread embankments have all been used to minimize compressibility. Compressible foundations under proposed buildings and similar structures are frequently improved by removing an amount of soil equal to the structure's weight so there is no net increase in stress in the natural soil after the structure is built.

c. Pressure-Compression Characteristics. When soils are gradually deposited layer upon layer, each layer is compressed by the layers above it; and, as time passes, the soil attains a state of consolidation in equilibrium with the superimposed pressure. Such a soil is described as *normally consolidated*. In such a soil, the density increases with depth or overburden pressure, and the void ratio decreases correspondingly. This relationship is nearly a straight line (virgin compression line) when plotted on a semilogarithmic scale (fig. 21). If all or a portion of the superimposed pressure is removed, rebound will occur, but the change in void ratio usually will be small compared to the void ratio change produced by initial compression and consolidation. Such a soil is *overconsolidated*. If an overconsolidated soil is again loaded, the change in void ratio will be small compared to that produced with similar pressure increments during initial consolidation. The final void ratio, however, will be somewhat smaller than that originally obtained. Where such a condition exists, either as a result of a glacier having overridden the deposit where extensive erosion has occurred or where desiccation has occurred, these materials often can function as a foundation without difficulty.

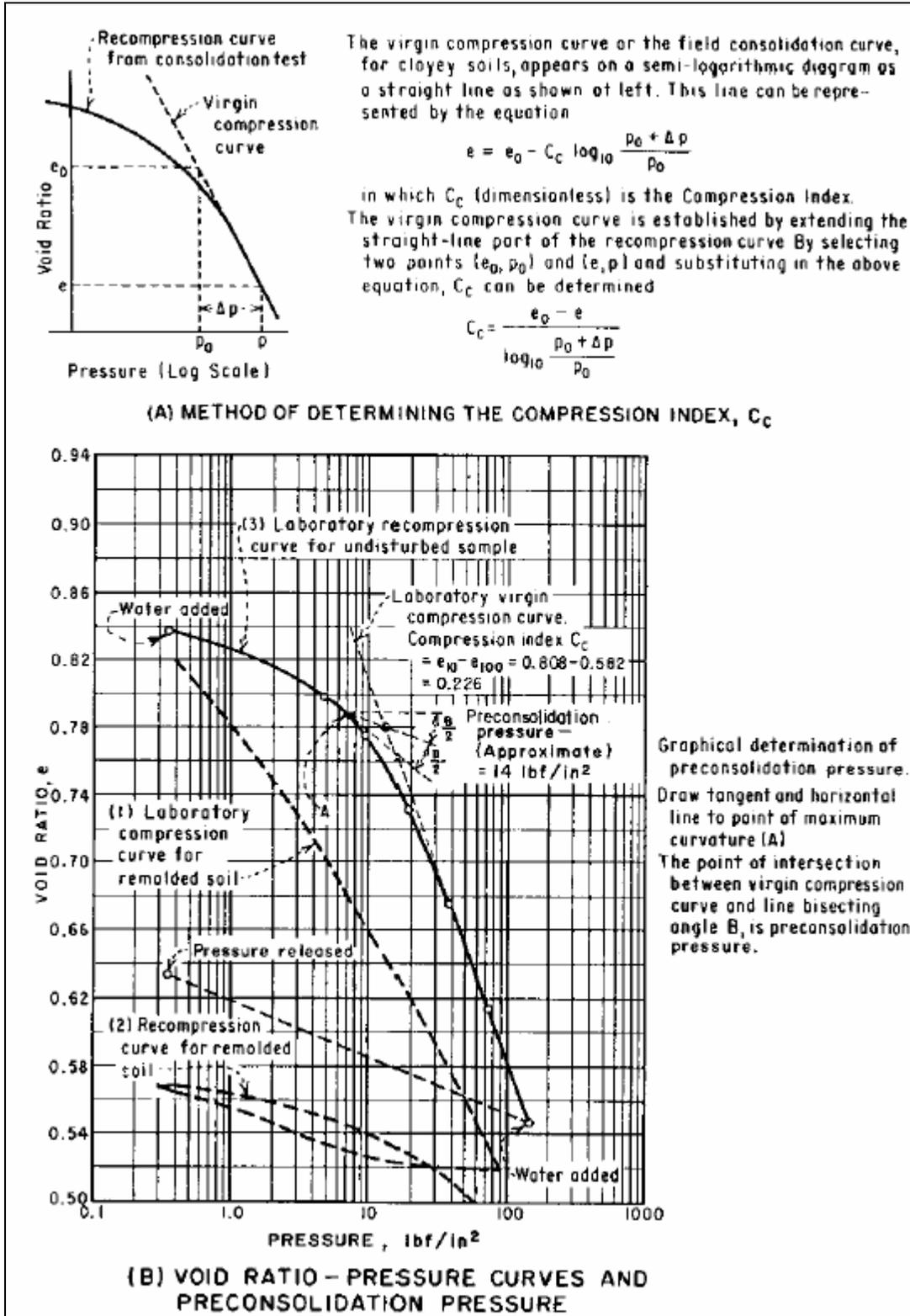


Figure-21.—Void ratio-pressure curve, compression index, and preconsolidation pressure.

In the arid and semiarid parts of the Western United States, unsaturated deposits of loose silt (ML), silty clay (CL-ML), or lean clay (CL) have caused special problems. These deposits include wind-deposited loess and loess-like soils and colluvial and alluvial soils deposited by flash flood runoffs, often in the form of mudslides. In these deposits, soils have never been completely wetted to allow breakdown of the loose depositional structure. Generally, these soils have relatively high dry strength created by well dispersed clay binder. It is known that loessial soils form high, near-vertical faces that are stable as long as the moisture content is low (fig. 22). However, upon wetting, strength is essentially lost and slope failures occur. Similarly, loessial soils support heavy structural loads on footings or piles when dry but lose their bearing capacity and resistance to compression when their loose structure collapses upon wetting (fig. 23). Certain interfan soil deposits adjacent to the southwestern foothills of the Central Valley of California as well as other areas in the Western United States with similarly formed deposits have similar characteristics. When dealing with hydraulic engineering works, where subsoils eventually will become saturated, recognizing such soil deposits is important so that measures can be taken to improve them before building structures upon them.



Figure-22.—A 34-foot cut on 1/2:1 slope in loess formation—Franklin Canal, Nebraska.

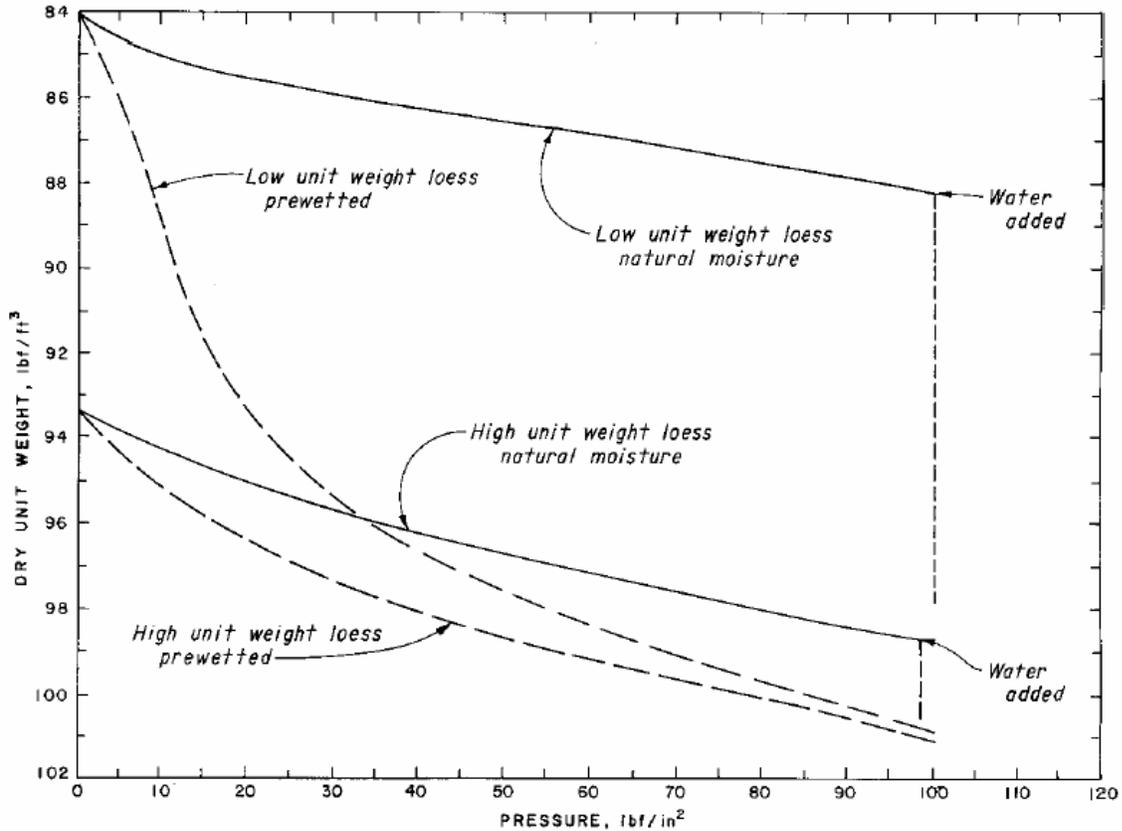


Figure-23.—Effect of loading and wetting high and low unit weight loess soil foundations.

Methods have been designed to identify these metastable soils. Certain relationships exist between in-place dry density, moisture content, and the volume change that may be anticipated for fine-grained soils. The relationship in which dry density is expressed as the ratio of in-place dry density to laboratory maximum dry density for various moisture contents, presented in relation to optimum moisture content, is shown on figure 24a. If the dry density ratio and moisture content difference of a soil plot above and to the left of the limit line (shown on fig. 1-24a), very small additional volume change will occur upon wetting, and treatment of in-place material is not normally required for small dams, canal embankments, or lightweight structures. If the dry density ratio and moisture content difference plot below and to the right of the limit line, significant volume change may occur on wetting of the in-place material even under low pressures; and treatment of in-place materials may be required. Rigid structures should not be placed on either wet loose soils or dry loose soils, subject to later wetting, as structure cracking may occur. Foundation treatment may be required for rigid structures whose foundation soil properties plot above the limit line.

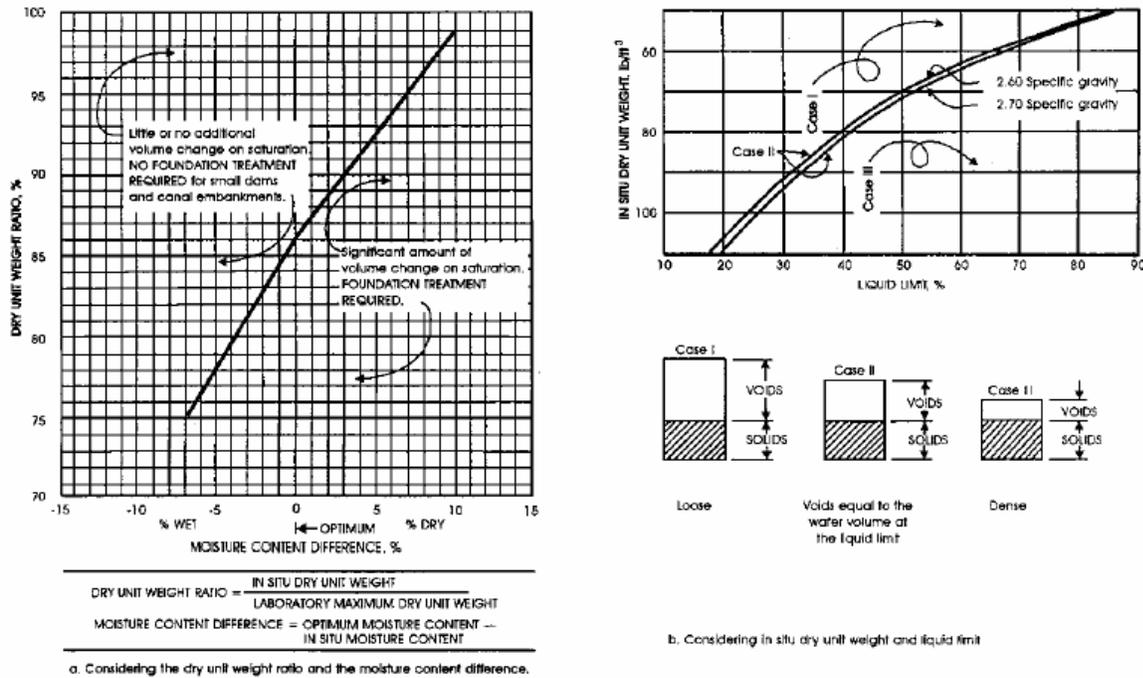


Figure-24.—Criterion for treatment of relatively dry fine-grained foundations.

Another criterion for predicting loose, fine-grained soils needing treatment involves in-place dry density and liquid limit as shown on figures 24b and 1-25 (from). The theory is simple: if in-place dry density is so low that the volume of voids is larger than that required to hold the volume of water needed to reach the liquid limit (as shown for case I), the soil can become saturated. Consistency is low, and the void space is sufficient to allow collapse. Conversely, if in-place dry density is high enough so that the volume of voids is less than that required to hold the volume of water needed to reach the liquid limit (as shown for case III), the soil will not collapse upon saturation but will reach a plastic state in which particle-to-particle contact always will take place. Therefore, soils with liquid limits and in-place dry densities that plot above the line show a critical dry density condition, while soils that plot below the line do not. In the latter condition, soils would only settle in a normal manner due to a pressure increase. The graph has additional usefulness in presenting data so the quality of denseness of soils in the case III category can be evaluated. For example, dry densities and liquid limits plotted close to the case II line may not be susceptible to collapse, but may be critical with respect to pressure increases; whereas soils with conditions plotted lower in the case III area would result in more firm material. In the case of expansive soils, with high liquid limits and very high in-place dry densities, plotting very low on the graph indicates susceptibility to future expansion. Therefore, in case III, a moderate range of dry densities exists that is most desirable.

Loessial soils in Kansas and Nebraska are quite uniform and have liquid limits between 30 and 40 percent. Therefore, in-place dry density and moisture content versus settlement upon wetting can be expressed in terms of in-place dry density and moisture content for these localities. Soils from other areas must be tested.

For loessial soils in Kansas and Nebraska:

- Loess with dry density less than 1,281 kg/m³ (80 lb/ft³) is considered loose and highly susceptible to settlement on wetting with little or no surface loading.
- Loess with dry density between 1,281 and 1,442 kg/m³ (80 and 90 lb/ft³) is medium dense and is moderately susceptible to settlement on wetting when loaded.
- Loess with dry density above 1,442 kg/m³ (90 lb/ft³) is quite dense and capable of supporting ordinary structures without serious settlement, even upon wetting.

- For earth dams and high canal embankments, a dry density of 1,362 kg/m³ (85 lb/ft³) has been used as the division between high, dry density loess requiring no foundation treatment and low, dry density loess requiring treatment. However, considerable compression may still occur, and defensive design techniques such as wide filters and drains may be required in the embankment.
- Generally, moisture contents above 20 percent will permit full settlement under load.

Where volume change is a potential problem, a foundation investigation must provide information not only as to soil types found, but also information on their present undisturbed state. Samples must be recovered in an "undisturbed" state.

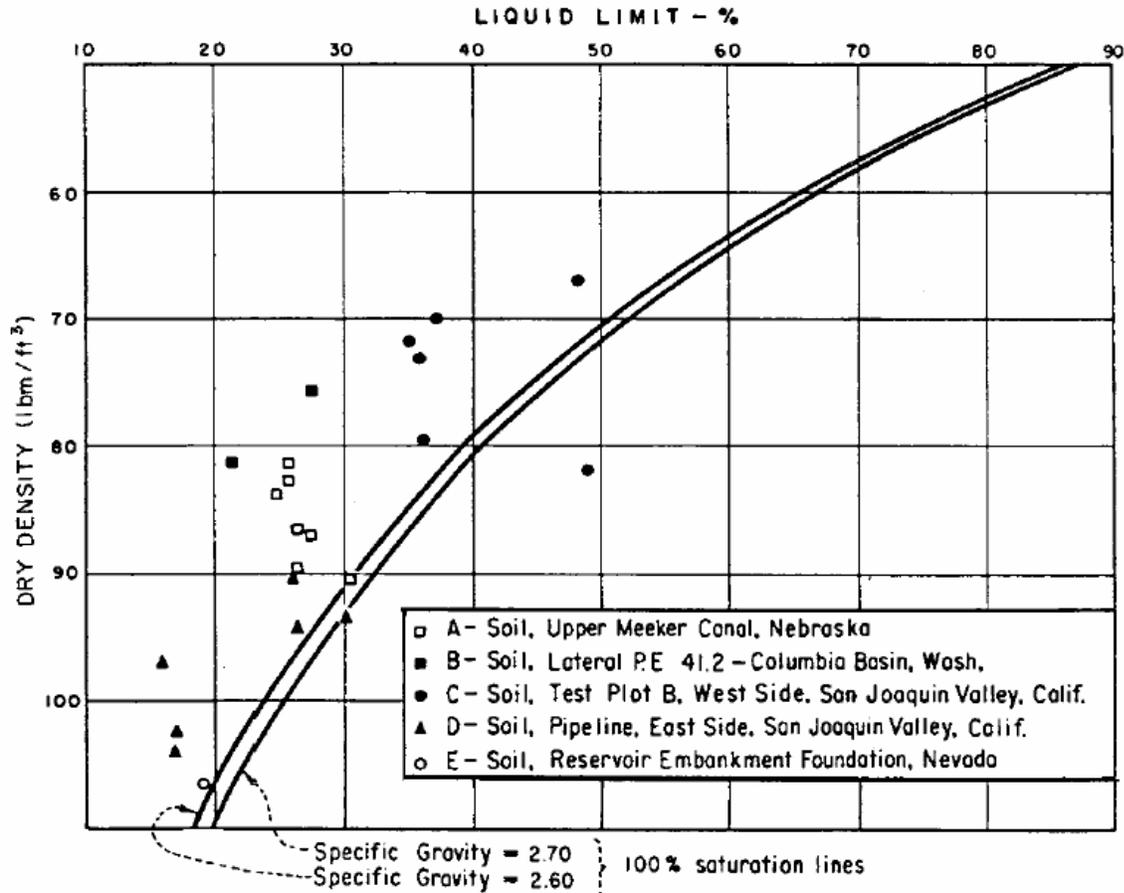


Figure-25.—Assessing collapse potential of various soils using dry density and liquid limit.

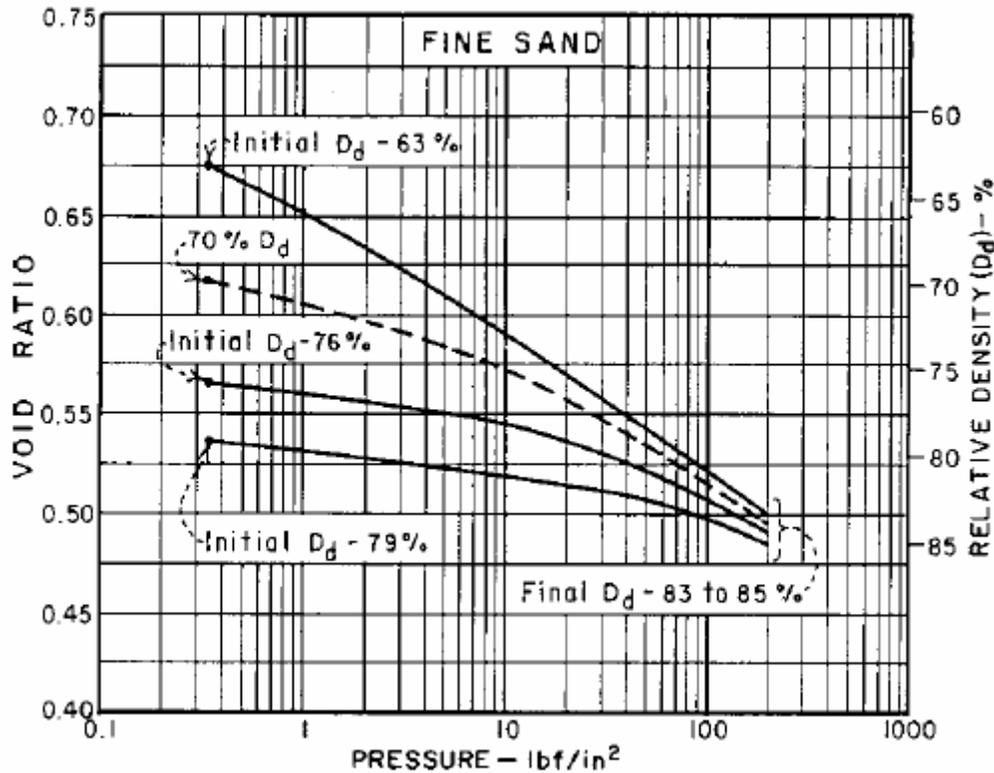


Figure-26.—Compressibility characteristics of a fine sand in relation to placement relative density.

Figure 21 shows the type of compression results obtained from testing cohesive soil. Figure 26 shows the relation of relative density and void ratio to compressibility for a compacted, fine sand.

d. Pressure-Expansion Characteristics. In addition to the normal rebound phenomenon, which occurs upon release of a compressive pressure as indicated on figure 21, certain types of clayey soils and clay shales exhibit expansive characteristics in the presence of water. The amount of expansion depends on the type of clay mineral, confining pressure, and the availability of water; expansion is a function of:

- time,
- confining pressure,
- initial density, and
- initial moisture content.

Clays containing montmorillonite are the chief sources of difficulty. Since hydraulic structures always provide a water source for soil expansion, clays must be identified and treated to avoid future failures. Figure 27 shows a comparison between pressure and expansion for two typical expansive clays. Figure 28 shows effects of placement moisture content and dry density on expansion characteristics for a typical compacted expansive clay. Graph (a) shows volume change under a pressure of 7 kPa (1 lb/in²), and graph (b) shows total uplift pressure developed when this clay was restrained from expanding. Figures 29 and 1-30 show canal lining failures caused by heaving of expansive soils. Table 2 gives criteria for identifying expansive clay.

An expansive clay can be modified by adding a small percentage of hydrated lime to the soil. Adding lime to soil has two major effects: (1) improves the workability and (2) increases the shear strength. The first effect is immediate and results from the following reactions of lime with soil:

- Soil plasticity is immediately reduced (fig. 31). The liquid limit (LL) of the soil changes very little while the plastic limit (PL) increases, thus reducing the plasticity index (PI) of the soil.
- The finer clay size particles (colloids) agglomerate to form larger particles.
- The large particles (clay clods) disintegrate to form smaller particles.
- A drying effect takes place caused by absorption of moisture for hydration of the lime, which reduces the moisture content of the soil.

The result of these reactions is to make the material more workable and more friable.

The second effect of adding lime to soil is a definite cementing action with strength of the compacted soil-lime mixture increasing with time. The lime reacts chemically with available silica and some alumina in the soil to form calcium silicates and aluminates.

The percentage of lime added to a soil depends on whether the purpose is for modification (small percent to increase workability) or for stabilization (sufficient lime to provide strength). For stabilization, lime percentage can be based on:

- soil pH,
- plasticity index reduction,
- strength gain, and
- prevention of harmful volumetric change.

When the soil-lime mixture reaches a pH of 12.4, sufficient lime has been added to react with all the soil. An optimum lime percentage exists beyond which addition of more lime slightly reduces the plasticity index of the mixture but cannot be economically justified. If a minimum strength material is needed, enough lime can be added to obtain that strength, or enough lime can be added to increase the shrinkage limit to placement moisture or higher to prevent excessive volume change through wetting and drying cycles.

23. Permeability.

a. Definition. Voids in a soil mass provide not only the mechanism for volume change, but also passages for water to move through a soil mass. Such passages are variable in size, and the flow paths are tortuous and interconnected. If a sufficiently large number of such paths act together, an average rate of flow through a soil mass can be determined under controlled conditions that will be representative of larger masses of the same soil under similar conditions.

Under gravitational forces, water movement through soil is called *percolation* or *seepage*. The measure of this movement is called *permeability*; the factor relating permeability to unit conditions is the *coefficient of permeability*, k , defined as:

$$k = \frac{Q}{A} \left(\frac{L}{\Delta h} \right) = \frac{v}{i}$$

where:

A = gross cross-sectional area through which

Q flows

Δh = head loss

i = hydraulic gradient, equal to $\Delta h / L$, or the ratio of head lost to distance over which the head is lost

k = coefficient of permeability

L = distance over which head is lost

Q = volume of water per unit time

V = discharge velocity

This equation is commonly known as Darcy's law. Temperature and viscosity of water affect the coefficient of permeability; these factors are usually accounted for in permeability determinations by correcting test results to a standard temperature of 20 deg. C.

Many units of measure are used to express the coefficient of permeability. In the past, Reclamation preferred to use feet per year (ft/yr), which is the same as cubic feet per square foot per year ([ft³/ft²]/yr) at unit gradient. Feet per day (ft/d) is a term used to some extent in canal design, while water-supply engineers favor gallons per square foot per day ([gal/ft²]/day). Most technical literature uses centimeters per second (cm/s) (fig. 32), and Reclamation will be converting to these units.

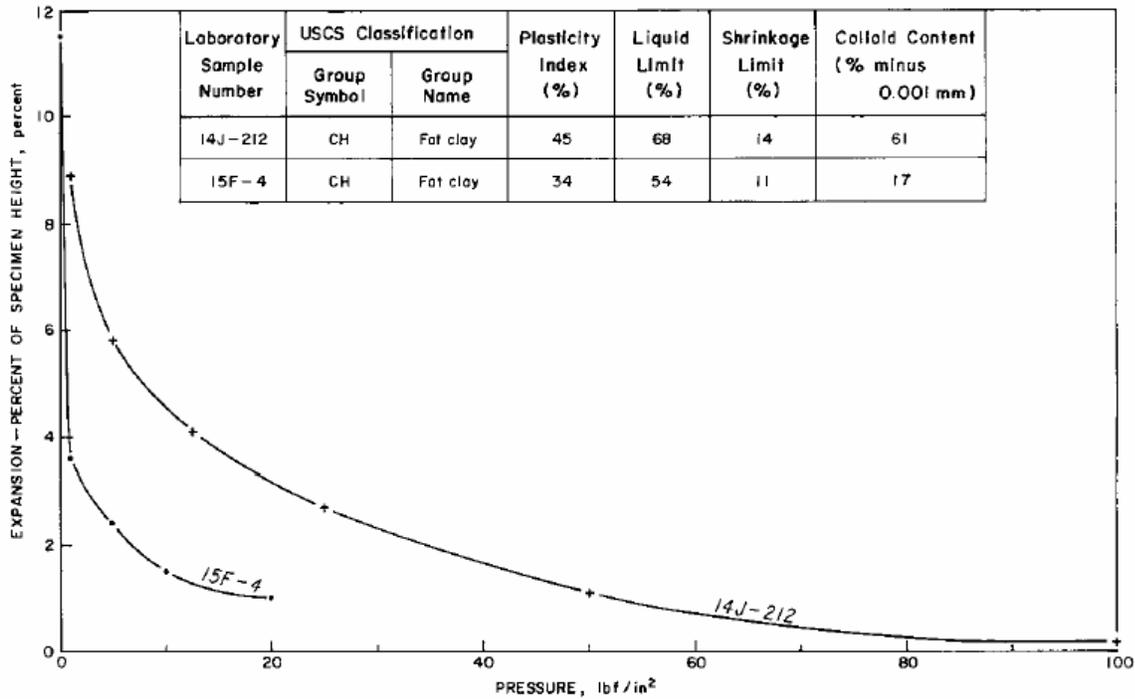


Figure-27.—Pressure-expansion curves for two typical expansive clays.

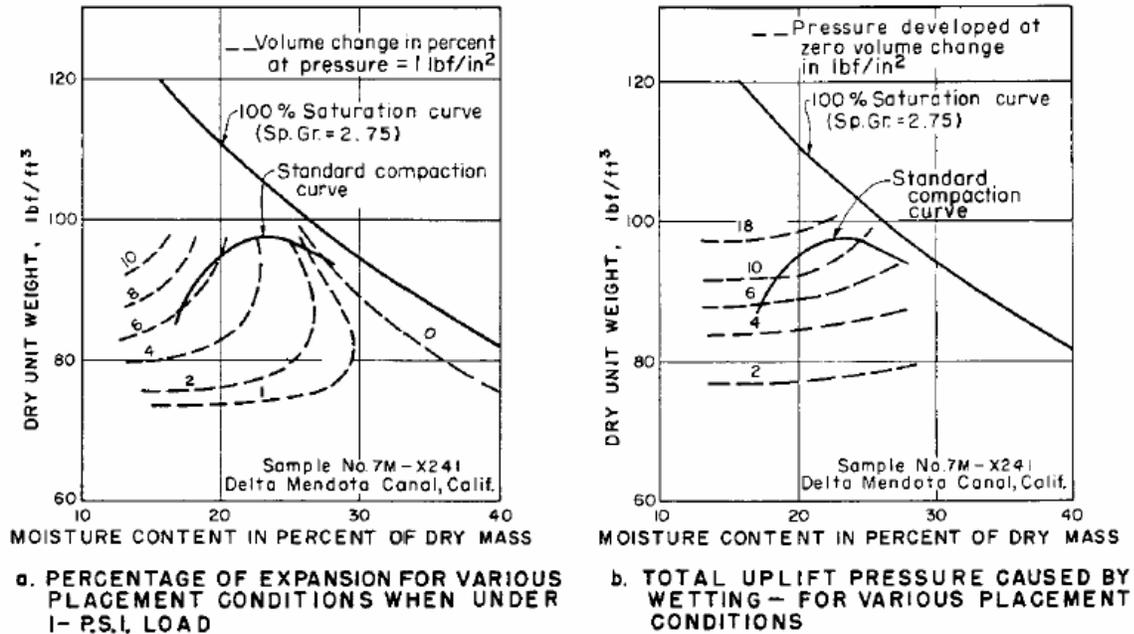


Figure-28.—Typical expansive clay. Effects of placement moisture content and dry unit weight on expansion characteristics.



Figure-29.—Slope failure and bottom heaving of Friant-Kern Canal, California.



Figure-30.—Heaving of canal bottom (about 300 mm at center), Mohawk Canal, Arizona.

Table-2.—Relation of soil index properties to expansion potential of high-plasticity clay soils. Data for making estimate of probable volume change for expansive materials

Data from index tests ¹				
Colloid content, percent minus 0.001 mm	Plasticity index, PI, %	Shrinkage limit, SL, %	Probable expansion ² , percent total volume change, dry to saturated condition	Degree of expansion
> 28	> 35	< 11	> 30	Very high
20 to 31	25 to 41	7 to 12	20 to 30	High
13 to 23	15 to 28	10 to 16	10 to 20	Medium
< 15	< 18	> 15	< 10	Low

¹ All three index tests should be considered in estimating expansive properties.
² Based on a vertical loading of 7 kPa (1.0 lb/in²)

b. Ranges of Permeability. The coefficient of permeability in natural soil deposits is highly variable. In many soil deposits, permeability parallel to bedding planes may be 100 or even 1,000 times larger than permeability perpendicular to bedding planes. An exception to this is loess in which vertical permeability is several times greater than horizontal permeability. Permeability of some soils is sensitive to small changes in density, moisture content, or gradation. In certain permeability ranges, a few percent variation in any one of these factors may result in a 1,000-percent variation in permeability. Because of possible wide variation in permeability, measurement with great accuracy is not possible for most design work; rather, the order of magnitude of permeability is important.

Soils with permeability less than 1×10^{-6} cm/s (1 ft/yr) as impervious, those with permeability between 1×10^{-6} cm/s and 1×10^{-4} cm/s (1 and 100 ft/yr) as semipervious, and soils with permeability greater than 1×10^{-4} cm/s (100 ft/yr) as

pervious. Figure 32 shows a compilation of some of the permeability testing performed at the U.S Bureau of Reclamation soils laboratory (Denver, Colorado) on compacted specimens of various soils. Figure 33 shows results of permeability tests on relatively clean, sand-gravel mixtures.

c. Control of Permeability. The determination of coefficient of permeability is important in water retention and water conveyance structures because water lost through the soil is an economic loss charged to the structure. Continuous movement of water through the soil of a structure may result in removal of soluble solids or may result in internal erosion called *piping*. Piping must particularly be guarded against because it occurs gradually and is often not apparent until the structure's failure is imminent. In nonhydraulic structures, permeability is important only when work below water table is required or when changes in water table occur in the area influenced by constructing the structure.

Seepage control is accomplished in a variety of ways that may be divided into three classes: (1) reducing the coefficient of permeability, (2) reducing the hydraulic gradient, and (3) controlling the effluent.

In embankments, permeability is reduced by selection of material through compaction control and on rare occasions by use of additives. Through foundations, seepage is reduced by several types of cutoffs formed by injection of a material into the foundation by grouting methods and densification by dynamic compaction, vibration, and loading often accompanied by changing the moisture content of the materials to hasten or enhance densification.

Hydraulic gradient reduction is accomplished either by reducing the head or increasing the length of the seepage path. Usually, some type of low permeability reservoir blanket is constructed to increase the seepage path.

Control of effluent requires a design that reduces seepage pressures sufficiently at all points so rupture of impervious zones or foundation layers is prevented and that provides filters to prevent piping. This is accomplished by zoning in earth dams and canal banks, using filters and drains, and installing drains and pressure relief wells. A number of combination control methods are in common use including wide embankment sections, impervious diaphragms, and linings. The control method selected depends primarily on cost of treatment compared to benefit received both in preventing water loss and in assurance against piping failure.

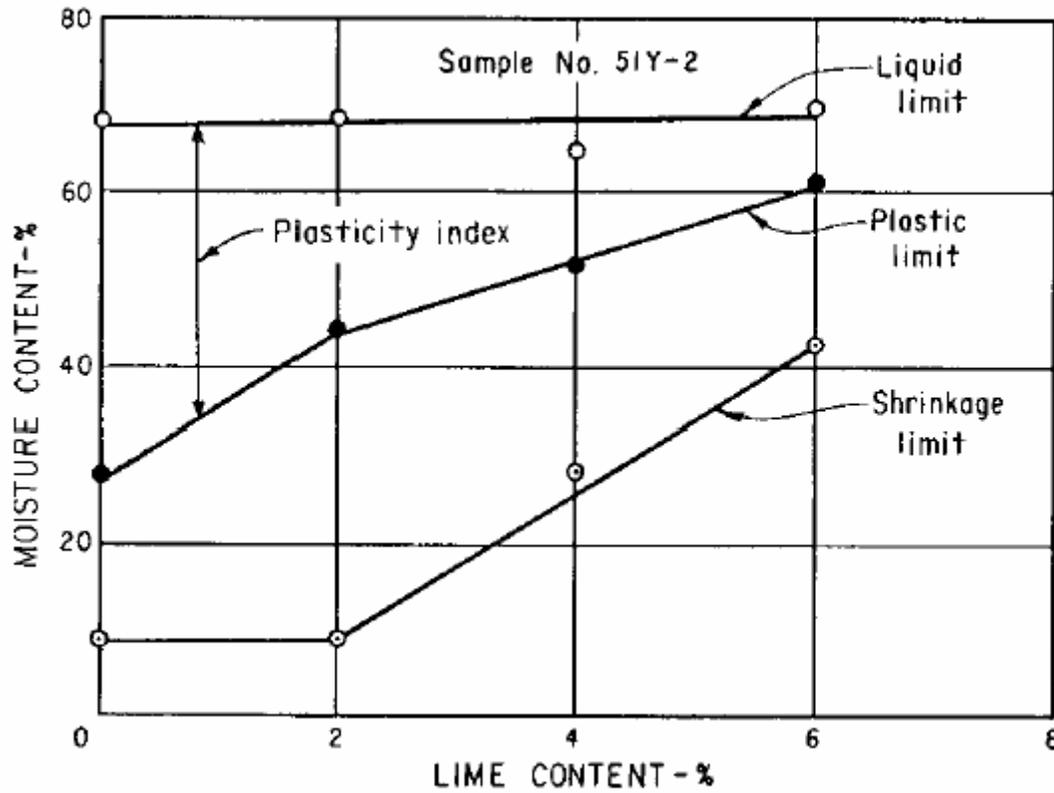


Figure 31.—Effect of lime on shrinkage limit, plastic limit, and liquid limit.

d. Determination of Coefficient of Permeability. Permeability is determined in the field by means of a variety of tests based either on forcing water through the material or removing water under controlled conditions. There are ranges of permeability for which none of these tests may be satisfactory. However, this happens so rarely that procedures have not been established for these special conditions. For groundwater studies, large scale aquifer testing may be necessary.

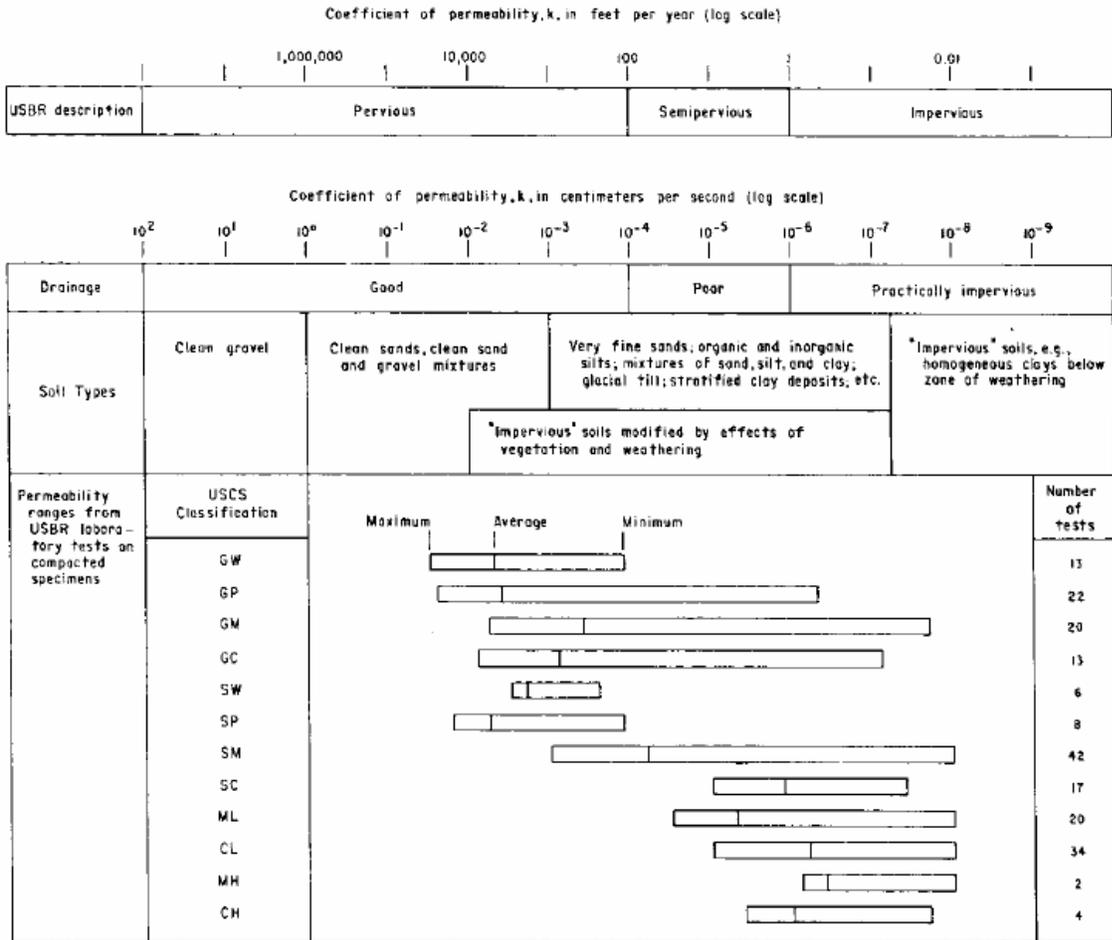


Figure 32.—Permeability ranges for soils.

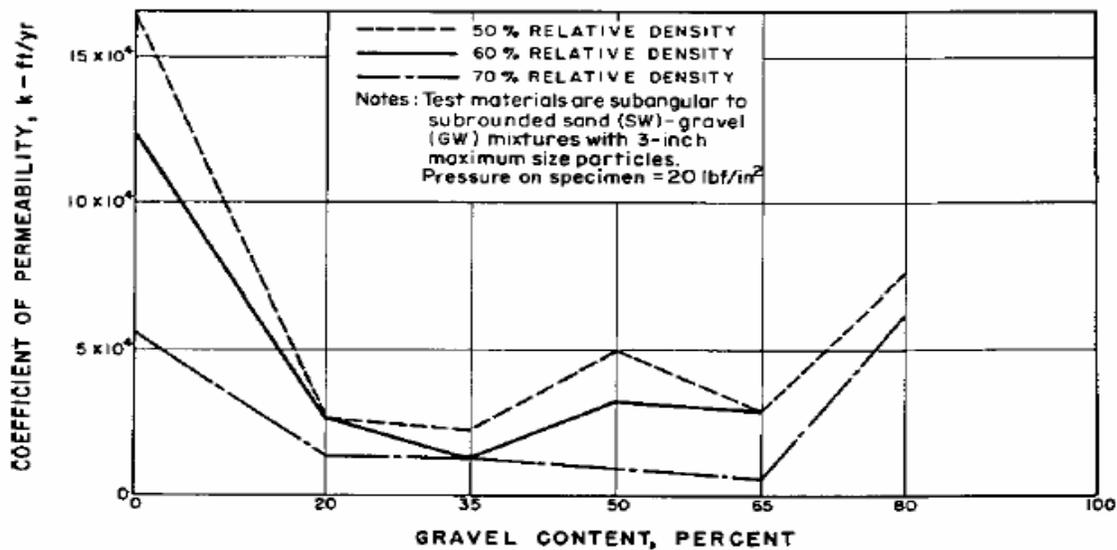


Figure 33.—Relationship of permeability to gravel content for specimens of various relative densities.

24. Engineering Characteristics of Soil Groups. Although substitutes for thorough testing have not been devised to determine the important engineering properties of specific soils, approximate average values for compacted specimens of typical soils from each USCS group are available based on statistical analyses of available data (table 3). The attempt to determine soil data from average values involves risks: (1) the data may not be representative and (2) the values may be used in design without adequate safety factors. In the early stages of project planning, when different borrow areas and design sections are being studied, these average values of soil properties can be used as qualitative guides. Because the values pertain to the soil groups, proper soil classification is of vital importance. Verification of field identification by laboratory gradation and Atterberg limits tests must be made on representative samples of each soil group encountered.

Table 3 is a summary of values obtained from more than 1,110 soil tests performed between 1960 and 1985 in Reclamation's Denver geotechnical laboratory. The data were obtained from reports on soil samples for which laboratory classifications were available and are arranged according to the USCS groups. The soils are from the 17 Western United States.

For each soil property noted in table 3, the average, minimum, and maximum test values; standard deviation; and number of tests performed are listed. Because all laboratory tests (except large-size permeability tests) were made on compacted specimens of the minus 4.75-mm (No. 4) soil fraction, data on average values for gravels were not available for most properties. The averages shown are subject to uncertainties that may arise from sampling variation and tend to vary widely.

The values for laboratory maximum dry density, optimum moisture content, specific gravity, and maximum and minimum index densities were obtained using the previously referenced test procedures. The MH and CH soil groups have no upper boundary of liquid limit in the classification system; therefore, the range of those soils is included in the table. The maximum liquid limits for the MH and CH soils tested were 82 and 86 percent, respectively. Soils with higher liquid limits than these have inferior engineering properties.

Two shear strength parameters are given for the soil groups under the heading c' and Φ' . The values of c' and Φ' are the vertical intercept and the angle, respectively, of the Mohr strength envelope on an effective stress basis as shown on figure 18. The Mohr strength envelope is obtained by testing several specimens of compacted soil in a triaxial shear apparatus in which pore-fluid pressures developed during the test are measured. Effective stresses are obtained by subtracting measured pore-fluid pressures in the specimen from stresses applied by the apparatus. Data used in compiling the values in table 3 were taken from unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial shear tests with pore-fluid pressure measurements and from consolidated- drained (CD) triaxial shear tests.

Table- 4 Engineering use data for compacted soils

Engineering Properties of Compacted Soil		Relative Desirability for Various Uses (No. 1 is considered the best)											
		Rolled earth dams					Canal sections			Foundations and fills			
		Homogeneous embankment	Core	Shell	Erosion-resistant blanket or belt	Compacted earth lining	Impervious	PerVIOUS	Resistance to frost heave				
Well-graded gravel	GW	PerVIOUS	Excellent	Negligible	Excellent	1	1	1	1	1	1	1	1
Poorly graded gravel	GP	PerVIOUS	Good	Negligible	Good	2	2	2	2	2	2	2	2
Silty gravel	GM	SemipervIOUS to impervIOUS	Good	Negligible	Good	4	4	4	4	4	4	4	4
Clayey gravel	GC	ImpervIOUS	Good to fair	Very low	Good	1	1	1	3	1	2	6	5
Well-graded sands	SW	PerVIOUS	Excellent	Negligible	Excellent	3 if gravelly	6	—	6	—	2	3	3
Poorly graded sands	SP	PerVIOUS	Good	Very low	Fair	4 if gravelly	7 if gravelly	—	7 if gravelly	—	5	4	4
Silty sands	SM	SemipervIOUS to impervIOUS	Good	Low	Fair	5	8 if gravelly	—	8 if gravelly	5 erosion critical	3	7	12
Clayey sands	SC	ImpervIOUS	Good to fair	Low to medium	Good	2	5	—	5	2	4	8	7
Silt	ML	SemipervIOUS to impervIOUS	Fair	Medium	Fair	6	—	—	—	6 erosion critical	6	9	11
Lean clay	CL	ImpervIOUS	Fair	Medium	Good to fair	3	9	—	9	3	5	10	9
Organic silt and organic clay	OL	SemipervIOUS to impervIOUS	Poor	Medium to high	Fair	8	—	—	—	—	7	11	—
Elastic silt	MH	SemipervIOUS to impervIOUS	Fair to poor	High	Poor	9	—	—	—	—	8	12	10
Fat clay	CH	ImpervIOUS	Poor	High	Poor	7	10	—	10	—	9	13	8
Organic silt and organic clay	OH	ImpervIOUS	Poor	High	Poor	10	—	—	—	—	10	14	—
Peat and other highly organic soils	PT	—	—	—	—	—	—	—	—	—	—	—	—

¹ Compacted to at least 95 percent of laboratory maximum dry density or to at least 70 percent relative density.
² ImpervIOUS: < 1 fl/yr (1 x 10⁻⁶ cm/s)
 SemipervIOUS: 1 to 100 fl/yr (1 x 10⁻⁶ cm/s to 1 x 10⁻⁴ cm/s)
 PerVIOUS: > 100 fl/yr (1 x 10⁻⁴ cm/s)

Three important engineering properties of soils typical of each classification group are listed on the chart adjacent to the group symbol. They are: (1) permeability when compacted, (2) shear strength when compacted and saturated,

and (3) compressibility when compacted and saturated. In addition, workability as a construction material is shown. Based on these properties, workability, and experience, the use chart lists the soil groups' relative desirability for use in rolled earthfill dams, compacted earthlined canal sections, and compacted backfill. The numerical ratings given in the chart are relative and are intended only

as a guide to aid the investigator in comparing soils for various purposes.

Gravelly soils are normally preferred construction and foundation materials because of their low compressibility and high shear strength. The GW and GP soils are pervious because they contain little or no fines to fill soil voids.

Ordinarily, good drainage is ensured. Their properties are not affected appreciably by saturation and, if reasonably dense, these soils have good stability and low compressibility. In these respects, GW soils are better than GP soils. The GW and GP soils are virtually unaffected by freezing and thawing.

As the sand, silt, and clay fractions increase, the matrix soils begin to dominate the gravel skeleton structure, and the total material assumes more of the characteristics of the matrix. When properly compacted, GC soils are particularly good material for homogeneous, small earthfill dams or other embankments, or for the impervious sections of high earth dams. Permeability of GC soil is low, shear strength is high, and compressibility is low.

An important factor in the behavior of gravelly soils is the gravel content at which interference between the large particles begins to influence total material properties. Extensive compaction studies demonstrated that particle interference begins to influence compaction at about 30, 35, and 45 percent gravel content, respectively, for sandy, silty, and clayey gravel soils tested, when placed using standard compactive effort. Similarly, at about the same gravel content, shear strengths show significant effects of particle interference. Soils having angular gravel particles, as compared to rounded particles, show interference characteristics at lower gravel contents.

A mixture's permeability is reduced as gravel content increases because solid particles replace permeable voids until the gravel content reaches an amount at which the matrix soils (i.e., sand, silt, or clay) can no longer fill the voids between the gravel particles. At that point, permeability increases with increase in gravel content. Structural characteristics of gravelly soils are largely controlled by density, amount and shape of gravel particles, and the amount and nature of the matrix soils. Usually, cohesionless gravel soils have high shear strength and low compressibility when placed at a relative density of 70 percent or above.

Structural characteristics of coarse sands approach those of gravelly soils, but the structural characteristics of fine sands are more like those of silty soils. As in gravelly soils, density and amount and nature of the matrix (silt and clay) control the structural properties of sand. The SW and SP soils are pervious; whereas, SM and SC soils are semipervious to impervious depending upon the amount and character of the fines. When adequately compacted, SC soils are good for impervious earthfill dams and other embankment materials because of their low permeability, relatively high shear strength, and relatively low compressibility.

Other engineering problems encountered with sands are normally related to density. Stability of pervious saturated sands remains high as long as adequate drainage is provided. Shear strength of saturated sands containing appreciable amounts of silty and clayey fines will be controlled by water content; thus, as density becomes lower and water content becomes higher, shear strength decreases.

One of the most troublesome problems encountered by geotechnical engineers is restricted drainage in saturated sands because of low permeability or impervious boundaries. If rapid loadings are applied and if soil density is sufficiently low to allow volume decrease, high pore pressure and reduced stability will result. This may cause strains of unacceptable magnitude or even total failure because of liquefaction. Seismic and equipment vibrations and vehicular traffic loads are examples of rapid loadings that must be resisted.

Coarse, cohesionless soils usually are not affected by moderate water velocities. However, fine sands can be loosened or rearranged by low velocities of waterflow. Therefore, when constructing on sand below ground-water table, the control of seepage is important to prevent particle displacement causing erosion, loosening, or quick action (quicksand).

Even small amounts of fines may have important effects on the engineering properties of coarse-grained soils. For instance, as little as 5 to 10 percent fines in sands and gravels may significantly reduce permeability and increase susceptibility to frost action.

Silts are nonplastic, fine-grained soils that are inherently unstable in the presence of water and, like fine sands, may become quick. Silts are semipervious to impervious, often difficult to compact, and are susceptible to damage by frost. Typical bulky grained, inorganic-silt soils having liquid limits of about 30 percent are less compressible than highly micaceous and diatomaceous silts that have flaky grains and have liquid limits above 100 percent.

25. Changes in Soil Properties. Although soil is commonly considered a stable material, it is constantly changing, either gradually from solid rock to increasingly finer particles or, conversely, gradually changing back to rock. In most soils, this change is sufficiently gradual that it is not a concern. However, in some soils, the change is rapid enough to be important in the life of an engineered structure. Soils where change may be important include those with appreciable quantities of: (1) organic matter, (2) soluble solids, or (3) minerals of volcanic origin. Residual soils may be in a state of chemical alteration such that during placement, they will have one set of characteristics; later, during the life of the hydraulic structure, they may have very different characteristics.

Frequently, existing soil deposits in their natural state have been stable for many years and give every indication that they will remain so. Nevertheless, human changes may result in failure of some soils. One of the soils prone to failure is called *sensitive clay*. This type of clay in an undisturbed condition has substantial shear strength, which to a large extent is lost upon being remolded. Very loose, saturated, fine sand and silt when subjected to dynamic loading such as an earthquake or vibration from machinery will lose strength and behave like a viscous liquid. This phenomenon is known as *liquefaction*. Another group of soils exists where minor changes in moisture content result in an abrupt change in shear strength. In some cases, these soils, such as loessial soils, have been deposited in a very loose state and exhibit change in shear strength and can collapse and subside when the moisture content is increased. Swelling clays frequently exhibit a change in strength characteristics caused by an increase in moisture.

Among the soils that, through desiccation, consolidation, and chemical action, have changed to forms commonly regarded as rock are varieties of shale, sandstone, and limestone. When these rocks are exposed to air, marked changes in characteristics can occur. Some shales flake off, air slake, or weather rapidly turning into soil. Some shales may dry out without any apparent effect, but if rewetted, they deteriorate rapidly into very soft clay. Some sandstones and limestones harden on exposure to air and retain their improved qualities, while other limestones and sandstones break down rapidly with fluctuating temperature and moisture content.

Although deterioration is rare, if unrecognized, failure can occur without advance warning. Engineering practices for treating soils that deteriorate are not well known. Where situations as described above are suspected, the situation should be reviewed by specialists in this field, and specialized treatment may be necessary.

26. Workability. Although laboratory testing indicates the maximum extent to which engineering properties such as shear strength, volume change, and permeability of a given soil may vary, achieving these limits in engineering practice is seldom practical. The ease with which satisfactory values of engineering properties can be economically reached is an important attribute of a soil, a soil deposit, or a foundation.

The cost to procure a unit volume of soil and place it (as in a structure) or for treating a unit amount of foundation varies widely, not only according to soil type but according to type and size of structure. Also, cost is influenced by the kind of equipment available and by current available labor. If a project is sufficiently large so that special equipment can be economically used, maximum efficiency in construction is most likely to be achieved. However, the soils selected must be workable by such methods.

Where separation of oversize is not required and where mixing requirements are minimized, borrow pits which can be preprocessed to optimum moisture content are preferable and usually more economical, even though longer haul distances may be required for their effective use. In practice, situations arise where separation of oversize is economical; also, cases exist where mixing two varieties of soil is worthwhile. Instances occur where extensive efforts to obtain maximum moisture control are justified; however, such operations should be avoided if possible.

Test procedures do not exist for measuring workability. Rather, all pertinent information concerning a soil, a borrow pit, or a foundation is tabulated so the various design possibilities can be evaluated. The engineering use chart (table 4) provides qualitative information on the workability of soils as a construction material and the relative desirability of various soil types according to structure. Borrow pits may be evaluated according to amount of work required.

Because equipment mobilization is charged against a soil deposit, unit cost decreases appreciably as the volume of work increases. The change in unit cost for excavation up to about 100,000 m³ (100,000 yd³) is noticeable; then to 1 million m³ (1 million yd³) it is gradual; and beyond that range, unit cost is nearly constant. Transportation costs are nearly constant above about 100,000 m³ (100,000 yd³), depending only on distance. Moisture control costs depend primarily on the uniformity and slope of the borrow area and the availability of water. Excavation costs are influenced somewhat by topography of the borrow area. Borrow pits slightly higher in elevation than the work structure are preferable to those below the work.

27. Frost Action. Heaving of subgrades caused by formation of ice lenses and subsequent loss of shear strength upon thawing is known as *frost action*. Water rises by capillarity and by thermal gradient toward the freezing zone and forms lenses of ice, which heave the soil. Soils most susceptible to frost action are those in which capillarity can develop but are sufficiently pervious to allow adequate water movement upward from below the freezing zone. Freezing of the pore water in saturated fine-grained soils, called closed-system freezing, decreases the density of soil by expansion but does not result in appreciable frost heave unless water movement can take place from below.

The severity of frost heave depends on three factors:

(1) type of soil, (2) availability of free water, and (3) time rate of fluctuation of temperature about the freezing point. Soils having a high percentage of silt-size particles are the most frost susceptible. Such soils have a network of small pores that promote migration of water to the freezing zone. Silt (ML, MH), silty sand (SM), and clays of low plasticity (CL, CL-ML) are in this category. Tests by the Corps of Engineers from 1950 to 1970 form the basis for figure 34; it relates frost susceptibility in terms of average rate of heave in percent by mass finer than the U.S.A. Standard 75- μ m (No. 200) sieve. The figure shows most soil types have a wide range of frost susceptibility without a sharp dividing line between frost-susceptible and nonfrost-susceptible soils. Nevertheless, silts, clayey silts, and silty sands have the highest potential for frost heave followed by gravelly and sandy clay, clayey sand, and clayey gravel. Soils with the lowest potential to heave are sandy gravels, clean sands, and silty sands with less than 3 percent finer than the 75- μ m (No. 200) sieve. In the absence of a source of free water, frost heave is limited to the increase in volume due to freezing of pore water. This upper limit amounts to about 9 percent of pore water volume.

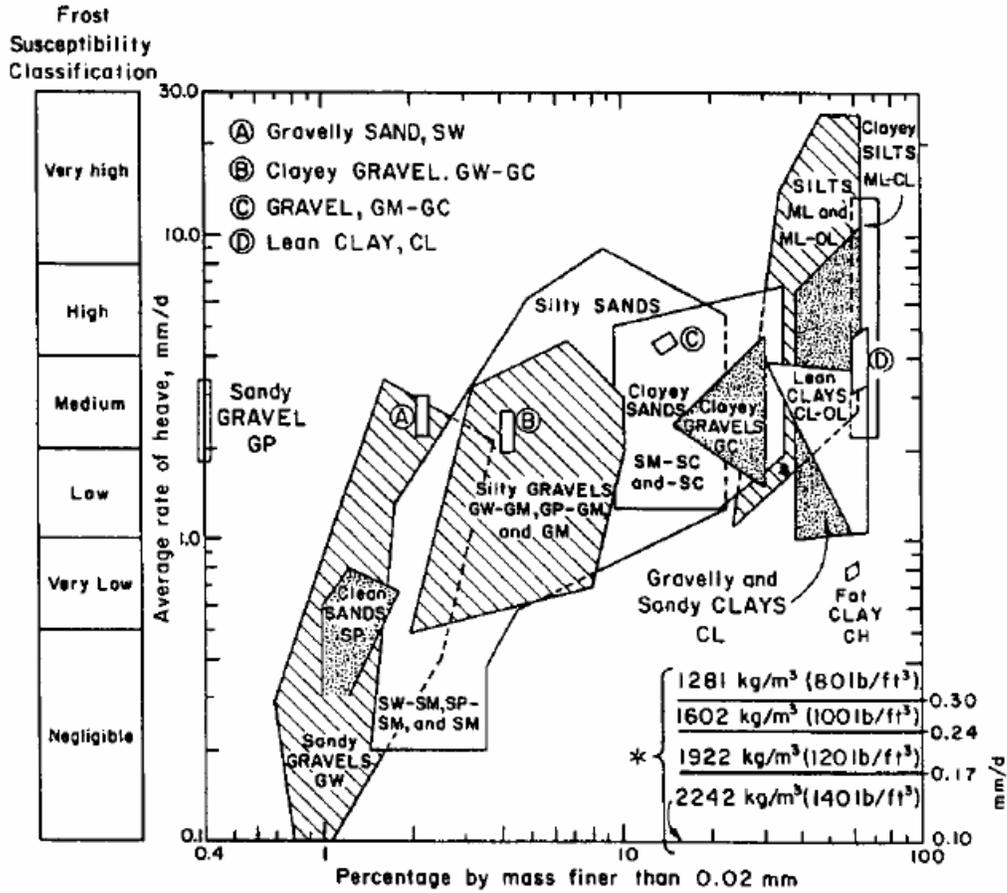
28. Erodibility. *Erosion* has been defined as ". . . a process of detachment and transport of soil particles or particle groups by the forces of water, wind, ice, and gravity." *Erodibility* is the susceptibility of a soil to erode.

The processes that influence erosion of *cohesionless* soil particles have been understood for many years. A cohesionless soil particle resting on the side or bottom of a stream or canal is acted on by gravity and by a tractive or erosive force caused by movement of water past the particle. Thus, erosion resistance of cohesionless soils depends on the applied tractive force and the mass of the particle expressed in terms of mean particle diameter (D_{50}). Figure 35 presents data collected by a number of investigators showing the relationship between critical tractive force, the force required to start erosion, and the mean particle diameter.

The processes that influence erosion of *cohesive* soils have been studied for a number of years but still are not completely understood. Over the years, investigators have attempted to correlate tractive force to various parameters and properties of cohesive soil. Strong, consistent correlation has not been found. Field performance data have been collected on operating canals and streams as well as laboratory data collected from various erosion devices, including flumes, erosion tanks, submerged hydraulic jets, and rotating cylinders. These studies have helped identify the properties that influence erosion of cohesive soils but have not provided quantifiable correlations between laboratory tests and erodibility in the field. Some soil and fluid properties that may influence the erosion process in cohesive soil include:

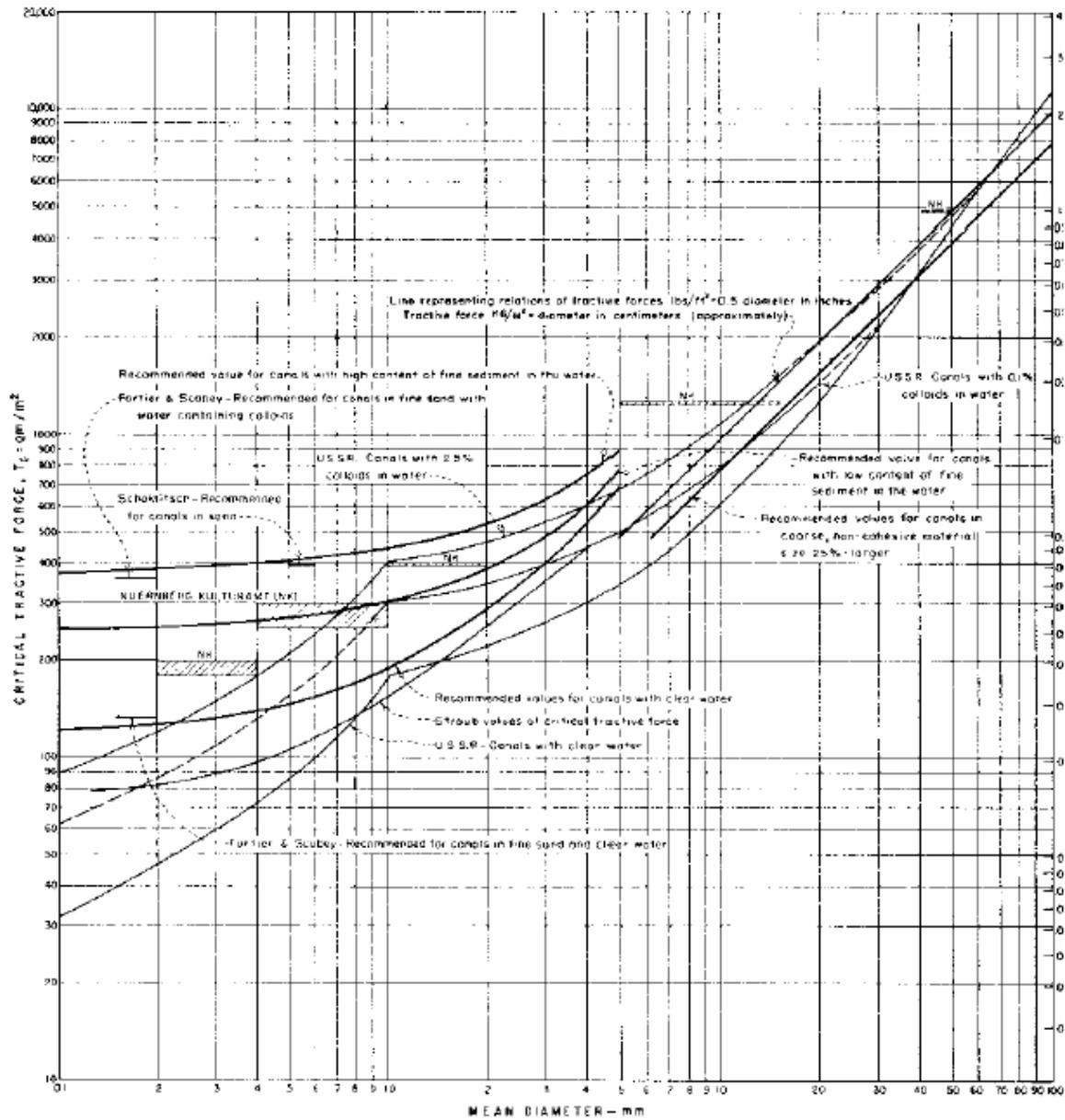
- dry density and moisture content,
- grain-size distribution,
- Atterberg limits,
- undrained shear strength,
- clay mineralogy,
- pore-water chemistry,
- stress history,

- soil structure and fabric,
- chemistry of the eroding fluid,
- temperature,
- viscosity of the eroding fluid, and
- applied tractive stress [55].



* Indicated heave rate due to expansion in volume if all original water in 100 percent saturated specimen was frozen, with rate of frost penetration 6.35 mm (0.25-in) per day.

Figure- 34.—Frost susceptibility classification by percentage of mass finer than 0.02 mm.



**LIMITING TRACTIVE FORCES
RECOMMENDED FOR CANALS
AND
OBSERVED IN RIVERS**

Figure-35.—Limiting tractive forces recommended for canals and observed in rivers.

29. Dispersive Clay. Some unique cohesive soils have been found to be highly erodible. These soils are designated *dispersive clay* because they erode when the individual clay particles disperse (go into suspension), even in the presence of still water. Dispersive clays cannot be distinguished from nondispersive clays by conventional index tests such as gradation, Atterberg limits, or compaction characteristics.

Dispersive characteristics are determined by performing three standardized tests on the questionable clay sample material. The three test results are combined to rate the clay as dispersive, intermediate, or nondispersive. The three tests are, (1) Determining Dispersibility of Clayey Soils by the Crumb Test Method, (2) Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method, and (3) Determining Dispersibility of Clayey Soils by the Pinhole Test Method. Chemical tests to determine quantity and type of dissolved salts in the pore water are also useful in determining dispersive potential of clay soils. Dispersive clays can be made nondispersive by adding a small percentage of hydrated lime (about 2 to 4 percent by dry mass of soil) to the soil. Detrimental effects of dispersive clay (in hydraulic structures) can also be minimized by proper zoning and by using designed granular filters to prevent piping failures.

30. Dynamic Properties. The response of soil to cyclic or dynamic stress application must be considered in the design of structures subjected to:

- earthquake loading,
- foundations subjected to machine vibrations, and
- subgrades and base courses for pavement.

A variety of laboratory equipment has been used to determine the dynamic properties of soil including:

- cyclic triaxial compression,
- cyclic simple shear,
- cyclic torsional shear,
- resonant column,
- ultrasonic devices.

The dynamic properties of most interest include:

- shear modulus,
- damping ratio,
- dynamic shear strength,
- pore-pressure response.

Detailed dynamic properties test procedures have not been included in this course.