contents over which a soil is plastic, and this range is called the plasticity index (PI).

Soils which will allow the formation of thicker adsorbed water layers will be plastic over a wider range of moisture contents and hence will have higher plasticity indexes. Coarse-grained soils such as sand will be plastic at no moisture content and therefore will have a PI of zero.

Engineering Characteristics:

Strength: The strength of a material is usually the first property considered in an engineering problem, and the strength of soil is an elusive amount for many reasons. For a given soil, the strength may vary with density, moisture content, lateral confinement, type of loading, and rate of loading. The problem is further complicated by the fact that soils are seldom subjected to simple tensile and compressive stresses. Rather, soil stresses occur in three dimensions and include both normal stresses and shearing stresses.

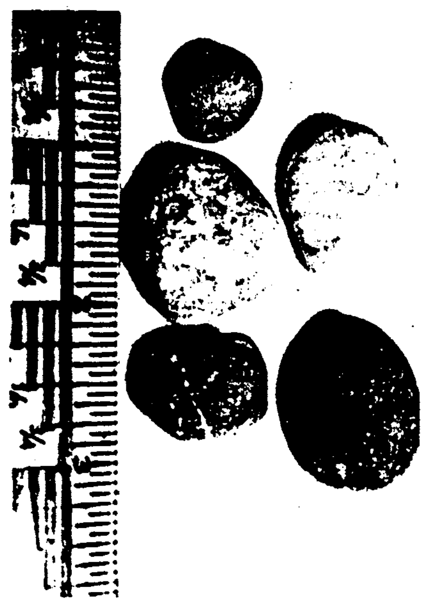
In 1776, a French scientist named Coulomb observed that the shearing resistance or strength of soil was made up of two parts; (1) one dependent upon confining pressure, and (2) one independent of confining pressure. That portion dependent upon confining pressure is called "internal friction," and the independent portion is called "cohesion." The equation for shearing strength of a soil may then be written as

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

s = shearing strength of soil
c = cohesion
σ = confining pressure or normal stress
ϕ = angle of internal friction of soil.

While many soils possess both cohesion and internal friction, it is common to classify soils as cohesive or cohesionless. Cohesive soils are those which have all or most of their shearing strength derived from cohesion. Cohesionless soils are those having little or no cohesion but which contain internal friction. Clay soils are cohesive while sands are cohesionless. Clays derive their cohesion from the large number of water films associated with these particles. These films cause an attraction of the particles for each other, giving the material cohesion.

On the other hand, sands are simple, inert particles with no attraction for each other and are cohesionless. However, sands are bulky grains which develop friction and interlock when they slide or tend to slide over each other. The greater the normal stress or confining pressure is applied, the greater is the friction and interlock. Well-graded sands and gravels possess more frictional strength than



Rounded

Angular

Subrounded

Subangular

do poorly graded ones. Other factors being equal, angular particles produce more frictional strength than do rounded ones. Frictional strength is quite dependent upon density — the more dense the soil, the more strength it possesses.

Shearing strength may be determined in the laboratory by the direct shear tests and by triaxial shear tests. The shearing strength of cohesive soils which have no frictional strength may be determined in the laboratory by the unconfined compression test and in the field by the vane shear test. When loaded quickly, saturated clays have no frictional strength. Under these conditions, the shearing strength of the soil is equal to its cohesion, which in turn equals one-half the unconfined compressive strength.

No discussion of shearing strength is complete without the inclusion of the effect of pore water pressure. When confining or normal stress is placed on a soil, it tends to force the individual grains closer together and to decrease the voids. When these voids are filled with water, pressure is developed if the excess water cannot drain. This pressure is called pore water pressure, and it tends to lessen the grain-to-grain pressure or normal pressure and hence reduces the shearing resistance. Coulomb's equation then becomes

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

where u = pore water pressure.

Pore water pressure may be taken into account by measuring it during the shear test or by performing loading at a rate slow enough to allow drainage and dissipation of pore water pressure. In determining shear strength for design, the conditions of the shearing tests should coincide with field loading conditions insofar as possible.

Three conditions are used in laboratory shear tests....

....In the quick test (or unconsolidated, undrained test), no time is allowed for drainage after placement of confining stress or after loading.

....In the consolidated, undrained test, time is allowed for consolidation or drainage from confining pressure but not from load increments.

....In the slow test (or consolidated, drained test), time is allowed for consolidation or drainage from both confining pressure and load increments. The test is performed slowly enough to allow drainage or dissipation of pore water pressures during loading.

The results of shear tests may be plotted as Mohr's circles of stress on a graph on which shearing stresses are shown as the ordinates and normal stresses as the abscissas. See Page B-22. Tests are performed at several confining pressures, and the Mohr circle for each set of stresses producing failure is plotted. An envelope or curve drawn tangent to these failure circles produces Mohr's envelope of failure. This envelope is usually assumed to be a straight line meeting the conditions of Coulomb's equation ($s = c + \sigma \tan \phi$). In this manner, the two parameters, cohesion and angle of internal friction, will define the strength of the soil. The intercept of the strength envelope with the zero normal stress line is the cohesion, and the slope angle of the envelope is the angle of internal friction. The failure or strength envelope for a partially saturated clay tested by the undrained or quick triaxial shear test is also shown on Page B-22.

The pore water pressure discussed above may also be in tension. If the voids in a soil mass are only partially filled with water, moisture films may cling to particles and form wedges between them. The surface tension of these films binds the grains together and will give sand an apparent but temporary cohesion. This phenomenon occurs on beaches at low tide.

The subjects of shearing stress of soils and pore water pressures are complex ones, and they have been simplified in this Appendix. For further information, reference is made to the Bibliography.

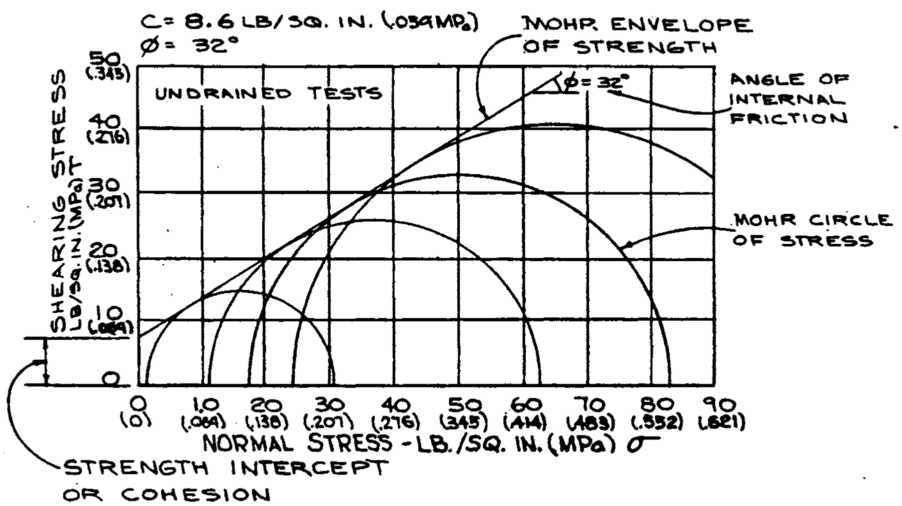
Compressibility: Fortunately, few actual shear failures occur in foundations for major structures. This type of failure, which stems from stresses being placed on a soil mass in excess of its shear strength, results in a "breaking" or shearing of the ground.

Much more common is distress to structures caused by settlement resulting from soil compressibility. This compressibility results from volume changes in soil masses and introduces problems which are peculiar to soils and which are not encountered in engineering problems with other materials.

Soil compression is produced by load and is associated with changes in soil moisture. It may also be a function of time. As previously noted, when load is placed on a soil mass, the individual grains are forced closer together, and the voids are decreased. When these voids are filled with water, pressure is developed if the excess water cannot drain immediately. If the stress continues, drainage of the excess water gradually occurs, the voids are decreased, and the soil compresses with time until the pressure in the pores is decreased to zero.

The time required for drainage of this excess pore water is a function of the permeability of the material and of the distance which the water must travel in the material. This compression which occurs over a period of time is called "consolidation." It is evaluated in the laboratory by the consolidation test, the results of which are usually plotted in the form of pressure-void ratio curves as shown on Page B-24.

If stress is increased to a certain point and time for consolidation is allowed, the material will not rebound to its original void ratio after the soil's load is removed. Instead, it will follow a somewhat flatter curve in volume increase called the decompression curve. If stress is again applied, the recompression of the soil will follow a flat curve until the stress to which the soil was previously loaded is reached. After reaching that point, consolidation will progress along a line almost joining the original or virgin curve. Thus it is evident that soil



MOHR DIAGRAM

consolidation is not a reversible process. Once a soil is consolidated or compressed, it tends to remain so even though the stress-producing compression is removed.

Pressure-void ratio curves are also plotted with the pressure shown in a logarithmic scale, as shown on Page B-24. It should be noted that the curve starts with a concave downward shape and soon reaches a straight line. The equation of the straight line portion of the curve is

$$e = e_o - C_c \log_{10} \frac{P_o + \Delta P}{P_o}$$

where C_c is a dimensionless property called the compression index,
P_o = original stress, and
ΔP = added stress.

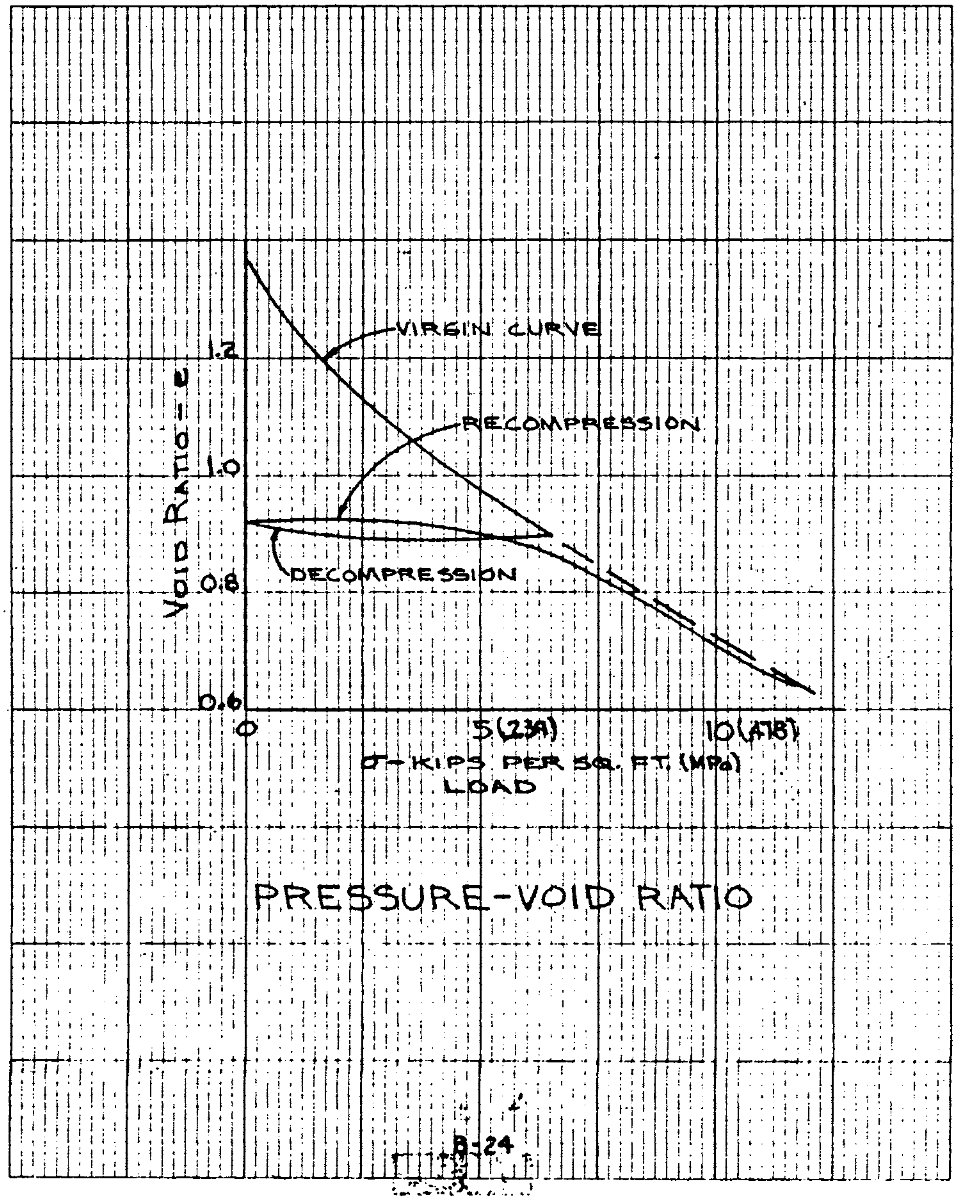
The steeper the curve, the more compressible the soil and the higher the compression index. A flat curve followed by a straight line in a semilog pressure-void ratio plot indicates a preconsolidated soil. The load or pressure at which the curve changes to a straight line roughly equals the preconsolidation load. It is easily seen that settlement is much less when loading does not exceed the preconsolidation load. Soils which have never been subjected to stresses greater than their present overburden load are termed normally consolidated. Those soils which have been subjected to greater loads (from previous overburden which has been eroded away, from glaciers, or from other means) are termed preconsolidated. These soils show less settlement than do normally consolidated ones.

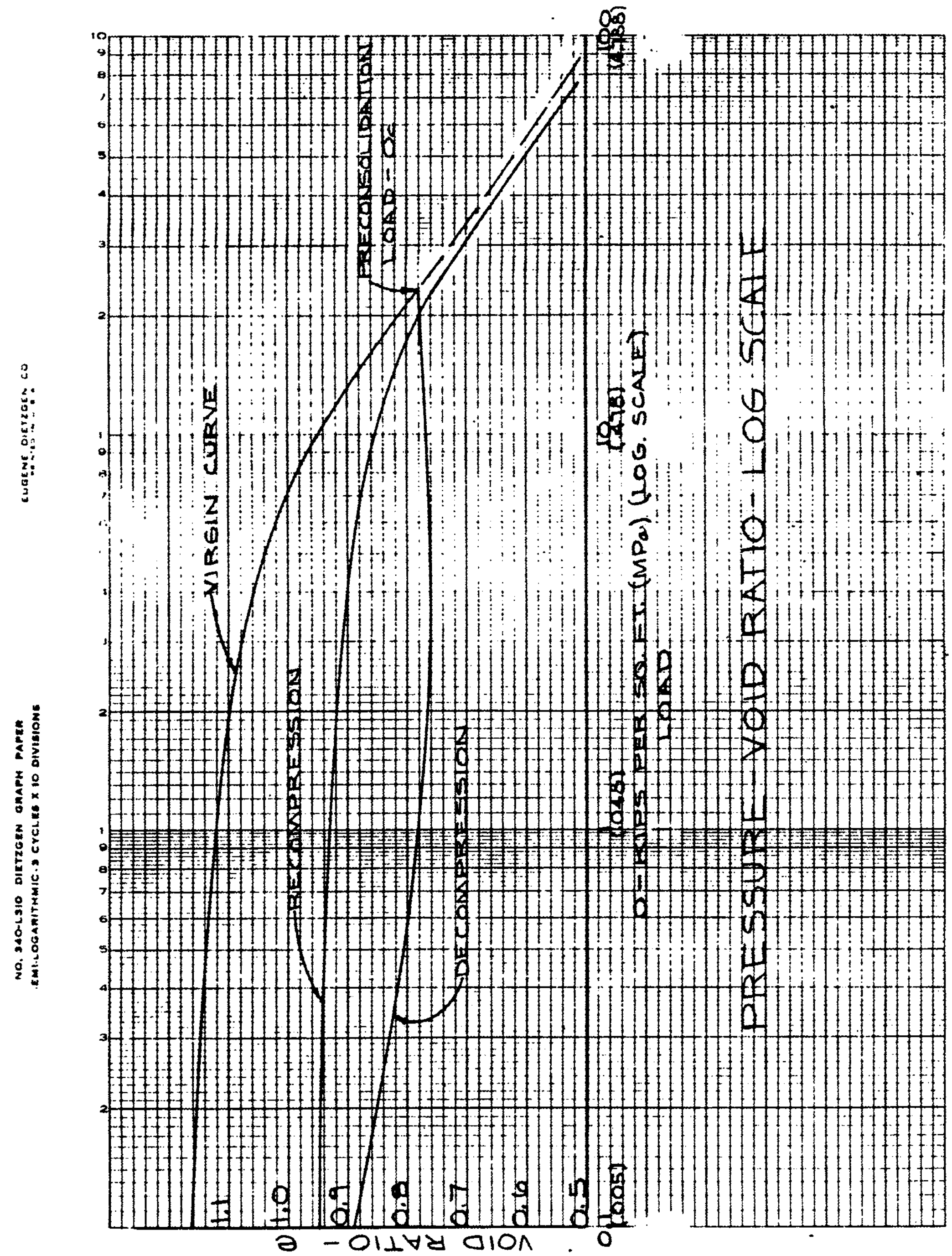
Once the change in void ratio for a stratum is determined, the change in stratum thickness or settlement due to that layer may be computed from the following equation

$$\Delta H = \frac{H \cdot \Delta e}{1 + e_o}$$

where ΔH = change in stratum thickness or settlement.
H = original stratum thickness.
e_o = original void ratio.
Δe = change in void ratio due to loading.

Soil compression includes not only the forcing of grains closer together but other actions as well. Included in these are both elastic and non-elastic processes. Under the effect of load, fracture or disintegration of the point-to-point contact of the grains is undoubtedly accomplished and is non-elastic. Some compression results from particle rearrangement which is also non-elastic. Soil compression may result from particle bending, especially in soils with flaky grain shape, and this process is probably elastic.


VIRGIN CURVE
RECOMPRESSION
DECOMPRESSION
VOID RATIO - e
1.2
1.0
0.8
0.6
0
5 (239)
10 (478)
σ - KIPS PER SQ. FT. (MPa)
LOAD
PRESSURE-VOID RATIO


NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC-3 CYCLES X 10 DIVISIONS
EUGENE DIETZGEN CO.
VIRGIN CURVE
PRECONSOLIDATION LOAD - σc
RECOMPRESSION
DECOMPRESSION
1.1
1.0
0.9
0.8
0.7
0.6
0.5
VOID RATIO - e
0.1 (.005)
1 (.048)
10 (.48)
100 (4.78)
σ - KIPS PER SQ. FT. (MPa) (LOG. SCALE)
LOAD
PRESSURE-VOID RATIO-LOG SCALE

The compressibility of a soil cannot be accurately predicted from its physical properties, but certain of these properties indicate compressible soils. It will be further emphasized in this Appendix that soils having a liquid limit in excess of 50 are classified as highly compressible. Terzaghi and Peck have derived an expression for the compressibility of normally consolidated clays as follows

$$C_c = 0.009\,(LL - 10)^*$$

Sowers has developed a relationship for soils of low plasticity and for porous rock relating the compression index to the void ratio, as follows

$$C_c = 0.75\,(e - a)^{**}$$

where a is a constant whose value is 0.2 for porous rock to 0.8 for highly micaceous soil.

Generally speaking, the higher the liquid limit and the higher the void ratio, the more dangerous the soil from the compression standpoint.

SOIL CLASSIFICATION: The types of soil and of soil deposits which cover the face of the earth are so numerous (and the properties of each deposit so variable) that the need for categories or classifications is easily recognized. The aim of any engineering classification system is to place soils having the same characteristics into one category. This task is not as easy as it may seem. If a system is complex enough to classify soil with a high degree of accuracy, it is too complicated, and the testing is too expensive for practical use. On the other hand, if the system includes simple, loose criteria, the soils within one category contain too wide a range of characteristics, and the purpose of soil classification is not accomplished.

The Unified Soil Classification System has been recommended by this Manual, and it is neither complicated nor perfect. However, the Unified System is logical; it groups soils fairly well by performance, and it lends itself well to field classification.

Unified Soil Classification System: The Unified System is based upon the sizes of the particles, the distribution of the particle sizes, and the properties of the fine-grained portion. First, the soils are divided into three

*K. Terzaghi and R. B. Peck, Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 1948.

**G. F. Sowers, Soil and Foundation Problems in the Southern Piedmont Region, Proceedings, ASCE, Vol. 80, Separate 416, 1953.

major categories; (1) coarse-grained soils, (2) fine-grained soils, and (3) highly organic soils. Second, the soil is subdivided by grain size, and then further subdivided by either gradation or plasticity characteristics.

As noted in SOIL PROPERTIES in this Appendix, under Grain Size, coarse-grained soil (sand and gravel) is that material which is retained on a No. 200 sieve, or having particle sizes larger than 0.074 millimeter. The smallest size in this category is about the smallest particle size which can be distinguished with the naked eye.

Fine-grained soil (silt and clay) is that material which passes a No. 200 sieve, or having particle sizes smaller or finer than 0.074 mm.

Highly organic soils are peat or other soils which contain substantial amounts of organic matter.

In the Unified System, soils having 50 percent or more material retained on the No. 200 sieve are classified as coarse-grained while those having less than 50 percent retained are classified as fine-grained. No laboratory criteria exist for the highly organic soils, but generally they can be identified in the field by their distinctive color and odor and by their spongy feel and fibrous texture.

Only particle sizes of 3 inches (.076 m) or less are considered in the Unified Classification. Fragments which are larger than 3 inches (.076 m) should be logged as cobbles or boulders.

Coarse-Grained Soils: The two major divisions of coarse-grained soils are gravel and sand. A coarse-grained soil having more than 50 percent of the coarse-grained fraction (fraction retained on No. 200 sieve) retained on a No. 4 sieve is classified as gravel, and it is denoted by the symbol G. A coarse-grained soil having more than 50 percent of the coarse-grained fraction passing a No. 4 sieve is classified as sand, and it is denoted by the symbol S. Coarse-grained soils are further subdivided either by their gradation (distribution of grain sizes) or by the properties of the fine-grained fraction of the soil. The classifications and criteria for each group are given in the Unified Soil Classification Chart on Page B-28. Also contained in this chart is a Plasticity Chart which is instrumental in classification by this Unified System. An additional chart, entitled Laboratory Identification Procedure, is on Page B-29, a fold-out page. This chart outlines the System in block fashion.

Less Than 5 Percent Pass the No. 200 Sieve: Those coarse-grained soils having less than 5 percent passing the No. 200 sieve are subdivided by their gradation and are given the classifications of GW, SW, GP, and SP meaning, respectively, Gravel — Well-graded, Sand — Well-Graded, Gravel — Poorly Graded, and Sand — Poorly Graded. These groups include those soils in which the fine-grained portion is so small that it should not effect engineering characteristics.

GW Group. Well-graded gravels and sandy gravels which contain little or no fines are classified as GW. In these soils, the presence of fines must have no effect on strength characteristics and on free draining characteristics. In addition to