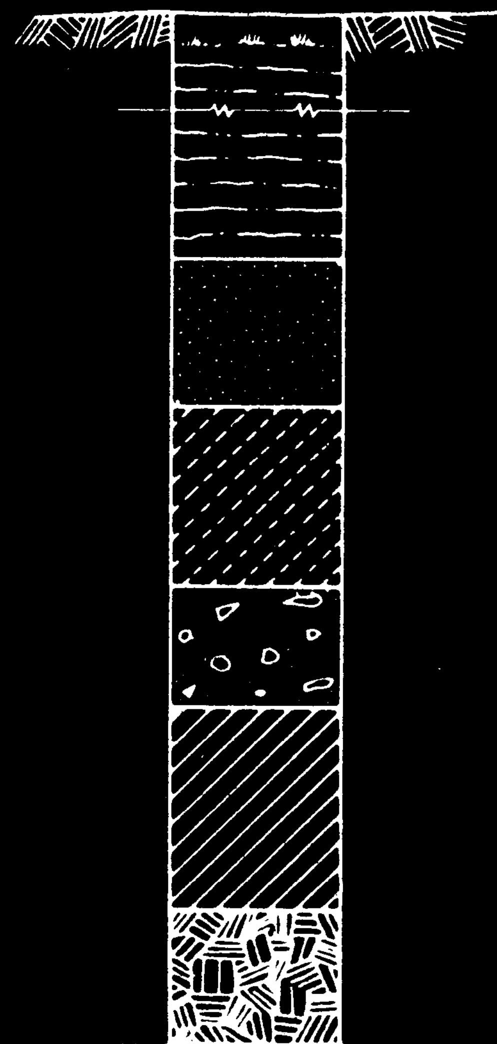
Soil as an Engineering Material



A Water Resources Technical Publication

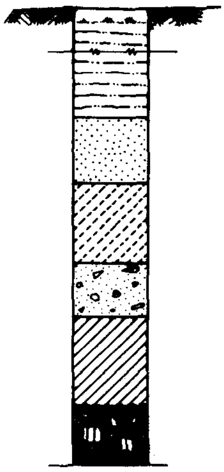
REPORT NO. 17

United States Department of the

INTERIOR

Bureau of Reclamation

Report No. 17 • A WATER RESOURCES TECHNICAL PUBLICATION



Soil as an Engineering Material

By Wesley G. Holtz
assistant chief research scientist
office of chief engineer

UNITED STATES DEPARTMENT OF THE INTERIOR

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian Reservation communities and for people who live in Island Territories under U.S. administration.

First Printing : 1969
Second Printing : 1974

UNITED STATES GOVERNMENT PRINTING OFFICE
WASHINGTON : 1974

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, or the Engineering and Research Center, Bureau of Reclamation, Attention 922, Denver Federal Center, Denver, Colo. 80225. Price $1.30.

PREFACE

Unlike metals, concrete, wood, and other common engineering materials, soils do not respond to the usual stress, strain, and strength relationships of the more elastic materials. Lacking uniformity because of the varied origins and heterogeneous compositions, soils must be sampled and tested and the data analyzed by special means and techniques.

Although soil is the oldest and most common material used by man for his works, only within recent decades has the science of soil mechanics been developed to its present state of capability. Despite the progress soils science has made, increased engineering requirements over the years to come demand ever more soils research.

The author traces this progress and the problems of the future in "Soil as an Engineering Material," originally given in abridged form as the 1968 Marburg Lecture at the American Society for Testing and Materials' (ASTM) 71st Annual Meeting in San Francisco. The purpose of the Marburg Lecture is to describe at the annual meetings, by leaders in their respective fields, the nature of a certain material and to point out its properties, current state of knowledge, outstanding developments, and/or the needs for future developments. Thus, in its treatment, this paper, which is the first Marburg Lecture on soils, is not too technical for those unfamiliar with the subject, but at the same time, is intended to provide interesting information for those familiar with the subject. Frequent use is made of the first person pronoun, in accordance with ASTM recommendations, to provide better identity between the author and his work. The entire lecture was published in the December 1968 issue of the ASTM Journal of Materials. The courtesies extended by the ASTM are gratefully acknowledged.

"Soil as an Engineering Material," while not a Research Report, has been placed in the Bureau's numbered series of Water Resources Technical Publications to provide easier classification and continuity of the series.

Much of the information in this book is derived from the Bureau of Reclamation's years of experience in sampling and testing materials, and constructing earth dams, canals, foundations, and other works.

Included in this publication are an informative abstract and list of descriptors, or key words. The abstract was prepared as part of the Bureau of Reclamation's program of indexing and retrieving the literature of water resources development. The descriptors were selected from the *Thesaurus of Descriptors,* which is the Bureau's standard for listing of key words.

Other recently published Water Resources Technical Publications are listed on the inside back cover of this report.

CONTENTS

TABLES

FIGURES

INTRODUCTION

Soil is often called our first engineering material because primitive man built his shelters in, of, and on this readily available natural material. On the other hand, we cannot predict its behavior in engineering works today with nearly the confidence that we can predict the behaviors of steel, concrete, wood, and other common building materials. This is not altogether surprising when we consider that soil deposits and formations are developed by many and varied geological processes and, therefore, are seldom homogeneous and are made up of components that have a variety of characteristics. Basically, the soils engineer must deal with this material as it exists, because it is normally not economically possible to make major modifications to its physical properties. Thus, the job of the soils engineer is one of investigation to determine the properties of the material and the reactions of a soil mass to imposed conditions.

The beginning of knowledge about the reactions of various types of soil materials to imposed loadings and other conditions undoubtedly dates back to early civilization. Artisans of the past gradually devised methods for avoiding difficulties when certain types of soils and structures were involved. Learning from experiences handed down from previous generations and from the artisans own trial-and-error methods, empirical relationships were gradually developed. These became the basis for early design and construction practices. However, these empirical methods were not always adequate, and the numerous failures that occurred are of historical record. The Campanile at Pisa, Italy, gained notoriety and became known as the Leaning Tower because of the inadequacy of its soil foundation (figure 1).

One of the first theoretical approaches to solving soil problems was made by Coulomb about 1773 [*1*] [1] in connection with determining pressures on retaining walls and the stability of banks. Coulomb recognized the importance of cohesion and friction in the solution of stability problems. He was followed by numerous investigators through the early part of the 20th century. Slope and settlement failures were studied and field soil-load tests and pile-load tests were made. Interpretations continued to be largely empirical because of the paucity of basic knowledge about the physical properties of soils. In the process, many poor concepts were propagated and used until experience proved them unsatisfactory. Notable among these were establishment of "allowable bearing values" for different types of soils and pile bearing values based on final driving resistance.

Documented failures clearly show that sound foundation practices cannot be based upon such rules. Similarly, while many earth dams, levees, and other embankments were constructed and served their purposes well, there were too many failures or partial failures which were directly related to the lack of understanding of soil behavior under the conditions imposed. Thus, confidence in these structures was lacking.

Under the pressure of necessity, soils engineering as an applied science began to develop in the early part of this century. The greatest impetus was given by Terzaghi, who was alarmed by the contrast of the high standards for concrete and steel structures and the state of ignorance which still existed in earthwork and foundation engineering. In the early 1920's, he made

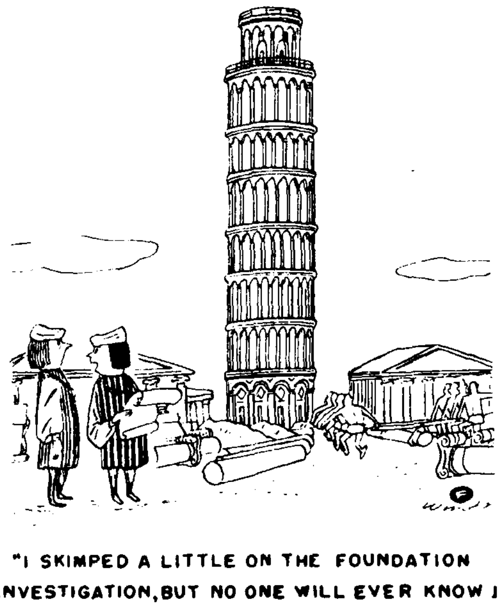


Figure 1.—**The Campanile at Pisa—a problem in soil foundations.**

[1] The numbers in brackets refer to the list of references at the end of this report.

the first attempts to coordinate and systematically apply the results of soils research and field performance data to foundation engineering practice [*2*, *3*]. He pioneered important research, initiated laboratory tests to demonstrate soil behavior, and first applied the term "soil mechanics" to the science. Much of the impetus which has led to better soils engineering practices, particularly since World War II, was due to the personal efforts and dynamic leadership of Terzaghi and his colleagues.

In the field of earth dam construction, Proctor published a series of articles in 1933 on the compaction of soils for embankment construction and the field and laboratory tests required for such earthwork construction control [*4*]. These articles formed the basis for modern earthwork construction control procedures which have provided confidence in the integrity of critical earthworks.

The practice of soils engineering has advanced greatly during the past four decades through education, research, and the development of many soils engineering laboratories. However, much is yet to be learned. An understanding of the relationships between stress, strain, and strength is an important part of applied mechanics. In the near-elastic building materials, such as steel, these relationships can be determined to a fairly high degree of accuracy. Even in the more imperfect elastic materials, such as concrete, these relationships can be determined to a fair degree of accuracy and certainly to a satisfactory degree for design purposes. In both instances, changes in these relationships with time and normal environments are usually not critical. In contrast, the stress-strain-strength relationships of soils are much more complex. Unfortunately, the simplified conditions which lend themselves to accurate analytical solutions do not exist in soils. A soil is made up of a three-phase system composed of solid matter, water, and air. The water may be bound in variable degrees, and the characteristics of the solid particles are affected by physical and chemical makeup, which may change with time and environment. When we add to these the variability of soil in place, the effects of geological history, and the critical effects of sampling and testing procedures on determined properties, obtaining usable design data becomes extremely difficult.

Although we now have developed the "science" to a point where rational designs are warranted for soil foundations and major soil structures, we must never lose sight of the fact that soil is a combination elastic-plastic material. Therefore, soil foundations and soil structures cannot be designed with the same elastic techniques that we utilize so effectively for most manufactured structural materials. Because of this and the heterogeneous nature of soils, we must add the factor of judgment based upon a great amount of experience. To achieve the confidence desired and utilize reasonable and economical safety factors, still another step is involved. This is construction control to assure that earth structures are constructed in a manner to achieve the soil properties assumed in the designs and to assure that the foundation conditions visualized during the design studies actually exist. Good soils engineering then embodies the use of the best practices in exploration, testing, design, and construction control.

THE FORMING PROCESSES OF SOILS

Geologically speaking, the earth's crust is made up of in-place rock and unconsolidated sediments. The soils engineer looks at the rock portion as a material which may resemble soil or solid rock depending upon its degree of weathering. The sediments are derived from rock formations through physical disintegration and chemical decomposition processes and deposited through wind, ice, gravity, or water action. Soil, as considered by the engineer, consists of solid particles with varying amounts of water, air or other gases, and organic material. Depending upon the geological processes, soil deposits can vary from loose to dense, from uncemented to highly cemented, and the particle distribution can vary from poorly graded (highly sorted) to well graded (little or no sorting). As transported soils are moved and reworked, abraded, further weathered, mixed with organic materials and soluble minerals, and leached, all in varying degrees, they eventually form the material that concerns the soils engineer.

Table I shows how various types of soils are developed from parent rocks and how such factors as climate, topography, and time influence the type of soil formed. The nature and extent of weathering as controlled by climatic conditions are of particular importance to the soils engineer. In the arid and semiarid portions of the United States, rainfall is sparse and solution by percolating water is of less importance than capillary water continually drawn to the surface. Thus, the concentration and deposition of calcium and magnesium are important to the soil formation. On the other hand, in moist, temperate climates with marked seasonal changes, percolating water plays an important part in soil development, solution and decomposition are dominant, and acid soils of the iron and aluminate varieties are formed. For instance, by utilizing certain ions available in the ground water, even granitic rocks, rich in potassium and sodium but typically in calcium, can alter to montmorillonite minerals in semiarid and arid climates; whereas, if rainfall exceeds evaporation, kaolinite minerals are formed. In humid, hot climates chemical processes are very active and organic influences are pronounced. The decomposition and removal of silica takes place, iron and alumina are concentrated, and lateritic soils are formed. Organic materials influence soils formed in the humid-cold and cold climates. These various climatic influences on soil development produce soils having differing properties and behavior characteristics. Thus, the interplay of materials and energy controls the exact product formed [5, 6].

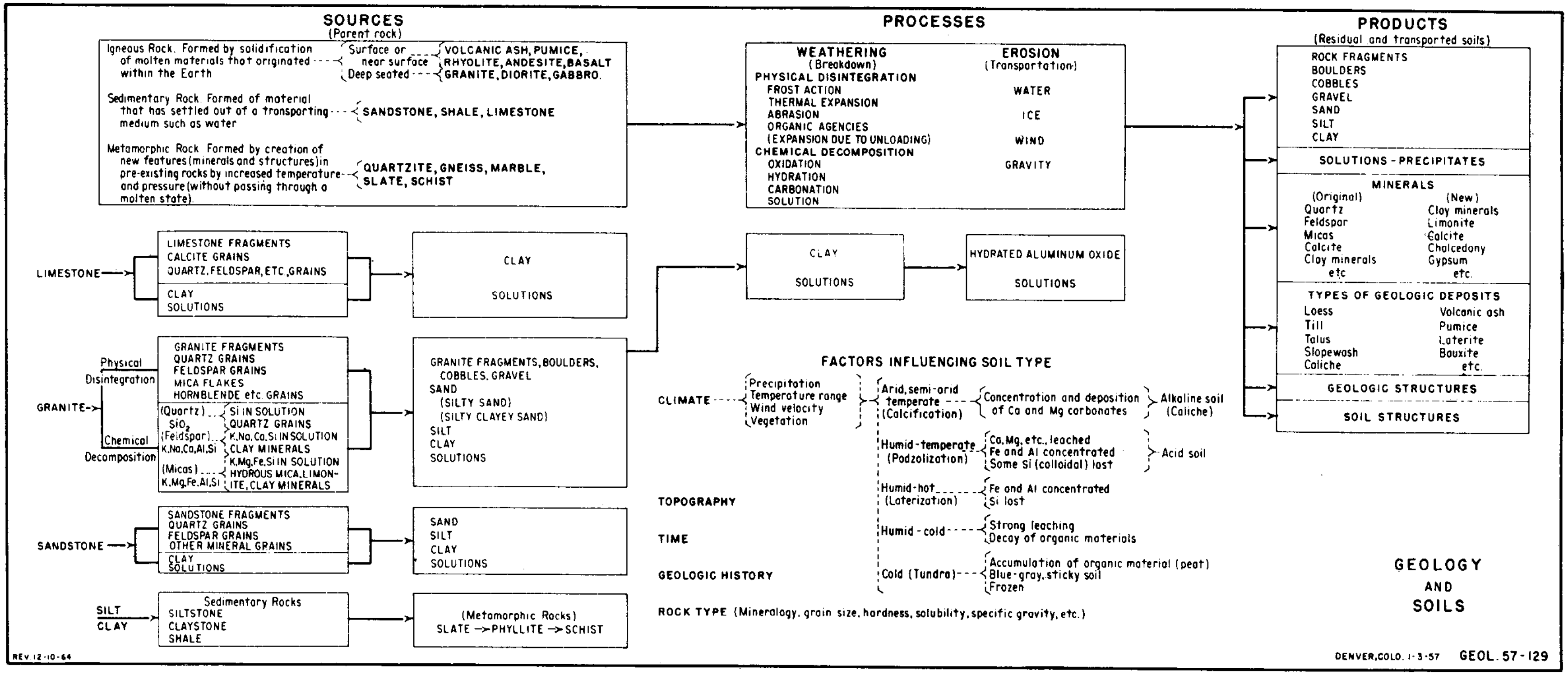


TABLE I.—*Forming processes of soils*

PROPERTIES OF SOILS

There are two groups of scientists having primary interest in the properties of soils. Agricultural scientists are interested in the crop growing ability of soils. This involves soil mineral constituents and characteristics related to water holding capacity, drainage, and workability. Soils engineers are interested in knowing how soil will act as an engineering material. Three structural properties are of primary interest to them: a. resistance of a soil mass to change in volume with changes in load or other conditions, b. ability of a soil mass to resist shear forces or lateral displacement under load, and c. permeability of the soil mass when it affects the load-volume change and shear characteristics, or when water movement must be controlled in hydraulic or other structures. Also, there are other properties which are of interest to soils engineers: gradation, specific gravity, water content, unit weight or mass density, consistency, and resistance to penetration.

Agricultural scientists, geologists, and engineers have defined four major soil components in which they all have a particular interest. These are gravel, sand, silt, and clay; natural soils are made up of various proportions of these components. Coarse-grained soils contain predominant amounts of gravel and sand, while the fine-grained soils contain predominant amounts of silt and clay. Sand-gravel soils are pervious and, if lacking in fine sizes, their engineering properties are not influenced greatly by water. Gradation, particle shape, and density largely control their engineering properties.

The silt, sand, and gravel fractions are often considered as the fabric of a soil mixture and contribute little to its plastic activity. Clay and humus are the active portions of a soil mixture because of their high specific surfaces and chemical composition. Clay soils are relatively impermeable to the passage of water, but their characteristics are greatly influenced by their water content. Their consistency may vary from hard to plastic and sticky to almost liquid as their water contents increase, and they may shrink when they become dry and swell when they become wet.

Structural Properties

Volume change.—Volume change may be in the form of volume decrease or increase, caused by loading or unloading various types of soils and is related to time, soil type, density, and water and air content. Volume decrease also can be produced to improve soils for construction purposes by rolling or vibrating. Volume increase also may occur as shear failure takes place or when clay minerals expand with increased moisture.

Effective stress and pore pressure.—When loads are applied to most common engineering materials which have reasonably elastic characteristics, the stresses and strains at any location in a structure or structural component can be determined rather directly on the basis of the applied loads and the stress-strain parameters of the material. In contrast, soils engineers must consider not only the total stress applied to a soil mass but also the effective stress which is applied to the soil grains and thus significantly controls its behavior.

When a soil is loaded, part of the load causes the soil grains to deform elastically and also to undergo non-elastic rearrangement but without significant change in their solid volume. This part of the load is carried on the soil skeleton as effective stress. The remaining part of the load is carried by the pore fluid consisting of water or air or both. The magnitude of pore fluid pressure, so developed, depends upon the relative compressibility of the soil structure and the pore fluid, the latter being related to the proportion of air and water. The unloading of a soil mass produces an opposite reaction. The permeability of the soil and its boundary conditions control the amount of pore pressure that will exist at any particular time.

If a soil is impervious or has impervious boundary conditions, a considerable length of time, perhaps years, is required for the pore fluid to escape. If a soil is free draining and has free draining boundary conditions, the pore fluid will escape as normal static loads are applied; thus, significant pore pressure will not develop. On the other hand, rapidly applied loads, such as pulsating or vibratory loads, may cause temporary pore pressure buildup.

Pore pressure in a soil may also be in the form of a capillary pressure or capillary tension (μ_c), in which the water films of a moist, unsaturated soil exert a force on the soil particles. This negative pore pressure is most significant in soils with appreciable amounts of fine-grained particles. This compressive stress on the soil particles (effective stress) can be lost as the soil becomes wetted and saturated. Thus, pore pressure can be in the form of pore air pressure (μ_a) for a soil mass devoid of water (rarely encountered in engineering problems), pore water pressure (μ_w) for a perfectly saturated soil mass, and pore air and capillary pressures ($\mu_a+\mu_c=\mu_w$) in partially saturated soils [*7–12*]. From an engineering standpoint, the latter condition is the most complex to investigate.

Normal hydrostatic pressure also can exist within saturated soil voids and must be taken into account in certain soil problems. Soils below ground water tables, soils below saturation lines in earth dams, and cut slopes in saturated soil deposits are examples of such conditions. Figure 2 shows pore pressures in an earth dam shortly after construction, which must be considered in stability analyses.

Since the pore pressure is an internal stress, the controlling factor in the stress-volume change behavior of a soil mass is not the total applied normal stress (σ), but the effective normal stress (σ') which is the total normal stress less the total pore pressure.

Compressibility, consolidation, and expansion.—Information on how much a soil mass will ultimately compress under load and what time will be required for a given portion of the ultimate compression to take place is required in the study of foundation settlement and the volume change within earthworks. The phenomenon of compressibility is associated with changes in the volume of the voids and only to a minor extent with the volume changes of the solid particles. If the voids of a soil mass are largely filled with air, the application of load will result in a relatively rapid compression; conversely, if the soil voids are filled with water, time is required for the pore water to drain from the soil mass. This time phenomenon is called consolidation.

In natural soils, particularly the fine-grained types, their past loading histories have an important bearing on their compression characteristics. When soils are gradually deposited grain by grain and layer upon layer, each element is compressed by the load of overlying elements; with the passage of time, they attain a state of equilibrium with the natural load conditions imposed. Such a soil mass is said to be normally consolidated. Different deposition processes and conditions lead to different soil structures. If part or all of superimposed load conditions, including past ice loadings, are removed through geological or manmade processes, some rebound will occur, although this may be small compared with the initial normal consolidation. Such a soil is said to be overconsolidated. In this state, if such a soil is again loaded, the volume change will be smaller than for normally consolidated soils up to the point where the applied load equals the past consolidating loads.

There are likely to be significant differences in the stress-strain characteristics of natural soils and remolded soils. The particles of remolded soils are artificially brought into their final position by working and moving the particles. Remolded soils will have a relatively young structure, while the constituents of the natural soils may not have changed their relative positions for extremely long periods of time. Thus, with the natural fine-grained soils, strong bonds may develop between particles, a phenomenon which may be insignificant in remolded soils. Figure 3 shows typical compression curves for a saturated clay soil for the normally consolidated, overconsolidated and compacted conditions. The original field compression line on this type of plot is considered to be a straight line and is referred to as the virgin compression curve.

There are times when, from a practical standpoint, it is desirable to know what length of time will be required for a certain degree of consolidation to take place. The Terzaghi consolidation theory, which is

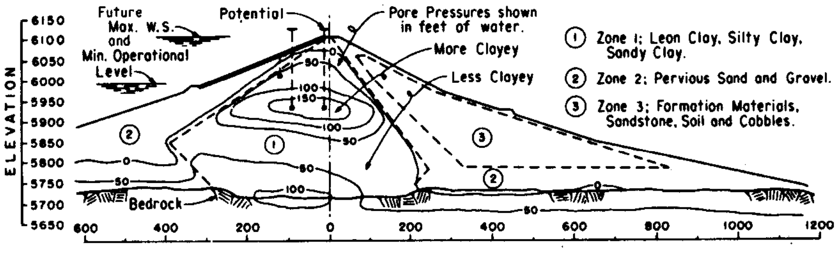


Figure 2.—**Observed pore pressures at completion of Navajo Dam, New Mexico.**

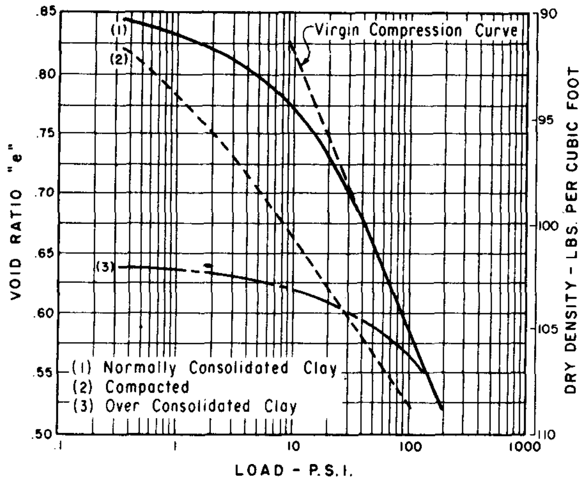


***Figure 3.*—Typical load-compression relationships for saturated clay soils.**

widely used for this determination, is based upon analyzing the time required for pore water to squeeze out of a uniform saturated clay strata into defined drainage boundaries [*3, 13, 14*]. However, such a determination becomes extremely complicated for nonhomogeneous soil deposits containing ill-defined pervious and semipervious boundaries, strata and lenses, and for unsaturated deposits and earth structures.

In addition to the normal rebound phenomena which occur in soil deposits when loads are removed, certain types of clay soils and shales exhibit expansive characteristics with water intake. The amount of expansion will depend upon the type and amount of clay mineral, initial density, water content, availability of water, load conditions, and time [*15*]. The shrinkage of these clays upon drying is an associated problem.

Quantitative compressibility, expansion, and consolidation properties of natural (undisturbed) and remolded (compacted) soils are most commonly determined in the laboratory by the one-dimensional consolidation or confined compression test for sand, silt, and clay soils (ASTM Test for One-Dimensional Consolidation Properties of Soils, D 2435–65 T). In this test, a sequence of incremental loadings is applied to a relatively thin element of soil, and the soil is prevented from moving laterally by the rigid walls of the specimen container. Porous stones at the top and bottom permit the drainage of the pore fluid during the test and also can be used to introduce water when saturation is desired. Rebound data are secured by removing the loadings in increments. The test presumes that the soil mass being studied will have no lateral displacement under the range of loadings being considered.

If meaningful data are to be secured, loading, unloading, and saturation sequences must approximate field stress and strain and other anticipated conditions before, during, and after construction. This allows the use of a stress-path type of analysis for computing settlement or uplift values [*16, 17*]. Figure 4 represents such a laboratory test on a soil core from a structure foundation. The soil was first loaded to the present in-place stress condition ①, the excavation load was removed ②, and the structure load applied ③. For general information purposes, additional loads were then applied, after which the final load was removed.

Several other types of confined compression tests are used to secure specific data. If it is desired to develop complete saturation under pressure, this may be done by utilizing a pressure chamber in which the internal water pressure in the specimen is applied in the amount necessary to dissolve all of the soil void air into the soil void water [*18, 19*]. Effective stresses in unsaturated soils can also be evaluated by utilizing very fine porous stones capable of measuring the initial capillary stresses [*18*]. The compression characteristics of gravelly soils can be determined by large-scale laboratory tests [*20*].

Shear strength.—Shear strength parameters are used to analyze the stability of embankment slopes, excavations, and footings. Compared to other building materials, the strength of soils can be classed as low, extremely variable, and subject to change with time and natural and operating conditions. The determination of meaningful shear strength parameters and their proper use constitutes the most difficult area in the practice of soils engineering.

The Coulomb equation $s = c + \sigma \tan \varphi$, where σ is the normal stress on the failure plane, c is cohesion, and $\tan \varphi$ is the friction coefficient, is used to evaluate the shear strength, s, of soil. Changes in water content, density,

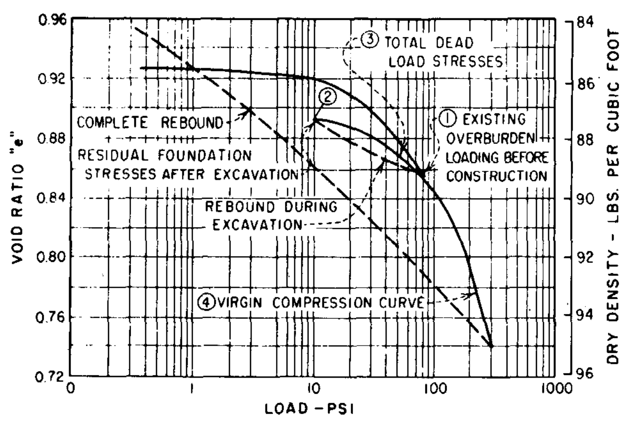


***Figure 4.*—Test load sequences for foundation soil at Dos Amigos pumping plant, California.**

and soil structure affect these parameters greatly. As in compression analyses, consideration must be given to past normal consolidation or overconsolidation, remolding, and all effects produced during the construction and operation of a structure.

Of the several types of laboratory shear tests which have been developed [*21, 22*], the triaxial shear test is the most widely used. In this test, a cylindrical soil specimen is enclosed in a thin watertight membrane, which is attached to end plates, and placed in a pressure chamber. Saturated porous end plates are provided for saturating or draining the specimen. Pore pressure measurements can be made through these end plates or through porous inserts. The specimen is surrounded by a liquid, and an ambient pressure is applied through the liquid. Incremental axial loads are applied to the specimen until failure occurs. The applied minor principal stress (σ_3) is considered to be that produced by the chamber pressure, and the applied major principal stress (σ_1) is that produced by the axial load and the chamber pressure. Several similar specimens are tested under different stress conditions, and the results are analyzed by Mohr's stress circles for each failure stress condition, or by plotting a continuous record of normal stress versus shear stress on the failure plane throughout the test. A line tangent to the Mohr circles, or through the plotted failure stress points, is called the envelope of limiting shear resistance. The tangent of the angle of the envelope, at any point, with the abscissa is known as the friction coefficient, and the intercept with the ordinate is the cohesion intercept as shown in figure 5. Two failure criteria are often considered; these are the maximum deviator stress ($\sigma_1 - \sigma_3$) and the maximum principal effective stress ratio (σ'_1/σ'_3) conditions.

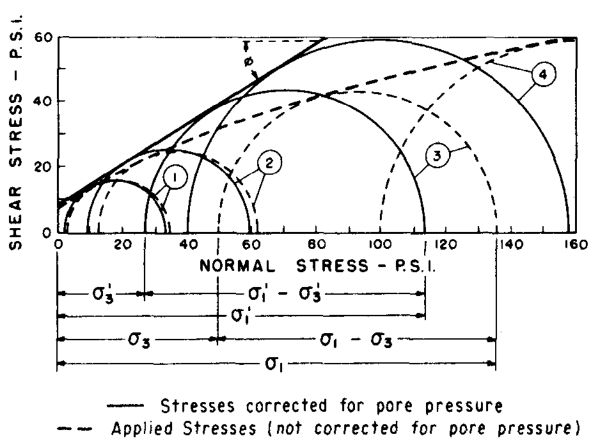


***Figure 5.*—Results of undrained triaxial shear tests with and without correction for pore air pressure.**

Triaxial shear tests can be performed with a variety of procedures, depending upon the data desired, and procedural differences will produce entirely different parameters [*23, 24*]. Therefore, it is extremely important that the procedures used be carefully programmed to represent all past and anticipated internal and external conditions. Three basic types of triaxial tests are most commonly used: a. The consolidated drained (CD) test in which the pore pressure, developed by the loadings, is dissipated by drainage during the application of each load increment; b. the consolidated-undrained (CU) test in which the pore pressure developed by the application of the chamber pressure is allowed to dissipate, and then the vertical load increments are applied at a desired strain rate or stress rate without further drainage; and c. the unconsolidated-undrained (UU) test in which the chamber pressure and vertical load increments are all applied at the desired stress rate or strain rate without drainage.

The CD test provides data for an effective stress analysis and, if this type of analysis is desired for the CU and UU tests, pore pressure measurements are made during the tests. No standard triaxial shear test has been developed in ASTM.

The design of earthworks or soil foundations for stability can be performed on total or effective stress bases. If the total stress analysis is used, complicated fixed drainage conditions must be closely duplicated in the test to provide meaningful parameters. For this reason, many soils engineers prefer to use the effective stress analysis. The available shear strength of a soil mass is then related to estimated existing pore pressures at some particular time during construction or operation by means of confined compression test data or data accumulated from field measurements of similar conditions.

A major amount of soils engineering research has been performed in the area of pore pressure measurements as related to triaxial shear testing. This has been found to be extremely difficult when testing partially saturated soils. In the effective stress analysis the Coulomb equation is written as follows: $s = c' + \sigma' \tan \varphi' = c' + \sigma - \mu) \tan \varphi'$, where the pore water pressure $\mu = \mu_a + \mu_c$. The pore air pressure, μ_a, can be readily measured through porous stone end plates or inserts if proper equipment and techniques are used. The measurement of capillary pressure, μ_c, has been difficult when the negative pore pressure exceeds the atmospheric pressure, at which point cavitation of water in the system occurs. Techniques have recently been developed which allow the measurement of large negative μ_c and μ values [25]. Very fine ceramic end plates having high air entry values are required. An example of μ_c, μ_a and μ (μ_w) values obtained during a shear test on a fine soil is shown on figure 6. What this means in terms of the shear strength parameters is shown by figure 7. This figure also illustrates how cohesion is dependent on the capillary (negative) stresses.

The field vane shear test (ASTM Method for Field Vane Test in Cohesive Soil, D 2573–67 T) is being used increasingly to determine the in-place strength of soft fine-grained soils that are difficult to sample and test in the laboratory. This procedure provides strength data which may be used for a total stress analysis and for controlling the rate of construction on soft foundations where strength gain due to consolidation is important [26].

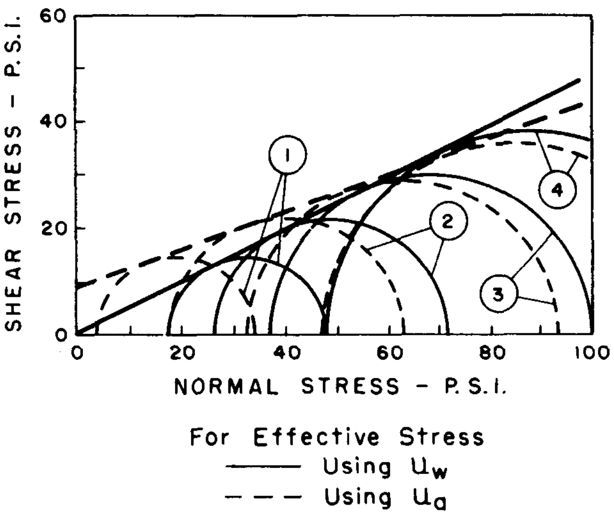


***Figure 7.*—Difference in effective shear strength utilizing pore air and pore water pressure analyses.**

Permeability

The determination of soil permeability is very important when designing hydraulic structures. The permeability of a soil also controls its drainage characteristics and, thus, is related to pore pressure dissipation, which in turn has an effect upon compression and shear strength properties at any particular time. The voids of a soil mass provide passages through which water may move. These passages are variable in size and, when the paths are considered to act together, an average rate of flow through a soil mass can be determined under controlled test conditions that will be representative of large masses of the same soil under similar conditions. The coefficient of permeability (k) is described as the rate of discharge through a unit area of soil at a unit hydraulic gradient. Field tests also have been devised to study the in-place permeability characteristics of soil deposits [27, 28]. While it is difficult to determine meaningful permeability data, these data are very important in investigations of seepage and saturation considerations for earth dams, other hydraulic earthworks, and seepage and water flow related to subsurface soil problems [29].

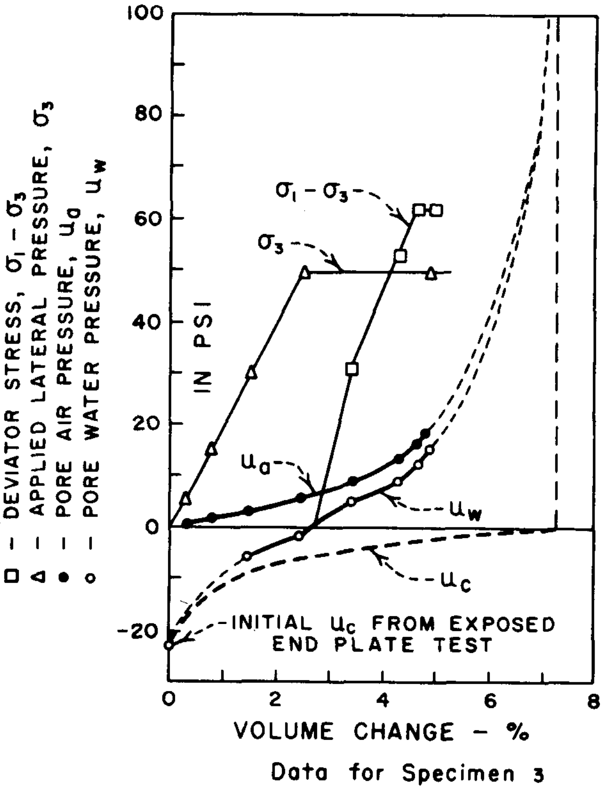


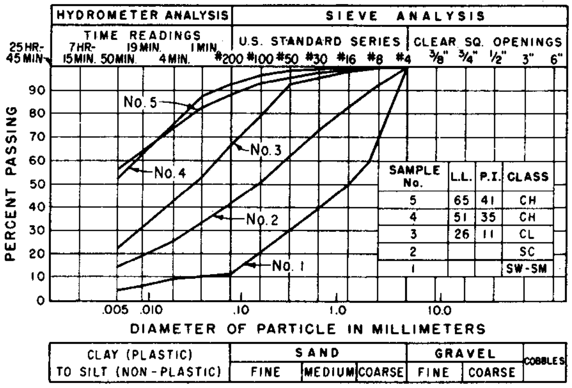
***Figure 6.*—Measured pore air, pore water, and capillary pressures during triaxial shear test.**

Gradation

The gradation, as applied to soils, is a descriptive term which refers to the distribution and sizes of grains (ASTM Method for Grain-Size Analysis of Soils, D 422–63). Figure 8 shows the gradation curves for several types of fairly well graded soils. A soil is said to be well graded if there is a good representation of all particle sizes from the largest to the smallest and poorly graded if there is an excess or deficiency of certain particle sizes within the size ranges, or if the range of predominant sizes is extremely narrow.

Specific Gravity

Specific gravity is a soil property usually determined for the purpose of evaluating the amount of solids con-



SAMPLE No.	L.L.	P.I.	CLASS
5	65	41	CH
4	51	35	CH
3	26	11	CL
2			SC
1			SW-SM

***Figure 8*.—Gradation curves for typical soils.**

tained in a soil mass and thus is used in computations for degree of saturation, porosity, and void ratio (void volume ÷ solid particle volume) (ASTM Test for Specific Gravity of Soils, Test for Specific Gravity and Absorption of Coarse Aggregate, C 127–59).

Water Content

The water content (or moisture content) of a natural or remolded soil is important because of its influence on other soil properties. It may be subject to change either from natural causes or from the operations of engineering structures. The control of water content is important in the compaction of soils. The control of water in soils used as construction or foundation materials often represents an important part of the structure cost. Water content is expressed as a percentage of the dry weight of the soil. Drying at a temperature of 110° C., which removes only the free, unbound water, is an accepted standard for determining the water content of soils (ASTM Method of Laboratory Determination of Moisture Content of Soil, D 2216–66).

Unit Weight

The unit weight or "density" of soils is an important property used as an index to soil behavior, as a physical value in soils engineering analyses, and to denote adequacy in earthwork construction. Generally speaking, both natural and remolded soils with high densities have better structural properties than soils with low densities.

The determination of accurate density values for natural soil deposits and compacted soil masses involves the use of good density-in-place testing when near the surface, or good sampling techniques when at some depth below the surface (ASTM Method for Thin-Walled Tube Sampling of Soils, D 1587–67; Test for Density of Soil In Place by the Sand-Cone Method, D 1556–64; and Test for Density of Soil In Place by the Rubber-Balloon Method, D 2167–66) [*30*]. The determination of the density of cohesionless soils is particularly difficult, especially if below the surface and below a ground water table, and advanced sampling procedures and drill mud techniques must be used. Nuclear moisture and density measuring devices have been developed during the last few years for both subsurface and surface measurements. While such density determinations may not be as accurate as those obtained by density-in-place or sampling methods, the results may be adequate for some purposes.

In 1933, Proctor [*4*] developed a compaction test which has become the most common standard for evaluating the density and moisture-density relationships of compacted cohesive soils and is often referred to as the "Standard Proctor Compaction Test" (ASTM Test for Moisture-Density Relations of Soils Using 5.5-pound Rammer and 12-inch Drop, D 698–66 T, A). Modifications of this method, utilizing different sizes of samples and different types and degrees of compaction, have been used for special purposes (ASTM Methods D 698, B, C, D, and Test for Moisture-Density Relations of Soils Using 10-pound Rammer and 18-inch Drop, D 1557–66 T, A, B, C, D. See also ASTM Method for Calibration of Mechanical Laboratory Soil Compactors, D 2168–66). The Standard Proctor Compaction Test is performed to determine the maximum density of a soil by compacting soil specimens at several water contents into a standard size cylindrical mold using a standard compactive effort. Figure 9 shows the Proctor compaction characteristics for several soil types.

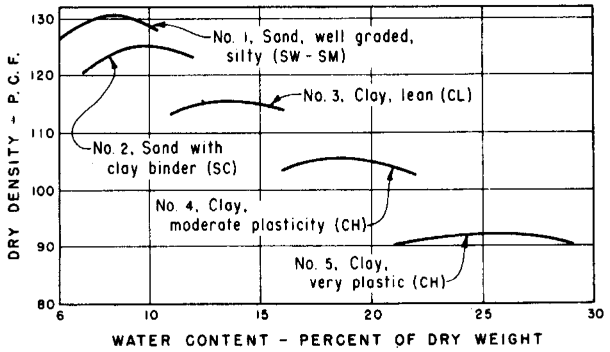


***Figure 9*.—Proctor compaction curves for typical cohesive soils.**

The Standard Proctor Compaction Test is performed on the soil fraction from which all gravel (+3/16-inch) particles have been removed. Thus, when utilizing or analyzing soils containing gravel, corrections for the weight and volume of the particles removed must be made in computing the density of the total material. Large compaction apparatus have been developed by several laboratories to determine the compaction properties of various types of gravelly soils [*31*].

Relative Density

Sand and sand-gravel soils with little or no fines do not produce definitive moisture-density relationships in the laboratory when compacted by Proctor-type impact methods. Furthermore, the densities when related to the "Proctor maximum density" do not correlate well with the engineering properties of these soils. For such soils, the "relative density" criteria have been found to be more meaningful and useful for evaluating the engineering properties of in-place subsurface soils, controlling laboratory samples, and for controlling construction quality. Relative density expresses the relationship of a soil density with the densities of the soil in its loosest and densest states and, thus, requires the determination of these two states for comparative purposes (ASTM Test for Relative Density of Cohesionless Soils, D 2049–64 T). Figure 10 shows the relative density characteristics of some cohesionless soils.

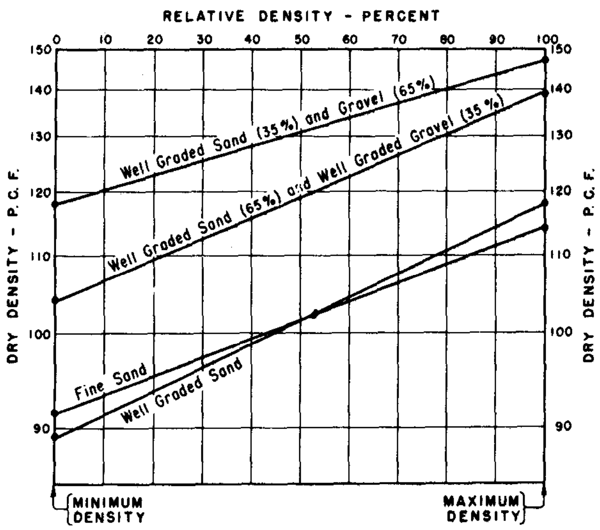


***Figure 10.*—Relation of relative density to dry density for typical cohesionless soils.**

Consistency

Depending upon the water content, fine soils or the fine fraction of coarse-grained soils can vary from a viscous liquid when extremely wet to a hard condition when dry. Four states are recognized for describing the consistency of soils. In terms of decreasing water content, these are: a. liquid, b. plastic, c. semisolid, and d. solid. Laboratory tests have been devised to define the water content limits for these states of consistency (ASTM Test for Liquid Limit of Soils, D 423–66, Test for Plastic Limit and Plasticity Index of Soils, D 424–59, and Test for Shrinkage Factors of Soils, D 427–61). As shown on figure 11, these are the liquid limit (LL), the plastic limit (PL), and the shrinkage

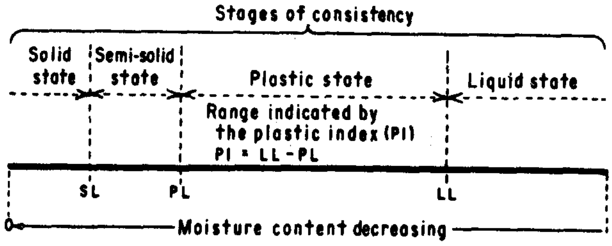


***Figure 11.*—Consistency limits.**

limit (SL), respectively. The water contents over which a soil is in the plastic state (LL-PL) is defined as the plasticity index (PI).

The consistency limits are useful to identify and classify soils and to estimate certain soil properties. The Liquid Limit-Plasticity Index Chart is useful for these purposes. Dr. Arthur Casagrande found that certain types of soils could be grouped in specific areas of the chart and established the "A Line," which divided medium- to high-plasticity soils, above the line, from low-plasticity soils, below the line (figure 12).

Penetration Resistance

The resistance to penetration of a rod or other device into a soil has been used over the years as a measure of the stiffness or denseness of a soil and is related to its shear strength. Probably the most used method for determining the in-place density or firmness of subsurface soils is known as the split tube penetration test (ASTM Method for Penetration Test and Split-Barrel Sampling of Soils, D 1586–67). The number of blows of a standard drop weight required to force the tube 12 inches into the soil at the desired test depth

562-379 O - 74 - 3

can be used to estimate the relative density of sands by the use of charts such as figure 13 [32, 33]. An indication of the relative firmness or consistency of saturated fine-grained soils can also be obtained [34, 35].

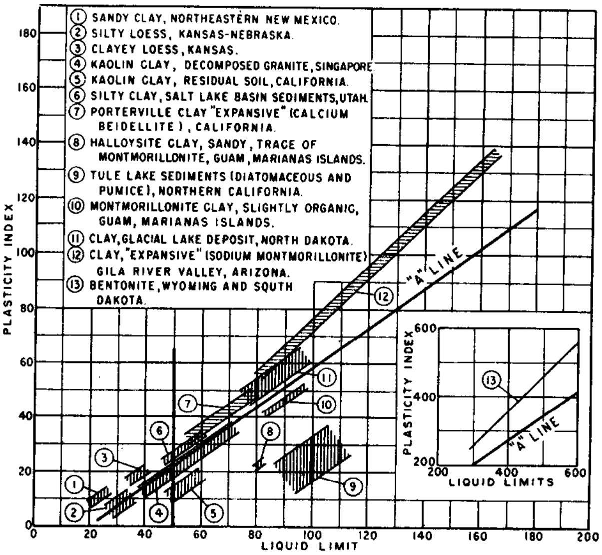


Figure 12.—**Consistency properties of various soil types.**

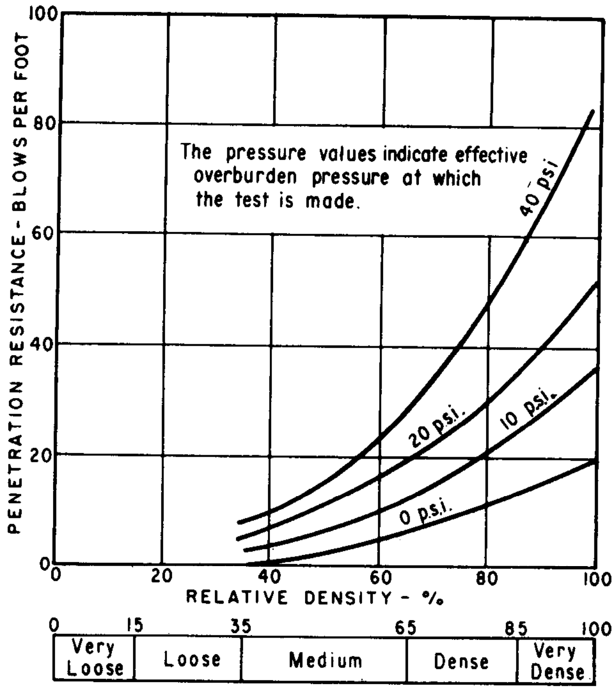


Figure 13.—**Relationship of relative density and split tube penetration resistance for various overburden load conditions.**

SOIL TYPES

Because of the varied nature of soils and their physical properties, it becomes important to consider soils on the basis of types which have certain inherent properties. For instance, normally from an engineering standpoint, it is not sufficient to evaluate a soil by its principal components, gravel, sand, silt, or clay, but some judgment must be made as to the relative amounts of the basic components and their influences on the physical properties of the total soil mixture.

Several systems have been devised over the years by which soils can be grouped or classified according to certain characteristics [*36, 37*]. Three of these have received widespread engineering use. These are known as the textural system, "A" type system, and the unified system. The textural method was first devised by agricultural scientists [*38*] and is based principally on the grain size characteristics of the soil; thus, for soils containing significant amounts of fine-grained fractions, the consistency characteristics are not sufficiently well described for engineering use. The "A" system, which was first developed in 1929 by Hogentogler and Terzaghi [*39*] for highway engineers, consisted of dividing soils into eight major groups according to texture and plastic characteristics. The Highway Research Board and the Bureau of Public Roads have been active in developing and modifying the system. The system was reviewed and revised in 1943 and further modifications have been continued [*40*]. This type of system has been reasonably well correlated with performance of subgrade and subbase soils and is generally favored by highway engineers.

Other soils engineers dealing with foundations and large earth structures were in need of a classification and grouping method which would be more descriptive of the soil groups and their properties, and thus more applicable to general engineering problems. During World War II, Dr. Arthur Casagrande developed an "Airfield Classification System" for the Corps of Engineers. This system appeared to best embody the requirements of a system for general soils engineering use. In 1952, the Bureau of Reclamation, the Corps of Engineers, and Dr. Casagrande together worked out certain modifications to the original airfield classification system as found desirable from experience and for general soils engineering purposes. This was named the Unified Soil Classification System [*41, 42*]. The system was adopted by ASTM Committee D–18 in 1966 and encompasses two standards, ASTM Method for Classification of Soils for Engineering Purposes, D 2487–66 T, and Recommended Practice for Description of Soils (Visual-Manual Procedure), D 2488–66 T. Table II summarizes basic information on the unified system, including the group symbols.

There are 15 groups in the unified system: 8 coarse grained, 6 fine grained, and 1 for soft peaty soils. Each symbol consists of two letters which may be considered as initials of the name of the most typical soil of that group. Any soil found on this earth—and on the moon—can be grouped into one of these categories.

TABLE II.—*Soil types as grouped by the Unified Soil Classification System*

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS			GROUP SYMBOLS [L]	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than No 200 seive size.	GRAVELS More than half of coarse fraction is larger than No 4 sieve size	CLEAN GRAVELS	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
	SANDS More than half of coarse fraction is smaller than No 4 sieve size.	CLEAN SANDS	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than No. 200 seive size.	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

[L] Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder

Coarse-grained soils are designated as gravel and gravelly soils with symbol G; sands and sandy soils with symbol S; and the fine-grained soils are designated as silt and silty soils with symbol M; clay and clayey soils with symbol C; and organic soils with symbol O; the peaty soils are designated by the symbol Pt. Gravel is defined as the component between the 3-inch and $\frac{3}{16}$-inch sizes; sand as that between the No. 4 ($\frac{3}{16}$-inch) and No. 200 sieve sizes; and silt and clay those components finer than the No. 200 sieve size and falling below and above the A-line, respectively (figure 12). The No. 200 sieve size is about the smallest individual particle size that can be distinguished by the eye.

Coarse-grained soils are subdivided according to their gradation and plastic characteristics. The symbol W is used to designate clean, well graded materials; P to designate clean, poorly graded materials; M to designate silty fines; and C to designate clayey fines. Thus, the symbols GW and SW, GP and SP, GM and SM, and GC and SC are used to describe these characteristics, respectively.

Each of the three types of fine-grained soils is subdivided according to its liquid limit properties. Soils of low to medium plasticity and compressibility, which have liquid limits less than 50, are designated by the symbol L; soils of high plasticity and compressibility, which have liquid limits greater than 50, are designated by the symbol H. Thus, the symbols ML, CL and OL, and MH, CH and OH are used to describe these characteristics, respectively.

Many natural soils have physical characteristics of two groups because they are close to the borderline between the groups either with respect to grain size or plasticity characteristics. In these cases, dual symbols are used, such as GW–GC, to describe a well graded, gravel-sand mixture with 5 to 12 percent clay binder, or as CL–CH for a plastic clay soil having a near 50 liquid limit. Note that the kaolin and halloysite "clays" shown on figure 12 are not true plastic clays but possess the low plastic properties of fine silts.

The flexible nature of the Unified Soil Classification System makes it particularly advantageous for engineering use for investigative, design, construction, specification, and research purposes. In describing soils for test hole logs and other purposes, the use of group symbols is not intended to substitute for any detailed descriptions which are necessary to convey a meaningful picture related to the behavior characteristics of the soil. Thus, descriptions of the natural characteristics of foundation soils are important; that is, looseness or compactness, firmness, perviousness, soil structure, and so forth. It is often important to supplement soil group names with other information such as geologic, pedologic, and local descriptions, using such words as "loess," "Pierre shale," and "Porterville clay," to fully describe known behavior characteristics.

One of the advantages of the Unified Soil Classification System is that the classification procedure can be performed with reasonable definitiveness in the field without testing equipment by the visual-manual procedures, or exactly in the laboratory utilizing standard test procedures for gradation and consistency. Normally, general logging is performed by visual-manual procedures on the site or on large groups of laboratory samples, while laboratory procedures are usually performed only as necessary on representative samples selected for detailed laboratory testing.

There are several important uses of good classification data in addition to providing a record of the types of soil in a deposit and communicating this information to other engineers. Because the classification data reflect performance characteristics, the data can be used for preliminary planning studies. When tests have been made on samples carefully selected on the basis of the classification, the data can be extended to cover nearby areas of the same material. Good test hole logs based upon this classification system not only help design engineers in their work, but also are of value to contractors, and are important from a legal standpoint when court cases involving soil problems and so-called "changed conditions" develop. In addition, the classification procedure develops a critical attitude in the minds of persons performing the work. Thus, there is a built-in tendency to develop a better philosophy about soils and to sense performance characteristics. This faculty and attitude are most essential in the practice of soils engineering.

SOIL PROPERTIES vs. SOIL PROBLEMS

The classes of soils provide a good means for delineating engineering problems which may be encountered in the overall field of soils engineering. Table III provides a very general grading as to appropriate uses of various types of soils for engineering purposes [*43*].

Gravelly Soils

Gravelly soils normally are preferred construction and foundation materials from the standpoint of their low-compressibility and high-strength characteristics. Because gravelly soils have not been considered to be problem soils, and costs for testing coarse soils are high, there has been some lack of research on these soils. However, in the past two decades there has been greater emphasis in this direction because of the desirability to use gravelly soils for earth dams and heavily loaded foundations. For these reasons, extensive studies have been conducted in Bureau of Reclamation laboratories on gravel soils containing variable amounts of sand, silt, and clay. The materials used are described in figure 14.

The GW and GP soils are pervious because they contain no fines or a very small amount of them. Good drainage ordinarily can be assured. Their properties are not affected appreciably by saturation and, if reasonably dense, good stability and low compressibility characteristics can be assured. The GW soils are better than the GP soils in these respects. Freezing and thawing conditions are not a problem.

As the sand, silt, and clay fractions are increased and this matrix begins to predominate over the gravel skeleton structure, the total material assumes more of the characteristics of the matrix. Borderline GW–GC soils are particularly good for homogeneous small earth dams or other embankments, or for the impervious sections of high earth dams when properly compacted. The permeability of this type of soil is low, friction and cohesive shear strength is good, and compressibility is low.

Looking at this from the other direction, an important factor in the behavior of gravelly soils is the gravel content at which the large particle interference begins

TABLE III.—*Engineering uses for various types of soils*

SOIL GROUP	IMPORTANT PROPERTIES				RELATIVE DESIRABILITY FOR VARIOUS USES *									
					ROLLED EARTH DAMS			CANAL SECTIONS		FOUNDATIONS		ROADWAYS		
												FILLS		
	PERMEABILITY WHEN COMPACTED	SHEARING STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	HOMOGENEOUS EMBANKMENT	CORE	SHELL	EROSION RESISTANCE	COMPACTED EARTH LINING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	FROST HEAVE NOT POSSIBLE	FROST HEAVE POSSIBLE	SURFACING
GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	1	1	—	—	1	1	1	3
GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD	—	—	2	2	—	—	3	3	3	—
GM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	2	4	—	4	4	1	4	4	9	5
GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1	1	—	3	1	2	6	5	5	1
SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	3 A	6	—	—	2	2	2	4
SP	PERVIOUS	GOOD	VERY LOW	FAIR	—	—	4 A	7 A	—	—	5	6	4	—
SM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	4	5	—	8 A	5 B	3	7	8	10	6
SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	3	2	—	5	2	4	8	7	6	2
ML	SEMIPERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	6	6	—	—	6 B	6	9	10	11	—
CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	5	3	—	9	3	5	10	9	7	7
OL	SEMIPERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR	8	8	—	—	7 B	7	11	11	12	—
MH	SEMIPERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	9	9	—	—	—	8	12	12	13	—
CH	IMPERVIOUS	POOR	HIGH	POOR	7 C	7	—	10	8 C	9 C	13 C	13 C	8 C	—
OH	IMPERVIOUS	POOR	HIGH	POOR	10	10	—	—	—	10	14	14	14	—
PT	—	—	—	—	—	—	—	—	—	—	—	—	—	—

NOTES: * - No. 1 is best. A - If gravelly. B - Erosion critical. C - Volume change critical.

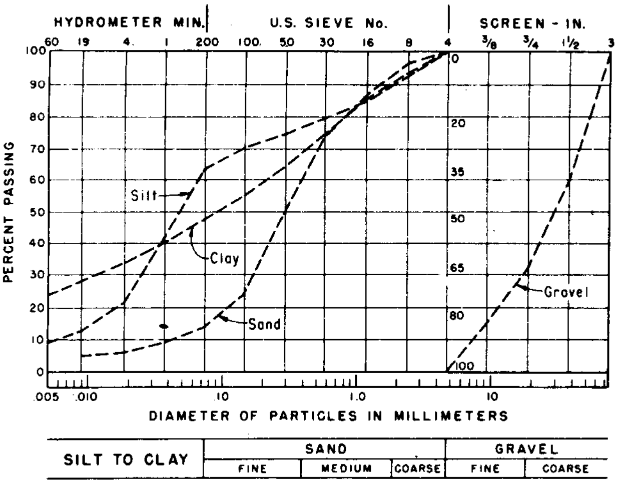


Figure 14.—**Test gradations for typical gravelly soils.**

to influence the matrix properties. From extensive compaction studies it was determined that particle interference began to influence compaction at about 30, 35, and 45 percent gravel contents, respectively, for the sandy, silty, and clayey gravel soils tested with standard Proctor compactive effort (figure 15). Similarly, at about the same gravel contents the strength characteristics show the significant effects of particle interference. Soils having angular gravel particles as compared with rounded particles have interference characteristics at lower gravel contents.

The gravel particles in the mixture reduce the permeability as gravel content increases, because solid particles replace permeable soil, until the gravel content reaches an amount at which the soil matrix cannot fill the voids between the gravel particles. At this point, permeability will increase with increase in gravel content. Figure 16 shows how the amount of gravel and density affect the permeability of sand-gravel mixtures.

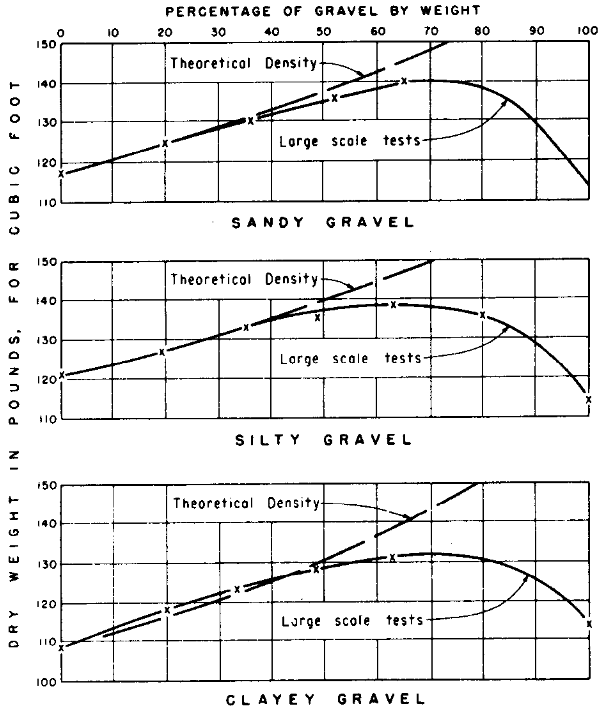


Figure 15.—**Effect of gravel content on the compaction characteristics of soils.**

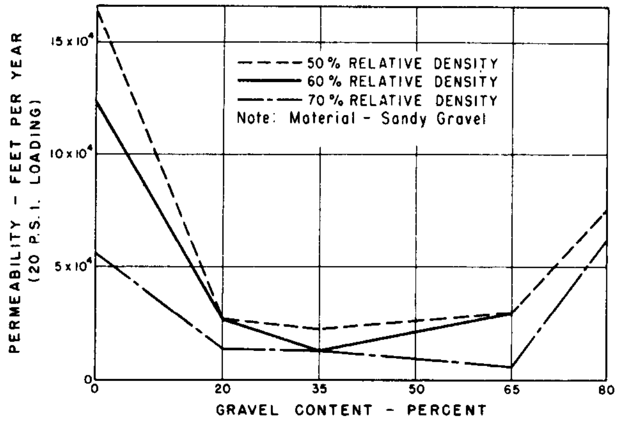


Figure 16.—**Effect of gravel content and density on the permeability of a sand-gravel soil.**

The structural characteristics of gravelly soils are then largely controlled by density, amount and shape of gravel particles, and the amount and nature of the soil matrix (figures 17–19). Generally speaking, soils which fall within the G groupings have good strength

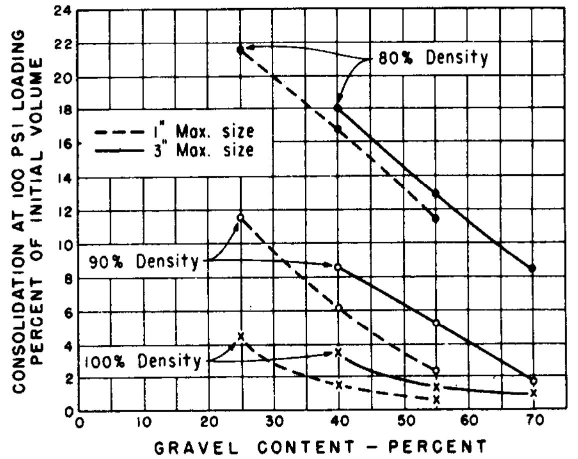


Figure 17.—**Effect of gravel content and relative density on the compressibility of a clayey-gravel soil.**

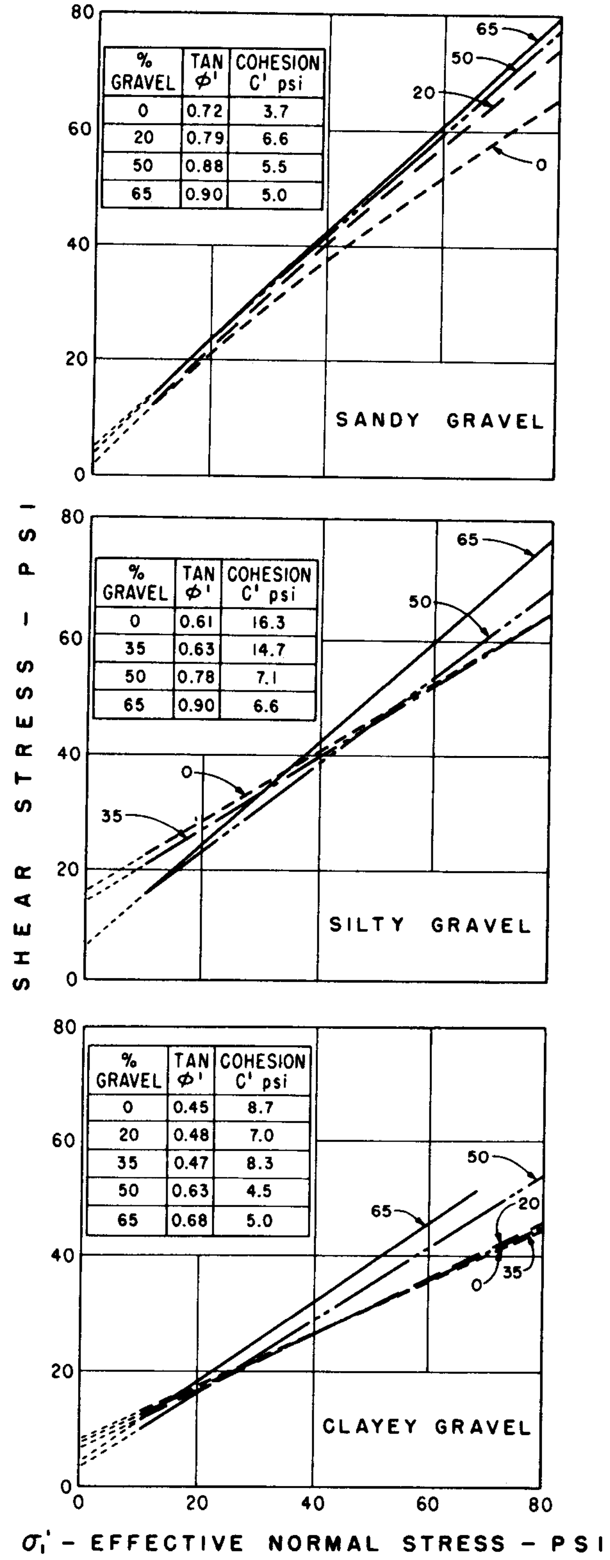


***Figure 18.*—Effect of gravel content on the shear strength of gravelly soils.**

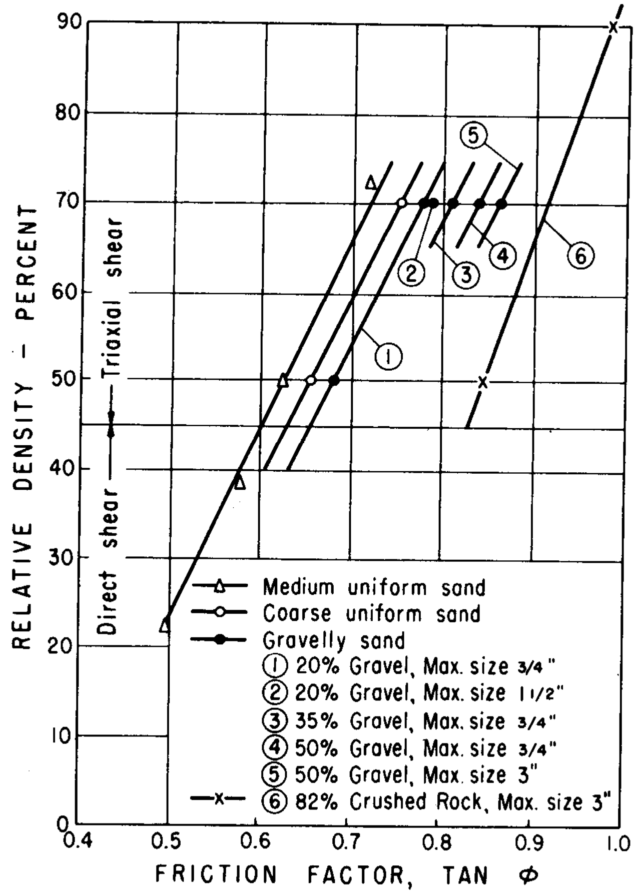


***Figure 19.*—Coefficient of friction of various sand-gravel soils.**

and low compressibility characteristics when at a relative density above 60 percent for cohesionless soils, or when at a density equivalent to that achieved by standard Proctor compactive effort for cohesive soils.

Sandy Soils

The structural characteristics of sands approach gravelly soils when they are coarse and approach silty soils when they are fine. Like gravelly soils, the density and amount and nature of the matrix (silt and clay) control the structural properties. The permeabilities of SP soils are very high, SW soils are high, and SM and SC soils are semipervious to impervious depending upon the amount and character of the fines. SW–SC and SC soils are good for impervious earth dam and other embankment materials because of their low permeability, relatively good shear strength, and relatively low compressibility, when adequately compacted.

The engineering problems encountered with sand soils, other than those related to permeability, are normally related to density. Figure 20 shows the critical

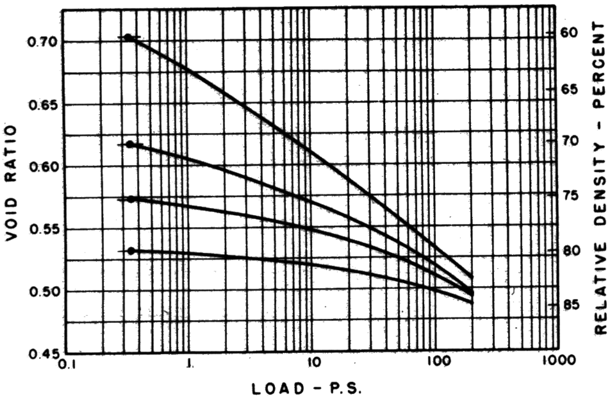


Figure 20.—**Effect of density on load-compression characteristics of a fine sand.**

nature of density of fine sand with respect to compressibility. The stability of pervious saturated sands will remain high as long as adequate drainage takes place. The strength of saturated sands containing appreciable amounts of silt and clay fines will be controlled by the water content; thus, as the density becomes lower and the water content becomes higher, the strength decreases.

The restriction of drainage in saturated sand soils, because of impervious boundaries or inadequate permeability, creates one of the most troublesome problems encountered by soils engineers. Fine sands and dirty sands with low plasticity fines are particularly troublesome. If rapid loadings are applied and if the soil density is sufficiently low to allow volume decrease, high pore pressure and reduced stability will result. This may cause unacceptable strains or complete failure. Seismic and equipment vibrations and vehicular traffic loads are examples of rapid loadings which must be resisted.

In structure foundation work, loose sands are commonly bypassed by utilizing pile footings which transmit the structure loads to underlying firm strata. The vibroflotation process, which utilizes very large (15-inch diameter) immersion vibrators, has been used successfully to consolidate relatively free draining sands to adequate density conditions. Coarse and medium sands normally exhibit good shear and low-compression characteristics if their relative densities are 70 percent or above, particularly if well graded, while fine sands require relative densities of 80 percent or above for comparable characteristics.

The most disastrous examples of this type of foundation and slope failures are related to seismic loadings caused by earthquakes. The recent damage caused by subsoil failures at Niigata, Japan, clearly illustrates the amount of damage that can result as these soils compress and lose shear strength.

The city of Niigata, having a population of 300,000 persons, is located on the Japan Sea Coast of Honshu Island. The city is largely built on fine alluvial sands deposited by the Shinano and Agano Rivers. On June 16, 1964, a major earthquake occurred which had an intensity of from 0.10 to 0.16 times the acceleration of gravity at the city. About one-third of the city, along the high ground water areas near the river, was damaged severely by liquefaction of the subsoils. Figures 21 to 24 show examples of the damage.

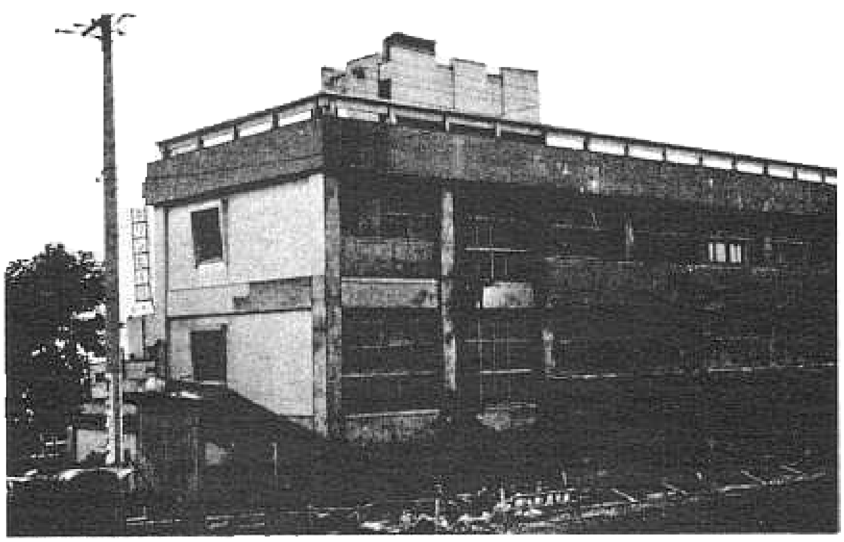


Figure 21.—**Typical structure settlement of about 2 meters during Niigata earthquake.**

Figure 22.—Loss of lateral foundation support to piles caused the collapse of Showa Bridge during Niigata earthquake.

Figure 23.—Foundation instability caused tilting, overturning, and settlement of Kawagishi-Cho apartment buildings during Niigata earthquake.

Investigations carried out by Japanese engineers and others resulted in a consensus that damage was related to the density of the sand, effective stresses existing at certain depth ranges, and the particular characteristics of the seismic disturbance [*44, 45*]. For instance, greatest damage to buildings on footings occurred when the immediate subsoil had relative densities in the 40 to 65 percent range. Even structures founded on piles 30 to 45 feet long suffered badly in sands having relative densities between 50 to almost 80 percent when liquefaction caused loss of lateral support and settlement.

Coarse cohesionless soils can resist moderate water velocities without damaging the soil density or structure. On the other hand, fine sand particles can be moved with low velocities and, thus, loosened. Therefore, when constructing on sand foundations below

Figure 24.—**Extrusion of subsoil pore water caused boils and ground slumping in school yard during Niigata earthquake.**

ground water table, it is important to control seepage water to assure that particle displacement will not occur and cause erosion, loosening, or quick action (that is, quicksand) as illustrated in the construction picture, figure 25.

Silt and Clay Soils

Low plasticity soils.—Even small amounts of fines may have important effects on the engineering properties of soils. For instance, as little as 5 to 10 percent in sand and gravel soils may significantly reduce permeability. Fines of this amount may also cause these soils to be susceptible to difficulties due to frost action.

Silt soils may vary from very hard compact and somewhat cemented siltstones capable of supporting heavy loads to very loose saturated silt deposits that in their natural state are not capable of supporting any structural load; in fact, they may, with time, consolidate under their own load. Like all soils, the higher the density the better will be the shear and compressibility characteristics.

Silts are the nonplastic fine soils. They are inherently unstable in the presence of water and, like fine sands, may become quick. Silts are semipervious to impervious, often difficult to compact, are highly susceptible to frost heaving, and have low cohesive strength. Typical bulky-grained inorganic silt soils having liquid limits of

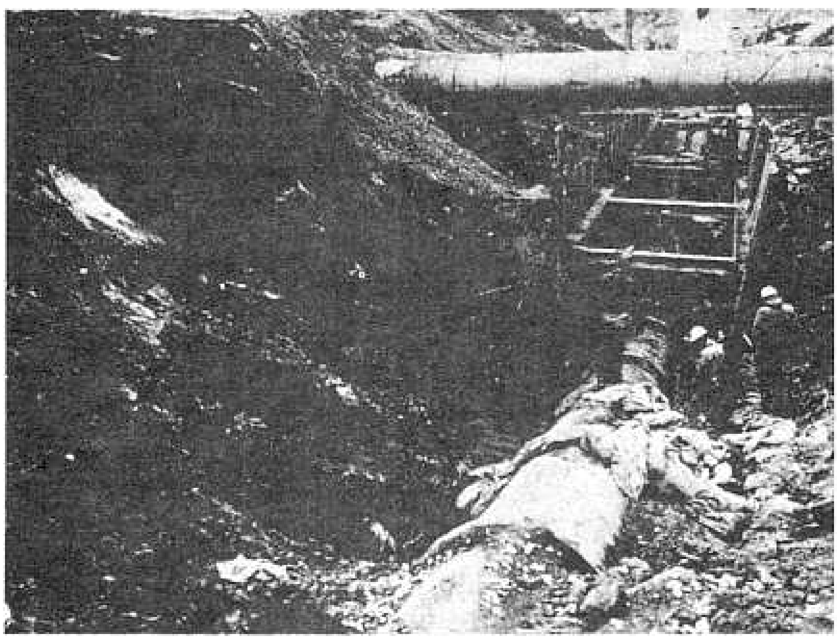


Figure 25.—**Failure of bank and liquefaction of foundation in construction excavation caused by improper control of ground water in fine sandy soils.**

the order of 30 percent are less compressible than highly micaceous and diatomaceous silts which have flaky grains and exhibit liquid limits of over 100 percent. Figure 26 shows the heaving of soil by ice lense formation.

In normal foundation engineering, when saturation exists naturally or is contemplated by operation of the engineering works, the loading of soft, compressible silty soils may be bypassed by driving piles through them to firm underlying strata [*46*]. Another method of treatment may be by preloading and draining to secure the desired consolidation and strength for the structure loads desired [*47*]. Excavation and refill with select compacted soils is a third method often used when the compressible strata are not overly deep. Organic compressible soils may be handled in the same manner, although the problem may be magnified [*48, 49*]. The San Francisco Bay muds are an example of unconsolidated compressible soils which pose foundation and settlement problems.

There are some instances when none of these methods are suitable for the engineering problem at hand. Examples of such situations were the constructing of a railroad fill and a 34-foot-high earth dam on the very low-density, fine sediments of the Great Salt Lake where split tube penetration resistance values were often zero. The procedures used are considered to be recent developments in the practice of soils engineering. In the first case, a rockfill pad was initially constructed upon which the embankment was later built as the soils consolidated and gained strength. Design modifications and the chance of some failures during construction were anticipated because of the extremely difficult foundation situation. The fill and one such failure that did occur are shown in figure 27 [*50*]. In the second case, Willard Dam was constructed on slightly firmer sediments in three separate stages, which provided adequate intervals of time for consolidation and strength gain of the foundation soils. The addition of loadings during any stage of construction was investigated by means of continuous measurements of the pore fluid pressure dissipation (figure 28) and related shear strength gain as indicated by the results of in-place vane shear tests [*51, 52, 53*]. From the beginning of construction in 1957 until the present time, the foundation of Willard Dam has settled approximately 11 feet without cracking.

The highly compressible volcanic lacustrine clay soil which underlies the central part of Mexico City offers a striking example of saturated clays which have a great capacity for compressibility. This high-plasticity porous clay soil has special properties related to the clay minerals from which it is formed. Because of ground water

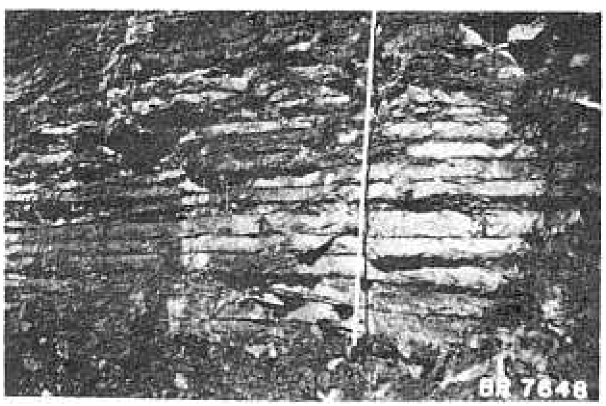


***Figure 26.*—Ice lense formation in varved silt deposit. (Photo courtesy National Research Council of Canada.)**

***Figure 27.*—Railroad fill and failure of a section during construction on a difficult soft foundation, Great Salt Lake, Utah. (Photograph by R. L. Collins, Ogden, Utah.)**

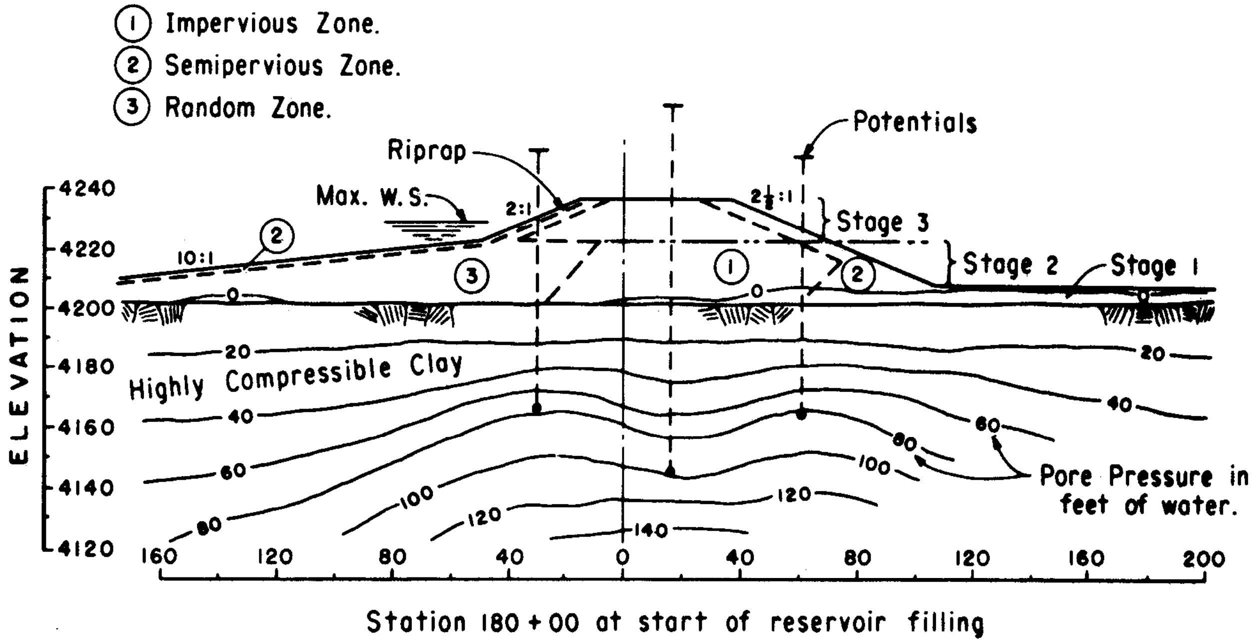


Figure 28.—Record of pore pressures in soft foundation soils at end of stage construction, Willard Dam, Great Salt Lake, Utah.

withdrawal and related increased effective stress, the surface of the ground is currently settling at the rate of about 2½ inches per year. Past settlement rates have been much higher (figure 29). Structural loads also produce large settlements. Thus, structures on mats or footings settle at a greater amount than the surrounding ground surface (figure 30), while structures on very long piles settle less and "rise" above the ground surface (figure 31). Only structures having the correctly designed pile lengths and flotation characteristics maintain a more-or-less relative position with respect to the surface level [*54*].

Similarly, a large basin, about 70 miles long and 20 miles wide, is developing in the southwestern portion of the San Joaquin Valley of California. Here, heavy pumping has lowered the ground water level, and the resulting increase in effective stress has caused compression of the Corcoran clay formation. Since 1920, the maximum subsidence, which has occurred in the lowest part of the basin, is of the order of 25 feet. Some parts of the basin are presently settling at rates up to 1 foot per year. Such large area subsidences cause difficult grade problems to engineers constructing canals, pipelines, and other grade-sensitive structures.

In the arid and semiarid parts of the United States, unsaturated deposits of loose ML, ML–CL, and low-plasticity CL soils cause special problems for the soils engineer [*55*]. These include wind deposited loess and loess-like soils and coalluvial and alluvial soils deposited by flash runoffs, often in the form of mudslides. In none of the cases have the soils been completely wetted to allow a breakdown of the loose structure so formed. Generally speaking, these soils have high dry strengths created by well dispersed clay binder [*56*]. Major loess deposits of this type are found in the Plains States of Kansas and Nebraska, but also can be found in other areas of the United States. It is well known that loessial soils form high vertical faces that are stable as long as the water content is low (figure 32). However, upon wetting, the strength is largely lost and bank 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 becoming wet (figure 33)

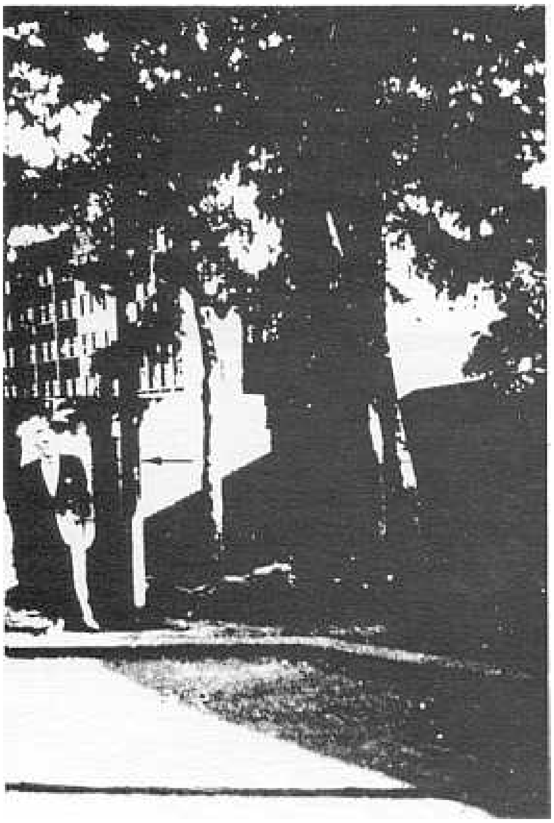


Figure 29.—Settlement of Mexico City subsoils left the old well casing extending well above the present ground surface.

[*57, 58*]. Certain interfan soil deposits adjacent to the southwestern foothills of the Central Valley of California have similar characteristics (figure 34). When dealing with hydraulic engineering works, wherein the subsoils will eventually become saturated, it is important to recognize these soils and to take measures to improve them before building structures on them.

As in the case of other low-strength compressible soils, the soils may be bypassed for certain types of structures by using piles or caissons to firmer strata below, unless very deep deposits are encountered. Pile driving, however, is difficult without jetting or wetting because of the high dry strength [*56*]. When the depth is not too great, the material may be removed and recompacted. Ponding to induce consolidation is an accepted method of treatment particularly when large structures, such as dams, and long in-line structures, such as highway embankments and canals, are involved.

A method for delineating these troublesome soils has been devised [*55, 58*]. If the soil density is sufficiently high so that upon saturation the soil will not have a water content close to its liquid limit (near zero strength), structure collapse will not be imminent. If, however, the soil density is sufficiently low, so that at saturation the soil water content is above its liquid limit water content, it can be said that structure collapse is imminent, and the soil deposit can suffer major settlement, even under its own weight. Thus, the non-

Figure 30.—Settlement and tilting of a structure on mat foundation, Basilica of Guadalupe, Mexico City.

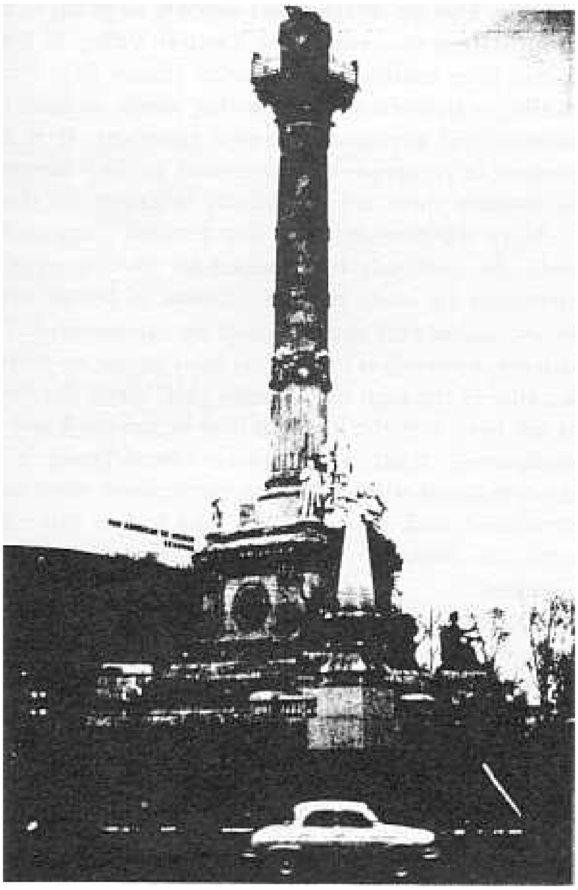


Figure 31.—**Structure on deep pile foundation has not settled as much as ground surface, Mexico City.**

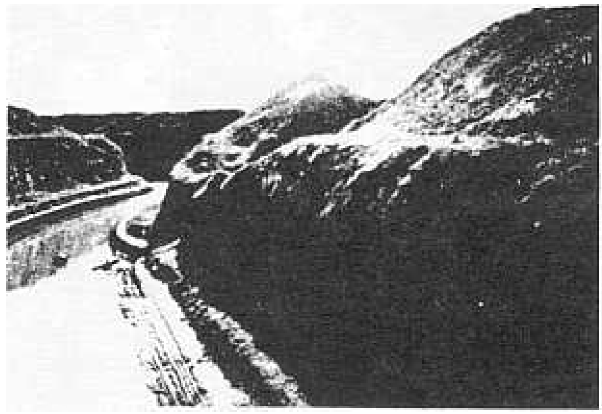


Figure 32.—**Stability of high steep slopes in natural dry loess soils.**

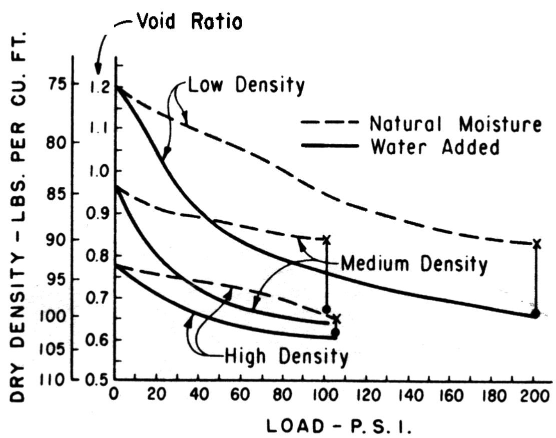


Figure 33.—**Effect of natural density upon compressibility of loess soils when loaded and wetted.**

sensitive and sensitive soil conditions can be defined on the figure 35 plot downward and to the right, and upward and to the left of the limit line, respectively.

A soil is said to be of high sensitivity if the ratio of the natural strength to the remolded strength, at equal density and water content, is high. Low-density soils having a liquid limit water content less than their water content at saturation would have the facility to lose their shear strength upon remolding. Thus, in figure 35, they would fall in the sensitive grouping above and to the left of the limit lines. In soils engineering, the sensitivity of clays is important when construction operations, pile driving, or overstressing may remold such a soil and produce extreme settlement or instability.

Marine clays in Norway, Sweden, and Canada present examples of extra-sensitive or "quick" clays that, when overstressing due to natural or other causes induces remolding, rapidly lose strength (figure 36). Such strength loss can result in tremendous slides that occur in a surprisingly short time [*59, 60*]. The landslide at Nicolet, Quebec (figure 37), is an example of this type of failure. Such clays have a high void content with a bonded structure that collapses. The accompanying decrease in volume causes high pore pressure conditions to develop and, if drainage is slow, effective stresses and available shear strength are reduced to small amounts.

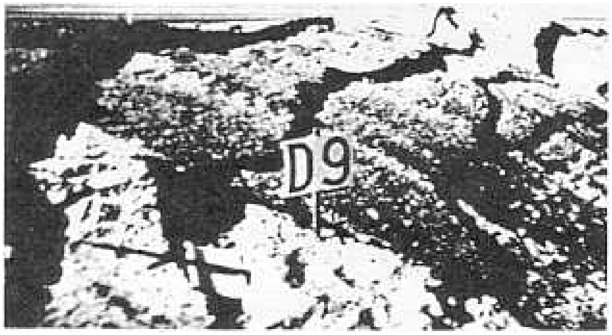


Figure 34.—**Collapse of loose interfan soils when ponded caused settlements up to 13 feet at Mendota test site, Central Valley, California.**

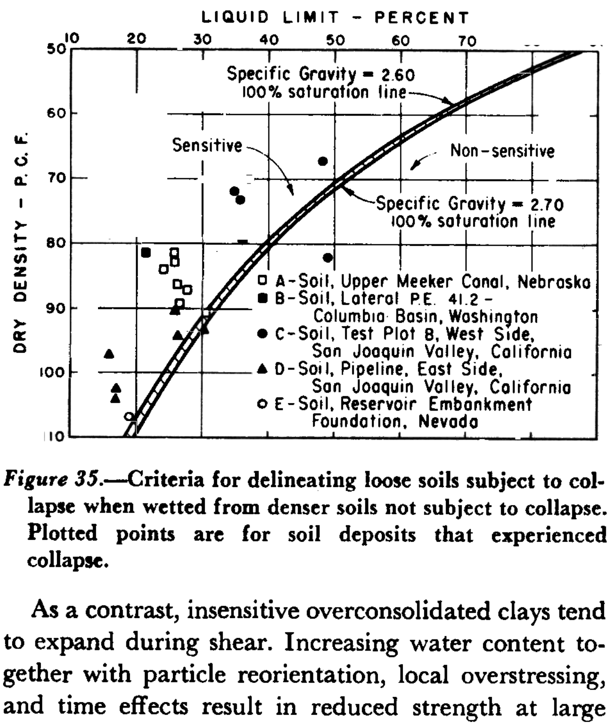


Figure 35.—**Criteria for delineating loose soils subject to collapse when wetted from denser soils not subject to collapse. Plotted points are for soil deposits that experienced collapse.**

As a contrast, insensitive overconsolidated clays tend to expand during shear. Increasing water content together with particle reorientation, local overstressing, and time effects result in reduced strength at large strains. The remaining long term or residual strength is much less than the peak strength determined by normal laboratory shear tests [*61, 62*].

Silt and clay soils may also be subject to undesirable changes in deformation and strength properties when subjected to dynamic loadings. In addition to dynamic forces created by earthquakes or other vibrations or rapidly applied loadings which must be resisted by the soil, the structural properties may be different under this type of loading than under static or slowly applied loadings [*63, 64*]. Silt and clay soils of low plasticity, low density, and high water content are most apt to cause such problems to earth structures and foundations.

Laboratory tests show that soil strength under cyclic load conditions is quite different from that mobilized under slowly applied load conditions and is a function of the number of stress cycles as well as the stress intensity. Figure 38 shows the normal shear strength of a moderately dense lean clay and its ability to withstand higher total sustained plus repetitive loads. Conversely, the normal strength of a lower-density lean clay soil is higher than the strength of the soil when sustained plus repetitive loads are applied.

Figure 39 shows a bank failure which took place in lean clay soils on the All-American Canal during the El Centro earthquake of 1940. Earth dams composed of suitable materials which are well compacted historically show good resistance to the seismic vibrations of earthquakes. Figure 40 shows the failure of

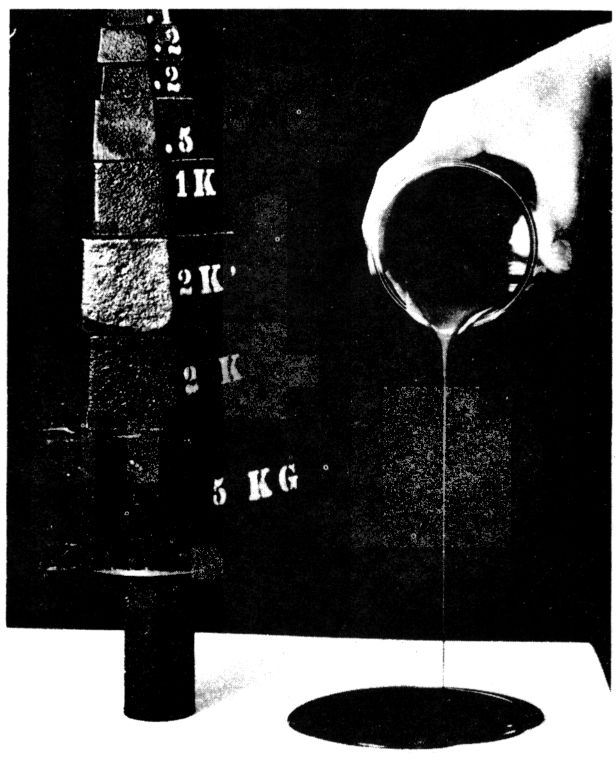


Figure 36.—**Illustration of strength of undisturbed extra-sensitive Canadian Leda clay and extreme strength loss upon disturbance by molding. (Photo courtesy National Research Council of Canada.)**

***Figure 37.*—Landslide in extra-sensitive marine clay, Nicolet, Quebec. (Photo courtesy Professor Jacques Beland, University of Montreal.)**

Sheffield Dam during the Santa Barbara earthquake in 1925. It is apparent that this dam, which was constructed in 1917, was not investigated and constructed by today's best standards. Adequate densification of the silty sand, which comprised the dam and its foundation, would have greatly increased its resistance to loss of shear strength and ultimate failure. The extensive land regression (figure 41) resulting from the Turnagain Heights landslide during the long-duration Alaska earthquake of 1964 was probably due in large measure to liquefaction of sand lenses and the weakening of sensitive clay zones [*65*].

High plasticity soils.—As the clay components of soils become more predominant, the mineral characteristics of the clays assume great importance. Soil properties such as cohesion, consistency, and water content are directly influenced by the mineral constituents of the clay. When soils are fairly moist, the clay particles are surrounded by water films. As dehydration takes place, these films become thinner and thinner until adjacent particles are held together by strong cohesive forces. As soils are wetted, the films become weaker. The film strength is also related to the fineness and specific surface of the material.

It is necessary to have a general idea of the chemical and mineralogical nature of clays in soils to understand their physical behavior. Chemically, clay minerals are complex crystalline hydrous alumnosilicates, often containing small amounts of potassium, sodium, magnesium, and iron. Briefly, two groups of clay minerals have been recognized, 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 adsorptive properties. In contrast, montmorillonite minerals have expanding lattices and exhibit a higher order of hydration and cation adsorption. The degree of lattice expansion is dependent upon the nature of the cations adsorbed. Illite, a common clay mineral, is sometimes described as a third type, but many investigators prefer to class it under the expanding lattice group. In illite there is

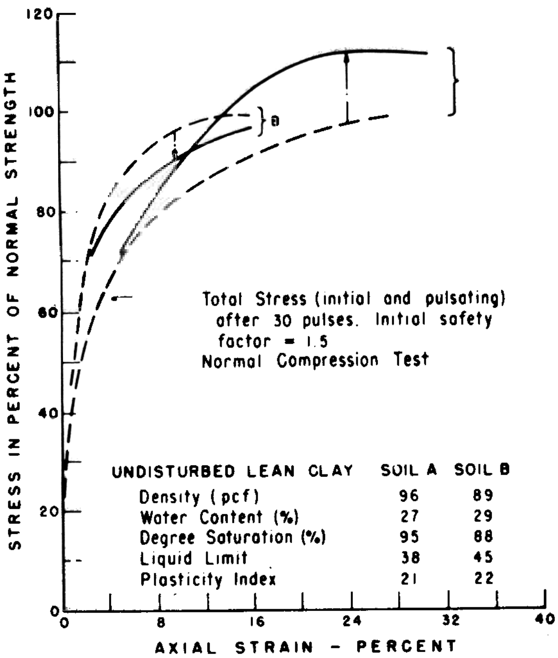


Figure 38.—**Comparison of normal strength to total strength "gain" or "loss" when sustained plus repetitive loadings are applied to cohesive soils.**

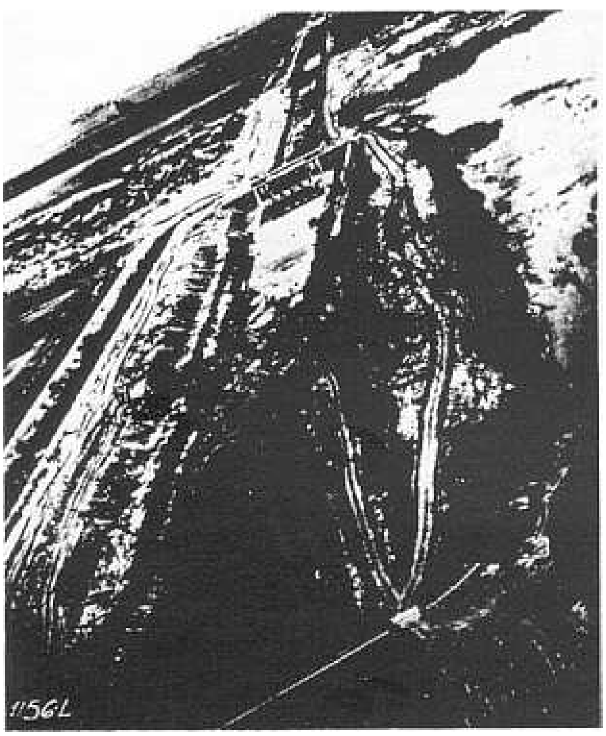


Figure 39.—**Bank failure in clay soil during El Centro earthquake, All-American Canal, California.**

Figure 40.—**Failure of Sheffield Dam, California, during Santa Barbara earthquake.**

a strong bonding of the silica sheets by means of potassium ions which reduces the expansion to very small amounts [*5*].

From an activity standpoint, then montmorillonite is greater than illite, and illite is greater than kaolinite, with sodium montmorillonite having the greatest activity of all common clays. The particle sizes vary in the opposite order; that is, kaolinite being the coarsest material. Figure 42 shows the classes of common clay from a mineralogical standpoint.

From an engineering standpoint, clays are not commonly found in a pure form but may involve mixed-layer structures. In addition, different types of clay soils may be intermixed during transportation and deposition. These factors make an identification and ultimate behavior prediction by petrographic analysis difficult.

The montmorillonite soils, with their expanding lattice structure and resulting capacity for wide ranges in water contents, can be particularly troublesome. Settlement from shrinkage, heave from swelling, and loss of stability caused by shrinkage or swelling can create major structural problems; this is often greatly magnified in the case of hydraulic structures [*15*].

The amount of volume change that occurs in an expansive soil is related to its initial density and water content, loading, soil structure (natural or remolded), amount of clay particles, and the nature of the clay minerals. The criteria shown in table IV may be used to identify clay soils as to their expansion and shrinkage potential. The effect of initial moisture and density on expansion and the effect of remolding on load-expansion properties are shown in figures 43 and 44, respectively. Figure 45 shows the heaving of a concrete lined canal caused by load removal and wetting in expansive clay soils. Figure 46 shows a typical bank failure in these soils caused by deep shrinkage cracks at the top of the slope and the loss of strength at the slope toe from expansion under light loading with resulting increased water content.

TABLE IV.—*Relation of soil index properties to expansion potential of high-plasticity clay soils*

Data for Making Estimates of Probable Volume Changes for Expansive Materials

Data from index tests [1]			Probable expansion [2] percent total volume change (dry to saturated condition)	Degree of expansion
Colloid content (percent minus 0.001 mm)	Plasticity index	Shrinkage limit (percent)		
28	35	11	30	Very high.
20–31	25–41	7–12	20–30	High.
13–23	15–28	10–16	10–20	Medium.
15	18	15	10	Low.

[1] All 3 index tests should be considered in estimating expansive properties.
[2] Based on a vertical loading of 1.0 p.s.i.

Figure 41.—**Destruction of the Turnagain Heights residential area of Anchorage by a major landslide, Alaska earthquake. (Photo courtesy U.S. Geological Survey.)**

Such heave and stability failures are not limited to hydraulic structures. For instance, highway pavements and building footings may move great amounts due to seasonal or other soil moisture changes, such as desiccation by tree roots. Many houses and other lightly loaded buildings have been literally torn apart by subsoil volume changes (figure 47). Expansive clay soils and shales are found throughout the western United States from the Dakotas to Texas and from Colorado to California. Design procedures to control structures on expansive subsoils include means for preventing moisture changes, removal and replacement with nonexpansive soil to adequate depth, designing footings to carry sufficient loads to counteract uplift, the use of anchor piles or caissons to resist uplift, and chemical treatment [*66*].

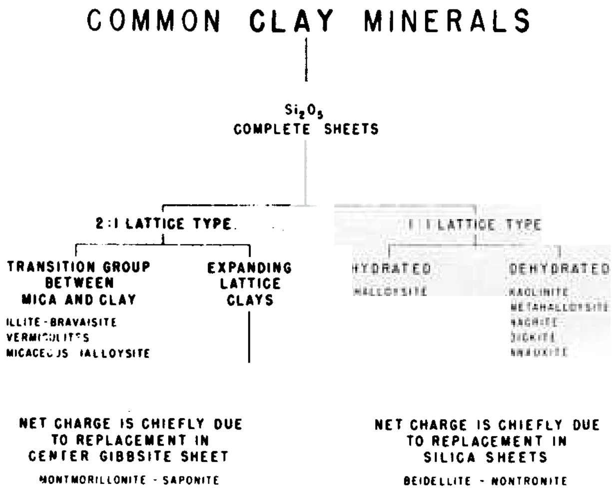


***Figure 42.*—Common clay minerals.**

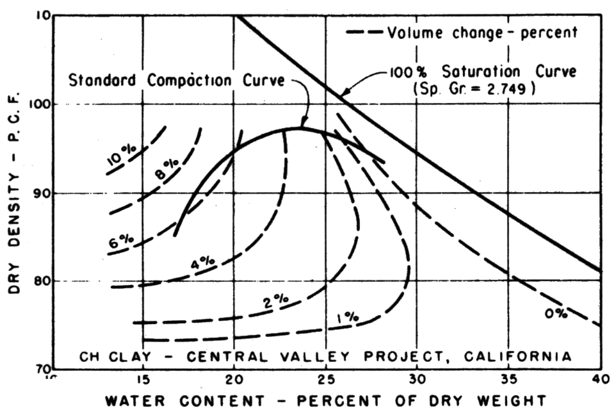


***Figure 43.*—Effect of initial moisture and density on the expansion properties of a compacted expansive clay soil when wetted.**

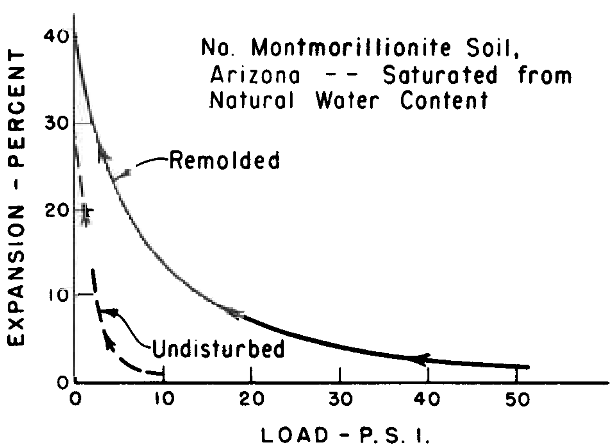


***Figure 44.*—Load-expansion properties of an expansive clay soil.**

***Figure 45.*—Heaving and cracking of a concrete canal lining caused by the saturation of expansive clay subsoils, Arizona.**

Treated Soils

In some cases it has been economical or necessary to treat soils with chemicals or other materials to improve their physical properties. For instance, plastic clay soils have been treated by mixing with lime or cement to lower plastic and liquid limits and thus improve stability. Shrinkage and swelling can also be reduced by this means. Cement, asphalt, and clay grouts have been used to reduce permeability in coarse open soils, and numerous successes have been recorded where chemical grouts were utilized to control water through finer pervious soils. Mixtures of soil and cement are now commonly being used to provide improved soil products for foundation and highway subgrades (ASTM Tests for Moisture-Density Relations of Soil-Cement Mixtures, D 558–57; Methods for Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures, D 559–57; Methods for Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures, D 560–57; Test for Cement Content of Soil-Cement Mixtures, D–806–

Figure 46.—Failure of a canal bank caused by the shrinking and swelling of expansive clay soils, California.

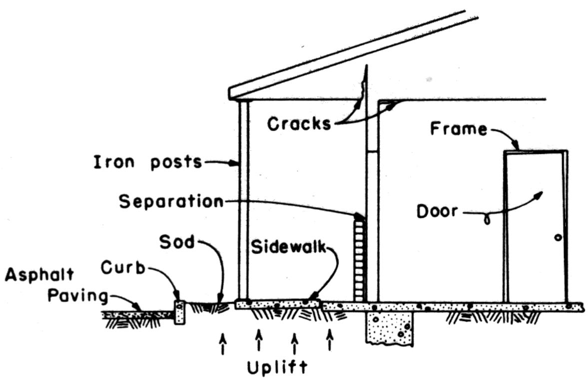


Figure 47.—Damage caused to building by uplift of expansive clay foundation.

57; Method of Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory, D 1632–63; Test for Compressive Strength of Molded Soil-Cement Cylinders, D 1633–63; Test for Compressive Strength of Soil-Cement Using Portions of Beams Broken in Flexure, D 1634–63; and Test for Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading, D 1635–63). Within the last 15 years, notable progress has been made in developing a suitable soil-cement product for the facing of earth dams and embankments. Research in this material development included extensive laboratory testing and long time field testing [67]. Since 1961, 11 major earth dams and 19 sizable reservoirs, embankments, and other earthworks have been constructed, or are being constructed, utilizing soil-cement as a facing material in place of rock riprap, at substantial savings.

CONSTRUCTION CONTROL

I would be remiss in this lecture if I did not mention the importance of good construction control for foundations and earthworks. A large portion of the work of ASTM Committee D–18 is in this area of soils engineering. Designs for earth foundations and earth structures are based upon certain assumptions and judgments which have been developed through study and analysis of field and laboratory investigative data. When very thorough investigations are made, there is less chance for wrong interpretations of soil behavior or the need to rely on costly overdesigns. Regardless of the thoroughness of investigations, there is a need to assess the structural properties of the earth foundation or earthwork materials as construction proceeds to assure that conclusions on which the designs were based are sound [*68*].

Foundation construction control also includes the enforcement of measures required to assure that construction procedures that would injure the foundation soils are not used. Poor pile construction practices can injure a foundation without developing the capability to bear the loads as required. Poor dewatering practices can loosen an otherwise dense foundation sand; thus, such control features as sheet pilings, cofferdams, and water removal facilities should be adequate to hold the ground water levels to safe distances below the structure foundation grades. Poor excavation and equipment handling, or overexcavation, can remold and destroy otherwise capable soft foundation soils. Poor bracing practices can cause excavation bank failures which may also extend into and disrupt a foundation (figure 25). Uncontrolled moisture loss can cause shrinkage, cracking, and slaking, which result in the destruction of the natural soil structure. Drilled piers and caissons, particularly when placed below ground water level, require extreme care to assure that the concrete placed after excavation will be continuously sound and will bear on firm undisturbed material [*69*].

Control of the placement and treatment of soils used as construction materials is necessary to assure the competence of the earthwork [*70, 71, 72*]. Without the advancements that have been made in construction equipment for placing and treatment and modern means for quickly checking the quality of the work, the very large earth dams such as the two California dams, Oroville, 770 feet high [*73*], and Trinity, 537 feet high, and other large earthworks being constructed today would not have been economically feasible. Figure 48

Figure 48.—**Self-powered sheepsfoot rollers used to compact impervious soils, San Luis Dam, California.**

shows a large self-powered sheepsfoot roller having a capability of compacting up to 900 cubic yards per hour of cohesive soil. Figure 49 shows a wheel-type excavator capable of excavating 3,000 to 4,000 cubic yards per hour of impervious Zone 1 material for the 78-million-cubic-yard San Luis Dam in California (figure 50). The bottom dump trucks shown being loaded by the wheel excavator in figure 49 averaged an operating capacity of 55 cubic yards and traveled at an average round trip rate of 20 miles per hour including loading and unloading time. The 50-cubic-yard 3-bowl electric excavators shown in figure 51 are powered by motors on each wheel. These excavators, which are capable of excavating a load in 30 seconds and hauling at a rate of 12 miles per hour, were used to construct the 13,100-cfs-capacity San Luis Canal.

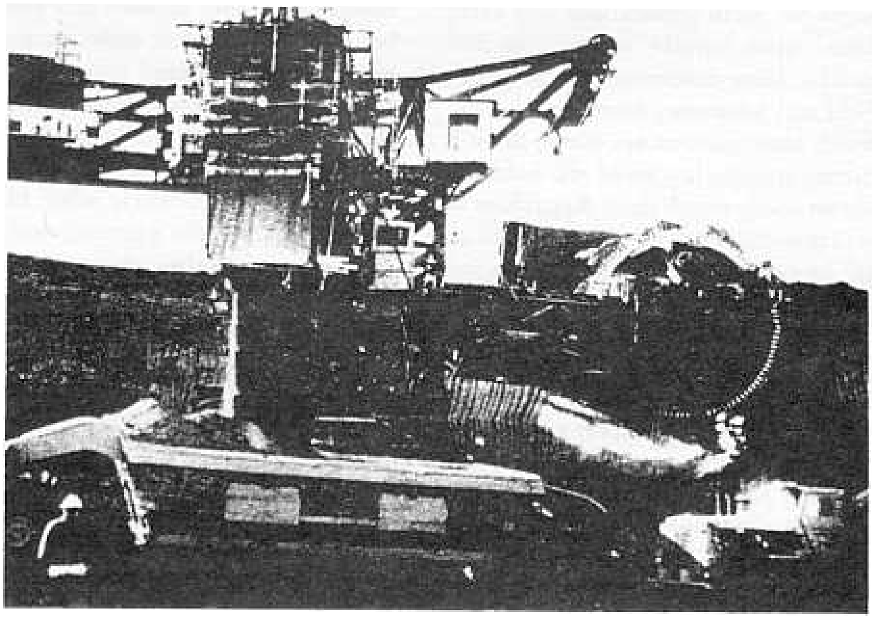


Figure 49.—**Wheel excavator used to excavate impervious soils, San Luis Dam, California.**

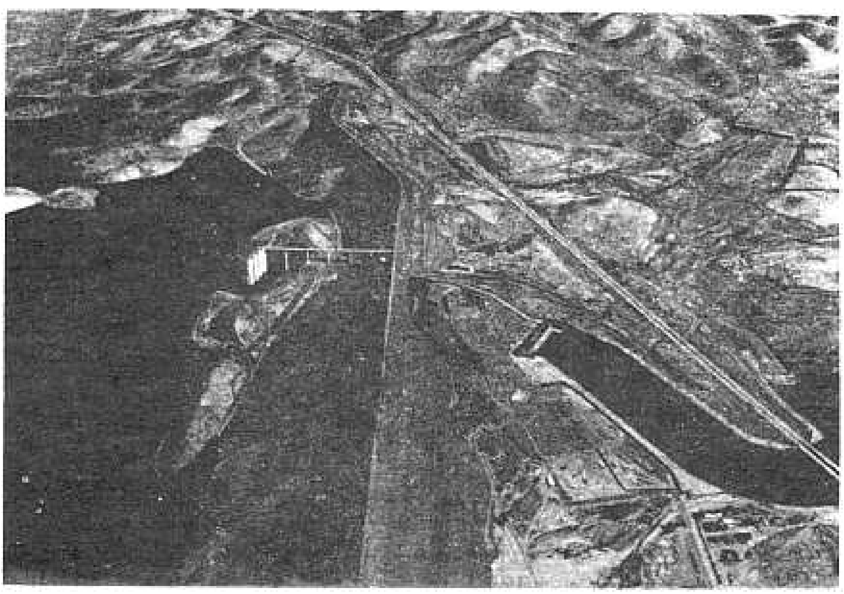


Figure 50.—**Air view of San Luis Dam after completion.**

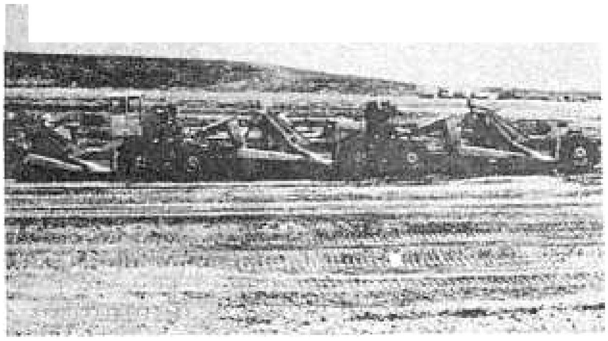


Figure 51.—**Three-bowl electric powered excavators used for construction of San Luis Canal, California.**

The control of construction for embankments, zones of earth dams, foundation refills, and structure backfill utilizing impervious and semipervious soils is based on controlling placement density and water content to some specified degree that will produce desired structural properties [*4, 72*]. It is the practice to specify dry density and water content in terms of standard maximum dry density (ASTM Method D 698), or modified maximum dry density (ASTM Method D 1557) and the respective optimum water contents. When free draining gravel and sand soils are used, it is a normal practice to specify dry density in terms of relative density.

It is a common practice to specify minimum acceptable soil densities or means for achieving these densities. For the control of compacting cohesive soils, specific limits of placement water contents are also specified. For evaluating the quality of the earthwork product for large concentrated earthworks, such as earth dams, statistical methods recently have been receiving more widespread use [*68, 71, 74*]. Such methods often provide a more realistic measure of the uncontrolled variation of the soil parameters in controlled construction of earthworks and the overall quality of a large structure. Statistical methods are also useful to indicate when significant changes occur in the materials or in some elements of the operation.

In the control of compacted earthwork construction, it is necessary to determine the in-place density of the compacted fill by sampling and weighing a known volume of soil taken from the fill, determining the water content, and computing the fill dry density. In the case of cohesive soils, the standard compaction test is then performed on the same material and the maximum dry density and optimum water content determined for comparison with the fill conditions. These tests are time consuming, several hours being required to determine the water content of cohesive soils. Today's high-speed construction operations make the job of assuring quality construction much more difficult than it was a few years ago. For this reason, rapid compaction control methods were developed. The rapid method of compaction control developed by Hilf [*75, 76*] provides precise procedures for determining percent compaction and variation from optimum water content in approximately 30 minutes.

Nuclear moisture and density meters are now available to determine, with fair accuracy, the wet density and water content of soils in a few minutes. However, information is still needed on the maximum density and optimum water content to determine the degree of compaction and variation from optimum water content. Thus, the total time to determine quality by this procedure remains lengthy.

In the case of free draining sand-gravel soils, the dry densities at the loosest and densest states are determined (ASTM Method D 2049) and the relative density of the in-place fill material computed. Field and laboratory tests for determination of the relative density of free draining sand-gravel soils can be made in 1 to 2 hours. Normally, this is sufficiently rapid.

SOIL SAMPLING AND TESTING

The general approach and objectives in the sampling and testing of soils are quite different from those of the other materials [*77*]. Judgment factors enter soil sampling and testing in programming, evaluating test conditions, modifying test procedures, and carrying out the tests. Good reliable test data can be obtained only when properly trained and competent personnel perform the sampling and testing work under the direction of a soils engineer having adequate knowledge of soils engineering principles [*78, 79*].

Many engineers believe that rigid standard procedures can be followed in conducting the "simpler" soil tests without encountering environmental effects. This is not correct. Values from such relatively simple tests as gradation, consistency, and compaction can be affected significantly by the techniques used. For example, with certain soils, particle breakdown during the gradation test can result in misleading data; the loss of moisture by air or oven drying can change liquid, plastic, and shrinkage limit values; and the sequence of drying and wetting can result in variations in compaction test data, as shown by figure 52.

When testing soils for compression, permeability, and shear properties, a multitude of values can be obtained depending upon the procedures followed. Here it is extremely important to follow closely a testing program designed to duplicate natural, construction, and operational conditions and sequences so that the results can be translated into reliable and significant predictions of soil responses in terms of the parameters needed for the particular type of design analysis contemplated. The selection of truly representative samples for testing is extremely important. It is more important to perform detailed and accurate tests on a few properly selected samples than to make sketchy tests on a large number of samples. Soils engineering is a big and complicated business today and, while each soil problem cannot be handled as an individual research problem, the above requirements for meaningful test data cannot be ignored.

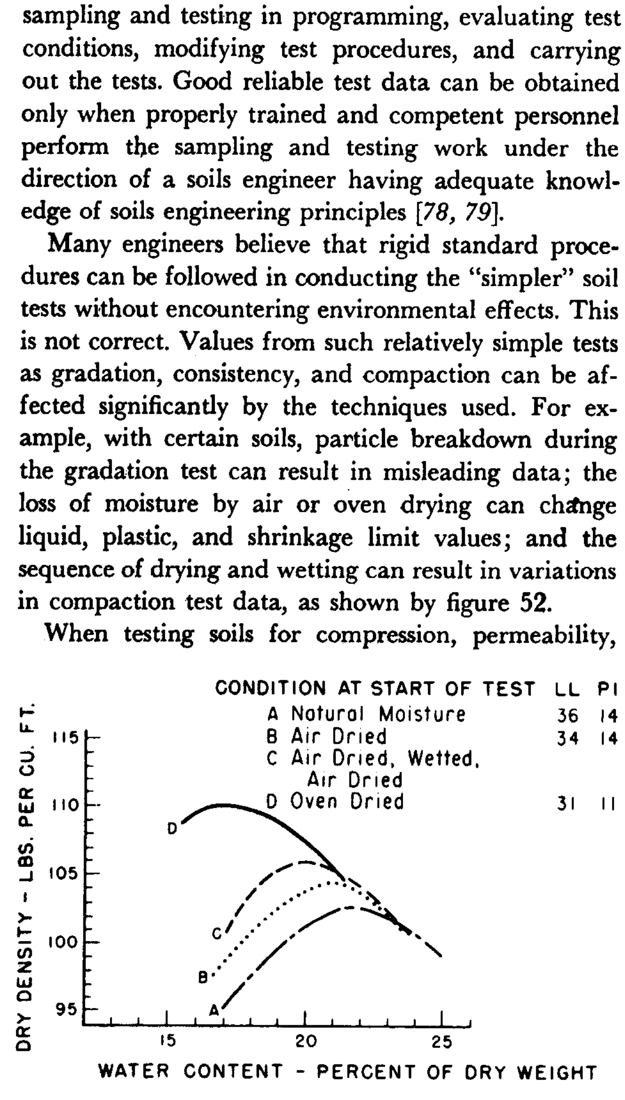


***Figure 52.*—Effect of drying and wetting sequence on compaction test data.**

The principal functions of ASTM in the soils engineering field are to develop testing procedures that will provide good usable and reproducible results, to assure that the test data are meaningful and properly used, and to encourage research leading to a better understanding of the behavior of soils as needed for the development of test standards. While standard soil tests cannot be written in terms of routine procedures covering all situations, it is important that test methods be developed for all soil tests which define and discuss: a. basic test requirements, b. suitable equipment and samples, c. required variations to fit various problems, and d. requirements for inclusion of *all* information necessary for the proper interpretation of the data for research, design, or construction purposes.

The American Society for Testing and Materials and its Committee D–18 on Soil and Rock for Engineering Purposes should be complimented on their accomplishments during the last 25 years. The large number of ASTM test designations, conference papers, and publications referenced in this lecture is strong evidence that the committee has not been idle.

CURRENT STATUS vs. RESEARCH NEEDS

The results of research and advancements made in the field of soils engineering during the past four decades, and particularly in the past decade, are certainly evidenced by the large earthworks that have been constructed and difficult foundation situations that have been successfully engineered. Thirty years ago, earth dams the height of Oroville and Trinity would not have been contemplated. Earth dams and other heavy structures are now being built on soft foundations that previously would have been considered impossible.

At the same time, soils engineers are the first to recognize that soils engineering is not an exact science wherein soil responses to definite conditions can be accurately predicted, nor will predictions ever be of the same order of precision as those for many common construction materials. However, if we are able to maintain or increase the present effort in research and development, we will be able, in the foreseeable future, to predict soil responses much more accurately. Some of the major problem areas which, if solved, will lead to safer and more trouble-free structures and more economical design and construction techniques, are discussed below [*80*].

The theoretical basis for soil and foundation engineering is rudimentary. For example, three-dimensional stability and settlement problems are treated using one- or two-dimensional analyses, no rational bearing capacity theory exists for layered soils, stress distributions are calculated using elastic theory, acceleration effects in earth embankments are computed by means of crude approximations, and earth pressures against various in-ground structures are estimated on the basis of empirical rules. The absence of sound, general theories robs the designer of a powerful tool. Without the most suitable parameters for soil properties and adequate analytical techniques, the advantages of computer advancements for solving complicated design problems are greatly lessened.

The general practice of isolating foundation design from that of the superstructure leads to uneconomical designs and occasionally to failures. In reality the soil, the foundation, and the superstructure interact to a degree that dictates their design as a single entity.

Greater knowledge regarding the strength, volume change, and permeability characteristics of gravel and rockfill materials and material breakdown under high confining pressures is needed for building high earth and rockfill dams. The improvement of knowledge of cracking within embankments is important, and current practices that use defensive design and construction procedures to minimize and control the effects of cracks if they should occur are costly. The stress states encountered in most field cases do not conform exactly to the stress states developed in the triaxial compression and one-dimensional compression tests which are almost universally used to measure stress-strain properties. During the last two decades, it has been recognized that the use of the effective stress analysis for determining the stability of embankment slopes and excavations usually provides the best means for determining that stability conditions are adequately considered. There has been a great deal of research activity in this area, including the measurement of total pore fluid pressures which take into account all of the soil-water phenomena, but much remains to be learned.

Certain "problem" soils cause considerable difficulties to soils engineers. These include soils containing colloidal organic matter and mineral constituents which exert a radical change on the soil properties. The removal, treatment, or bypassing of such soils requires a tremendous annual expenditure of time, labor, and equipment which may be attributed largely to ignorance of the fundamental physical properties of these questionable materials.

In this rich country the usual practice for founding buildings on soft ground is to be conservative and use piles. The cost of constructing pile foundations is often a major economical consideration of the design. The use of load tests on individual piles (ASTM Test for Load-Settlement Relationship for Individual Piles under Vertical Axial Load, D 1143–61 T) provides a common method for determining pile bearing values. However, pile groups are usually involved, and it is necessary to interpret these results with respect to the pile groups. There are many theories which relate the bearing capacity of an individual pile to a pile in a group with recommended values varying from 25 to 75 percent of the test pile safe load. Further research to evaluate this relationship is vitally needed; this would require expensive large-scale field tests on pile groups. Further, there is ample evidence that more daring foundation treatments (preloading, floating, etc.) have been and can be more satisfactory and economical than piling. The obstacle to application of these techniques is the lack of examples by which response predictions by the best laboratory and theoretical techniques have been compared with the actual response.

The extensive landslides which developed during the Alaskan earthquake and the foundation failures and landslides at Niigata illustrate the need for research studies of earthquake ground motions and the response of natural soil deposits and embankments to these ground motions and the need to determine the induced stresses and properties of soils under these stresses. Current methods of design to check the stability of slopes and evaluate foundation competence during earthquakes are largely empirical. In seismically active areas there are uncertainties involved in the use of many attractive construction sites when potentially active faults are encountered. At the present time, it is not certain what principles should be followed in such construction.

There are many soil problems such as those related to water flow, methods for stopping flow, and others too numerous to mention, which need to be solved for safety and economic reasons. A long term research effort, which is well organized and well supported, could enhance the practice of soils engineering by providing sounder approaches for three general areas: a. more economical designs could be made for everyday common problems by removing the need for conservatism; b. engineering projects, whose feasibility is currently in doubt, could be undertaken; and c. entirely new design approaches and construction procedures could be developed.

It has been estimated that a progressive 10-year program of needed research would require additional funds of the order of $35,000,000 per year, built up to this amount during the first 5 years and continuing at that level for the following 5 years. A large buildup in trained manpower and testing facilities would be required. The need for conducting large scale field tests and performing measurements on structures and foundations during and after construction is considered of major importance.

Research funds for soils engineering suffer to a significant degree because there is no product, as such, to market as there is in the case of other construction materials. Research funds are provided largely by governmental sources, with some participation by large engineering firms and organizations interested in additives and construction products. Construction firms and construction equipment manufacturers who make a livelihood from earthwork projects have not contributed in proportion to their interest in the discipline. There is a need to interest all segments of the soils engineering business in the support of needed research.

CONCLUSION

In this lecture, I have attempted to describe how soils differ from other construction materials because of their heterogeneous nature and because past geologic history and construction and operating sequences affect their responses to imposed conditions. While advances have been made in theory, in determining meaningful parameters, and in understanding soil mass behavior, practitioners of the discipline still rely heavily on empirical procedures and judgment developed by training and experience. All of us would prefer to solve soils problems on a more rational basis, although we know we will always be required to face problems resulting from the heterogeneity of the material. From the problems that harass practitioners of this discipline, which I have tried to point out, I think you might agree that the job of a soils engineer is a difficult one and embodies three important work phases: a. a complete understanding of the purpose and effects of the engineering structure, b. a full development and assessment of thorough investigative information, and c. assurance that proper construction methods and workmanship are obtained.

REFERENCES

[1] Coulomb, C. A., "Essai sur une Application des Regles de Maximis et Minimis a Quelques Problemes de Statique, Relatifs a l'Architecture," *Memoires de l'Academie des Sciences (Savants Etrang)*, Vol. 7, 1773, pp. 343–382.

[2] Terzaghi, Karl, "Erdbaumechanik," Vienna, F. Dueticke, 1925.

[3] Terzaghi, Karl, and Peck, R. B., *Soil Mechanics in Engineering Practice,* Wiley, New York, 1948, pp. 56–78.

[4] Proctor, R. R., "Fundamental Principles of Soil Compaction," *Engineering News Record,* Vol. III, August 31–September 28, 1933, pp. 245–248, 286–289, 348–351, 372–376.

[5] Rich, C. I., and Kunze, G. W., *Soil Clay Mineralogy,* University of North Carolina Press, 1964, pp. 3–112.

[6] Baver, L. D., *Soil Physics,* Wiley, New York, 1946, pp. 11–26.

[7] Gibbs, H. J., Hilf, J. W., Holtz, W. G., and Walker, F. C., "Shear Strength of Cohesive Soils," *Research Conference on Shear Strength of Cohesive Soils,* American Society of Civil Engineers, 1960, pp. 69–76.

[8] Gibbs, H. J., "Pore Pressure Control and Evaluation for Triaxial Compression," *Laboratory Shear Testing of Soils, ASTM STP 361,* American Society for Testing and Materials, Philadelphia, pp. 212–225.

[9] "Measurement of Negative Pore Pressure of Unsaturated Soils," *Laboratory Report No. EM-738,* Bureau of Reclamation, Denver, 1966.

[10] Bishop, A. W., and Henkel, D. J., *The Measurement of Soil Properties in the Triaxial Test,* Arnold, London, 1962, pp. 183–190.

[11] Hamilton, L. W., "The Effects of Internal Hydrostatic Pressure on the Shearing Strength of Soils," *Proceedings,* American Society for Testing and Materials, Vol. 39, 1939, pp. 1,100–1,122.

[12] Hilf, J. W., "An Investigation of Pore Water Pressure in Compacted Cohesive Soils," *Technical Memorandum No. 654,* 1956, Bureau of Reclamation, Denver.

[13] Gibbs, H. J., "Estimating Foundation Settlement by the One-Dimensional Consolidation Test," *Engineering Monograph No. 13,* 1953, Bureau of Reclamation, Denver.

[14] Terzaghi, Karl, and Peck, R. B., *Soil Mechanics in Engineering Practice,* Wiley, New York, 1948, pp. 233–242.

[15] Holtz, W. G., and Gibbs, H. J., "Engineering Properties of Expansive Clays," *Transactions,* American Society of Civil Engineers, Vol. 121, 1956, pp. 641–663.

[16] Lambe, T. W., "Methods of Estimating Settlement," *Conference on Design of Foundations for Control of Settlement, Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, September 1964, pp. 47–71.

[17] Bara, J. P., and Hill, R. R., "Foundation Rebound at Dos Amigos Pumping Plant," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, September 1967, Vol. 93, SM5, Part 1, pp. 153–168.

[18] Gibbs, H. J., et al, "Shear Strength of Cohesive Soils," *Research Conference on Shear Strength of Cohesive Soils,* American Society of Civil Engineers, 1960, pp. 87–88.

[19] Lowe, John III, Zaccheo, P. F., and Feldman, H. S., "Consolidation Testing with Back Pressure," *Conference on Design of Foundations for Control of Settlement,* American Society of Civil Engineers, 1964, pp. 73–90.

[20] Gibbs, H. J., "The Effect of Rock Content and Placement Density on Consolidation and Related Pore Pressure in Embankment Construction," *Proceedings,* American Society for Testing and Materials, Vol. 50, 1950, pp. 1,343–1,363.

[21] Sowers, G. F., "Strength Testing of Soils," *Laboratory Shear Testing of Soils, ASTM STP 361,* 1963, pp. 3–31.

[22] Bishop, A. W., and Henkel, D. J., *The Measurement of Soil Properties in the Triaxial Test,* Arnold, London, 1962, pp. 2–32.

[23] Lambe, T. W., *Soil Testing for Engineers,* Wiley, New York, 1951.

[24] LOWE, JOHN III, "Stability Analysis of Embankments," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, SM4, July 1967, pp. 1–33.

[25] GIBBS, H. J., "Pore Pressure and Effective Stress in Soil Tests," *Proceedings,* Third Annual Symposium on Engineering Geology, Boise, Idaho, 1965.

[26] *Earth Manual,* Bureau of Reclamation, Denver, 1963, Designation E–20.

[27] *Earth Manual,* Bureau of Reclamation, Denver, 1963, Designation E–19.

[28] *Design of Small Dams,* First Edition, Bureau of Reclamation, Denver, 1960, pp. 144–147.

[29] CEDERGREN, H. R., *Seepage, Drainage, and Flow Nets,* Wiley, New York, 1967.

[30] *Earth Manual,* Bureau of Reclamation, Denver, 1963, Designation E–2.

[31] HOLTZ, W. G., and LOWITZ, C. A., "Compaction Characteristics of Gravelly Soils," *Conference on Soils for Engineering Purposes, ASTM STP 232, American Society for Testing and Materials,* Philadelphia, 1957, pp. 67–101.

[32] "Second Progress Report of Research on the Penetration Resistance Method of Subsurface Exploration," *Laboratory Report No. EM–356,* 1953, Bureau of Reclamation, Denver.

[33] HOLTZ, W. G., and GIBBS, H. J., "Research on Determining the Density of Sands by Spoon Penetrative Testing," *Proceedings,* Fourth International Conference of Soil Mechanics and Foundation Engineering, 1957, Vol. 1, pp. 35–39.

[34] PECK, R. B., HANSON, W. E., and THORNBURN, T. H., *Foundation Engineering,* Wiley, New York, 1953, p. 109.

[35] *Earth Manual,* Bureau of Reclamation, Denver, 1963, pp. 313–314.

[36] CASAGRANDE, ARTHUR, "Classification and Identification of Soils," *Transactions,* American Society of Civil Engineers, Vol. 113, 1948, pp. 901–991.

[37] *Symposium on the Identification and Classification of Soils, ASTM STP 113,* American Society for Testing and Materials, 1950.

[38] *Soil Survey Manual,* Handbook No. 18, U.S. Department of Agriculture, Washington, D.C., 1951.

[39] HOGENTOGLER, C. A., and TERZAGHI, KARL, "Interrelationship of Load, Road and Subgrade," *Public Roads,* Vol. 10, No. 3, 1929, pp. 37–64.

[40] *Interim Recommended Practice,* Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, American Association of State Highway Officials, Designation M145–661.

[41] *Earth Manual,* Bureau of Reclamation, Denver, 1963, pp. 1–23 and 379–400.

[42] "Unified Soil Classification System," *Technical Memorandum No. 3–357,* Office of Chief of Engineers, Waterways Experiment Station, Vol. 1; Appendix A, Volume 2; and Appendix B, Volume 3, 1953.

[43] WAGNER, A. A., "The Use of the Unified Soil Classification System by the Bureau of Reclamation," *Proceedings,* Fourth International Conference on Soil Mechanics and Foundation Engineering, 1957, Vol. 1, pp. 125–134.

[44] *Soil and Foundation,* The Japanese Society of Soil Mechanics and Foundation Engineering, Vol. VI, No. 2, March 1966.

[45] SEED, H. B., and IDRISS, I. M., "Analysis of Soil Liquefaction: Niigata Earthquake," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, No. SM3, May 1967, pp 83–108.

[46] "Pile Supported Structures in Lake Deposits," *Water Resources Technical Publication,* Research Report No. 11, 1968, Bureau of Reclamation, Denver.

[47] ALDRICH, H. P., "Precompression for Support of Shallow Foundations," *Symposium on Design of Foundations for Control of Settlement,* American Society of Civil Engineers, 1964, pp. 471–506.

[48] JONAS, EARNEST, "Subsurface Stabilization of Organic Silty Clay by Precompression," *Conference on Design of Foundations for Control of Settlement,* American Society of Civil Engineers, 1964, pp. 447–470.

[49] CASAGRANDE, LEO, "Construction of Embankments across Peaty Soils," *Proceedings,* Boston Society of Civil Engineers, Vol. 53, No. 3, 1966, pp. 272–317.

[50] CASAGRANDE, ARTHUR, "The Calculated Risk," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 91, No. SM4, Part 1, July 1965, pp. 1–40.

[51] WALKER, F. C., "Willard Dam—Behavior of a Compressible Foundation," *Journal of the American Society of Civil Engineers,* Soil Me-

chanics and Foundations Division, Vol. 93, No. SM4, July 1967, pp. 177–198.

[52] "Willard Dam," *Technical Record of Design and Construction,* Bureau of Reclamation, Denver, January 1967.

[53] LEONARDS, G. A., and NARAIN, S., "Flexibility of Clay and Cracking of Earth Dams," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 89, SM2, Part 1, March 1963, pp. 84–91.

[54] ZEEVAERT, LEONARDO, "Foundation Problems Related to Ground Surface Subsidence in Mexico City," *Field Testing of Soils, ASTM STP 322,* 1963, pp. 57–66.

[55] HOLTZ, W. G., and HILF, J. W., "Settlement of Soil Foundations Due to Saturation," *Proceedings,* Fifth International Conference on Soil Mechanics and Foundation Engineering, Vol. I, 1961, pp. 673–679.

[56] GIBBS, H. J., and HOLLAND, W. Y., "Petrographic and Engineering Properties of Loess," *Engineering Monograph No. 28,* 1960, Bureau of Reclamation, Denver.

[57] HOLTZ, W. G., and GIBBS, H. J., "Consolidation and Related Properties of Loessial Soils," *Symposium on Consolidation Testing of Soils, ASTM STP 126,* 1951, pp. 9–33.

[58] GIBBS, H. J., and BARA, J. P., "Predicting Surface Subsidence from Basic Soil Tests," *Field Testing of Soils, ASTM STP 322,* 1962, pp. 231–246.

[59] CRAWFORD, C. B. and EDEN, W. J., "Stability of Natural Slopes in Sensitive Clay," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, No. SM4, July 1967, pp. 419–436.

[60] BJERRUM, L., "Stability of Natural Slopes in Quick Clay," *Norwegian Geotechnical Publication No. 10,* 1955, Oslo, Norway.

[61] BJERRUM, L., "Progressive Failure in Slopes of Over-Consolidated Plastic Clay and Clay Shales," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, No. SM5, Part 1, September 1967, pp. 1–49.

[62] SKEMPTON, A. W., "Long Term Stability of Clay Slopes," *Geotechnique,* Vol. 14, No. 2, 1964, pp. 77–102.

[63] SEED, H. B., and CHAN, C. K., "Clay Strength under Earthquake Load Conditions," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 92, No. SM2, March 1966, pp. 53–78.

[64] ELLIS, WILLARD, and HARTMAN, V. B., "Slope Stability during Earthquakes, San Luis Unit, California," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, No. SM4, July 1967, pp. 355–375.

[65] SEED, H. B., and WILSON, S. D., "The Turnagain Heights Landslide, Anchorage, Alaska," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 93, No. SM4, July 1967, pp. 325–353.

[66] HOLTZ, W. G., "Expansive Clays—Properties and Problems," *Quarterly of the Colorado School of Mines,* Vol. 54, No. 54, Oct. 1959, pp. 90–125.

[67] HOLTZ, W. G., and WALKER, F. C., "Soil-Cement as Slope Protection for Earth Dams," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 88, No. SM6, Dec. 1962, pp. 107–134.

[68] *Earth Manual,* Bureau of Reclamation, Denver, 1963, pp. 181–326.

[69] MANSUR, C. I., "Engineered Control of Foundation Construction," *Conference on Quality in Engineered Construction,* American Society of Civil Engineers, June 1965, unpublished.

[70] *Design of Small Dams,* First Edition, Bureau of Reclamation, Denver, 1960, pp. 144–147.

[71] TURNBULL, W. J., ET AL, "Quality Control of Compacted Earthwork," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 92, No. SM1, 1966, pp. 93–103.

[72] "Panel Discussion on Compaction, Testing, and Test Results," *Compaction of Soils, ASTM STP 377,* 1965, pp. 80–135.

[73] GORDON, B. B., and MILLER, R. K., "Control of Earth and Rockfill for Oroville Dam," *Journal of the American Society of Civil Engineers,* Soil Mechanics and Foundations Division, Vol. 92, No. SM3, 1966.

[74] DAVIS, F. J., "Quality Control of Earth Embankments," *Proceedings,* Third International Conference on Soil Mechanics and Foundation Engineering, 1953, Vol. 1, pp. 218–224.

[75] HILF, J. W., "A Rapid Method of Construction Control for Embankments of Cohesive Soil," *Conference on Soils for Engineering Purposes, ASTM STP 232,* 1957, pp. 123–158.

[76] *Earth Manual,* Bureau of Reclamation, Denver, 1963, Designation E–25.

[77] Holtz, W. G., "Introduction to Laboratory Shear Testing of Soils," *Laboratory Shear Testing of Soils, ASTM STP 361,* 1963, pp. 1–2.

[78] "Introduction to Soil Testing," *Procedures for Testing Soils,* American Society for Testing and Materials, Philadelphia, 1958, pp. 1–9.

[79] Burmister, D. M., "Judgment and Environment Factors in Soil Investigations," *ASTM Bulletin No. 217,* October 1956, pp. 55–57.

[80] "10-Year Research Needs in Soil Mechanics and Foundation Engineering," *Report of the Committee on Research,* American Society of Civil Engineers, Soil Mechanics and Foundations Division, September 1966, unpublished.

ABSTRACT

The forming processes of soils are many and varied. These processes produce natural deposits and formations that are seldom homogeneous and soil components having a variety of characteristics. Thus, the job of the soils engineer is largely one of investigation to determine the physical properties of the material and the reactions of the soil mass to imposed conditions.

Compared with other common building materials, soils are difficult to sample and test. To obtain meaningful data for the design of earth structures and foundations, consideration must be given to geological history, present conditions, construction sequences, and anticipated operating conditions. Proper construction control procedures are important to assure that the properties assumed in the design are obtained.

Although soils were used for man's earliest structures, the use of scientific approaches to solve soils engineering problems was begun only about four decades ago. Continuing research during this period has greatly increased knowledge and capability. Today, we are able to better define the soil problems that can be anticipated for specific types of soils. However, there are many problem-soil situations which are extremely difficult to solve with present knowledge.

The ever-pressing needs to build structures more economicallv, to construct ever-larger and more complex structures, and to construct in areas having critical soil conditions and environments, continue to impose more requirements for increased knowledge about soil in general and the parameters which describe its behavior. Because of the complex nature of soils, the relatively short era of the science, and the increased engineering requirements, the overall needs for soils engineering research during the next decade are extensive.

DESCRIPTORS—soils engineering / materials / testing / soil mechanics / research / construction control / earthworks / foundations / soil formation / soil properties / soil types / soil problems / earthquakes / evaluation.

Printed on Recycled Paper

☆U.S. GOVERNMENT PRINTING OFFICE: 1995—841-544

Partial List of
WATER RESOURCES TECHNICAL PUBLICATIONS*

ENGINEERING MONOGRAPHS

No.

31 Ground-Water Movement
32 Stress Analysis of Wye Branches
33 Hydraulic Design of Transitions for Small Canals
34 Control of Cracking in Mass Concrete Structures
35 Effect of Snow Compaction on Runoff from Rain on Snow
36 Guide for Preliminary Design of Arch Dams
37 Hydraulic Model Studies for Morrow Point Dam
38 Potential Economic Benefits from the Use of Radioisotopes in Flow Measurements Through High-head Turbines and Pumps

RESEARCH REPORTS

1 Research—Engineering Methods and Materials
2 Aquatic Weed Control Studies
3 Soils Tests Computer Programs
4 Hydraulic Downpull Forces on Large Gates
5 Park Range Atmospheric Water Resources Program—Phase I
6 Annual Report of Progress on Engineering Research—1965
7 Effects of Monolayers on Insects, Fish and Wildlife—A Reservoir Evaporation Reduction Study
8 Synthetic Resin Primer for Coal-Tar Enamel
9 Vibration Studies of Monticello Dam
10 Annual Report of Progress on Engineering Research—1966
11 Pile Supported Structures in Lake Deposits
12 Buried Asphalt Membrane Canal Lining
13 Removal of Saline Waters from Aquifers
14 Comparison of Analytical and Structural Behavior Results for Flaming Gorge Dam
15 Annual Report of Progress on Engineering Research—1967
16 Structural Model Tests of Arch Dams—Glen Canyon and Morrow Point Dams

TECHNICAL RECORDS OF DESIGN AND CONSTRUCTION

Trinity River Division Features of the Central Valley Project
Twin Buttes Dam
Morrow Point Dam and Powerplant Foundation Investigations

*For a pamphlet listing other Bureau of Reclamation publications, address Office of Chief Engineer, Attention 841, Denver Federal Center, Denver, Colo. 80225.