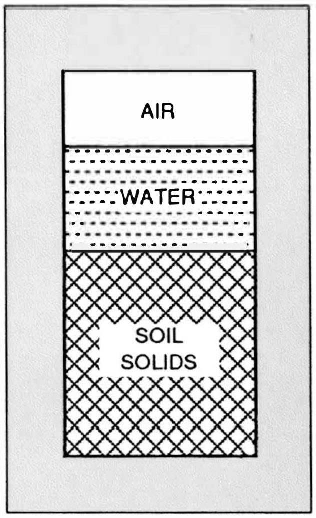
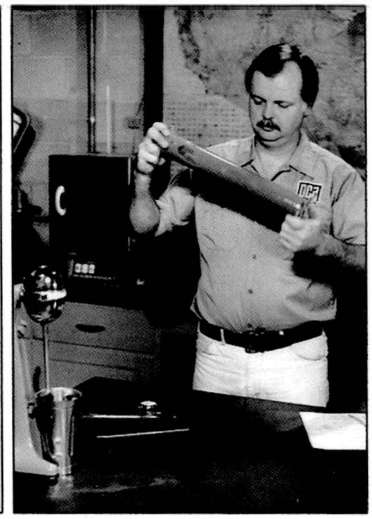
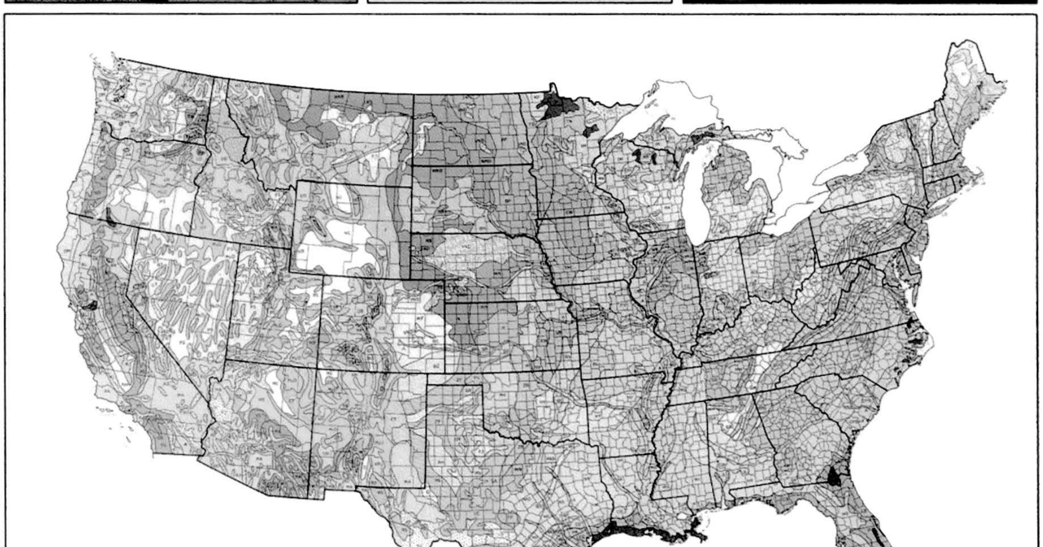
PORTLAND CEMENT
ASSOCIATION

PCA Soil Primer







FOREWORD

The *PCA Soil Primer* was prepared to furnish engineers with basic information on soil with regard to its influence on the design, construction, and performance of concrete, soil-cement, and other types of pavement. Definitions of soil terms are given and tests commonly employed by soil technicians are described, with particular emphasis on the practical meaning and application of these terms and tests.

No attempt has been made to present a complete technical treatise or to discuss the technology of soil science in relation to foundations for bridges, buildings, dams, and similar structures. The aim has been to assemble in one booklet the substance of accumulated knowledge of soil technology as related to pavements, and to reduce this material - now widely scattered throughout technical literature - to the simplest and most useful terms.

Many articles published by geotechnical engineers and agencies have served as an excellent source for much of the information used here. Liberal use has also been made of the publications of the U.S. Department of Agriculture. The information on soil classification and design used by the Department of the Army, Corps of Engineers, and the Federal Aviation Administration was obtained from their published engineering manuals. Reference is also made to the published testing procedures of the American Society for Testing and Materials and the American Association of State Highway and Transportation Officials. The Portland Cement Association gratefully acknowledges these sources of information and expresses appreciation to all these agencies for their continuous efforts to improve pavement design and construction by the application of soil technology.

This primer is only an introduction to geotechnology. A further and continuing study of the literature and developments in this field is necessary for attainment of some technical competence.

This publication is based on facts, tests, and authorities stated herein. It is intended for the use of professional personnel competent to evaluate the significance and limitations of the reported findings and who will accept responsibility for the application of the material it contains. The Portland Cement Association disclaims any and all responsibility for application of the stated principles or for the accuracy of any of the sources other than work performed or information developed by the Association.

TABLE OF CONTENTS

INTRODUCTION

CHAPTER 3. Soil Classification Systems

CHAPTER 4. Engineering Properties of Soils

CHAPTER 5. Soil Surveys and Soil Sampling

CHAPTER 6. Examples of Soil Surveys, Tests, and Analyses

TABLES AND FIGURES

INTRODUCTION

Earth materials are everywhere and are, at least casually, familiar to us all. Since the materials most commonly found on the surface of land masses are those we collectively refer to as soils, we have concern for their nature and behavior.

The development of soil science has a long history in relation to support of agriculture, but the development of engineering soil science, or soil mechanics, is of more recent origin. As a result, many aspects of soil mechanics – or currently geotechnology – find their beginnings in earlier soil science. The published literature of agricultural soil science – especially soil maps – in relation to engineering soil (geotechnical) concerns is particularly useful. And while this is generally true, it is especially useful for roads, streets, and other "horizontal" structures.

CHAPTER 1

SOIL TERMINOLOGY AND SOIL IDENTIFICATION PROPERTIES

Most of us are aware that soil includes clay, silt, sand, and perhaps gravel. Loam is an agricultural term applied to mixtures of sand, silt, and clay amenable to cultivation. It does not imply engineering or mechanical attributes and is not employed in geotechnical work.

The terms, "soil" and "rock," are in common usage, but a definitive distinction between the two terms is not provided. It is common to refer to natural gravel deposits as soil but to consider cobbles and boulders as rock materials. Contrariwise, processed gravel is not generally thought of as soil, and glacial "boulder clays" can include cobbles and even boulders and still be considered as soils.

SOIL

Soils derive from rock or in some cases other accumulations of hard materials such as marine shells, coral, or the like. The process is one of abrasion and fracturing to smaller and smaller size particles. Agencies contributing to this break-down process are wind, water, freezing, slides and rock impact, root growth, wetting and drying, heating and cooling, glacial action, and man's activities.

This simple break-down action is but one aspect of the soil forming process. Of far greater significance for fine grained soils are modifications by chemical processes, plant and animal additions, and man's impact, as the soils are transported by flowing water or subject to moist-to-wet conditions in place.

Wind and water can move soil great distances and sort particle sizes in the process. Slides, avalanches, and rock falls move material downslope and mix sizes. Glacial action can move and mix materials over great distances and broad areas, but melt waters also contribute to sorting and low-fines depositions.

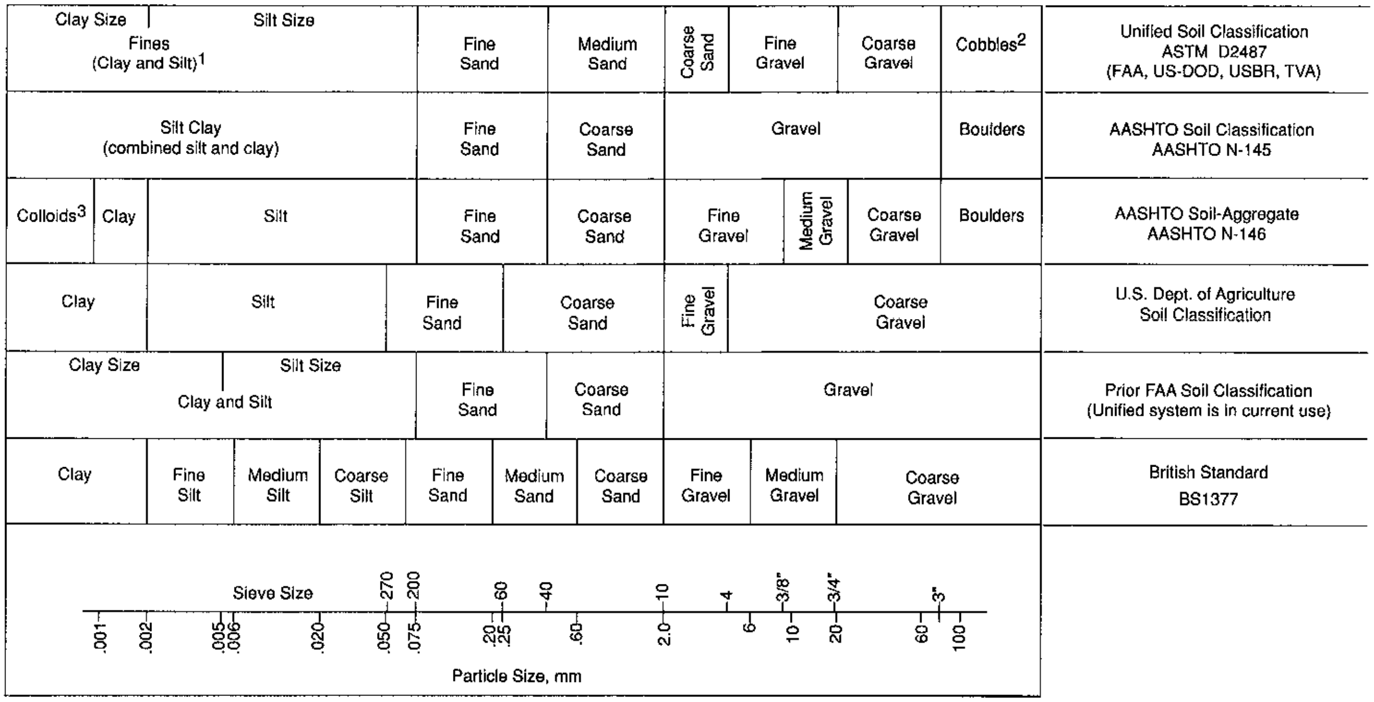
Recognition of these soil forming processes (break-down and modification) can be of value in preliminary site surveys or in extending the information from a limited sampling operation. Near their mountain or upland sources, soils materials will be coarser and more closely related to the source rocks, while far down, slow streams soils will be fine grained, greatly modified, and subject to sorting processes. Also the break-down process will apply more directly in arctic areas, while chemical modification of soils will be greatest in tropic areas.

SOIL PARTICLE SIZES

Soil particle sizes range from cobbles, to gravel, to sand, to silt, to clay, and ultimately to colloids at the fine end. It has become the practice to define these various particle size ranges for purposes of describing moisture characteristics and for identification and classification. Fig. 1 shows these size ranges as standardized and adopted by various groups. The Unified Soil Classification and AASHTO Systems are engineering classifications. The AASHTO methods grew from needs and developments of highway engineering.

These systems extend from U.S. Bureau of Public Roads (USBPR) origins, through Federal Highway Administration (FHWA) adjustments, to AASHO, then AASHTO* standards. The Unified Soil Classification System (USCS) was formulated in support of developing soil engineering technology; originally soil mechanics and more recently geotechnology. It began, however, with a system devised by Dr. Arthur Casagrande for the U.S. Army Corps of Engineers (USCE) for use in classification of materials for military airfields. With the Casagrande system as a basis, the "unified" system was jointly developed by

* American Association of State Highway (and Transportation) Officials.



1Clay: PI≥4 and plot of PI vs. LL falls above "A" line in Table 4.
Silt: PI<4 or plot of PI vs. LL falls below "A" line in Table 4.
2Boulders are particles retained on a 12-in. square opening sieve.
3Colloids are part of the clay fraction.

Fig. 1. Soil-separate size limits of several agencies

the Corps of Engineers, the U.S. Bureau of Reclamation (USBR), and the Tennessee Valley Authority (TVA). It has since been subject to minor adjustments and has been adopted by many other organizations both in and out of the United States. It is now an ASTM standard, D 2487.

The U.S. Department of Agriculture's (USDA) method is a development supporting agricultural technology. While it does not have strong engineering application, it is included here to permit comparison with engineering classifications. Mapping of soils is far more extensive for agricultural purposes and is commonly an excellent source of soil information for some particular site. But the reported agricultural attributes need translation for engineering applications.

The Federal Aviation Administration (FAA) system was an engineering-oriented procedure. It has been supplanted by the Unified Classification as FAA standard, but the prior method may still be encountered. Therefore, the former FAA system is included for comparison.

As an example of methods having somewhat less consistent size range boundaries, the British standard soil sizes are included for comparison. There are quite similar standard size ranges employed by U.S. geological groups, and geological maps are another source of soil information.

The amounts of each particle size group in a soil are determined by laboratory tests usually referred to as the mechanical analysis of a soil. The amounts of the gravel and sand fractions are determined by sieving; silt, clay, and colloid contents are determined by sedimentation tests. The distribution of particle sizes that compose a soil is called the gradation of the soil. The standard methods of tests prescribed by the American Association of State Highway and Transportation Officials* and the American Society for Testing and Materials,** which include the hydrometer test for the fraction of the soil passing the No. 200 sieve, have been widely used in highway engineering.

SOIL TYPE

While soil names are used to designate soil particle size ranges, actual soils as found in the field are much too varied to be limited to these specific particle size ranges. A sand soil, for instance, can include limited quantities of silt, clay or gravel sizes, or combinations and be classified merely as sand. The same is true for silt, for clay, or for gravel. Size ranges are standardized so that the quantities of other sizes that can be present and still class the soil a sand, silt, clay, or gravel can be determined. When greater quantities of other sizes are present, that basic soil type has a combined designation, such as clayey sand, sandy gravel, clay-silt, or the like. The determination and designation of such mixtures in soil classification is referred to as "soil texture."

SOIL TEXTURE

The amount of each soil separate contained in a soil mixture will determine its texture or feel. The textural terms used for various combinations of soil separates are defined by several agencies. The amount of each soil separate in the soil is determined by laboratory tests. These test results are then compared with the definitions of texture in use to determine the textural name.

Both the U.S. Department of Agriculture (USDA) classification and the Federal Aviation Administration (FAA) soil classification schemes employed triangular soil texture plots. These are shown as Figs. 2 and 3. They provide not only good examples of textural soil designations (soil types), but they also permit a comparison between the agricultural classes and an engineering classification system.

* AASHTO T88.
** ASTM D422.

With laboratory experience in testing and classifying the texture of a soil after its gradation is determined, it is possible to make approximations of texture by the feel of moist soil when rubbed and ribboned between the thumb and index finger.

The texture of a soil is given to tell as much as possible about that soil in a few words. With texture determined, approximations and estimates can be made of many properties of a soil, such as bearing value, water-holding capacity, susceptibility to frost heave, and adaptability to soil-cement construction.

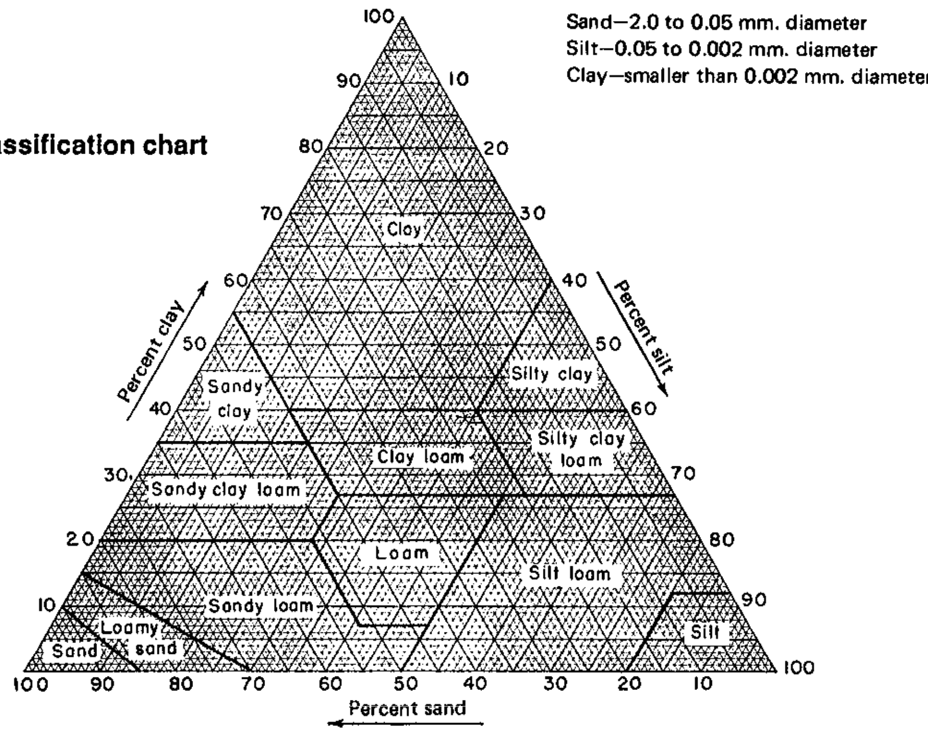
To permit approximate textural classification, many practical shortcuts can be devised to determine the amount of silt and clay in a soil. However, since the range in clay content for the textural groups is not large, accurate weighing of samples is needed, and this requires some laboratory facilities.

ASTM D2488, "Description of Soils (Visual-Manual Procedures)" describes a procedure for the identification and description of soils for engineering purposes based on visual examination and simple manual tests.

Field Identification of Texture

The feel and appearance of the textural groups illustrate factors used in determining the texture of a soil in the field and also assist in field classification work. Note that forming a cast of soil, dry and moist, in the hand and pressing or rolling a moist

Fig. 2. USDA textural classification chart



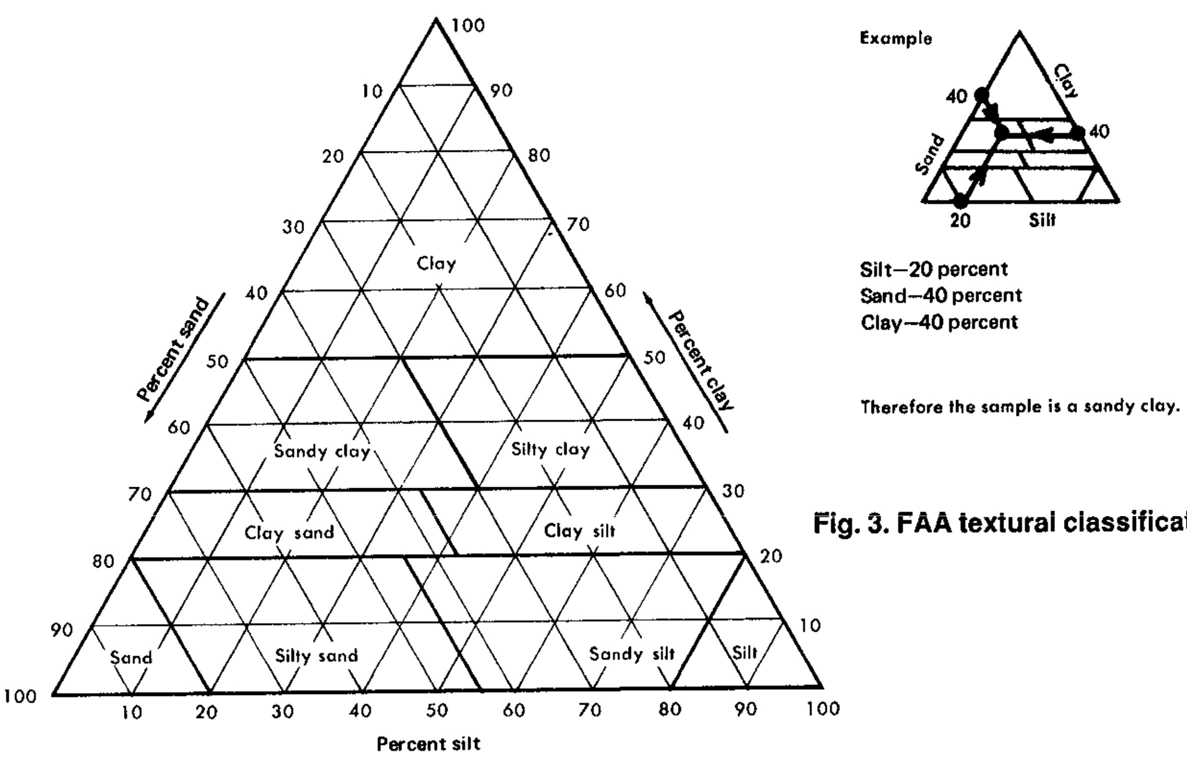


Fig. 3. FAA textural classification chart

ball of soil between the thumb and finger constitute two significant field tests to judge soil texture.

Sand. Includes only small amounts of fines or no fines. These are found on beaches, in dunes, or in stream bar deposits. Individual grains can be seen and felt readily. Squeezed in the hand when dry, this soil will fall apart when the pressure is released. Squeezed when moist, it will form a cast that will hold its shape when the pressure is released but will crumble when touched.

Silty-sand. Consists largely of sand, but has enough silt and clay present to give it a small amount of stability. Individual sand grains can be seen and felt readily. Squeezed in the hand when dry, this soil will fall apart when the pressure is released. Squeezed when moist, it forms a cast that will not only hold its shape when the pressure is released but will also withstand careful handing without breaking. The stability of the moist cast differentiates this soil from sand.

Silt. Consists of a large quantity of silt particles with none to small amounts of sand and clay. Lumps in a dry, undisturbed state appear quite cloddy, but they can be pulverized readily; the soil then feels soft and floury. When wet, silt loam runs together and puddles. Either dry or moist casts can be handled freely without breaking. When a ball of moist soil is pressed between thumb and finger, its surface moisture will disappear, and it will not press out into a smooth, unbroken ribbon but will have a broken appearance.

Silty-clay. Consists of plastic (cohesive) fines mixed with a significant quantity of silt. It is a fine-textured soil that breaks into hard clods or lumps when dry. When a ball of moist soil is pressed between the thumb and finger, it will form a thin ribbon that will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will withstand considerable handling.

Clay. A fine-textured soil that breaks into very hard clods or lumps when dry and is plastic and unusually sticky when wet. When a ball of moist soil is pressed between the thumb and finger, it will form a long ribbon.

Fat or Heavy Clay. A highly plastic clay strongly exhibiting the characteristics indicated for clay.

Lean or Lighter Clay. A moderately plastic clay showing the characteristics indicated for clay much less strongly.

AGRICULTURAL SCIENCE ASPECTS

Since agricultural soil technology and soil map coverage provide an important source of information for pavement engineering, various aspects of this technology deserve comment.

Texture

Texture classifications for agriculture are not the same as for engineering purposes but can be compared approximately. Field identification means are applicable in either case.

Sand, Silt, and Clay. These are directly similar for agricultural purposes to sand, silt, and clay as earlier discussed for engineering application under "Soil Texture."

Mixed Soils. Nominally equivalent soils, as discussed for engineering purposes, would be: sandy loam (loam is defined below) for silty-sand, clay loam for silty-clay, and silt loam for silt mixed with moderate amounts of fine sand and some clay.

Loam. Consists of an approximately equal mixture of sand, silt, and clay. It is easily crumbled when dry and has a slightly gritty, yet fairly smooth feel. It is slightly plastic. Squeezed in the hand when dry, it will form a cast that will withstand careful handling. The cast formed of moist soil can be handled freely without breaking.

Soil Color

The color of a soil varies with its moisture content. While it is standard practice to determine color of a soil in a moist condition, the moisture condition of the soil when color is determined must always be recorded. Color of mottled soils must be determined at their natural moisture contents because manipulation will blend and destroy individual colors. The apparent color of a soil, both wet and dry, is one of the tools used to locate different soils and to determine the limits of each soil horizon (layer). The individual horizons are defined under "Soil Profile," below.

Color indicates possible presence of certain compounds. Black to dark brown colors are indicative of organic matter. Reddish soils indicate the presence of unhydrated iron oxides (hematite) and are generally well drained. Yellow and yellowish brown soils indicate presence of iron, perhaps hydrated iron, and are poorly drained; otherwise, the iron would be in a different chemical form with a different color, perhaps redder. Grayish blue and gray and yellow mottled colors indicate poor drainage. White colors indicate presence of considerable silica or lime, or in some cases aluminum compounds.

Soil Structure

A soil mass in its natural state tends to break or form a structure of a rather definite shape resembling a geometric figure. Thus a soil may have a prismatic, block, granular, crumb, or floury structure. Structure is indicative of drainage characteristics and is one of the tools used to locate different soils and to determine the limits of soil horizons. Soil structure should not be confused with the structural (strength) characteristics of a soil.

Soil Profile

A vertical cross section of soil layers constitutes the soil profile, which is composed of several major layers as shown in Fig. 4. Over the years, the system of letter designations of the different horizons have been changed and extended several times. The designations shown in Fig. 4 are termed Master Horisons. There are 22 further subdivisions within the Master Horizons that are termed Subordinate Distinctions. A complete description of these horizons and their subordinates is given in the following references:

- **"Designations for Master Horizons and Layers of Soil,"** U.S. Department of Agriculture, Agency for International Development, October 1986.
- **"Soil Survey Manual,"** 430-V, Issue 1, U.S. Department of Agriculture, Soil Conservation Service, June 1981.

Since this system of designations is too extensive to describe here, only the general characteristics of the O, A, B, C, and R horizons are summarized below and referred to in other parts of this publication. The O, A, and B horizons are layers that have been modified by weathering, while the C horizon is unaltered by soil-forming processes. The R horizon, below the other soil layers, is the underlying material in its original condition of formation.

O horizon. The top layer composed primarily of organic litter, such as leaves, needles, twigs, moss, and lichens, that has been deposited on the surface. This layer, as well as underlying layers, may not exist due to erosion.

A horizon. The original top layer of soil having the same color and texture throughout its depth. It is usually 10 to 12 in. thick but may range from 2 in. to 2 ft. Removing native cover of timber by lumbering operations or of grasses by farming may introduce erosion that removes this top layer as well as underlying layers. The A horizon is also referred to as the topsoil or surface soil when erosion has not taken place.

B horizon. The soil layer just below the A horizon that has about the same color and texture throughout its depth. It is usually 10 to 12 in. thick but may range from 4 in. to 8 ft. In regions of humid or semihumid climate, the B horizon is a zone of accumulation in the sense that colloidal material carried in suspension from overlying horizons has lodged in it. The B horizon is also referred to as the subsoil.

C horizon. The soil layer just below the B horizon having about the same color and texture throughout its depth. It is quite different from the B horizon. It may be of indefinite thickness and extend below any elevation of interest to the highway engineer. At the beginning of the soil profile development, the C horizon constituted the entire depth, but time, weather, and soil-forming processes have changed the top layers into the A and B horizons described above. The C horizon (mother soil) may be clay, silt, sand, gravel, combinations of these soils, or stone. The C horizon is also referred to as parent material or soil material.

R horizon. The layer of solid bedrock underlying the C horizon. It is of indeterminate depth and is in its original condition of formation.

U.S. Department of Agriculture Soil Classification System

A system of soil classification was devised by Russian agricultural engineers about 1870 to permit close study of soils with the same agricultural characteristics. Around 1900 this system was adopted by the U.S. Department of Agriculture, which has since classified and mapped the soils in most of the agricultural areas in the United States. Many agricultural and geological departments of state universities and colleges use a similar system.

Highway engineers found that this system and the resulting valuable soil information could be used in identifying soils,

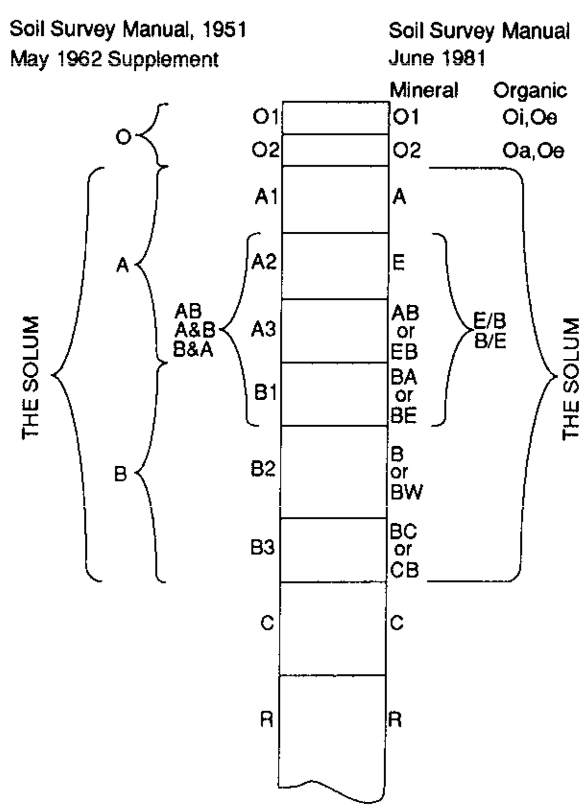


Fig. 4. Pedalogical soil profile showing the major horizons

after which they could classify them for engineering purposes in their own work. Therefore, while the U.S. Department of Agriculture system is called a soil classification system for purposes of nomenclature and use by the agricultural engineer, it is used as a soil identification system by the highway engineer. This system is based on the fact that soils with the same weather (rainfall and temperature ranges), the same topography (hillside, hilltop, or valley), and the same drainage characteristics (water-table height, speed of drainage, and so forth) will grow the same type of vegetation and be the same kind of soil. This is illustrated by the fact that the black wheat-belt soils of the West are the same as the black wheat-belt soils of Russia, Argentina, and other countries.

The system is important basically because a subgrade of a particular soil series, horizon, and grain size will perform the same wherever it occurs since such important factors as rainfall, freezing, groundwater table, and capillarity of the soil are part of the identification system. In no other system in use are these important factors employed directly. The system's value and use can be extended widely as soon as the engineering properties, such as load-carrying capacity, mud-pumping characteristics, and cement requirements for soil-cement, are determined for a particular soil. This is because soils of the same grain size, horizon, and series are the same and will function the same wherever they occur. Hence, a North Carolina engineer and a Texas engineer, after each has identified a soil in his or her own area by this system, could exchange accurate pavement design and performance data.

This system can be used only as an initial step in soil classification since the engineering properties of a soil must be determined after it is identified.

In 1965, the USDA system was improved and extended by adoption of the principles of soil taxonomy (discussed later). Terminology of the older system may still be encountered so it is included here for that purpose.

In the pre-1965 system, soils were divided into three main orders—zonal, intrazonal, and azonal—depending on the amount of profile development.* The zonal soils are mature soils characterized by well-differentiated horizons and profiles that differ noticeably according to the climatic zone in which they occur. They are found in great areas where the land is well drained but not too steep.

Intrazonal soils are those with well-developed characteristics resulting from some influential local factor of relief or parent rock. They are usually local in occurrence. Bog soils, peats, and salt soils are typical examples.

Azonal soils are relatively young and reflect to a minimum degree the effects of environment. They do not have profile development and structure developed from the soil forming processes. Alluvial soils of flood plains and dry sands along large lakes are examples.

Great Soil Groups. The three major divisions in the pre-1965 system are subdivided into suborders and then further subdivided into great soil groups on the basis of the combined effect of climate, vegetation, and topography. For example, the great chernozem soil group is developed under grass vegetation in temperate subhumid areas, while the laterite group is formed in areas of abundant rainfall and high temperature. The great

* These three divisions of the top order replace the two categories (pedalfers and pedocals) previously used by the Department of Agriculture. See James Thorp and Guy D. Smith, "Higher Categories of Soil Classification: Order Suborder, and Great Soil groups," *Soil Science*, Vol. 67 January to June 1949, pages 117-126.

soil groups falling in the zonal, intrazonal, and azonal orders are given in Table 1.

Soil Series. Soils within each great soil group are divided into soil series, and the soil series are further broken down into soil types.

Similar soils within a great soil group that have uniform development (the same age, climate, vegetation, and relief) and similar parent material are given a soil series designation. All soil profiles of a certain soil series, therefore, are similar in all respects with the exception of a variation in the texture of the topsoil, or A horizon. Each soil series was originally named after a town, county, stream, or similar geographical source, such as "Norfolk" or "Hagerstown," where first identified. This method of naming a series is not necessarily used now since it may in some cases interfere with the Department of Agriculture's present system of correlating a number of series over wide areas.

Soil Taxonomy. In 1951 when soil taxonomy was initiated, there were approximately 5500 soil series recognized in the United States. However, these soils were classified by the USDA system of 1938 in which classes were loosely defined. Differing experiences of soil scientists resulted in differences of opinion into which classification many soil series fell. Some series seemed to fit into a number of classes in a category, while others did not fit into any class. Consistency in classification was difficult to maintain. As the number of defined series continued to increase, it was recognized that a more logical and precise classification system was needed, and the principles of soil taxonomy were employed.

Soil taxonomy is intended to be a logical, well-defined classification system. It is a comprehensive soil classification system developed between 1951 and 1965 and continually updated to the present. It conveys usable and applicable soil data and interpretations between competent soil scientists by using nomenclature devised from Greek and Latin roots to make the class names as connotative as possible. By knowing the nomenclature, the engineer can deduce the basic properties of the soil and its suitability for given applications. For example, by knowing that "aqu" indicates wetness and "ents" denotes the soil order, entisols, one can determine that ""aquents"" are recently deposited, wet soils with few or no diagnostic horizons that have been subjected to very little weathering.

In 1965, USDA adopted what was then known as the 7th Approximation as its soil classification system. Soil surveys completed since then have used soil taxonomy as its basis for

Order	Suborder	Great soil groups
Zonal soils	1. Soils of the cold zone	Tundra soils
	2. Light-colored soils of arid regions	Desert soils Red desert soils Sierozem Brown soils Reddish-brown soils
	3. Dark-colored soils of semiarid, subhumid, and humid grasslands	Chestnut soils Reddish chestnut soils Chernozem soils Prairie soils Reddish prairie soils
	4. Soils of the forest-grassland transition	Degraded chernozem Noncalcic brown or Shantung brown soils
	5. Light-colored podzolized soils of the timbered regions	Podzol soils Gray wooded or Gray podzolic soils* Brown podzolic soils Gray-brown podzolic soils Red-yellow podzolic soils*
	6. Lateritic soils of forested warm-temperature and tropical regions	Reddish-brown lateritic soils* Yellowish-brown lateritic soils Laterite soils*
Intrazonal soils	1. Halomorphic (saline and alkali) soils of imperfectly drained arid regions and littoral deposits	Solonchak or Saline soils Solonetz soils Soloth soils
	2. Hydromorphic soils of marshes, swamps, seep areas, and flats	Humic-glei soils* (includes wiesenboden) Alpine meadow soils Bog soils Half-bog soils Low-humic-glei* soils Planosols Groundwater podzol soils Groundwater laterite soils
	3. Calcimorphic soils	Brown forest soils (braunerde) Rendzina soils
Azonal soils		Lithosols Regosols (includes dry sands) Alluvial soils

*New or recently modified great soil groups.

From "Higher Categories of Soil Classification: Order, Suborder, and Great Soil Groups," by James Thorp and Guy D. Smith, *Soil Science*, Vol. 67, January to June 1949, pages 117-126.

Table 1. Soil classifications in the higher categories

classification. Although not specifically designed for highway engineers, engineers can obtain useful information by becoming familiar with the taxonomy and recognizing the key formative elements in the soil class name. These key elements give specific information on such items as soil moisture, texture, particle size and mineralogy, climate, relief, vegetation, etc.

The six category classification system has a few classes in the highest categories and an increasing number in each succeeding class so that the lowest category has the largest number of classes. The six category levels of soil taxonomy are: orders, suborders, great groups, subgroups, families, and series.

Each order is classified according to the complete soil horizon and differentiated by the diagnostic surface and sub-surface horizons. Generally, the degree of weathering plays a major role in which order a soil belongs. The ten orders and their general properties are given below.

1. Histosols - soils derived mainly from organic soil materials.
2. Entisols - recently deposited or recently exposed soils that have not been in place very long, and therefore have had very little weathering.
3. Vertisols - clayey soils that occur in environments where the soils develop deep, wide cracks during periods of dryness. These soils have a high volume-change potential.
4. Inceptisols - characterized by indistinct horizons.
5. Aridisols - distinguished by being dry or at least physiologically dry because of high salt content.
6. Mollisols - contain dark-colored surface horizons that are rich in bases; most are developed under grass.
7. Spodosols - contain either a horizon in which amorphous mixtures of organic matter and aluminum have accumulated, or less commonly, a thin, black or dark reddish pan cemented by iron or iron-manganese, or an iron-organic matter complex is present.
8. Ultisols - contain translocated clay, but are relatively low in bases.
9. Alfisols -contain translocated clay, but are relatively high in bases.
10. Oxisols - weathered soils that have low cation exchange capacity of the fine-earth fraction, low cation retention, and no more than traces of primary alumino-silicates in the first 2 meters, or they have iron-rich mixture of clay, quartz, and other diluents with a mottled appearance that forms a continuous phase within 30 cm of the surface.
11. Andisols - in-situ weathering of volcanic materials into amorphous components.

In the United States, there are 11 orders, 53 suborders, 261 great groups, about 1900 subgroups, approximately 6755 families, and over 17,000 series. As knowledge and experience increase, and as new soils are observed, this system allows definitions to be elaborated and classes to be redefined or expanded without creating confusion. This system does not use surface horizons (A or O) that are thin enough to be obliterated by normal plowing or fires. All classifications are based on permanent soil profiles and soil characteristics.

As indicated, the nomenclature of soil taxonomy, except for soil series, is designed so that the class names are indicative of the category of the system. All order names have "sol" for the final syllable from the Latin solum. The suborder names are two syllables. The first gives common characteristics of the suborder and the second distinguishes the order. Great group names are formed by prefixing another formative element to the suborder name. Subgroup names are formed from great group names with one or more modifiers that indicate properties intergrading to some other class or to some aberrant soil property. Family names, the fifth category, have a polynomial name based on criteria used to differentiate families. The sixth category, soil series, are usually named after a community or a geographic feature in the vicinity where the soil was first defined.

Each categoric class name also describes certain characteristics of the soil with the most basic characteristics in the upper levels, and the more specific characteristics in the lower categories. As stated, orders are classified according to the complete soil horizon and differentiated by the diagnostic surface and sub-surface horizons. Suborder and great group classes are distinguished by such items as moisture content, organic content, temperature, pH, composition, stratification, disturbance by man, presence or absence of certain minerals or horizons, coarse fragments, chroma of the horizons, cation exchange capacity, percentage base or sodium saturation, etc. Subgroups are modifiers to the great groups by identifying a feature or features that fall on the outside of the great groups central concept. Subgroups fall into three basic categories: typics, intergrades, and extragrades. Typics are subgroups that show no distinguishing characteristics from the great groups. Intergrades are subgroups with certain properties associated with other orders. Extragrades have all the properties of the great group or higher category, or another subgroup, except for one. Families are classed by particle size, mineralogy, structure, texture, calcareousness, pH, depth, slope, coatings of silt and clay, and cracks. Two to four differences are commonly used to distinguish classes. Finally, series are differentiated by all the parameters in the upper classes that are appropriate for the series.

To demonstrate how soil taxonomy may be used, the following example is given. The Miami soil series is fine-loamy, mixed mesic Typic Hapludalf, which is a fine-loam soil, with many minerals and particle sizes (mixed), that has an annual soil temperature between 8° and 15° C (mesic), from the order alfisol (alf). It is dry less than 90 days a year (ud), it has a normal horizon development (hapl), and it is typical of the soil profile in that class (typic).

This is only a brief outline of soil taxonomy. To learn more about soil taxonomy, the following are excellent sources of information.

Keys to Soil Taxonomy, Soil Management Support Services Survey Staff, fourth edition. SMSS Technical Monograph No. 6, Blacksburg, Virginia, 1990 (this reference is updated approximately ever 2 years).

Philipson, W.R., et al., "Engineering Values of Soil Taxonomy," *Highway Research Record No. 426*, Highway Research Board, 1973.

Johnson, W.M., and McClelland, J.E., "Soil Taxonomy: An Overview," *Transportation Research Record No. 642*, Transportation Research Board, 1977.

Fernau, E.A., "Application of Soil Taxonomy in Engineering," *Transportation Research Record No. 642*, Transportation Research Board, 1977.

McCormack, O.E., and Flach, K.W., "Soil Series and Soil Taxonomy," *Transportation Research Record No. 642*, Transportation Research Board, 1977.

Basic soil class	Subclass	Soil separates: Very coarse sand, 2.0-1.0 mm.	Coarse sand, 1.0-0.5 mm.	Medium sand, 0.5-0.25 mm.	Fine sand, 0.25-0.1 mm.	Very fine sand, 0.1-0.05 mm.
Sands	Coarse sand	25% or more		Less than 50%	Less than 50%	Less than 50%
	Sand	25% or more			Less than 50%	Less than 50%
	Fine sand	Less than 25%		—or—	50% or more	Less than 50%
	Very fine sand					50% or more
Loamy sands	Loamy coarse sand	25% or more		Less than 50%	Less than 50%	Less than 50%
	Loamy sand	25% or more			Less than 50%	Less than 50%
	Loamy fine sand	Less than 25%		—or—	50% or more	Less than 50%
	Loamy very fine sand					50% or more
Sandy loams	Coarse sandy loam	25% or more		Less than 50%	Less than 50%	Less than 50%
	Sandy loam	30% or more; Less than 25%		—and—	Less than 30%	Less than 30%
	Fine sandy loam	Between 15 and 30%		—or—	30% or more	Less than 30%
	Very fine sandy loam	Less than 15%		—or—	More than 40%*	30% or more

*Half of fine sand and very fine sand must be very fine sand.

Table 2. Percentage of sand sizes in subclasses of sand, loamy sand, and sandy loam, basic textural classes as defined by the USDA

The basic textural groups based on particles smaller than 2 mm in diameter as defined by the Department of Agriculture are given in Fig. 2. Three of the basic textural groups—sand, loamy sand, and sandy loam—are further subdivided as shown in Table 2. The terminology and size limits of the soil separates are given in Fig. 1. The textural soil group has a "gravelly" prefix if it contains 20% or more gravel. The basic textural class name, however, is based on the size distribution of the material smaller than 2 mm in diameter. The sum of the percentages of each of the soil separates, therefore, equals 100 after the gravel material has been excluded.

Application to Soil-Cement Testing. The Department of Agriculture soil classification system has proved very helpful in soil-cement testing and construction work. It has been found that the cement requirement of a definite soil series and horizon is the same regardless of where it is encountered. Once the cement requirement has been determined by laboratory tests, no further soil-cement tests for that particular soil are needed when it is used on another project. Thus, by identifying the soil proposed for use by series and horizon, the need for conducting soil-cement tests can be sharply reduced or eliminated altogether for large areas. An increasing number of engineers are making use of this system of classification to reduce their soil-cement testing work.

Thompson, P.J., et al., "An Interactive Soils Information Systems User's Manual," USA-CERL Technical Report N-87/18, U.S. Army Corps of Engineers, CERL, July 1987.
National Soils Handbook, United States Department of Agriculture, Soil Conservation Service, 1983.

Agriculture Handbook No. 436, (currently being revised, to be issued approximately 1995.)

Soil Type. As already mentioned, the texture of the surface soil, or A horizon, may vary slightly within the same soil series. The soil series is, therefore, subdivided into the final classification unit, called the soil type. The soil type recognizes the texture of the surface soil and is made up of the name of the soil series plus the textural classification of the A horizon. For example, if the textures of the A horizon of a soil series named Norfolk are classified texturally as sand and sandy loam, the soil type in each case would be Norfolk sand and Norfolk sandy loam. Both of these soil types would have the same B and C horizons (parent material) and would have been found under the same conditions of climate, vegetation, and topography.

Availability of Soil Maps. A large portion of the United States has been surveyed and mapped by the Department of Agriculture in cooperation with state agricultural experiment stations and other federal and state agencies. At the completion of a soil survey, which usually covers an area of one county, a soil map is made and a report is written that describes the soil types occurring. These reports and maps are available to the public and can be viewed at or obtained from the U.S. Department of Agriculture, county extension agents, colleges, universities, libraries, and the state conservationist of the USDA's Soil Conservation Service. A tabulation of the counties in the United States for which maps have been published as of February 1991 is given in *List of Published Soil Surveys*, 1990, U.S. Department of Agriculture, Soil Conservation Service, revised February 1991. This publication may be obtained from Public Information Division, Soil Conservation Survey, P.O. Box 2890, Room 0054-S, Washington, D.C. 20013.

In addition, the U.S. Bureau of Reclamation has made surveys using the agricultural soil classification system in 17 western states. Inquiry can be made at local offices of the Bureau of Reclamation for the availability of soils data for these areas.

CHAPTER 2

SOIL CONDITION AND RELATED TESTS

The pavement engineer has a particular concern with soil strength. And while strength is much a matter of type of soil, the strength of any individual soil is largely a matter of its moisture condition and, to a degree, its density or unit weight.*

SOIL WATER

Moisture content. The moisture or water content of a soil is normally expressed as a percentage of the oven-dry weight of the soil. It is determined by weighing the moist soil, oven-drying it to constant weight at 110 deg C (230 deg F), and reweighing. The difference in weights is the weight of water the soil contained. This weight divided by the oven-dry soil weight and expressed as a percentage is the moisture content. ASTM standard D2216 or AASHTO T265 describe the test method. In common usage, the terms "moisture content" and "water content" are synonymous.

Soil moisture is of three different types: gravitational water, capillary water, and hygroscopic water.

Gravitational water. Water free to move under the influence of gravity. This is the water that will drain from a soil. For in-place soils it is water at and below the ground- water table and is often termed groundwater. Groundwater is unbound or "free" water.

Capillary water. Water held in the soil pores or "capillaries" by "capillary action." This is the result of attraction between fluids and solid surfaces, which, because of stronger attraction to water than to air, results in the upward curving of a meniscus at the water's edge and to actual rising of water in a narrow tube. As a contrary example, air has stronger attraction than mercury, and mercury shows an inverted meniscus. The "lifting" of water in a capillary tube has been represented as "surface tension" effects and does lift the water in tension. Water pressure is zero at the groundwater level or phreatic surface. It is under pressure below this surface and in tension above. Note that capillary water cannot exist directly in the presence of gravitational water. Effects of gravity on a mass of water result in pressure or compression from the water weight. This overrides the tension and relieves the capillary attractions. Capillary water is not generally considered to be "free" water since it is, at least weakly, bound by the surface tension action. However, because it is not strongly bound to soil particles directly, it has sometimes been described as free water in older and especially in agriculturally-oriented soil references. Capillary moisture can be considered to be absorbed into the soil pores in the same way wet ink would be considered to be absorbed by a blotter.

Hygroscopic water. Moisture retained by soil after gravitational and capillary moisture are removed. It is held by each soil grain in the form of a very thin film adsorbed on the surface by molecular attractions involving both physical and chemical affinity. Hygroscopic moisture can include water taken into the crystal lattice of soil grains by physio-chemical attractions. Adsorbed moisture, while removable by oven drying, tends to remain after air drying. It can be described as the air-dry moisture content. This film is in equilibrium with the moisture content of the air and increases or decreases with changes in humidity. Since the hygroscopic water is in surface films, the quantity relates to surface area of soil grains. Because each dividing of a grain results in two additional surfaces, the smaller the soil grains the greater the surface area of soil grains and the greater the hygroscopic moisture.

SOIL-WATER CONSISTENCY

Most soils include a fine fraction of silt, clay, or a combination. The consistency of these soils can range from a dry solid condition to a liquid form with successive addition of water and mixing as necessary to expand pore space for acceptance of the water. The consistency passes from solid to semisolid to plastic and to liquid as illustrated in Fig. 5.

About 1911, A. Atterberg, a Swedish scientist, defined moisture contents representing the limits dividing the states of consistency. The shrinkage limit (SL) separates solid from semisolid, the plastic limit (PL) separates semisolid from plastic state, and the liquid limit (LL) separates plastic from liquid

* It is common practice to refer to the weight per unit volume of soil as "density." In the strictest sense, the term should be "unit weight" expressed as mass per unit volume.

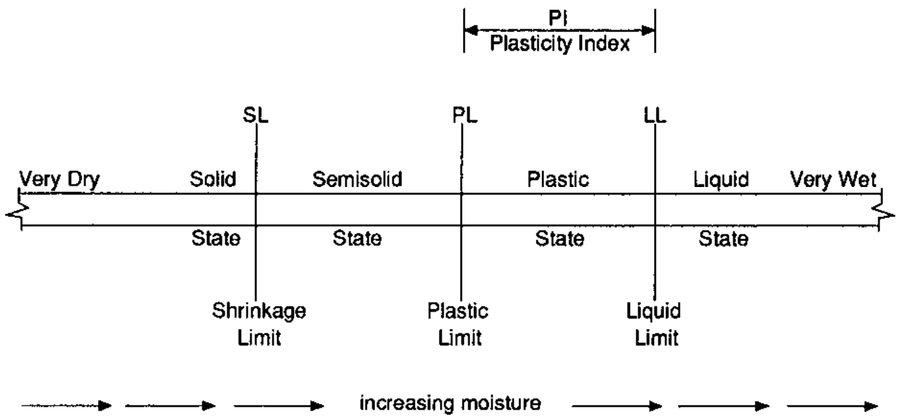


Fig. 5. Soil states and consistency limits (Atterberg limits)

state. The width of the plastic state (LL minus PL), in terms of moisture content, is the plasticity index (PI). The PI is an important indicator of the plastic behavior a soil will exhibit.

Standard procedures have been developed so that consistent determinations can be made by anyone employing these procedures to establish the dividing limits. Since it is the more plastic or finer soils or soil fractions that reflect this pattern of response to moisture variation the standard tests are performed on the portion of a soil that will pass a No. 40 mesh sieve.

Shrinkage Limit. This limit separates the solid state from the semisolid state. It is represented by the point in a drying process at which no further shrinkage takes place while drying continues. Standard test procedures can be found in ASTM D427. While this limit is an element of the soil-water consistency pattern it has less significance or application than the other limits in relation to soil engineering.

Plastic Limit. This limit separates the semisolid state from the plastic state. It is represented by the moisture content at which the soil when rolled to a 1/8 in. cylindrical ribbon will begin to break into short sections. Standard test procedures are described in ASTM D4318 and AASHTO T90.

Liquid Limit. This limit separates the plastic state from the liquid state. It is represented by the moisture content at which the soil when separated by a standard (1mm) groove in a standard cup will flow back together (1 cm length) under 25 standard (1 cm fall impact) taps or blows. Standard test procedures are described in ASTM D4318 and AASHTO T89. The liquid limit is considered to relate directly to soil compressibility; the higher the LL, the greater the compressibility.

Plasticity Index. The PI is the numerical difference between the LL and the PL each expressed as moisture content in percent. ASTM D4318 and AASHTO T90 are standards for PI determination. This index is a significant indicator of soil behavior. The higher the index number, the more plastic the soil will be. Low PI soils are very sensitive to moisture change since only a few percent (equal to the PI) moisture can change the soil from a plastic to a liquid state.

NONPLASTIC SOILS

The soils considered in the prior section had compositions including a fine fraction of silt and clay, which provided them a plastic consistency. Soils composed almost entirely of sand sizes, gravel, coarse silt, or combinations of these have a nonplastic consistency. The Atterberg limits tests, which involve the minus No. 40 sieve fraction, rolling to a 1/8-in. cylinder, or flowing a standard groove closed by tapping, cannot be conducted on these type soils. The PI is then designated as NP (nonplastic).

Sands. Coarse sands and fine gravels, which include little or no particle sizes that would pass the No. 40 sieve, are clearly nonplastic (NP). These will show no significant consistency variation with moisture variation. Finer sands do display a consistency response to moisture variation. Dry sands have no cohesive element to join grains together. The individual particles respond with only mass, shape, and gravity. When excavated or placed in piles, they will show characteristic maximum slopes at their "angle of repose." Moist sands are bound by capillary moisture films at contact points between grains. This bonding is zero when dry, increases through a maximum as moisture is increased, and returns to zero on complete saturation. This moisture variation does not cause swelling or shrinkage in undisturbed sands, but when moist sands are moved or disturbed by construction operations, the capillary fringes will compete with gravity forces. The result is increased voids and reduced density. This phenomena is termed "bulking," and it can lead to settlement problems, especially in light construction when not properly considered and treated.

Silts. Coarse silts and silty fine sands can often be subjected to the Atterberg limits tests. They will tend to show a PL equal to—occasionally somewhat higher than—the LL, so that the PI will be zero (or less). These are classed as nonplastic (NP). Some finer to very fine silts are encountered that include no clay fraction. It was earlier noted that a primary soil-forming process was to break or grind rock to finer and finer grain size. When this process results in silt size particles with no added influence of accumulated additives or chemical changes toward clay formulation, the resulting soils are "rock flour" silts. These will show a moisture variation consistency response and permit Atterberg limits testing. The result for most finer silt deposits is a PL close to the LL and a PI less than 10. Some extremely fine "rock flour" silts are to be found, particularly in arctic areas where decay processes are minimal, that have strong plasticity attributes. These are found to have quite high moisture contents at both the PL and LL. Thus, while the PI may be double digit, it would not reflect the high plasticity commonly associated with a high liquid limit.

MOISTURE EQUIVALENT

Both capillary moisture and hygroscopic moisture are to a degree "bound" and represent a capacity for the soil to hold water against forces tending to remove it. Measures of this "water-holding capacity" are the "moisture equivalent" moisture contents. Low values are associated with coarse grained soils, which are not moisture sensitive and are highly permeable. High values are associated with plastic clays, which are very moisture sensitive and are of low permeability.

Field Moisture Equivalent. The field moisture equivalent (FME) is the minimum moisture content at which a smooth surface of soil will absorb no more water in 30 seconds when the water is added in individual drops. It shows the moisture content required to fill all the pores in sands, when the capillarity of cohesionless expansive soils* is completely satisfied and when cohesive soils approach saturation. The test procedure is covered by AASHO T93.

Centrifuge Moisture Equivalent. The centrifuge moisture equivalent (CME) is the moisture content of a soil after a saturated sample is centrifuged for one hour under a force equal to 1000 times the force of gravity. This test, ASTM D425, is used to assist in structural classification of soils.

Low values, such as 12 or less, indicate permeable sands and silts; high values, such as 25, indicate impermeable clays. High values indicate soils of high capillarity, and low values indicate soils of low capillarity. Study of soils and test results shows that when the FME and CME are both more than 30 and the FME is greater than the CME, the soil probably expands upon release of load and is classified as elastic.

SOIL MOISTURE SUCTION

FME and CME have origins in agricultural soil technology, but they found early applications in relation to highway subgrade assessment and right-of-way soil surveys. They continue in some use, but the technology concerned with subgrade moisture-strength in place is now more focused on "soil moisture suction." This is the moisture tension associated with capillarity.

Water in soil above the watertable has a pressure less than atmospheric. It arises from the surface tension (capillary) and adsorption forces by which the water is bound or held in the soil. This is termed soil moisture suction or soil suction.

Soil Suction. Moisture tension or suction ranges from zero at saturation to quite large values for relatively dry soil. The suction can be expressed in units of (negative) pressure. Relation between the suction and moisture content is very dependent on the soil type. A test standard for measurement of soil suction is presented as AASHTO T273 and ASTM D3152.

pF Scale. The pF scale was introduced by Schofield** to simplify the treatment of the broad pressure ranges involved. On the pF scale the soil suction is represented as the common logarithm of the length in centimeters of an equivalent suspended water column.

Tensiometer. Soil suction or moisture tension is a measure of the negative pore pressure in soils in place above the water table. Tensiometers have been developed to measure the negative pore pressure in place in subsurface installations. With calibration of the negative pressure to moisture content for the soil involved, the tensiometer provides a measure of water content variation.

DENSITY, POROSITY, VOID RATIO, AND DEGREE OF SATURATION

A soil mass is a porous material containing solid particles interspersed with pores or voids. These voids may be filled with air, with water, or with both air and water. There are several terms used to define the relative amounts of soil, air, and water in a soil mass.

Density. The weight of a unit volume of the soil. It may be expressed either as a wet density (including both soil and water) or as a dry density (soil only). Soil density is discussed further in Chapter 4.

Porosity. The ratio of the volume of voids to the total volume of the mass regardless of the amount of air or water contained in the voids. Porosity may also be expressed as a percentage.

Void ratio. The ratio of the volume of voids to the volume of soil particles. The porosity and void ratio of a soil depend upon the degree of compaction or consolidation. Therefore, for a particular soil in different conditions, the porosity and void ratio will vary and can be used to judge relative stability and load-carrying capacity with these factors increasing as porosity and void ratio decrease.

Degree of saturation. The ratio of the volume of water to the volume of voids–usually expressed as a percentage.

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* Mica or diatomaceous soils. Diatomaceous soils are largely made up of the siliceous remains of small marine algae called diatoms.

** Schofield, R.K., "The pF of the Water in Soil," *Transactions, Third International Congress of Soil Science*, (Oxford), 1935.

WATER-AIR-SOLIDS RELATIONS

A commonly employed conceptual diagram is helpful in considering the inter-relations of the weights and volumes of water, air, and solids in a volume of soil.

Air, Water, Soil Diagram. Fig. 6 shows a conceptual diagram of relative volumes of air, water, and soil solids in a volume of soil: The pertinent volumes are indicated by symbol to the left while weights of these material volumes are indicated by symbol to the right. This treatment helps with concepts of inter-relationships and derivation of simple expressions for important soil parameters.

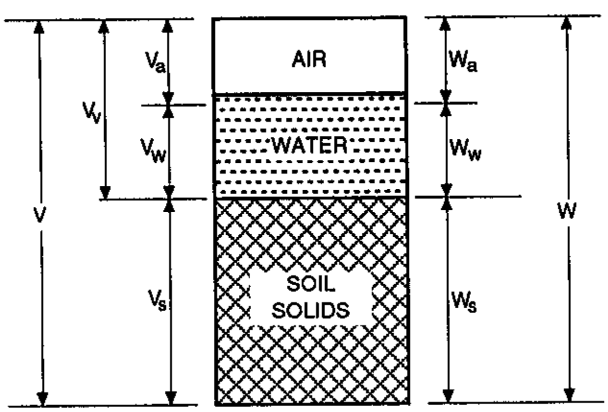
Limitations to Soil-Water Consistency. It was earlier indicated that soil consistency can range from dry solid to liquid as moisture is added. It is important to recognize that the amount of water that can be added to a soil is limited by the volume of the soil voids. Consistencies beyond this voids-filled condition can only be gained by disturbing the soil to reduce density and increase voids.

It can be very instructive, based on only limited soil tests, to employ the concepts and relations of Fig. 6 to examine the maximum possible (voids full) moisture content for densities of concern.

Plastic Fines in Coarse Soils. There is another instructive opportunity using the concepts of soil-water consistency and of Fig. 6. Soil behavior where coarse sands and gravels are involved will be greatly dependent on the relative quantities of coarser particles and plastic fines.

Coarse soils with substantial fines. Consider a quantity of coarse soil particles free of any fines. The soil structure would be one of particles in contact with adjacent surrounding particles and with voids in the structure. This would be a quite strong structure since forces would be transmitted directly from particle to particle of the sand or gravel present. If, however, plastic fines are added sufficient to more than fill voids in the coarse particle soil structure, the coarse particles will be separated and no longer in contact. Such a soil structure would behave in much the same way as the plastic fines alone. Obviously a transition between the more stable coarse grain structure and the plastic fines structure or texture would occur with the addition of fines approaching voids-full in the coarse matrix to the over-full condition.

Coarse soils with limited fines. The coarse particles of a low fines soil would form a quite stable or strong grain to grain structure. Low plasticity fines dispersed in the voids–less than void filling–would have little or no effect on the stable coarse matrix. Response to moisture variation (consistency) of the fines would not be significant. If, however, the fines were plastic, they could act as a lubricant reducing inter-particle friction. While the effect on the stable coarse matrix would not be great, the soil-water consistency of the fines would have an impact.



V—Total volume

V_v—Volume of voids

V_a—Volume of air

V_w—Volume of water

V_s—Volume of solids

W—Total weight

W_a—Weight of air = 0

W_w—Weight of water

W_s—Weight of solids

Unit weight of water, $\gamma_w = \frac{W_w}{V_w} = 62.4 \text{ lb/ft}^3$

Moisture content, $w = \frac{W_w}{W_s} \times 100(\%)$

Specific gravity of solids, $S_s = \frac{W_s}{V_s \gamma_w}$

Dry density (unit weight), $\gamma_d = \frac{W_s}{V}$

Void ratio, $e = \frac{V_v}{V_s}$

Wet density (mass unit weight), $\gamma_m = \frac{W}{V}$

Note: Common practice is to term weight-per-unit-volume density.
The more correct term is unit weight.

Fig. 6. Conceptual diagram of air-water-solids relationship

CHAPTER 3

SOIL CLASSIFICATION SYSTEMS

In order that soils may be evaluated, it is necessary to devise systems or methods for identifying soils with similar properties, and then to follow this identification with a grouping or classifying of soils that will perform in a similar manner when their densities, moisture contents, and relations to water tables, climate, and so forth, are similar. Such procedures are common practice where a variety of soil types exists. A clear understanding of the relation of soil identification to soil classification is necessary to prevent confusion about many factors involved in soil work.

In general, certain soil tests such as gradation and Atterberg limits are used to assist in the identification of a soil. Then these same tests are used to assist in classification. Several systems are in use for both processes.

The primary purpose of soil identification is to describe a soil in sufficient detail to permit engineers to recognize it and, if need be, to obtain samples in the field.

The most widely used system of engineering soil classification for highways was devised a number of years ago by the Public Roads Administration (later the U.S. Bureau of Public Roads and now the Federal Highway Administration) for subgrade soils. In this system, AASHTO M145, soils are classified in one of seven groups, A-1 through A-7.

The U.S. Army Corps of Engineers adopted a classification system that uses texture as the descriptive term such as "GW—gravel, well graded"; "GC—clayey gravel"; and "GP—gravel, poorly graded." This classification was expanded in cooperation with the USBR and the TVA and was referred to as the Unified Soil Classification System. It is now identified as ASTM D2487. The U.S. Federal Aviation Administration has also adopted this system.

AASHTO CLASSIFICATION SYSTEM

The American Association of State Highway and Transportation Officials system of classifying soils is an engineering property classification based on field performance of highways. It was originally referred to as the Public Roads Administration Soil Classification System since it was devised by that organization in 1931 (*Public Roads*, Vol. 12, No. 5, July 1931) and revised in 1942 (*Public Roads*, Vol. 22, No. 12, February 1942). The system was revised further by a subcommittee of the Highway Research Board in 1945 (Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, Vol. 25, 1945, pages 375-392). In the same year, it became a standard of AASHO—AASHO M145. It has been called the HRB Classification System and the AASHO Classification System. Highway Research Board has become Transportation Research Board (TRB) and AASHO has become AASHTO. The classification standard is now AASHTO M145.

Grouping together soils of about the same general load-carrying capacity and service resulted in seven basic groups that were designated A-1 through A-7. The best soils for road subgrades are classified as A-1, the next best A-2, and so on, with the poorest soils classified as A-7.

Members of each group have similar broad characteristics. However, there is a wide range in the load-carrying capacity of each group as well as an overlapping of load-carrying capacity in the groups. For example, a borderline A-2 soil may contain materials with a greater load-carrying capacity than an A-1 soil, and under unusual conditions may be inferior to the best materials classified in the A-6 or A-7 soil group. Hence, if the

AASHTO soil group is the only fact known about a soil, only the broad limits of load-carrying capacity can be stated. As a result, the seven basic soil groups were divided into subgroups with a group index devised to approximate within-group evaluations. Before 1966, group indexes ranged from zero for the best subgrades to 20 for the poorest. Increasing values of the index within each basic soil group reflect (1) the reduction of the load-carrying capacity of subgrades and (2) the combined effect of increasing liquid limits and plasticity indexes and decreasing percentages of coarse materials.

In 1966 the AASHO Recommended Practice was revised so that there is now no upper limit of group index value obtained by use of the formula. The adopted critical values of percentage passing the No. 200 sieve, liquid limit, and plasticity index are based on an evaluation of subgrade, subbase, and base course materials by several highway organizations that use the tests involved in the classification system.

Under average conditions of good drainage and thorough compaction, the supporting value of a material as a subgrade may be assumed as an inverse ratio to its group index, that is, a group index of zero indicates a "good" subgrade material and group index of 20 or greater indicates a "very poor" subgrade material.

The charts and table used in AASHTO M145, the Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, are shown in Figs. 7 and 8 and Table 3. In addition to the charts and table given here, the AASHTO Recommended Practice includes detailed descriptions of each classification group and the basis for the group index formula. Examples of the determination of the group index are also included.

Classification of materials in the various groups applies only to the fraction passing the 3-in. sieve. Therefore, any specification regarding the use of A-1, A-2, and A-3 materials in construction should state whether boulders, retained on a 3-in. sieve, are permitted.

ASTM (UNIFIED) SOIL CLASSIFICATION SYSTEM

The American Society for Testing and Materials' Soil Classification System is based on the system developed by Dr. Arthur Casagrande of Harvard University for the U.S. Army Corps of Engineers during World War II. The original classification was expanded and revised in cooperation with the U.S. Bureau of Reclamation (USBR) and the Tennessee Valley Authority (TVA) so that it now applies to embankments and foundations as well as to roads and airfields. This system is a standard of ASTM D2487. The system is used by these agencies as well as the FAA.

The ASTM Soil Classification System identifies soils according to their textural and plasticity qualities and their grouping with respect to their performances as engineering construction materials. The following properties form the basis of soil identification:

1. Percentages of gravel, sand, and fines (fraction passing the No. 200 sieve).
2. Shape of the grain-size distribution curve.
3. Plasticity characteristics.

The soil is given a descriptive name and letter symbols, as shown in Table 4, indicating its principal characteristics.

Three soil fractions are recognized: gravel, sand, and fines (silt or clay).

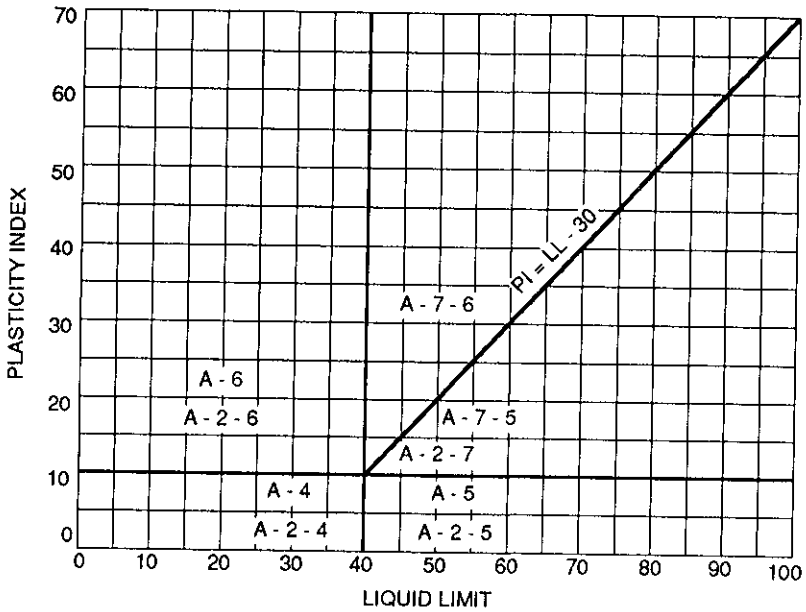
The soils are divided as (1) coarse-grained soils, (2) fine-grained soils, and (3) highly organic soils. The coarse-grained soils contain more than 50% material retrained on the No. 200 sieve, and fine-grained soils contain 50% or more passing the No. 200 sieve.

If the soil has a dark color and an organic odor when moist and warm, a second liquid limit should be performed on a test sample that has been oven-dried at 110 + 5 deg C for 24 hours. The soil is classified as organic silt or clay (O for organic) if the liquid limit after oven drying is less than three-fourths of the liquid limit of the original sample determined before drying.

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (More than 35% passing No. 200)			
Group classification	A-1			A-2							A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve analysis, *percent passing:*											
No. 10	50 max.	–	–	–	–	–	–	–	–	–	–
No. 40	30 max.	50 max.	51 min.	–	–	–	–	–	–	–	–
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit	–		–	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

*Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Table 3. AASHTO classification of highway subgrade materials



Note—A-2 soils contain less than 35% finer than 200 sieve.

Fig. 7. Liquid limit and plasticity index ranges for AASHTO soil classes

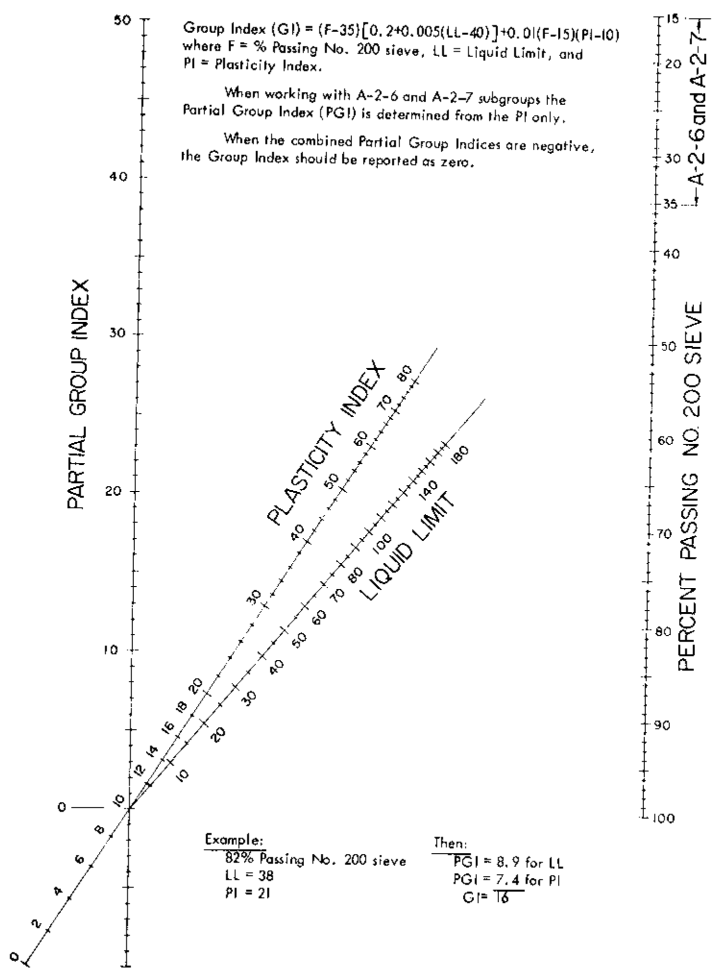
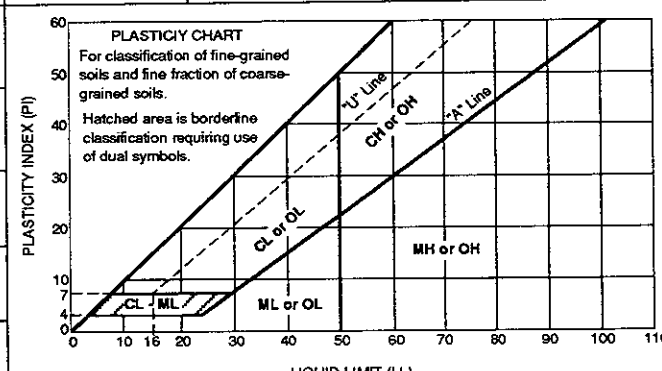


Fig. 8. Group Index chart

Major Divisions			Group Symbols	Typical Descriptions	Classification Criteria		
Coarse-Grained Soils More than 50% retained on No. 200 sieve*	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve: GW, GP, SW, SP More than 12% pass No. 200 sieve: GM, GC, SM, SC 5% to 12% pass No. 200 sieve: Borderline classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures		Fines classify as ML or MH	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Fines classify as CL or CH	
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands, and gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with Fines	SM	Silty sands, sand-silt mixtures		Fines classify as ML or MH	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Fines classify as CL or CH	
Fine-Grained Soils 50% or more passes No. 200 sieve*	Silts and Clays Liquid limit less than 50		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and Clays Liquid limit 50 or more		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
Highly Organic Soils			PT	Peat, muck, and other highly organic soils	Visual-Manual Identification, see ASTM Designation D2488		

*Based on the material passing the 3-in. (75-mm.) sieve.

Table 4. ASTM (Unified) soil classification system

The coarse-grained soils are subdivided into gravels (G) and sands (S). The gravels have 50% or more of the coarse fraction (that portion retained on the No. 200 sieve) retained on the No. 4 sieve, and the sands have more than 50% of the coarse fraction passing the No. 4 sieve. The four secondary divisions of each group—GW, GP, GM, and GC (gravel); SW, SP, SM, and SC (sand)—depend on the amount and type of fines and the shape of the grain-size distribution curve. Representative soil types found in each of these secondary groups are shown in Table 4 under the heading "Typical Descriptions."

Fine-grained soils are subdivided into silts (M) and clays (C), depending on their liquid limit and plasticity index. Silts are those fine-grained soils with a liquid limit and plasticity index that plot below the A line in the diagram in Table 4, and clays are those that plot above the A line. The silt and clay groups have secondary divisions based on whether the soils have relatively low (L) or high (H) liquid limit (greater than 50).

The highly organic soils, usually very compressible and with undesirable construction characteristics, are classified into one group designated PT. Peat, humus, and swamp soils are typical examples.

In addition to the "Group Symbols" given here (GW, SP, CH, etc.) the system also gives definitions of "Group Names" (silty gravel, clayey sand, etc.) as well as more precise definitions of organic soils (OL and OH). This information is too extensive to include here. See ASTM D2487.

PRIOR FEDERAL AVIATION ADMINISTRATION CLASSIFICATION SYSTEM

The FAA now uses the ASTM D2487 Soil Classification System. However, the prior system may still be encountered and is included here for that purpose.

The older FAA Soil Classification System is based on the gradation analysis and the plasticity characteristics of soils.

The textural classification is based on a grain-size determination of the minus No. 10 material and the use of Fig. 3, that employs definitions of sand, silt, and clay sizes. These are shown in Fig. 1.

The mechanical analysis, liquid limit, and plasticity index data are referred to Table 5, and the appropriate soil group, ranging from E-1 to E-13 inclusive, is selected.

Two modifications of this procedure may be required. In one case, test results on fine-grained soils, groups E-6 through E-12, may place the soil in more than one group. When this occurs, the test results are referred to Fig. 9, where the appropriate soil group is determined.

The other modification is used when considerable material is retained on a No. 10 sieve since the classification is based on the material passing the No. 10 sieve. Upgrading the soil one to two classes is permitted when the percentage of the total sample retained on the No. 10 sieve exceeds 45% for soils of the E-1 to E-4 groups and 55% for the remaining groups, provided the coarse fraction consists of reasonably sound material. Further, it is necessary that the coarse fraction be fairly well-graded from the maximum size down to the No. 10 sieve size. Stones or rock fragments scattered through a soil are not considered of sufficient benefit to warrant upgrading.

		Mechanical analysis					
			Material finer than No. 10 sieve				
Soil group		Retained on No. 10 sieve,* percent	Coarse sand passing No. 10, retained on No. 40, percent	Fine sand passing No. 40, retained on No. 200, percent	Combined silt and clay passing No. 200, percent	LL	PI
Granular	E-1	0-45	40+	60 –	15 –	25 –	6 –
	E-2	0-45	15+	85 –	25 –	25 –	6 –
	E-3	0-45	–	–	25 –	25 –	6 –
	E-4	0-45	–	–	35 –	35 –	10 –
Fine grained	E-5	0-55	–	–	45 –	40 –	15 –
	E-6	0-55	–	–	45+	40 –	10 –
	E-7	0-55	–	–	45+	50 –	10-30
	E-8	0-55	–	–	45+	60 –	15-40
	E-9	0-55	–	–	45+	40+	30 –
	E-10	0-55	–	–	45+	70 –	20-50
	E-11	0-55	–	–	45+	80 –	30+
	E-12	0-55	–	–	45+	80+	–
	E-13	Muck and peat—field examination					

*If percentage of material retained on the No. 10 sieve exceeds that shown, the classification may be raised provided such material is sound and fairly well graded.

Table 5. Prior FAA classification of soils for airport construction

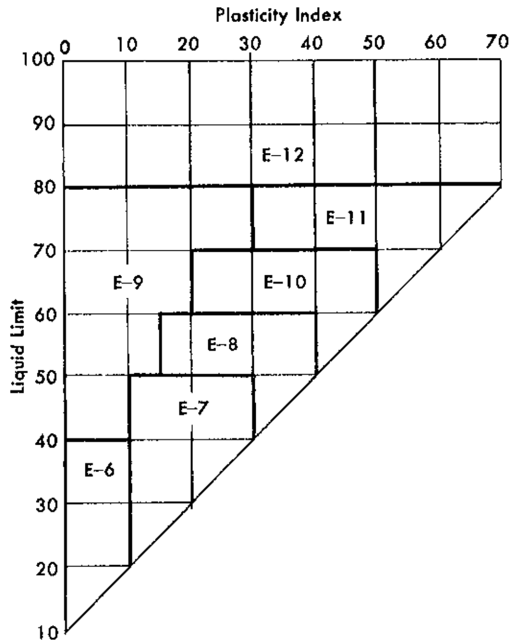


Fig. 9. FAA classification for fine-grained soils

CHAPTER 4

ENGINEERING PROPERTIES OF SOILS

The basic characteristics of soils—internal friction, cohesion, compressibility, elasticity, capillarity, and permeability—combine to indicate the mechanical and hydraulic properties that determine the suitability of soils for engineering use. In most applications, the strength of the soil—its load-carrying capacity and resistance to movement or consolidation—is of primary importance. Depending on the proposed use, other properties such as volume-change characteristics or drainage may be considered in evaluating suitability.

These engineering properties are influenced most by the soil type, its gradation and composition. Thus, it is possible to know, in a general way, whether a soil will be strong or weak, free-draining or impermeable, if we know its gradation, texture, or classification grouping.

For a particular soil, the engineering properties are greatly affected by the degree of compaction, the moisture content at time of compaction, and the existing moisture content. Therefore, the discussion of engineering properties and corresponding test methods in this chapter is preceded by a section on soil compaction.

SOIL COMPACTION

The term "compaction" refers to the practice of artificially densifying or increasing the unit weight of a soil mass by rolling, tamping, vibrating, or other means. There is no other single treatment that produces so marked a change in physical properties at so low a cost as does properly controlled compaction.

The density of a soil is measured in terms of its volume-weight and usually expressed as pounds of wet soil or dry soil per cu ft. These volume-weights are designated as wet density and dry density respectively.

Several factors influence the value of density obtained by compaction. Of primary importance are: (1) the moisture content of the soil; (2) the nature of the soil—that is, its gradation and physical properties; and (3) the type and amount of compactive effort.

Moisture-Density Relationships. A basic principle in soil analysis is that for a given compaction effort and a given compaction moisture content, a soil will attain a corresponding density. For any particular compaction effort the density resulting will be greater as moisture increases from the dry-side condition, until a maximum is attained. The density will then decrease with further moisture increase toward the wet-side. The maximum for the effort being employed is termed "maximum density," and the corresponding moisture is termed "optimum moisture content" (OMC).

Moisture-Density Tests. Standard tests were first developed by R.R. Procter* in 1933, which involved a standard compaction effort representing the construction densities common for highway work. This compaction came to be known as "Standard Proctor" compaction or density, and because it was standardized early by AASHO it was also variously known as "Standard AASHO" density. Current test standards for this compaction effort are ASTM D698 and AASHTO T99. The standard test involves a 5.5 lb drop hammer, 12-in. drop, 25 blows per layer, 3 layers, in a 4-in. diameter mold. This provides 12,375 ft-lb per cu ft of compaction effort. Alternates of a 6-in. diameter mold and of other test parameters can be employed, but the compaction effort must be the same.

At the outset of World War II, the U.S. Army Corps of Engineers in developing design methods for heavy aircraft found need for a higher density as a construction standard. The test standard devised came to be known as "Modified Proctor" or "Modified AASHO" (now "Modified AASHTO") density. Current test standards for this compaction effort are ASTM D1557 and AASHTO T180. The standard test involves a 10 lb drop hammer, 18-in. drop, 56 blows per layer, 5 layers, in a 6-

* Proctor, "R.R., Fundamental Principles of Soil Compaction," *Engineering News-Record*, Vol. 59, 1933.

in. diameter mold. This provides 56,000 ft-lb per cu ft of compaction effort. Alternates of a 4-in. diameter mold and of other test parameters can be employed.

Further discussion of moisture-density tests and related matters can be found in *Soil-Cement Laboratory Handbook** and in *Subgrades and Subbases for Concrete Pavements.**

Typical mositure-density curves are shown in Fig. 17 (page 28) where the effects of moisture and density on soil strength are discussed.

The maximum density of a soil gives approximate information on its gradation; the optimum moisture gives approximate information on the clay and silt content. The shape of the moisture-density curve, which may vary from a sharply peaked parabolic curve to a flat one or to one sloping irregularly downward as the moisture content increases, gives additional valuable data showing the influence of moistu-re on the load-supporting value of the soil. For example, a flat curve indicates a soil that will have the same load-supporting power over a wide range in moisture contents.

ASTM D4253 and D4254 are specialized tests for some cohesionless, free-draining soils for which a well-defined moisture-density curve is not apparent.

Field Density Determination. The performance of pavement structures depends to a great extent upon proper, uniform compaction of the subgrade and pavement components. Therefore, roadbuilding agencies usually control compaction by specifying minimum requirements based on (1) soil density, (2) compactive effort, or (3) a combination of the two. Most agencies specify some minimum density and limit the range of moisture content. In most instances, AASHTO T99 or T180 form the basis for these specifications. For example, 95% of maximum density and a moisture content of 2% moisture less to 2% greater than optimum moisture content. Some methods commonly used to determine in-place densities are discussed in the following paragraphs.

The basic procedure used to determine in-place soil densities consists of removing a sample of the compacted soil and determining its wet weight, moisture content, and the volume of the cavity previously occupied by the soil.

The soil sample is removed from an area approximately 4 to 5 in. in diameter and extending the full depth of the layer being tested; the resulting cavity should be approximately cylindrical in shape. The moist soil is weighed, and its moisture content determined.

The volume of the cavity or hole is determined by accurately measuring the amount of material of known unit weight required to fill the hole. Sand (sand-density cone method), water (water-balloon method), and oil have been used for this purpose. Some of these methods are described in AASHTO T191, T205, and T214 and in ASTM D1556 and D2167.

In-place densities can also be determined by means of undisturbed samples, and there are several methods involving nuclear devices (AASHTO T238 and T239, and ASTM D2922 and D3017). Rapid field test methods for determining degree of compaction and moisture content include: ASTM D4643, D4944, D4959 and D5080.

STRUCTURAL STRENGTH OF SOILS

In general the strength of soil is a matter of shear strength or resistance to shearing. Shear resistance (τ) of soil is the sum of two factors: the cohesion (c) and the "internal friction" (N tan ϕ). The "angle of internal friction," ϕ, and thus tan ϕ, is virtually a friction coefficient in a common friction sense. It represents a resistance to shearing (friction) along a shear path or surface. The N is the normal stress (unit force) on the shear surface, which, acting with the tan ϕ factor, creates the internal friction or shear resistance. The total shear resistance is thus the sum of the cohesion and internal friction:

$$\tau = c + N \tan \phi$$

Cohesion and Internal Friction

The cohesion and the angle of internal friction are soil properties, but the normal stress, N, is independently induced by external loads on the soil mass or self-weight of overlying soil. Because of this, it is not possible to relate shear resistance to a particular soil type and condition in more than a general way (except for soils for which $\phi = 0$).

Cohesion is the result of molecular attractions involving both the soil particles and moisture films. This is an inherent bonding together, which provides shear resistance independent of external forces on the soil. Cohesion is related to plasticity in that highly plastic (high PI) soils are considered to be highly cohesive. The cohesion of a particular soil, however, is greatly dependent on its moisture condition and to some extent its density.

Note that any soil in a liquid condition—moisture above the LL—would have no shear resistance and, therefore, no cohesion. With drying the cohesion increases, and a highly plastic clay can become rock-like when quite dry. In this condition it would have very high shear resistance and cohesion.

Conversely a low PI fine sand when dry would have no inherent bonding and no cohesion. When moist such a dry sand would enjoy some capillary bonding of particles. This is called "apparent cohesion."

Internal friction is the resistance to shearing in a soil mass from the "angle of internal friction" of the soil and the normal stress induced on potential shear surfaces by external loads or self-weight of the soil. The friction coefficient represented is the tangent of the angle of internal friction (tan ϕ), so that the larger the angle the larger the coefficient. The internal friction is the product of the tan ϕ (coefficient) and the normal stress (N) on a potential shear surface.

The angle of internal friction is the angle whose tangent is the ratio between the resistance offered to sliding along any plane in the soil and the component of the applied force acting normal to that plane. Values are given in degrees and range from 0° for highly plastic clays to as high as 45° for aggregate materials having quite angular particles.

Shearing Resistance

The shearing resistance of a soil is the sum of its cohesion and the internal friction. Plastic soils that have shear strength largely from the cohesion element are generally taken to be the weaker soils. Nonplastic soils that have shear strength predominently from internal friction are generally taken to be the stronger soils. An anomaly to this general pattern is in regard to granular soils with no binding fines when unconfined. One can handle a chunk or clod of plastic soil but not of dry sand. The dry sand has no strength or shear resistance.

Mohr Diagram. The Mohr diagram and Mohr circles provide a means for demonstrating the shear strength behavior of different types of soil. This diagram is a plot of shear strength against normal stress. The circles are defined by the difference between a vertical or major principal normal stress and a lateral or minor principal confining stress. See Fig. 10.

Figs. 11, 12, and 13 illustrate the relation of cohesion and

* Available from Portland Cement Association.

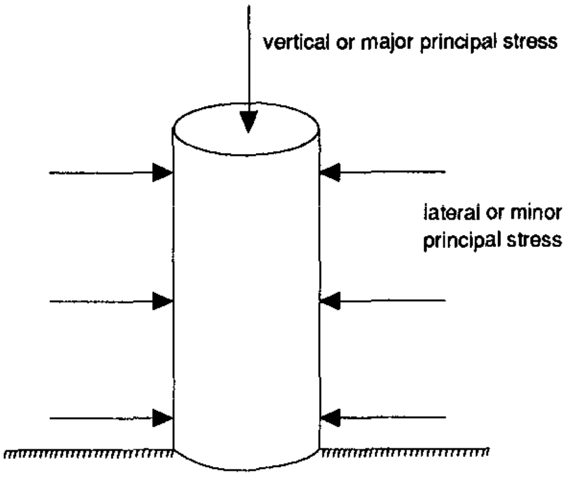


Fig. 10. Triaxial compression test

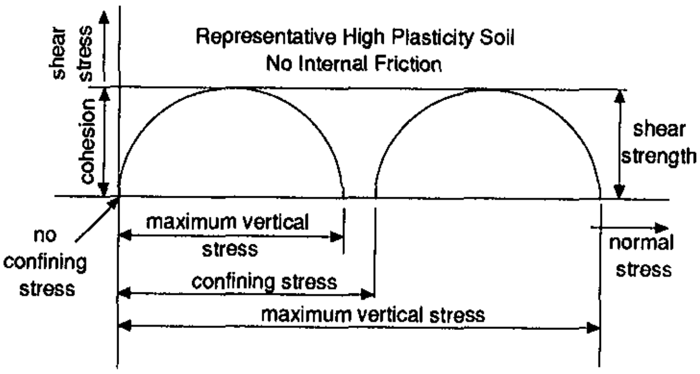


Fig. 11. Mohr's diagram for a highly plastic clay soil

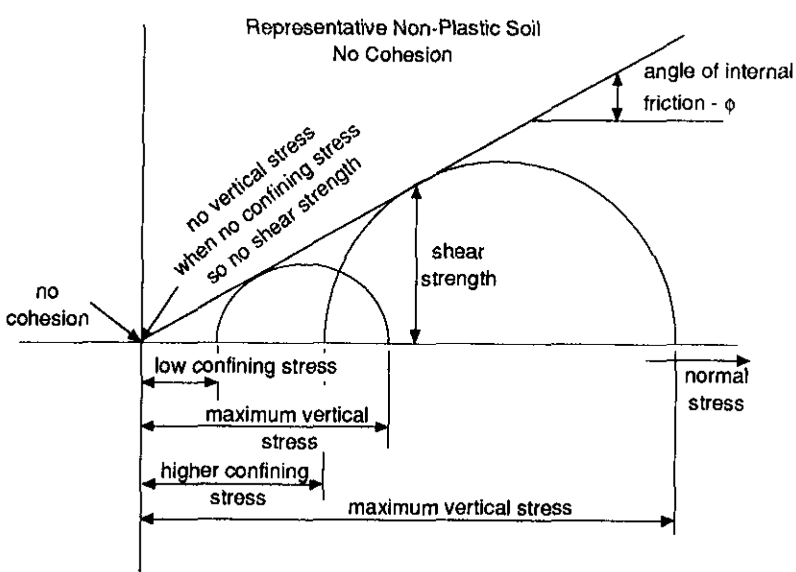


Fig. 12. Mohr's diagram for a nonplastic soil

internal friction to shear resistance for various soil types. Fig. 11 is a diagram illustrating the shear strength of a highly plastic clay soil with a zero angle of internal friction. It thus has no internal friction, and shear strength is due only to cohesion. External forces have no effect on shear strength.

Fig. 12 is a diagram illustrating the shear strength of a nonplastic soil with zero cohesion. Shear strength is entirely due to internal friction. Thus with no (lateral) confinement from external forces (or self-weight) shear strength is zero. However, the shear strength increases rapidly with increasing confinement.

Fig. 13 is a diagram illustrating the shear strength pattern for most soils. Shear strength is the sum of the inherent cohesion plus the internal friction due to the combination of confinement from loading and the angle of internal friction. These soils can range from high cohesion plus low internal friction (small ϕ) to low cohesion plus high internal friction (large ϕ).

Shear Strength Tests

Various laboratory tests have been devised to determine the shearing strength of soils: the direct shear test, the triaxial compression test, and the unconfined compression test. These are briefly discussed in the following sections.

Direct Shear Test. (See Fig. 14.) A soil specimen is placed in a split mold and shearing forces are applied to cause one portion of the specimen to slide in relation to the other portion. The test is conducted on specimens at several different loads normal to the shearing force. The unit normal forces applied and the shear stresses of failure are plotted to determine the internal friction and cohesion of the soil. The direct shear test is used for both cohesive and cohesionless soils.

Triaxial Compression Test. (See Fig. 10) A soil specimen is encased in a rubber membrane and subjected to a constant lateral pressure through a liquid or gas around the specimen. A vertical axial load is then applied and increased to failure of the specimen. The test is repeated with different lateral pressures. The test data are analyzed graphically by use of Mohr circles to determine the cohesion and internal friction of the soil. The results are used in various formulas to determine the load-carrying capacity of the soil for dams, buildings, pavements, and the like. Several types of equipment and variations in test procedures have been developed. The test is described in ASTM D2850.

Unconfined Compression Test. (See Fig. 15). The unconfined compression test is similar to the triaxial compression test except that no lateral pressure is used. A vertical axial force is applied until the specimen fails along a shear plane or by bulging. The vertical strains or deformations are measured along with the applied load increments. The shear strength is usually assumed to be half of the compressive strength. Details of the test procedure are given in ASTM D2166 and AASHTO T208.

INDEX TYPE TESTS

In earlier times, and to a degree still, the complexity of the pavement design problem prevented the direct use of shear strength for design. Design methods were devised based on tests that provided an index number related to soil strength. This was most commonly, but not always, considered to represent shear strength. Several of these tests and methods have come into common use and continue to be employed.

California Bearing Ratio (CBR) Test. This is an index type test that measures the force required to penetrate a soil surface by a 3-sq-in. end area round piston. The index (CBR) value is the percent of an established reference values for 0.1 and 0.2-in.

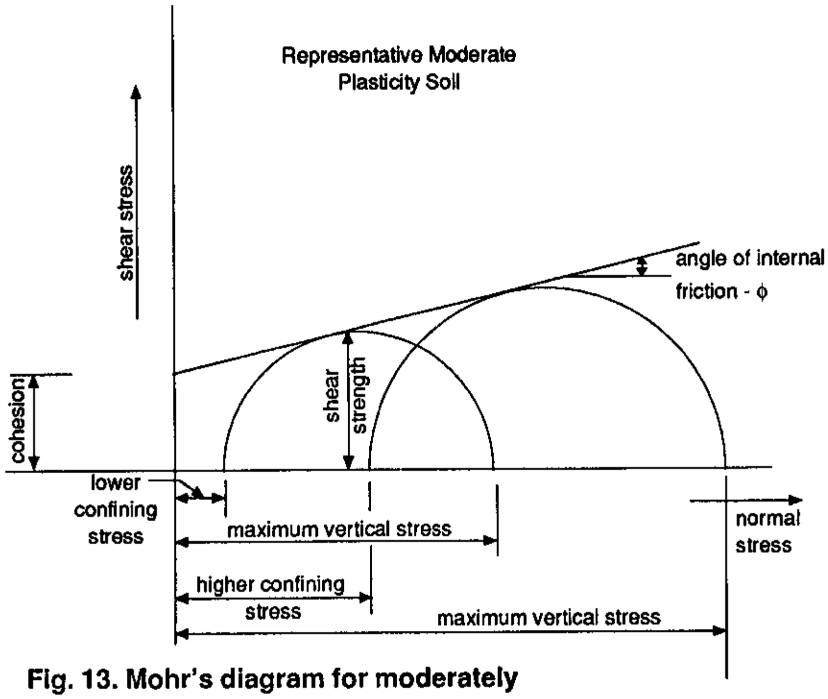


Fig. 13. Mohr's diagram for moderately plastic soil

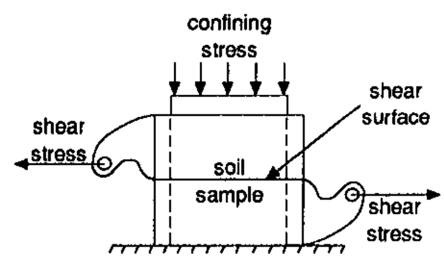


Fig. 14. Direct shear test

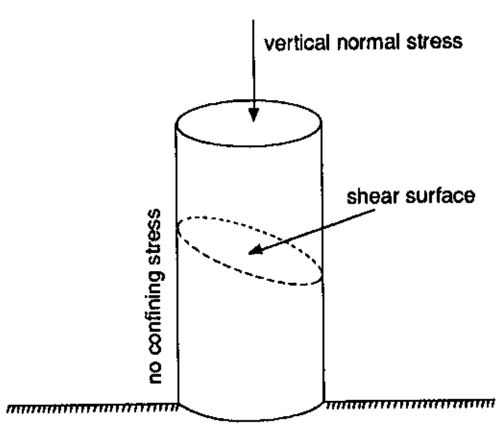


Fig. 15. Unconfined compression test

penetration. The reference value of 100 was originally considered to represent the resistance of a well-graded crushed stone. This test and design methods were originally devised by O.J. Porter for the California Division of Highways. It was called the California Bearing Ratio and thus CBR. Both the test and CBR design methods were further developed and modified by the U.S. Army Corps of Engineers for applications to World War II airfield design and later similar uses. Typical CBR values may range from 2 to 8 for clays and 70 to 90 for crushed stones.

The test and design methods have been widely employed for flexible pavement design both within the United States and worldwide. It is no longer considered proper to relate penetration loads to crushed stone resistance or to show the CBR number as a percentage. It is also proper to use only the CBR acronym without identifying it with the California Bearing Ratio.

The Army Corps of Engineers and some highway departments use the CBR principle in conducting tests to evaluate the bearing value of materials. Methods of preparing specimens and conducting the test are given in ASTM D1883 and AASHTO T193. Several agencies have their own modifications. Numerous papers in Transportation Research Board publications and in other engineering publications give details on various testing techniques and data interpretation.

Stabilometer Test. This laboratory test was developed by F.N. Hveem of the California Division of Highways. The stability of a soil can be determined by means of the Hveem Stabilometer, which measures the transmitted horizontal pressure due to a vertical load. The stability, expressed as the "resistance (R) value," represents the shearing resistance to plastic deformation of a saturated soil at a given density. The test is described in AASHTO T190 and ASTM D2844.

The R value may vary from zero to 100—zero representing a liquid and 100 representing a material that transmits no horizontal pressure from an applied load. The R value is used in flexible pavement design.

Cohesiometer Test. This laboratory test was also developed by F.N. Hveem of California. The cohesiometer test* provides a measure of the cohesive resistance or tensile strength of a material. The sample is clamped in the testing machine directly over a hinge. One end is fixed and the other end is loaded through a cantilever arm until rupture occurs over the hinge at midpoint of the specimen. The load required to cause rupture is used to calculate the cohesiometer value. The cohesiometer value is used in the design of flexible pavements.

Modulus of Subgrade Reaction, k. This is a bearing test, conducted in the field, which provides an index to rate the support provided by a soil or subbase layer directly beneath the concrete slab.

Practically all concrete pavement design is based on the modulus of subgrade reaction, k, used in the Westergaard formulas and in the PCA methods contained in the booklets, *Thickness Design for Concrete Highways and Streets,*** and *Design of Concrete Airport Pavements.***

The modulus of subgrade reaction, k, is defined as the reaction of the subgrade per unit of area of deformation and is given in lb per sq in. (psi) of area per in. of deformation. The unit load for a deformation of 0.05 in. is generally used in determining k. However, the Army Corps of Engineers determines k for the deformation obtained under a load of 10 psi. For realistic test results neither of these limits should be exceeded.

The determination of k for concrete pavement design is made in the field on the subgrade in place, or on the subbase—if one is used—under conditions that will approximate reasonable mean service conditions. A 30-in.-diameter plate is recommended. The plate size influences bearing-test results because the forces resisting deformation consist of shear around the plate perimeter as well as consolidation under the area of the plate. With plates of 30-in. diameter and greater, the shear-resisting forces around the perimeter are of minor importance.

For heavy-duty airport pavement design where a strong stabilized subbase is planned, a modification in the interpretation of the action of the subbase is required. This is described in Appendix B of PCA's *Design of Concrete Airport Pavement*.

Details for plate-bearing field tests are given in ASTM D1195 and D1196, in AASHTO T221 and T222, and in the Department of the Army Technical Manual TM-5-824-3. ASTM D1196 is a nonrepetitive load test that determines a gross k (k_g). Most pavement designs have been based on the k_g value. The elastic k (k_e) value as determined from the repetitive plate-bearing test, ASTM D1195, is a higher value since most of the inelastic deformation is eliminated in the repetitive test.

When performing plate-bearing tests on stabilized subbases, the loading equipment may not be able to produce a deflection of 0.05 in. Even if it were, the resulting pressure on the subbase may far exceed the pressures exerted under the concrete slab by the traffic loads, and this would not represent service conditions. As a result, a maximum pressure of 10 psi is recommended for plate-loading tests.

Cone Penetrometers. Cone Penetrometers, such as the WES Cone Penetrometer and the Dynamic Cone Penetrometer (DCP), are devices used to measure the strength of a soil in place. Test results can be used to estimate the soil shear strength, CBR, and modulus value. Since the tests are rapid and essentially nondestructive, they are ideally suited for on-site construction evaluation and testing and can be used over large areas to evaluate uniformity. The penetrometers consist of a small cone with an apex angles between 30° and 60° mounted to a steel rod. The projected area of the base of the cones is approximately 0.5 sq in. The penetrometers are driven into the ground at either a constant rate (WES) or by dropping a specific hammer weight over a given distance (DCP). Measured values are the load needed to drive the penetrometer or blow counts per unit of depth. These values are then correlated to CBR, shear strength, or soil modulus value. Also, by plotting load or blow counts against depth, one can obtain profiles of changing soil strengths. This can be used for such things as checking the depth of stabilization and finding soft or stiff layers.

FIELD DETERMINATION OF SOIL BEARING VALUES

Soil bearing values are determined in the field for (1) soils under buildings, bridges, and dams; and (2) subgrade soils and pavements in place. Various direct loading procedures are used.

For large structures, field tests on soils are done to determine the sizes of footings or foundations, with or without piling, needed to support the design loadings or structure in service, without obtaining uneven or excessive settlement during or after construction.

Pore pressures built up by consolidation in the presence of moisture may also require analysis. The field test is usually conducted on the soil in place at the elevation of the proposed footing or foundation. The size of the loaded area is determined by the problem at hand, as is the type of area loaded; in some cases a footing itself may be loaded. In tests of this nature the primary

* Test Method No. 306B, "Testing and Control Procedures," *Materials Manual*, Vol. 1, State of California, Department of Public Works, Division of Highways.

** Available from Portland Cement Association.

data obtained consist of the unit load and time-deformation curve under load. Repetitive loading may or may not be required by the design problem. ASTM procedures and textbooks on soil mechanics can be consulted for additional details.

RÉSUMÉ OF BEARING VALUE OF SOIL

The foregoing discussion shows that much study and experience are required to arrive at a final figure for the bearing value of a subgrade soil for use in pavement design at a particular location. However, general ideas of a soil's bearing value can be obtained from published data by a general correlation of soil classifications with bearing values. This has been done in Fig. 16. The beginner as well as the specialist in soils will find this chart most valuable for approximate relationships.

The soil-bearing-value chart, Fig. 16: (see next page)′

1. Compares the AASHTO, ASTM (Unified), and FAA Soil Classification Systems.
2. Shows clearly the wide range in bearing values possible in the various soil classifications and the wide overlapping of classifications; hence, the need for specific test data for each soil on each specific project.
3. Gives general limits of bearing values for soils ranging from poorest to best. Thus, after laboratory tests are available permitting a close estimate as to where a soil will fall in a specific classification, it is possible to estimate the bearing value after consideration of drainage, rainfall, and other factors that influence subgrade performance.

When it is not practical to make a plate-bearing test on a section of subbase, an estimate of the k value on granular and cement-treated subbases can be determined from tables and figures given in the PCA design publications cited earlier in this section.

SOIL STRENGTH EVALUATION

While the potential strength of soil for design purposes is very much a matter of the type of soil, the condition or confinement of an individual soil has great impact on its strength. As earlier noted the confinement of low fines, high friction angle (ϕ), granular soils largely determines their in-place strength. Much more common, however, are subgrade soils that have substantial plastic fines. The strength of these soils can range widely from dry through their plastic range to liquid. This range is restricted in actual construction circumstances by the limits to voids—and thus to moisture—as density is increased. It follows from this that the strength of common soils is very dependent on the density and moisture content.

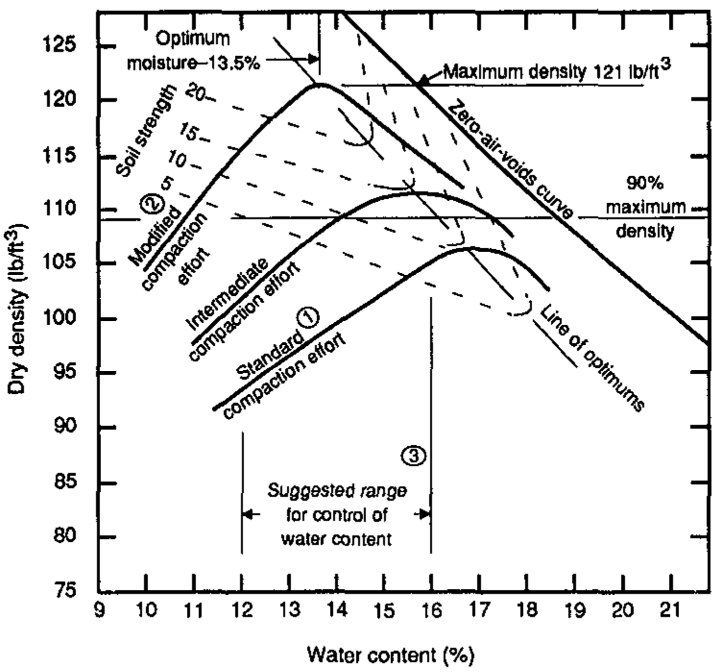
Even for a fixed density the soil moisture can vary from voids empty to voids full. It has been found that the soils beneath pavements will tend to a condition of nearly full voids—a 90 to 95% saturation condition. This becomes a virtually constant condition away from the edges of wide pavements. Subgrades under narrower highway pavements can respond to some seasonal variation.

Moisture, Density, Strength. The pattern of moisture, density, and strength can be examined over pertinent ranges of density and moisture by conducting strength tests on soil specimens prepared for moisture-density testing. Specimens compacted by a standard compaction effort at a variety of moisture contents from below to above optimum will attain densities to form a curve rising to maximum density at optimum moisture content then falling. If these specimens are then conditioned to near saturation (soaked) and tested for strength, they will indicate a portion of the moisture-density-strength pattern. Repeating this process for three compaction efforts—commonly standard AASHTO, and intermediate, and modified AASHTO—an entire moisture-density-strength pattern can be portrayed. This is most commonly done using CBR as the soil strength, but other strength tests will provide a similar pattern. Fig. 17 is an example of the development of this type plot.

The moisture contents plotted are those at time of compaction, and the densities are those attained. The soil strengths, however, are not those for the moisture at compaction but are for the near saturation moisture conditions resulting from soaking. These moisture contents better represent the highest moisture and lowest strength to be expected after field placement, and the strengths are, therefore, those to be considered for design. Strengths determined at compaction moisture contents would plot a somewhat similar pattern, but the strengths would be much greater.

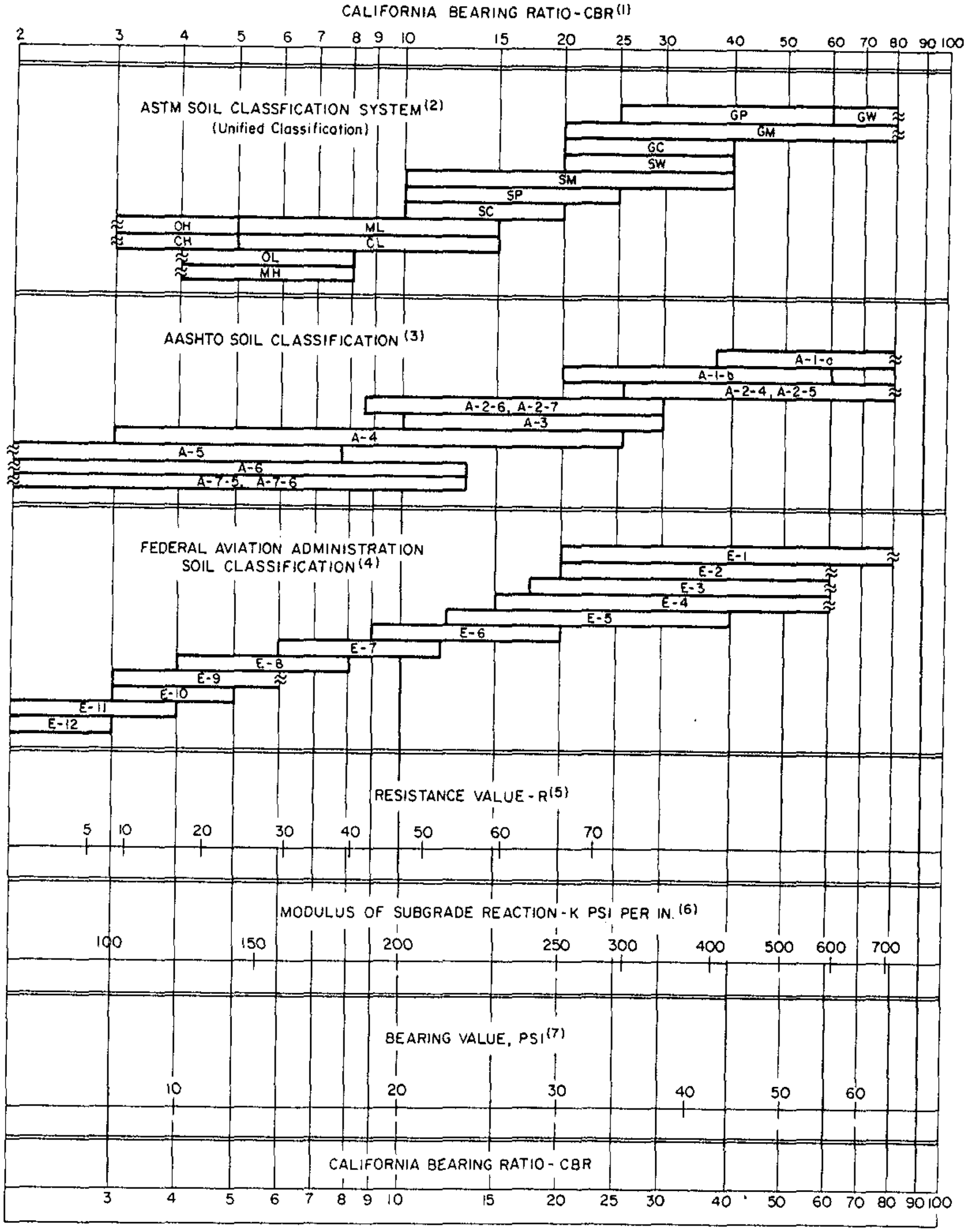
EXPANSION AND SHRINKAGE TESTS

The volume changes of highly expansive clays found in some areas of the world cause serious damage to pavements and structures. This is particularly true in regions where these soils remain in a relatively dry condition until wetted by an infrequent rainy period. The resulting expansion can be substantial and differential from point to point down a roadway or airport pavement.



① Standard proctor or standard AASHTO density.
② Modified proctor or modified AASHTO density.
③ For a specified minimum density of 90% modified AASHTO maximum density this suggested range for control of water content will reasonably assure the compacted soil will have a strength of 10 or better.

Fig. 17. Moisture content versus density with soil strength (CBR) contours. Reprinted with permission from *Construction Guides for Soils and Foundations* by Richard G. Ahlvin and Vernon Allen Smoots, Copyright ©1988, John Wiley & Sons, Inc.



(1) For the basic idea, see O. J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, pages 100-136.
(2) ASTM Designation D2487.
(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, 1945, Vol. 25, pages 376-392.
(4) *Airport Paving*, U.S. Department of Commerce, Federal Aviation Agency, May 1948, pages 11-16. Estimated using values given in FAA *Design Manual for Airport Pavements*. (Formerly used FAA Classification; Unified Classification now used.)
(5) C. E. Warnes, "Correlation Between *R* Value and *k* Value," unpublished report, Portland Cement Association, Rocky Mountain-Northwest Region, October 1971 (best-fit correlation with correction for saturation).
(6) See T. A. Middlebrooks and G. E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, page 152.
(7) See item (6), page 184.

Fig. 16. Approximate interrelationships of soil classifications and bearing values

These high-volume change soils are composed in moderate to large part of laminar, platy clay particles, which can draw water into this laminar crystal lattice. The result is a significant increase in volume or increase in pressure if the volume change is prevented. This can represent severe problems for pavement designers, and tests to evaluate potential expansion are necessary.

Index Tests. Several simple tests that indicate the volume change potential of soils are given in ASTM D427 (Shrinkage Limit, Shrinkage Ratio, Volumetric Shrinkage, and Linear Shrinkage). Test method ASTM D4829 gives an expansive index of soils and, based on the test results, evaluates soils from very low to very high expansion potential.

California Bearing Ratio (CBR) and Resistance R-Value Tests. Expansion tests are usually conducted in conjunction with the CBR (ASTM D1883) and R-Value (ASTM D2844) tests. In both instances, the test specimen is compacted to a predetermined density, at proper moisture content, in a mold, and a supply of water is made available. Surcharges, equal to the weight of the cover material that will overlay the soil in the ultimate pavement structure, are applied to the top of the specimen. The expansion that occurs during some given soaking period is measured as the actual change in length of the specimen, or the pressure exerted by the expanding soil can be measured by means of a calibrated restraining gage. The same specimen is then used for the CBR or R-value determination.

Sand Equivalent Test. A rapid field method, known as the sand equivalent test, has been developed to detect the presence of undesirable claylike materials in soils and aggregate materials. This method tends to magnify the volume of clay present in a sample somewhat in proportion to its detrimental effects.

The sand equivalent test is a sedimentation-type test in which a sample of the test material, in a prepared solution, is thoroughly agitated in a 100-ml. glass cylinder. After setting for 20 minutes, the sand and clay fractions settle into layers. The heights of these layers are measured by taking readings with a specially calibrated rod. The sand equivalent (SE) is calculated as follows:

$$SE = \frac{\text{sand reading}}{\text{clay reading}} \times 100$$

Concrete sands and crushed stone have SE values of about 80; very expansive clays have SE values of zero to 5.

Details of the test procedure are given in AASHTO T176. The test was formerly designated as ASTM D2417, but is no longer listed by ASTM.

RESILIENT MODULUS TEST

The earlier empirical or experience based flexible pavement design methods, which made use of index type strength tests, are being replaced by methods using theoretical models. These methods employ a property called the resilient modulus (M_R), which relates stress to strain. Soils have been found, on direct initial loading, to show strain response including an inelastic part. With repeated cycling of the loading this inelastic element is reduced and becomes more consistent cycle to cycle. The resilient modulus test is a triaxial type test, which is repeatedly loaded (increments of 200 cycle loading) at a particular confining stress and applied (deviator) stress. The "elastic" or "recovering" strain is determined for each cycled deviator stress. The resilient modulus is the deviator stress divided by the elastic strain. AASHTO T274 gives test details.

The following rough correlation has been developed between MR and CBR (see page 25):

$$MR = 1500 \times CBR$$

It is considered reasonable for soils with a CBR of 10 or less.

CONSOLIDATION TEST

A consolidation test was devised by Dr. Karl Terzaghi, an international authority on soil mechanics for foundations, to determine the consolidation or settlement that would take place in a soil under specific loadings. Sometimes called a compression test and one of the first soil load-bearing-value tests evolved, it is used to estimate the settlement that may take place in soil under large structures, such as buildings and bridge piers, and in very high earth embankments.

The test apparatus consists principally of a small, short cylinder that is filled with soil placed between two porous stones. The soil specimen is consolidated by a piston placed on the upper porous stone; any moisture forced from the specimen can escape through the porous stones. The piston is mounted on the short end of a lever arm, with weights on the opposite end. Ames dials are mounted to measure consolidation.

To conduct the test, the sample is loaded and deformations recorded at stated time intervals. The loads correspond to the anticipated field loads, and the time interval is plotted against the consolidation as a percentage. Results are analyzed in terms of determined field conditions.

Since the soil sample is completely confined, the test is applied only to field conditions of a similar nature—on building foundations, high fills, and the like, as previously mentioned.

A procedure for determining the rate and magnitude of consolidation of soil when it is unrestrained laterally and loaded and drained axially is given in AASHTO T216 and ASTM D2435.

PERMEABILITY AND CAPILLARITY

Permeability—that property of a soil allowing it to transmit water—depends on the size and number of continuous soil pores. Determined by test on a representative sample of soil, permeability is expressed as the coefficient of permeability. It equals the apparent velocity of water flow under a hydraulic gradient of 1, which exists when the pressure head (or height of water) on the specimen divided by the depth of the specimen equals unity.

The permeability of a soil varies with such factors as void ratio, grain size and distribution, structure, degree of cementation, and degree of saturation. It will also vary with the degree of compaction, since this influences the size of the soil pores. A particular soil loosely packed will be more permeable than the same soil tightly packed. Nature produces these same differences: (1) by freezing action in the surface in winter, loosening a soil, and (2) by repeated wetting and drying in the summer, consolidating the soil, in connection with shrinkage forces that may be present.

The coefficient of permeability, k, is used to determine the quantity of water that will seep through a given cross section of soil in a given time and distance under a known head of water. The formula:

$$Q = k \frac{H}{L} At$$

where

Q = quantity of water
k = coefficient of permeability
H = hydrostatic head
L = thickness of soil through which flow of water is determined under hydrostatic head, H
A = cross-sectional area of material
t = time

Very porous soils, such as sands that have a k value, in centimeters per second, of 1.0 to 10^{-3} can be drained. Silty and clayey sand soils have a k value of about 10^{-3} to 10^{-7}, and highly cohesive clays have a k value of less than 10^{-8}. It is difficult, if not impossible, to reduce the water content of soils by drains when the k value is less than about 10^{-3}. Generally speaking, for earth dams, the U.S. Bureau of Reclamation classifies soils with k values about 10^{-4} as pervious and soils with k values below 10^{-6} as impervious.

Capillarity is the action by which a liquid (water) rises in a channel above the horizontal plane of the supply of free water. The number and size of the channels in a soil determine its capillarity. This soil property is measured as the distance (ranging from zero to 30 ft or more) moisture will rise above the water table by this action.

Moisture in clay soils may be raised by capillarity for vertical distances as great as 30 ft, considered by the highway engineer to be "high capillarity." However, a long period of time is often required for water to rise the maximum possible distance in clay soils because the channels are very small and frequently interrupted. Silts have high capillarity, but maximum capillary rise occurs in a few days rather than over a long period because the pores are larger. The capillary rise in gravels and coarse sands varies from zero to a maximum of a few inches.

Capillarity of a soil and the elevation of the water table under the pavement determine whether the subgrade will become saturated. Whether or not the subgrade becomes saturated from capillary action (or from condensation, seepage, and the like) determines the bearing value of the soil to a considerable extent. Subgrade wetting by capillarity also determines whether frost heave needs to be considered in design requirements for the subgrade and pavement.

CHAPTER 5

SOIL SURVEYS AND SOIL SAMPLING

Soil surveys are made to obtain necessary information concerning the types and extent of soils that will be encountered on a project. Representative soil samples are taken for analysis. The extent of the survey and sampling work will depend on the size of the project, the character and variation of the soils in the area, and other factors.

SOIL SURVEYS

Experience and a good working knowledge of soils are prerequisites for a satisfactory soil survey. While all engineers are not expected to make soil surveys, detailed procedures will be presented in the following paragraphs to give the inexperienced engineer an understanding of the work required .

A soil survey includes an examination of soils existing over a definite area, a description of these soils, and a location of the limits of extent of the various soils. Soil surveys of airports and roads can be divided into two general types:

1. Surveys of existing roadway or airport-pavement subgrades that are at the present time at proper grades.
2. Soil surveys of new locations where the grade line has been plotted on paper but has not yet been set in the field.

In either case, the first step in making a soil survey is to obtain a general layout map of the project, the grading plans, and the ground-profile plans that were used or that are to be used in construction. The next step is to obtain all available soil maps of the area, particularly the U.S. Department of Agriculture county survey report and map. With the map as a basis, the soil surveyor can check the soil profile over various areas on the project and locate the soils described in the report.

The availability and status of Department of Agriculture soil maps were discussed under "Availability of Soil Maps," Chapter 1.

Two major objectives should be kept in mind when a soil survey is being made:

1. To obtain complete information so that samples representative of each soil type and horizon can be taken into the laboratory and tested.
2. To obtain sufficient information concerning the location of the various soil types and horizons, so that laboratory findings can be properly interpreted and used in design and during construction. This will include the possible use of selective grading to place the best soils in the upper portion of the subgrade.

Existing Roadway and Runway Subgrades

In the soil survey of a graded roadway or runway, the soil surveyor first drives over the project to become acquainted with the topography and general drainage condition. He/she locates a number of cuts, if possible, in which a study of the undisturbed natural soil profile can be made. The soil profile is studied through the A, B, and C horizons—the C horizon to a sufficient depth to include all material that has been excavated for the project. Ordinarily there are marked physical characteristics in the horizons that accompany the soil-forming processes. These characteristics are expressed in the profile by differences in color, texture, structure, and consistency.

These differences are used to establish the boundaries of the A and B horizons, and the thickness of each is measured. Then in the engineer's notes each horizon is described with respect to color, texture, structure, consistency, and depth.

Where distinct minor differences occur within any horizon, they are indicated as subhorizons, designated as shown in Fig. 4.

Notes should also be taken on the character of the topography, the general drainage condition, the types of vegetation, the depth of roots, and the gravel content of the soil throughout the profile. When the soil-survey notes are studied by the laboratory or office engineer, they offer valuable information for checking soil test and design analysis.

After the soil horizons are identified, located, and described, an inspection is made of the upper 10 in. of the subgrade at sufficiently close intervals to locate the position of each change in soil type and horizon, or to locate the limits of fill sections, probably composed of a mixture of the adjacent soil types and horizons. If the soil survey is being made to determine the suitability of the soils for subgrades, it is necessary to investigate the soil condition in greater detail and to a much greater depth than 10 in. (See ASTM D420 and AASHTO T86 for additional information on subgrade soil surveys.)

Locations of New Roadways and Runways

When a survey for a new roadway or runway is being made prior to grading, the soil surveyor studies the survey report on the soil types within the area and the soil map. After marking the centerline of the project on the map, he/she notes the soil types that will be traversed. Then a tentative grade line can be set, thus determining the cut and fill required and the horizons that will occur in the upper portion of the roadway. This preliminary work can best be done in the office. The soil surveyor can then go into the field and follow survey procedures similar to those previously described.

A study is made of exposed soil profiles along roadways or railroad rights-of-way in the immediate vicinity of the project in question. This is supplemented by auger borings along the proposed centerline of the project at sufficiently close intervals and to sufficient depth to locate soil type changes that will occur in the final graded project.

SOIL SAMPLING

Soil samples for complete testing are necessarily large; therefore, the minimum number representing the project is desirable. In selecting large samples, it is sometimes good practice to take small preliminary samples on which to conduct exploratory soil tests that will permit definite identification of certain soil types. These exploratory tests will vary in detail. Usually grain-size, liquid-limit, and plastic-limit tests are made to segregate one soil type from another and to assist in final sampling. The number of small samples taken will vary with the soil surveyor's familiarity with the soils and his/her confidence in identifying them. In many instances, it will be possible to take only one sample of each horizon of each soil series. When complete, these exploratory test data can be analyzed and locations chosen for taking the large soil samples. Soil samples for exploratory identification tests should weigh about 10 to 15 lb.

If the soil surveyor is familiar with the soils, he/she can forego small samples for exploratory tests. By visual inspection the surveyor can choose which of the soils should be taken for complete testing in the laboratory. It may be necessary to take only samples representative of the natural horizons of each soil series as it occurs on the project and to use the information obtained as the basis for designing the roadway. Soil samples for complete soil-cement testing should weigh about 50 to 75 lb.

Soil samples for roadways not yet constructed are taken from the various soil horizons in exposed cuts or by boring with an auger from the surface.

When soils are being sampled, it must be remembered that in the natural profile at a single location there is a greater change in soil character with increase in depth than with increase in longitudinal distance. For instance, the A horizon soil usually is similar over a considerable horizontal area, whereas the B horizon soil, only a short vertical distance below, may be entirely different from the A horizon material at that point. Also, of course, the B horizon soil is usually similar over a considerable horizontal area, whereas it may be entirely different from the underlying C horizon material at any single location. Samples should be taken so that only one horizon is represented by each sample.

From this discussion it is obvious that composite (mixed) samples of soils taken from different depths are not satisfactory

Similarly, it is not good practice to take composite samples of the same soil horizon at different points, since data obtained for composite samples do not apply to any single location and may be very misleading. If the soil is the same throughout the area, one sample at one point will suffice. If soils from the same horizon are slightly different, this fact should be noted.

The need for accurate and scientific sampling cannot be overemphasized. If the samples are not truly representative of the job, testing is a complete waste of time, and the project is jeopardized.

Complete identification should be supplied with each soil sample. This information should include:

1. Date sampled.
2. Name of sampler.
3. Location of project—county or city.
4. Sampling location.
5. Name of builder.
6. Sender's soil number.
7. Number of bags included.
8. Soil series or soil type.
9. Horizon, color, apparent texture.

If the sample is taken from a natural soil profile, depth below ground surface should be given, and if it is taken from a roadway or runway, an estimate of its original location in the natural soil profile should be included. Excellent details of procedures and equipment for making soil surveys and obtaining soil samples are presented in ASTM D420 and AASHTO T86.

SOIL SURVEYS AND SAMPLING FOR SOIL-CEMENT PROJECTS

Since soil-cement* utilizes soils occurring on or near projects, it is necessary to identify and sample each soil type accurately, as discussed above. The samples are tested in the laboratory to determine the minimum amount of cement required to harden them adequately. **

Most soils are suitable for soil-cement construction and can be readily pulverized and mixed with cement and water under a wide range of weather conditions. Some clayey soils, however, are harder to pulverize and generally require more cement for adequate hardening than is required by the more friable soils. Also, construction with these soils is more dependent on weather conditions.

For economy, when the more friable sandy and silty soils are available nearby, they can be borrowed and placed on top of the heavy clay soil. In some cases, selective grading is employed to place the better soils on the surface for processing with cement.

Almost any normally reacting friable material can be used as borrow. While well-graded granular materials make excellent soil-cement, their use will generally not be necessary since lower-cost materials such as dirty sands and gravels, silty or clayey sands can often be found along the roadway or in the vicinity. The use of low-cost borrow materials will reduce the cost of the soil-cement project and conserve the rapidly depleting supply of good granular pit materials. Estimated cement requirements and distance of haul of the borrow material should be included in the survey report.

Soil maps are of immense value in locating borrow materials. Aerial photographs and geology maps will also prove valuable.

Individual horizons are sampled in the borrow area. Various combinations of the horizons can then be made and tested in the laboratory as required. If the borrow material will be removed from a vertical face with a power shovel, a representative mixture of all horizons in the pit will result. A representative sample taken from the full face of the pit will be adequate in such instances.

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* Soil-cement is a mixture of pulverized soil with measured amounts of portland cement and water, compacted to high density. As the cement hydrates, a hard, durable paving material is formed, which is used primarily as a base course for roads, streets, and airports. A bituminous surface is placed on top of the base course to complete the pavement.

** Soil-cement tests are discussed in Soil-Cement Laboratory Handbook, available from Portland Cement Association.

CHAPTER 6

EXAMPLES OF SOIL SURVEYS, TESTS, AND ANALYSES

The following examples show highway engineering application of information given in previous chapters. The first example describes a soil reconnaissance survey for an airport. The second covers a detailed soil survey, sampling, testing, and classification procedure for the same airport. The third example analyzes soil tests in terms of the design and performance of concrete, soil-cement, and granular base pavements.

EXAMPLE 1. SOIL RECONNAISSANCE SURVEY

Assume that an airport is to be built near a small town in northern Illinois. All construction is to be at a new location that will require grading, drainage, and paving. The property has been acquired, and the direction of the winds predominating during the year has been determined. Two runways, NE-SW and NW-SE, intersecting each other at the center at right angles, will be built as the initial improvement. A soil reconnaissance is needed as a preliminary to a detailed soil survey with attendant soil sampling and testing that will give the information needed for detailed pavement designs.

The engineer assigned to soil reconnaissance was given a general plan showing the exact location and boundaries of the airport site. After brief study the engineer went to the local library to locate a soil map and survey report for the county; no reports were available. Inquiry to the county agricultural agent disclosed that a survey and report made by the state agricultural experiment station were on file at the state university. A telephone call to the university library verified this, and arrangements were made to visit the library to copy required information.

Since the construction engineer needed some soil information at once to permit general analysis of probable construction, the soils engineer drove to the airport site to obtain preliminary information that would be used in studying the soil survey report later at the library. While driving to the airport, the soils engineer gave close attention to the lay of the land, to crops in the fields and natural vegetation, and to the appearance of soil exposed in the back slopes of cuts in the roadway.

The soils engineer noticed that the ground was gently rolling near the airport. Corn, oats, clover, alfalfa, and soybeans were common crops. There were small stands of oak and hard maple in corners of fields too irregular for farming. Stopping at a few cuts, the engineer studied the soil profile. The A horizon, about 1 ft thick, followed the ground surface except on the crests of hills where it had been washed away. This black surface layer graded in color down to a yellowish brown layer that could be broken into small fragments, generally angular.

The engineer carefully examined the black surface soil, which could be broken up by the fingers into dust with a little manipulation. When dampened and squeezed in the hand, it formed a cast that could be handled considerably without breaking. However, only thin pats of soil could be formed by pressing between the thumb and forefinger; it would not ribbon out. All these factors indicated the soil to be a silt loam, probably an A-4 AASHTO soil classification. Such a soil would have rapid, high capillary properties and would be susceptible to frost heave if water were available by capillary action. It might drain readily with the water table below the capillary-rise height; it would have good supporting value as a subgrade if above capillary-rise height but poor value if within capillary-rise height.

Lumps from the 1-1/2- to 2-ft layer of yellowish brown B horizon could be crushed against each other in the hand with difficulty, forming generally angular fragments. When the soil was moistened, a 3/8-in.-wide ribbon could be formed by squeezing between thumb and forefinger, but the ribbon would barely sustain its own weight. These factors indicated a clay loam, an A-7 AASHTO soil classification. Such a soil would possess high capillarity and low load-supporting value and would be subject to frost heave in the presence of capillary water.

The C horizon was quite similar to the B horizon except that it was more yellowish in color.

After arrival at the airport site, the engineer found it to be a field of half corn and half oats. From the highest point of ground, a short distance southeast of the center of the tract, the engineer studied the area's general features within sight. The surrounding area was gently rolling, with a small stream about a mile south and another small stream about a mile north. The water table would probably be at least 30 to 40 ft below ground elevation at the low points on the site, and, hence, frost-heave problems would probably be negligible. Later, farmers in the area told the soils engineer that these streams carried water only during spring rains. (These physical field conditions are of the greatest importance; all terrain must be critically inspected for unusual conditions such as springs and bog areas).

In surrounding fields, crops were the same as those in the neighboring country leading to the site. A small grove of oak and hard maple trees could be seen to the northeast.

The soil at the crest of the rise on which the engineer was standing was brownish or yellowish drab, grading down the slope into dark brown or black. This showed that most of the A horizon at the crest had been washed away. The color was determined from moist soil. All features found were similar to those noted in surrounding country and would probably apply at the site.

The engineer noted that roads bordered the site on all sides. These were visited at once. Some cuts existed, and the exposed soil profiles looked like the ones studied previously. The engineer picked up samples of the soil horizons and studied them as before, but more critically. A moist ball of A horizon soil evidenced some grittiness when bitten, indicating the presence of some sand. This was also true of the B and C horizon soils, but to a lesser extent—indicating a higher silt and clay content.

Returning to the high crest, the soils engineer studied general elevations of the surface to estimate probable grading requirements on the site. The field's southwest and northeast corners were level, with a generally wide, flat ridge running from the southeast corner to the northwest corner. In the absence of levels and contour data, it was estimated that a maximum cut in the ridge of about 10 ft would supply sufficient earth to produce a site meeting grade-lime requirements. Hence, the NW-SE runway would be built on the B and C horizon soils in cut, and the NE-SW runway would be built on fill comprised of mixtures of the A, B, and C horizons, with the B and C horizon soils making up a large part of the mixture. The thick layer of A horizon material might be at runway grade for short distances at some locations. Under existing drainage conditions, the B and C horizon soils would make up the most unfavorable subgrades, and design requirements would probably be based on them.

Summing up all observations made during the soil reconnaissance, the soils engineer concluded:

1. Most of the NW-SE runway would be in light cut.
2. Most of the NE-SW runway would be on light fill except through the low ridge, where it would intersect with the NW-SE runway.
3. Most of the thin layer of A horizon silt had been eroded or would be lost in grading operations because it was too thin to warrant salvaging.
4. The subgrades would probably be the B and C horizon A-7 clay loams, with pavement design requirements dictated by these soils. The engineer estimated (referring to Fig. 16) that their average k value would be about 150.

After reporting the above conclusions to the construction engineer, the soils engineer proceeded with the detailed soil survey, sampling, and testing. A level-survey party was sent to the project at once to obtain data for a contour map and related data.

EXAMPLE 2. DETAILED SOIL SURVEY, SAMPLING, AND TESTING

The soils engineer next visited the state university and obtained the soil map and survey report of the area. A sketch was made of the airport area (Fig. 18), with the site boundaries shown by heavy black bands. Detailed study revealed that the area was largely comprised of three soil types, with small areas of two other soil types on the crest of high ground in the center of the site.

The descriptions of the soil types were copied from the soil survey report for use in field and laboratory identification. The

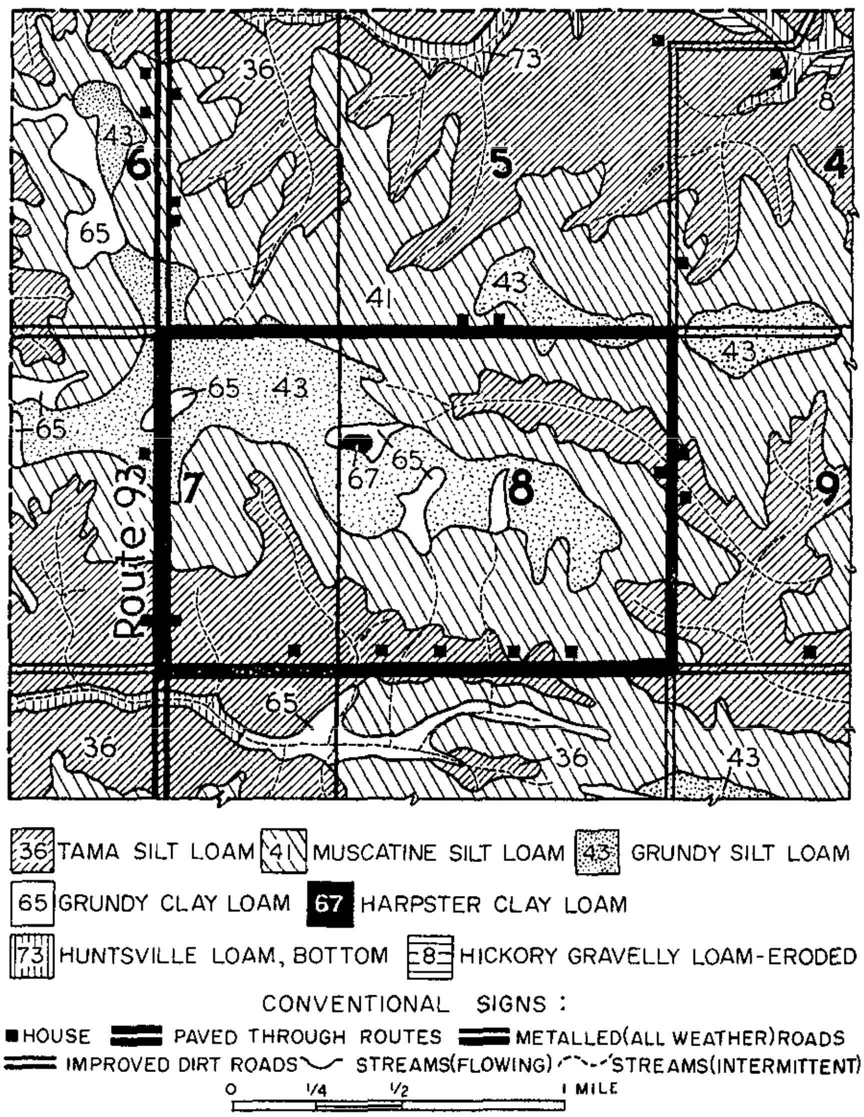

Fig. 18. Detailed soil survey map

following descriptions of two soil types, copied from the report, illustrate the extensive information given in such reports:

"Grundy silt loam (Type 43). A dark soil developed on nearly level topography. It occurs, for the most part, in association with the larger areas of Muscatine silt loam (Type 41). The surface varies from a dark brown, faintly granular silt loam to silty clay loam 8 to 10 in. thick. (This is the A horizon.) The subsurface extends to a depth of 16 or 18 in. and is a little heavier and usually darker than the surface. (This is the B horizon.) The subsoil is a brownish or yellowish-drab clay loam having dark-coated and angular structural particles. The lower part of the subsoil becomes more friable. (These are components of the C horizon.) Surface drainage of this soil type is slow because of its smooth topography, and underdrainage is good where a satisfactory outlet is available. The dark color of the A horizon is indicative of high organic matter."

"Harpster clay loam (Type 67). It is a dark soil that occurs chiefly in depressions in association with Grundy clay loam (Type 65) and Drummer clay loam (Type 152) and is high in organic matter. Many areas are too small to be shown on the map, but they are easy to recognize and should be looked for in clay-loam areas. The surface, 5 to 10 in. thick, is a black clay loam that usually appears somewhat grey when dry because of the large amount of shell fragments present. (This is the A horizon.) Ease of identification is due to shell fragments. Such fragments do not occur in the Grundy clay loam (Type 65) or the Drummer clay loam (Type 152). The latter frequently contains pebbles, which assist in its identification. The subsurface soil, as well as the subsoil, is a greyish-drab clay loam that usually, though not always, contains shell fragments. (These are the B and C horizons.) Lime concretions are nearly always present somewhere in the profile." (This gives a most reliable index for identification since study of the descriptions of other soil types in this area shows that none has shells present or the lime-concretion characteristic.)

The soils engineer was well pleased with the wealth of data obtained from the state agricultural soil survey report. Otherwise, at least a week of hard fieldwork would have been required in making soil borings and studies to duplicate the information obtained from two hours' study of the report.

The engineer then learned that the contour map on 2-ft elevations was available. A copy was obtained and the engineer proceeded to the field equipped with the contour map, the soils map previously copied, and a soil auger and shovel, to make a detailed soil map of the area. The engineer decided to determine and plot first the limits of the small areas of Harpster clay loam and Grundy clay loam on the crest. The area covered by the Grundy silt loam would be determined and plotted as the second

Soil sample and identification	Gradation: Plus No. 10 sieve, percent	Material passing No. 10 sieve: Sand, percent, 2.0-0.05 mm.	Silt, percent, 0.05-0.005 mm.	Clay, percent, 0.005mm.	Material passing No. 200 sieve	Test constants (Material passing No. 40 sieve): LL	PI	SL	FME	Volume change at FME
1. Grundy silt loam, A horizon (silt loam)	0	23	59	18	79	35	12	21	25	6
2. Grundy silt loam, B horizon (silty clay loam)	0	7	66	27	94	40	14	23	29	9
3. Grundy silt loam, C horizon (silty clay)	0	7	61	32	94	44	20	20	29	16
4. Muscatine silt loam, A horizon (silty clay loam)	0	16	63	21	86	34	9	23	26	5
5. Muscatine silt loam, B horizon (silty clay loam)	0	7	66	27	94	55	33	16	32	30
6. Muscatine silt loam, C horizon (silty clay loam)	0	8	67	25	93	53	32	15	31	29
Tama silt loam, A horizon						Not sampled; will be covered with fill.				
Tama silt loam, B horizon						Not sampled; will be covered with fill.				
Tama silt loam, C horizon						Not sampled; will be covered with fill.				
7. Grundy clay loam, A horizon (silty clay loam)	0	5	75	20	96	31	6	26	28	3
8. Grundy clay loam, B horizon (silty clay loam)	0	7	65	28	94	41	14	23	29	9
9. Grundy clay loam, C horizon (silty clay)	0	6	60	33	95	45	20	21	29	16
Harpster clay loam, A horizon				Not sampled; does not occur on runway location.						
Harpster clay loam, B horizon				Not sampled; does not occur on runway location.						
Harpster clay loam, C horizon				Not sampled; does not occur on runway location.						

Table 6. Test results on soils from airport site

Soil No.	AASHO	ASTM (unified)	FAA
1	A-6(9)	OL	E-7
2	A-6(15)	ML-CL	E-7
3	A-7-6(21)	CL	E-7
4	A-4(8)	OL	E-6
5	A-7-6(34)	CH	E-8
6	A-7-6(33)	CH	E-8
7	A-4(6)	OL	E-6
8	A-7-6(15)	ML-CL	E-7
9	A-7-6(22)	CL	E-7

Table 7. Classification of soils from airport site

step. The third step would be the determination and plotting of the area covered by the Tama silt loam. The procedure would locate the area covered by the Muscatine silt loam occurring between the Grundy silt loam and Tama silt loam, except for small areas of Grundy silt loam occurring on the north boundary, which would be surveyed last.

The lime concretions and shells permitted rapid identification and location of the one area of Harpster clay loam soil occurring on the site, and this was plotted accurately on the contour map.

Borings were made in areas of Grundy clay loam and Grundy silt loam, and small samples of each horizon were studied carefully to permit rapid identification on successive borings. With the firsthand knowledge just obtained on the appearance and feel of the soils, the original soil map and descriptions, and the contour map, the soils engineer then determined and plotted the Grundy clay loam pockets. Two test pits were dug 10 ft deep to expose possible subgrade for critical examination.

Next borings were made in the Muscatine silt loam to obtain small samples for careful study and for comparison with the Grundy silt loam, thus permitting ready and rapid identification of these two soils. Again using the original soil map and identifications and the contour map, the engineer determined the Grundy silt loam limits and plotted them accurately on the contour map. Two 10-ft test pits were dug in this soil type also, to permit critical inspection and study.

After all plotting was completed, the field was divided into 1000-ft squares, and a soil sample was lifted and inspected at each corner to uncover any irregularities. Visual inspection of these soil borings, which were about 3 ft deep, was sufficient since the soil survey personnel could readily identify the various soil types due to the preceding detailed work. No irregularities were found.

The soils engineer then took the new soil map to the design engineer for a conference on runway location, amounts of cut and fill, and so forth, to determine the soil types and horizons to sample and test.

At this conference, it became obvious that on this relatively level location, with a difference in elevation of only about 30 ft on the site, the high-ground cut would be sufficient to give the fill required on the NE-SW runway. Since the NW-SE runway would be to the south of the one pocket of Harpster clay loam, that would not need to be sampled.

This review of runway location showed the need for soil samples of the A, B, and C horizons of the Grundy clay loam, Grundy silt loam, and Muscatine silt loam. Samples were lifted accordingly and taken to the laboratory for testing. From these test results (Table 6), the soils were classified (Table 7) according to the details given in Chapter 3.

All the foregoing information was submitted to the design engineer to serve as a basis for design.

EXAMPLE 3.
ANALYSES OF SOIL TESTS

Many times the soils engineer is called on to analyze the value and performance of a soil from laboratory data alone. The following examples will illustrate such analysis of specific test data. The abbreviations for the tests, previously illustrated and given, are used.

PCA Soil No. 3937, AASHTO Group A-1-b(0)

Gradation	**Percent**
Coarse sand (No. 10 to No. 60 sieve)*	71
Fine sand (No. 60 to No. 270 sieve)*	18
Silt (0.05 to 0.002 mm.)*	5
Clay (<0.002 mm.)*	6
Passing No. 10 sieve	100
Passing No. 40 sieve	46
Passing No. 200 sieve	12
Passing No. 270 sieve	11
Physical test constants	
LL	17
PI	(NP nonplastic)
SL	16
FME	17
Volume change at FME	3

The first step is to classify the soil:

The AASHTO soil classification will be found to be A-1-b(0) by referring the above data to Table 3.

The ASTM (Unified) classification will be found to be SW-SM by referring the data to Table 4.

The old FAA classification will be found to be E-1 by referring the data to Table 5.

The next step is to interpret this soil in terms of the general characteristics of the soil group to which it belongs.

The general characteristics of this soil are given in AASHTO M145, the *Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes* (see Chapter 3). Comments on the significance of physical test constants given in Chapter 2, assist in analyzing the soil. Characteristics of SW and SM soils, as defined by the ASTM classification, are given in Table 4.

Discussion of Soil No. 3937

The grain size data show the preponderance of sand-size grains and indicate at once that the characteristics of sands will predominate to produce a good to excellent subgrade. The textural classification is coarse sand as defined by the U.S. Department of Agriculture textural classification.

The LL of 17 is typical of sands and shows little cohesion. This is substantiated by the grain size data, which reveal that the soil contains only 11% silt and clay combined. The lack of PI also indicates little or no cohesion.

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* USDA size limits of soil separates.

It will be noted that the LL, PL, SL, and FME are essentially identical, which shows that the soil has no expansion properties (other than bulking below PL). This coincidence of water-holding properties reveals that this soil, when dry, will readily absorb free water until the voids are filled and that it will drain very rapidly and dry readily.

Since the FME and LL are the same, further indications are given of very low cohesion, and the soil can become "quick" (quicksand) quite easily with upward flow of water.

Subgrade Characteristics

For flexible pavements: This sand will make a good subgrade, as shown by its grain size distribution and low liquid and shrinkage limits. However, since it lacks cohesion, it must be confined to give good supporting value or it will rut readily under traffic. Further, during construction it will be necessary to add binder to a surface layer or to provide tracks so that the trucks can operate over it without bogging down. Also, binder may be required in a surface layer (1) to prevent granular base material from being worked into the sand during construction and (2) to give a suitable stability to the subgrade surface to permit compacting the granular base material to required densities. Once the sand has been confined, it will have good supporting value.

For concrete and soil-cement pavements: This sand will make a good subgrade. On soil-cement construction, if compaction with tamping rollers is not effective, pneumatic-tire rollers may be used, along with some surface ironing utilizing flat-wheel rollers to eliminate marking left by the pneumatic tires.

PCA Soil No. 3977, AASHTO Group A-4(7)

Gradation	Percent
Plus No. 10 sieve size	0
Sand (No. 10 to No. 270 sieve)	5
Silt (0.05 to 0.002 mm)	82
Clay (<0.002 mm)	13
Passing No. 200 sieve size	96
Physical test constants	
LL	31
PI	6
SL	26
FME	28
Volume change at FME	3

The first step is to classify the soil:

The AASHTO soil classification will be found to be A-4(7) by referring the data to Table 3 and Fig. 7.

The ASTM (Unified) classification will be found to be ML, silt, by referring the data to Table 4.

The old FAA classification will be found to be E-6 by referring the data to Table 5.

The next step is to interpret this soil in terms of the general characteristics of the soil group to which it belongs:

The general characteristics of this soil are given in the AASHTO classification of soils, AASHTO M145 (see Chapter 3). Comments on the significance of physical test constants, given in Chapter 2, assist in analyzing the soil. General characteristics of ML soils, as defined by the unified classification, are given in Table 4.

Discussion of Soil No. 3977

The grain size data show the preponderance of silt-size grains and indicate at once that the characteristics of silts will predominate and that the soil will no doubt classify as an A-4 soil. The textural classification is silt loam as defined by the U.S. Department of Agriculture textural classification chart, Fig. 2.

The LL of 31 indicates that there is little clay in the soil or that it is inactive. This is verified by the low PI of 6, which also shows that the soil's cohesion is little more than that of sandy soils.

The low PI also shows that there is a very limited moisture range separating the plastic condition from the liquid condition. When such soils occur where surface or capillary water is available, they will change quickly to a very unstable condition above the LL, particularly when subjected to manipulation or vibration such as from passing wheel loads. The LL of 31 minus the PI of 6 gives 25, which is the PL of the soil. This coincides with the SL of 26.

The SL of 26 shows that the soil can absorb a fair amount of moisture before its volume begins to increase because of absorption of moisture and before it begins to lose its high stability and load-carrying capacity. Also, since the SL and PL are about equal and the PI is 6, the soil will lose stability very rapidly with the addition of only a small amount of water after it reaches the SL.

The FME is indicative of the moisture content that can be readily absorbed from the surface by an exposed soil in its natural, undisturbed condition. This soil with an FME of 28, which is above the PL, will readily become plastic after rains, and with soaking from rain will reach a moisture condition very close to the LL of 31. Hence, only a little manipulation or vibration would be required to carry it over the LL. This high FME also indicates that the silt will be subject to considerable frost heave in the presence of capillary water.

The volume change at FME of 3 indicates high silt content and very little volume change resulting from moisture increases.

Subgrade Characteristics

For flexible pavements. This silt will readily absorb surface moisture and approach the liquid limit. In this condition, flexible base material will be easily driven into the soil under traffic, and the liquid soil will enter any pores in the base material. Hence, the flexible base materials must be protected by an extra layer of well-graded sand, stone chips, or similar material to prevent infiltration of the soil under traffic.

Should the soil be located where a high water table and freezing occur, subbases will be required to aid in compensating for loss of subgrade support during the spring thaw period.

For concrete and soil-cement pavements: The bridging or load-distribution characteristics of concrete and soil-cement reduce pressures on the subgrade to safe limits. As a result, these pavements are not as sensitive to weakening of the subgrade during the spring thaw period. Control is needed, however, to achieve reasonably uniform subgrade conditions. But since infiltration of the soil into the pavement is impossible, special subbase precautions to counteract that possibility are not required.

This soil is subject to pumping on the more heavily traveled main roads of concrete used by heavy truck traffic. A subbase of well-graded granular material or a cement-treated subbase is provided on such heavily traveled roads to blanket the subgrade. Such a subbase will also be adequate, under most climatic conditions, to control problems of frost heave. It is not required on the less traveled roads where concrete and soil-cement are used, since the occasional heavy truck will not create a condition that produces mud-pumping.

PCA Soil No. 3948, AASHTO Group A-7-6(21)

Gradation	Percent
Plus No. 10 sieve size	0
Sand (No. 10 to No. 270 sieve)	7
Silt (0.05 to 0.002 mm)	65
Clay (<0.002 mm)	28
Passing No. 200 sieve size	94
Physical test constants	
LL	44
PI	20
SL	20
FME	29
Volume change at FME	16

The first step is to classify the soil:

The AASHTO soil classification will be found to be A-7-6(21) by referring the above data to Figs. 7 and 8 and Table 3.

The ASTM (Unified) classification will be found to be CL (clay) by referring the data to Table 4.

The old FAA classification will be found to be E-7 by referring the data to Table 5 and Fig. 9.

The next step is to interpret this soil in terms of the general characteristics of the group to which it belongs:

The general characteristics of this soil are given in the AASHTO classification of soils, AASHTO M145 (see Chapter 3). Comments on the significance of physical test constants, given in Chapter 2, assist in analyzing the soil. The general characteristics of CL soils, as defined by the ASTM classification, are in Table 4.

Discussion of Soil No. 3948

The textural classification is silty clay loam as defined by the U.S. Department of Agriculture textural classification.

A-7 soils are elastic and rebound after removal of load or compaction force. They have high volume changes accompanying moisture variations above the SL, and they have low bearing value.

The LL of 44 is in the lower range of this value for clays, which may run as high as 80 or 100. Therefore, this soil belongs to the better clays, although it is still an inferior subgrade soil.

The PI of 20 shows that a considerable increase in moisture content may take place before it changes from a plastic to a liquid condition.

The PL of 24 and the SL of 20 show that only a small amount of moisture need be absorbed to change the load-carrying capacity of the soil from a high value at the SL to a rapidly decreasing value at the PL.

The FME of 29 is higher than the PL of 24, showing that the soil will absorb free surface water sufficiently to exceed the PL, where load-carrying capacity decreases very rapidly.

Subgrade Characteristics

For flexible pavements: Since A-7 soils are elastic and rebound after removal of load, they are difficult to compact. When they serve as a subgrade for a flexible pavement, it is also difficult to compact the granular base course material. Of more importance, after construction, each passing load tends to compact the base and the subgrade, but subsequent rebound tends to loosen and open up the granular base; this permits easy entrance of water and leads to loss of load-carrying capacity. The low load-carrying capacity of A-7 soils requires maximum thickness of granular base materials. A-7 soils also have high volume change with moisture changes.

For concrete and soil-cement pavements: The bridging or load-distribution properties of concrete and soil-cement are valuable engineering properties since pressures transmitted to the subgrade are low. A reasonably uniform subgrade compacted at proper moisture content is needed to minimize differential volume change. Proper design and construction of subgrades and subbases are discussed in *Subgrades and Subbases for Concrete Pavements.**

Protection from pumping—discussed for soil No. 3977—is also necessary for this soil.

SOIL-CEMENT DESIGN AND CONSTRUCTION

The details of soil-cement tests, plus the meaning and explanation of these details, are given in *Soil-Cement Laboratory Handbook.** Similar information on construction is given in *Soil-Cement Construction Handbook** and will not be repeated here. Familiarity with these details is needed to permit a complete understanding of the following comments.

The test data for the three soils given in the preceding pages supply many of the answers to problems of soil-cement testing, design, and construction. Highlights of the analysis of the use of these soils for soil-cement follow.

PCA Soil No. 3937, AASHTO Group A-1-b(0)

From Table 1 in Soil-Cement Laboratory Handbook, it is seen that a cement content of 6% by weight will probably prove adequate.

The high sand content, 89%, shows that the soil will require little pulverizing effort and that mixing of water and cement will be a rapid, efficient operation. The maximum density will be about 120 lb per cu ft and the optimum moisture will be about 11%, using ASTM D558 or AASHTO T134. Air-dry moisture content of a soil of this gradation will probably be about 2%, and for a 6-in. compacted thickness, approximately 6-1/2 gal of water per sq yd will be required.

Soil-cement made of this soil will have excellent quality and strength.

Required densities can be easily obtained with pneumatic-tire and steel-wheel rollers.

PCA Soil No. 3977, AASHTO Group A-4(7)

From Table 1 in Soil-Cement Laboratory Handbook, it is seen that A-4 soils require from 7 to 12% cement for adequate hardening. Since this A-4 contains very little sand or clay, the higher cement content, 12%, should be selected for cement estimates.

This soil will pulverize readily under a wide range of moisture conditions since the silt itself has little or no cohesion and there is little cohesion imparted to the soil by the low clay content. Mixing operations will be easy and rapid. The maximum density will be about 106 lb per cu ft, and the optimum moisture will be about 17%, using ASTM D558 or AASHTO T134. Air-dry moisture content of a soil of this gradation will probably be about 5%, and for a 6-in. compacted thickness, approximately 5 gal of water per sq yd will be required in construction.

Soil-cement made of this soil will have good quality and strength.

The mixture will compact readily with tamping rollers, and it will finish well. Normal attention to the production of 1-in.-thick surface mulch will eliminate surface compaction planes

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* Available from Portland Cement Association.

produced by the sheepsfoot rollers, tractor plates, motor patrol wheels, and so forth. Final rolling with pneumatic-tire and steel-wheel rollers, with the mulch at optimum moisture or slightly above, will produce a tight, even surface.

PCA Soil No. 3948, AASHTO Group A-7-6(21)

From Table 1 in Soil-Cement Laboratory Handbook, it is seen that a cement content of 13% by weight for an A-7 soil will probably prove adequate.

This soil will pulverize above the shrinkage limit and below the plastic limit. As its moisture content decreases below the shrinkage limit, clods tend to form. These can be pulverized by moistening for 24 hours to bring them above the shrinkage limit, or they can be crushed with sheepsfoot rollers.

Mixing operations will be rapid and efficient when the soil is air-dry. Cement should not be added when the percentage of moisture in the soil exceeds the quantity that will permit a uniform, intimate mixture of soil and cement during mixing operations. The pulverized soil can be protected from rains by maintaining good crown and surface grade. This permits rapid runoff of surface water before soil-cement processing.

The maximum density will be about 110 lb per cu ft, and the optimum moisture will be about 16%. Air-dry moisture content of this soil will be about 8%, and for a 6-in. compacted thickness, approximately 5 gal of water per sq yd will be required during construction.

Soil-cement made of this soil will have good quality and strength.

The mixture will compact readily. Normal attention to production of a 1-in. surface mulch will produce a tight, even surface after rolling.

SUMMARY

The foregoing examples of soil reconnaissance, detailed soil survey, sampling, testing, and design analysis for flexible, concrete, and soil-cement paving are offered to show not only the physical steps and work involved, but also the mental processes followed by soils and design engineers in arriving at the required answer. This latter phase is the key to success in soil work because it requires selecting all the soil properties that have specific bearing on the problem at hand. In the interest of brevity, no effort has been made to bring all factors influencing design into the discussion, and several points on drainage, capillarity, frost heave, and so forth, have not been included.

CONCLUSION

This primer has been intended to serve as a starting point for obtaining a working knowledge of soils as they apply to pavement design and construction.

After the substance of this handbook has been absorbed, the engineer can begin talking soil language. By continued study, by discussions of specific points with others, and by fieldwork on specific problems, the engineer will make soil information a useful and essential tool in the adequate and economical design of pavements.

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