DEPARTMENT OF SOILS

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GENERAL ENGINEERING APPROACH TO THE CLASSIFICATION AND IDENTIFICATION OF SOILS

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SYNOPSIS

In *Transactions*, American Society of Civil Engineers, 1923, the writer said (*1*)[1]:

"The classification of soils should be along the lines of strain characteristics rather than those of stress, namely, elastic strain, plasticity and fluidity, for the phenomena are evident and measurable, and embrace within them the results of all the stresses which may act." The stresses acting within a soil mass are caused by force of gravity, gravitation (attraction between the particles), surface tension, adhesion, and molecular attraction, all of which may be considerably varied by the presence of comparatively small amounts of foreign constituents. To permit proper use of a soil classification system, where the results obtained in engineering structures with one soil can be applied to similar problems at other locations, the system must be based on an identification of soils along strain characteristics. A study of all of the proposed classification systems in the past 25 years as reported in the voluminous literature on this phase of soil mechanics, indicates that none permit certain identification of the soil properties for application to general engineering problems. Most of the proposed systems are either entirely or to a great degree dependent upon equality of textural composition. Although they have found good use in specific problems involving soils, such as pavement base design, there is little value in these systems for other engineering problems.

This paper plans to approach the problem from basic considerations and to outline the general classification of soils on the criteria of measurable strains of elastic, plastic and fluid nature. In this way the results of forces acting on a soil are the basis of classification, not what constituents make up the soil. The complete system will also require a study of effects of foreign ingredients, giving the information necessary for artificial control of soils as well as for the determination of maximum deviation in strains under known forces. When these factors can be completely evaluated, the problem of soil classification and identification will be solved. Meanwhile, the paper outlines the gaps in information and suggests research problems to fill such gaps.

All true answers to a question are always implied in the question. For instance, if one asks: "What type of soil is this sample," the answer can only be true, if soils can be classified by type. Similarly, the same is true for color, smell or any other means of classification by the senses or by measured criteria. Soil classification systems have been devised on the basis of average grain size, even though grain shape may be the more important criterion. Similarly, chemical, geological and agricultural heredity have been used. But, when the desired result is to group unlike soils into classes which have like engineering characteristics, the answers are false. That is the crux of the difficulty in applying present soil classification and identification systems to the solution of engineering problems.

The purpose of a soil classification and identification system for engineering control and use of soils is to permit the extrapolation of known physical actions of some soils and thereby prophesy the action of other soils under similar exterior loading, temperature and moisture conditions. The answers desired are physical strains and stresses. The

[1] Italicized figures in parentheses refer to the list of references at the end of the paper.

question must include that possibility. The proper classification must therefore be, not on size, color, taste or feel, but on the basis of numerical physical characteristics.

There is a possibility that physical characteristics may be correlated with other apparent similarities. Just as it is possible that all fat people walk slowly, or that all New Yorkers walk rapidly. But such classifications are not reliable and the attempt to apply them for prophecy ("design" is the word used in engineering) often leads to failure. Timber classification, until recently, was on the basis of species with the assumption that permissible stresses were alike for all wood of the same species. Only a few years ago, this long old practice was discarded for the more proper grading on the basis of stress characteristics. Likewise concrete was classified on the basis of cement percentage with the assumption that, for example, all 1:2:4 mix was graded at the same strength. This method, too, has been replaced by strength specification for the mix proportions. There is no necessity for citing the similar examples of steel stress specifications. There is no proper reason why soils should not also be classified along strength or stress criteria.

It might be best first to list and analyze the probable objections to a stress classification of soils. The most often heard objection is that soils are not homogeneous materials and do not have isotropic properties. In reply, it is to be noted that the theory of elasticity deals only with homogeneous strain, which has a linear relation to the stress causing the strain. And it is assumed, only partially confirmed by test, that all solids comply with such linear relationship and are never stressed beyond the elastic limit. It is also assumed that the removal of stress in solids is accompanied by a simultaneous elimination of the strain. Although it is known that these assumptions are in many cases considerably in error, yet the elastic theory is applied to the design of all solids. Heterogeneous strains (which do not bear a linear relationship to the stress) occur in soils, as also in many solids. But any heterogeneous strain is homogeneous in its smallest parts, so that within fixed limits, the elastic method of design can be applied, using the respective physical properties in each range of limits. Furthermore, it should be recognized that design values of maximum soil stresses are very low, for example 3 tons per sq. ft. equals 42 lb. per sq. in. Laboratory and field tests should be limited to the range of values normally used, and much of the nonconformity to approximate linear relationship will be eliminated.

A second objection is that soil problems often deal with stresses after failure, and motion is not permissible in elasticity. That is not true. Every strain is a movement, and only in perfectly elastic materials is that movement compensated by an equal reversal upon the removal of the force causing the strain. If the force is not removed, the strain persists. Experience with the flow and creep of such elastic materials as the metals, and of concrete, indicates that the existence of movements does not necessarily void the possibility of application of elastic formulas.

The third objection that soils may be of so many different types and classifications falls away when it is noted that the difficulty is in the method of classification. If soils were classified as to their physical characteristics then soils in the same class would act alike physically.

The question then resolves itself into a choice of physical characteristics as a framework or index of classification for engineering designs involving soils. All soil problems can be grouped into cases where:

1. Original soil is being studied for determining its strength to support additional load, (foundations for structures, roads, etc.)
2. Original soil is being operated upon, either by cutting externally (canals, abutments, retaining walls,) or by internal removals (tunnels).
3. Artificial soil is being prepared with desired load or drainage properties, (dams, fills, sub-bases).
4. Original soil is being altered (dewatering, electro—and chemical-osmosis).

All problems may involve soils of physical state varying from almost perfect liquid (river mud), through plastic clays, viscous silty mixtures, and clean-granular sands to rock-like hardpans. Yet, each material has a measurable, although in some cases possibly negligible, fluidity, plasticity and solidity.

It seems, therefore, that it is necessary and also that it will be sufficient, for the purpose of identification and likewise as a system of class fixation to record for the soil in question:

1. Routine data, such as local name and field descriptions by the use of human senses.
2. Weight and composition.
3. Fluidity characteristics.
4. Plasticity characteristics.
5. Solidity characteristics.

ROUTINE DATA AND SENSORY TESTS

The purpose of recording routine data is to provide clues for correlation with local description often used in the literature of foundation experiences. By itself, such identification cannot be used as a means of classification of soils for engineering purposes, since soils of very similar behaviour often have quite different names in various countries, and even in fairly close localities. (For instance, the midwestern "gumbo" and the eastern "bulls' liver" show almost identical physical reactions in construction work.) The local soil name can be often made more descriptive by geological identification of the parent material and the historical formation which resulted in the soil.

The method of pedology as a means of identification, and its limitations, are completely described by Hans F. Winterkorn (*2*). The use of local names as a sole means of classification was apparently common among the Romans, and John Evelyn (*3*) in 1678, translated the list as: "chalk, marle, fullers-earth, sandy, gravelly, stony, rock, shelly, coal or mineral." Many of the more recent attempts at soil classification are merely geological or agricultural names for the same local descriptions.

The color of a soil may be an unalterable local characteristic, but it has no bearing on the physical properties beyond the immediate locality where certain soils may be so identified. In a given locality, color may become a useful tool. For instance, in the foundations for the Northwestern University stadium in Evanston, Illinois, excavations uncovered soils of apparently similar type but of definite varying colors. Load tests were made on each of the different colors and the bearing values for the same settlement were found to be unequal. Schedules of footing sizes were prepared for each value over the range of column loads, and the actual size chosen in the field as the subgrade was exposed, using color of the soil as the sole criterion. However, any attempt to carry the experience at that site over to any other locality would be extremely dangerous.

Color may often be a clue to chemical composition or even to the lack of stability of the soil or of the rock. Rock colors were described by Werner in 1814 as: white, gray, black, blue, green, yellow, red and brown. The U. S. Geological Survey used the Color Standards and Nomenclature developed in 1912 by Robert Ridgway, who listed 1,115 colors. This list was abridged by Goldman and Merwin to 114 colors for field geologists and their charts were published by the National Research Council in 1928. The report of the Rock-Color Chart Committee (*4*) is based on the A. H. Munsell Color notation as a three dimensional classification along the axes:

a. Value as measured by the reflection, from 0 to 10.
b. Hue or color of the spectrum, using 10 hues.
c. Chrome or the inverse of grayness, from 1 to 10.

Sensory tests were often the only ones used in soil classification and identification with similarly acting materials previously encountered. The great number of successful foundations designed and constructed by the engineers of only one generation ago is proof of the sufficiency of such methods. However, it should also be noted that it required many years of practice in a rather restricted area for the development of an expert foundation engineer. The geographically widening field of work and the increase in congestion and in size of modern construction do not permit continuation of such training method. The sensory testing of soils followed the method recommended both by John Evelyn (*3*) and by Sir Humphrey Davy (*5*), the latter also advising that a soil is to be classified by how it acts and not on what it contains. (A summary of these early soil classification recommendations appears in the writer's discussion of Arthur Casagrande's paper on "Classification and Identification of Soils" (*10*)).

Among the more recent papers on sensory testing methods, is P. C. Rutledge's "Description and Identification of Soil Types," (*6*), in which are listed six simple field tests by manual and visual observation, and correlation between soil names in common use and the results of the recommended tests.

WEIGHT AND COMPOSITION

The weight and composition of the soil should always be determined and recorded, not only for the help such information may give in identification, but usually because the physical properties are functions of these values. They are a description of what a soil contains, and the desired goal is to express these constituents numerically. The weight of the soil in the natural state, the percentages of solid matter and of removable water are comparatively simple items to determine for grains retained on the 200 mesh screen. The presence of absorbed water or occluded gels may cause some error in the values, but not of serious magnitude, if the soils contain minor amounts of the finer materials. When such is not the case, weight and composition of the soils are insufficient data for identification and often lead to wholly erroneous conclusions of the expected physical action.

The weight of soil varies with many factors, and comparatively little is known of the variation in situ, with depth. Artificial classification by standardized tests (Proctor, modified Proctor, AASHO, etc.) provide some data as a guide. But what is really required, is the variation in density with changes of climate, seasonal and in some cases even daily, effect from soil moisture movement, considering both the introduction of foreign dissolved materials in the soil water and the residue when the soil moisture evaporates, and variation with age for artificially prepared soil bodies. Some work on this phase of the problem was recently reported by Hans F. Winterkorn (*7*).

The separation of a soil into grain sizes by sieving is an extrapolation of a procedure used in concrete technology. That certain combinations of grain size percentages are better than others in the aggregates used for a skeleton to be bound by cement or bituminous binders is a well established fact. However, identical fractional combinations of grains used as aggregates do not necessarily give identical concrete strengths in compression, in tension and in shear. Similarly, identity of mechanical analysis of soils does not necessarily imply identity of physical reaction to like forces under like conditions, and almost always there will be unlike variation in strain characteristics under the same variations in exterior conditions. For example, identical gradation of apparently clean sands from Wading River and Port Jefferson, both on the north shore of Long Island, reacted entirely differently in railroad fills. The former material contains a thin coating of silicic acid, causing a very rapid solidification of loosely placed fills; while the latter remains perfectly granular in response to external loads and to vibration.

Typical of the size range classification systems is the report of the Sub-Committee on Sediment Terminology of the American Geographical Union (*8*), in which all materials from 160-in. boulders to the very fine clay of 0.24 microns are grouped into 24 named soils. D. M. Burmister has published recommendations to use the shape of gradation curve, plotted on logarithmic paper, together with the fineness of the soil and dispersion of the grain size, as criteria for classification.

The development of the several modifications of the Bureau of Public Roads grouping of materials for subgrade and granular subbases, first by C. A. Hogentogler, later in collaboration with E. S. Barber, moved from a purely textural basis to one in which physical action as determined by tests was also a criterion. This classification has found considerable use in the highway and airport field, but a more accurate application of the results would be possible, if numerical values of elastic compression, plastic flow and fluidity were used as the identifying symbols, rather than the grouping names.

In 1945, the Highway Research Board Committee on Classification of Materials for Subgrades and Granular Type Roads (Harold Allen, Chairman) reported the three methods for the classification of subgrade materials for highways and airfields (*9*). The highway group method was a further modification of the Hogentogler-Barber classification and required the determination of the mechanical analysis of the materials and also, for the fraction passing the No. 40 sieve, the liquid limit and the plasticity index. The airport use groups recommended by the Corps of Engineers were identified in the case of coarse grained soils by gradation, grain shape and dry strength, and in the case of fine grained soils by dry strength and examination in the plastic range. The airport use grouping presented by the CAA was based on mechanical analysis; on liquid limit, plasticity index and volum change, for the fraction passing the No. 4 sieve; capillary rise on the fraction passing the

No. 10 sieve; and California Bearing Test for the granular soils when saturated. No claim was made for any of the proposed classification methods that the application was beyond the designated use of determining suitability for subgrades.

The paper presented by Arthur Casagrande (*10*) to the American Society of Civil Engineers, on "Classification and Identification of Soils" reviews the existing soil classification methods and describes in detail the U. S. Engineer Department airfield soil grouping, similar to the second of the three methods in the 1945 report of the Allen Committee. Casagrande develops in some detail the range of known correlation between the grouping as recommended and such physical properties as compressibility, permeability, rate of volume change, toughness near plastic limit and dry strength.

There seems to be no complete solution to the problem in sight, if the methods of classification are based chiefly on weight and composition, even with the addition of the usual plasticity investigations. An analogy can be drawn with the classification of metals as now used in industry. If an attempt were made to classify by percentage content of the various elements, no physical properties could be deduced unless the combination of those elements in the same percentage ratios was tested under desired exterior conditions of temperature, time of loading, etc. for the property in question. Likewise, to classify metals into groups without consideration of the minor constituents which in some instances have so large a control on physical properties, would result in misleading information. And yet again, the relative location of a small minor constituent can affect many properties. For example, a very small percentage of some elements on the surface of iron will prevent corrosion, while other elements will increase the surface resistance to abrasion, and in neither of these cases have the basic strengths of the body been altered.

Similarly in soils, a small percentage of colloid material covering the larger grains may have a decided influence on the physical properties, and the type and kind of colloid is also an important factor, and yet many times that amount of the same ingredient as a coagulated mass (balled clay for instance) may have no affect on the general physical properties of the soil in question.

The answers can only be determined by asking the proper questions, and in the case of soils that is equivalent to field or laboratory testing, upon natural or undisturbed samples where soil action in natural state is being studied, for unique and characteristic strains. Eventually, certain soil constituents will become ear-marked as necessary for the existence of a characteristic strain condition, but only after considerable development and accumulation of data. It will be much simpler to correlate the expected strain and stress distribution within a soil mass when the characteristic strains are known, especially so when prototype structures are proof of sufficiency of design with soils of similar characteristic strains. Such characteristic strains for all soils must fall into groups of fluid, plastic and solid reactions to external forces.

FLUIDITY CHARACTERISTICS

Example of soils behaving like liquids are often encountered and result in many internal soil failures because the action is not expected. A few common problems in which fluid character of soils must be taken into consideration are: internal lateral pressures in hydraulic fills, lateral pressure of some clay fills when suddenly saturated, loss of internal resistance of soil-water combinations in which a decrease of volume makes the combination a liquid suspension (phenomenon of pore pressure), and the condition known as quicksand. From the engineering point of view, sufficient data are only necessary to classify soils as liquid within definite limits of external conditions.

A perfect fluid can be defined by its characteristic that the internal stress at any depth of the body in any direction is equal to the weight of the material above that depth. In soil mechanics language, the lateral pressure always equals the lateral resistance to pressure and both are equal to the vertical weight of the liquid from the free surface to the depth considered. The unit weight of the fluid remains constant under all surcharged pressures, except as the volume varies with thermal changes. The pressure at any depth is constant no matter what the extent of the fluid volume, and such pressure is exhibited instantaneously and continuously, and is accompanied by no change in volume. An infinitesimal force will start motion in a liquid and such motion continues as long as the force exists.

Water is not the only fluid and water does not always act as a fluid. If the existence of the fluid state is recognized, under limiting conditions, the stress distribution in any problem can be easily determined. The possibility of the limiting conditions must be studied and in some problems can be controlled so that the fluid state cannot exist. The necessary tests for the determination of when a soil sample becomes fluid, using the characteristic that no volume change occurs under pressure addition, can be made with presently available equipment.

Soils in fluid state will always have considerable viscosity, varying with temperatures and amount of dissolved and dispersed matter. Coulomb in 1801 first measured the coefficient of viscosity by rotating a horizontal disk in the fluid. The coefficient of viscosity is the reciprocal of the coefficient of fluidity, and is defined as the force necessary to move a unit area at unit velocity when separated from a similar surface by a unit thickness of fluid material.

The viscosity of the liquid soil is only pertinent when the problem concerns the relative motion of soil and structure. Examples of such problems are the flow of the soil through an orifice, hydraulic fill movement through pipes, movement of a pile or caisson through soil, relative movement of soil surfaces in slides and internal slips and any soil problem in which viscous flow occurs.

Ek-Khoo Tan *(11)* proved that in granular soil masses failure occurs along surfaces where the soil grains are so dispersed in the air and water void medium, that it acts as a viscous material. In the analysis of "Jacked-in-Place Pipe Drainage" *(12)* and in a paper presented before the Rotterdam Soil Mechanics Conference on "Soil Resistance to Moving Pipes and Shafts," *(13)* the writer concludes from all the evidence collected that the soil in the immediate vicinity of the moving surfaces is in the viscous state. Skempton and Golder *(14)* also indicate that along the slip surfaces in stability problems the internal friction is zero and the value of internal resistance during failure is independent of depth. The above examples are cited to show how often the viscous nature of soils must be recognized in engineering problems.

The laws of friction of viscous flow are rather simple: independence of the loads applied, proportionality with relative velocity and zero when the velocity is zero, proportionality to area in contact. Mobility of the soil cannot occur until the particles are so separated by the liquid medium that the solid particles do not touch during the motion. Mobility is considerably influenced by temperature. Bingham *(15)* in 1922, showed the effect on a 50 percent clay suspension. The ratio of mobilities at 40 deg. and 25 deg. C is $\frac{7.88}{5.11}$ or 1.54. Incidentally, the ratios of fluidity of water for the same temperatures is $\frac{166.9}{111.7}$ or 1.49, substantially the same as for the clay suspension.

Identification of the liquid condition in nature is quite simple—one has only to step on the soil to recognize the condition. To determine whether a soil can become liquid under varying external factors, laboratory tests of a specimen, not necessarily an undisturbed sample, should be carried out to fix the minimum moisture content, with and without remoulding, necessary to change the soil into a liquid under a reasonable range of pressures. The classification is therefore also simple: the soil becomes liquid under certain pressure and fluid filled void content combinations. It should be noted that air and other gases are permissible ingredients of the fluid which separates the solid grains. Within the liquid range, soil problems resolve themselves into either a determination of how to prevent the condition or else providing the necessary structural resistances to the liquid pressure.

PLASTICITY CHARACTERISTICS

Examples of soils behaving like plastic materials are well known, as is evident in many reports on clays. Engineering problems in plastic soils are common and if the plastic deformations are expected, proper design precautions can be provided, as for example, in consolidation and settlement of fills, settlement of foundations, plastic creep of soils into tunnel excavations both at the heading and in the invert, and failure of piles in certain soils. Many of these problems seem complicated and indeterminate because the inequality of strains under the same loading is not taken into consideration, especially where only part of the soil volume is in the plastic state. When the strain:time relationships for component parts of a structure are not the same, distress must be expected, and often failure results.

The characteristic of the plastic state is that a definite shearing force will start a deformation which continues indefinitely. A shearing force of lesser amount will not start such deformation. The removal of the shearing force after deformation starts will not cause the deformation to stop. Bingham (*15*) defines plasticity as a condition of friction and mobility acting simultaneously. (When the internal friction is zero, the material is in the viscous state.) In the plastic state, the solid particles touch each other and the voids are filled with liquid, gas or amorphous solid, permitting a flow which is the result of the solid particles sliding over each other.

In addition to this concept of plastic resistance in the direction of the surfaces of sliding, there is also a tackiness or adhesion which acts normal to that surface. The values of such normal forces are measured by the tensile strength of the soil. In this state of matter, the characters of the solid particles have little effect, as long as the non-solid medium in which they float is continuous. Such non-solid matter is best studied in the light of its colloidal properties. In this connection, it must be noted that all solids tend to adsorb or condense upon their surfaces any gas or liquid with which they come into contact, and that a change in this adsorption property can be arranged by chemical, electrical or thermal treatment. Therefore the specific surface in many soils is more important than specific gravity or size gradation as an indication of probable action.

With all other factors constant, pressure sufficient to cause solid particles to intercog will convert the plastic soil into one having solidity characteristics; release of pressure sufficient to eliminate the contact of the particles (or internal pore pressure within the liquid medium causing the same change) will convert the soil into a viscous mass.

Much progress has been made in the determination of plastic strains and correlation with various external conditions. From the physical-chemical point of view, the work of Hans F. Winterkorn (*16*) shows that the study of soil and water film reactions can explain some soil properties and also provide methods for the control of properties, for example, the tensile strength of soil mixtures will be greater than that of either component when they are oppositely charged. The surface area of the soil particles can be accurately determined by gas absorption techniques (*17*) and should be considered as a possible major factor in correlation for similar physical action.

The use of the plasticity index, based on tests on the soil fraction passing the No. 40 sieve, for classifying soil action, must be restricted only to those soils having sufficient percentage content of that sized fraction so that the larger grains are merely fillers and not in contact. Several fairly constant relationships have been determined which may form the basis of identification criteria.

Skempton (*18*) indicates that the relationships between

(a) liquid limit and percentage of minus 2 micron sized particles and

(b) plasticity index and liquid limit, are both linear.

This agrees with A. Casagrande's formula (*10*) for the plasticity index in terms of the liquid limit, and the equation or graph of the linear curve could well become the criterion of plastic material classification.

Another likely approach is the curve of major principal stress against density, or against void ratio at failure. P. C. Rutledge has shown that both of these curves are of single types for clays. The strength test in simple compression of a test specimen at natural water content and then again at successive increases in density as the specimen is allowed to dry out, gives the former curve quite readily.

Identification of plastic materials should be by the existence of plasticity in the whole soil, not in a fraction separated by mechanical or other means. Classification of the soils for physical action seems to be most fruitful from the numerical constants of the strength:volume change relationship, using either density or void ratio change as the measure of volume change. Control of property possibilities are dependent upon the specific surface (total surface area per unit mass) of the solid fraction and the colloidal properties of the non-solid medium of the soil and both can be identified and used as further classification criteria.

SOLIDITY CHARACTERISTICS

Solidity character of soils is not to be confused with elasticity, which latter is a particular characteristic only approximated within definite stress limits in all solid materials. Most of the stress problems in foundation engineering deal with soils beyond the stress limits within which the property of elasticity is exhibited.

Soils are in the solid state when the individual grains are interlocked (Coulomb described it as intercogged) and the void spaces are filled with gases, liquids, amorphous solids or mixtures thereof. Within all compressive stress limits of the problem, the interlocking continues to exist, even though some of the points and edges may be crushed. Shear resistance results from the summation of the physical shearing of grains, the gravitational pull when grains are caused to rise over other grains as they relatively move past each other plus adhesions and cohesions of the fluid and colloidal filling in the void space. Compressive resistance results from the strain in the solid particles, the void spaces being sufficiently vacant or containing compressible gases to permit the necessary linear and volume reductions of the mass.

Statics is the study of strains (motions) and their relation to the stresses causing the strains, and is a special case of dynamics. In the theory of elasticity, assumption is made of homogeneous strains, which result in no fractures or internal separations. In purely elastic bodies, a strain resulting from compression, tension or shear, disappears entirely when the stress is removed. The work done is ultimately zero because the rebound neutralizes the work of temporary deformation.

Even in materials which are normally classed as elastic, a part of the deformation remains, and some work is done to overcome internal friction. Such strain is called heterogeneous, and the application of the theory of elasticity is only justified because each heterogeneous strain is homogeneous in its smallest parts. However, the accumulation of the residual unbalanced strains becomes evident as a settlement, consolidation, expansion or displacement. With these ideas in mind, the usual methods of design employed for structural solids can be applied to soil problems. At all times, the point of change to plastic or fluid condition must be watched, and the corresponding laws will then apply.

The laws of direct stress in solid soils are simple—strain and stress bear linear relationship within rather narrow limits, and the slope of relationship will usually differ in each range. Maximum tensile strain limits the greatest tensile stress; beyond that point rupture occurs and the soil mass is no longer a continuous structural unit. Maximum compressive strain occurs at the boundary between the solid and plastic state; failure will not occur, but the strain will then continue without a further change in stress.

Shear resistance follows the laws first set by Coulomb, namely—direct variation with the stress existing normal to the surface of shear, independent of the area of the surface of shear, and maximum just before motion starts. The ratio of shear resistance to load acting normal to the surface of shear is called the coefficient of internal resistance which consists of a cohesion plus the tangent of the angle of internal friction. It is usually assumed that the angle of internal friction for any material is a constant, independent of the materials within the voids; however, the value of the cohesion is dependent upon temperature, chemical and physical changes of the pore filling.

Internal deformation by shearing takes place along two sets of parallel planes, which do not originate simultaneously and are not necessarily uniformly distributed. The angle between the sets of planes is constant for any material and is independent of the nature or intensity of stress. This angle differs the more from a right angle, the harder and more brittle the material. Karman in 1911 showed that the angle of shear increases when side pressures are simultaneously applied with the load, i. e. radial pressure makes a material less brittle. If tension is taken as minus compression, Hartmann states the law that the acute angle formed by shear planes is bisected by the axis of maximum compression and the obtuse angle by the axis of minimum compression. Mohr connects the angle of shear with the ultimate values of maximum and minimum stress by the relation that the cosine of the angle of shear equals the ratio of the difference in the stresses to the sum of the stresses. In the application of these principles, local overstress may be safely disregarded if adjacent areas show possibilities of taking the extra stresses, the structure however to be designed to safely transfer either distribution of loads.

Many field observations corroborate the existence of solidity characteristics in soils. Hogentogler (*19*) states that in the Bates Road tests the soils below the test pavements were found to act as homogeneous solids within the limit of 6 lb. per sq. in. loading. According to Middlebrooks, the Waterways Experiment Station has shown a definite relationship between shearing strength and density of sands, and he also finds that the elastic theory method is adaptable to clay foundation design since it guards against local interior failures.

Density of soil in place cannot be determined except possibly by electrical or thermal conductivity measurements after the soil has been calibrated for relationship between density and conductivity. However, such simple methods as Hvorslev's piston sampler permit very close determinations of weight per unit volume in samples removed at various depths. Modifications of soil density, in artificial soil deposits, are studied by means of curves of density: moisture percentage on dry basis, using usually "modified AASHO" standard for compacting the test specimen. The maximum possible density, for the compaction employed, is then determined. If maximum strength of soils and maximum density were dependent upon the same percentage of void filling matter, then a great deal of testing for soil classification could be eliminated. However, it is generally found that "maximum density" does not occur at the same void content as does maximum compressive strength or maximum toughness of soil.

The compressive strength of soil has no meaning, nor any possible application in engineering design, unless the strain at any loading concentration is also known. For that reason, several recently suggested "consistency classifications," defining: very soft, firm or medium, stiff and hard, etc. in terms of arbitrary limits of unconfined compressive strength at failure, can have little use in design. A "stiff" soil which will fail at 2 tons per sq. ft. at a strain of 0.04 ft. bears no relation to a "very stiff" soil which will fail at 3 tons per sq. ft. at a strain of 0.25 ft. There is no reason why the "consistency" classification is not a definite numerical value of maximum strain at maximum strength, and for more important operations, a curve showing the strain: stress relationship. Hveem's stability test procedure can be used to determine lateral deformation as well as linear vertical deformation for confined and unconfined compression. Hvorslev's compression test apparatus permits a field determination rapidly. The Dutch method of driving a 60-deg. cone plunger in the bottom of the soil exploration casing, by static loads, determines a compression resistance of the soil at varying depths. A modification of the procedure, employing flat disk in lieu of the cone plunger should give compressive strengths of soils in situ, at definite strain values, with expenditures comparable to the cost of obtaining "undisturbed" soil samples.

For artificial soils, the Proctor needle and the California Bearing Ratio tests can measure relative resistance under various densities and moisture contents. Correlation of the CBR of soil at maximum density under optimum moisture and at full saturation with field observations of strength under use has been proved possible in definite locations. Variations in climatic conditions must be considered before such results are used elsewhere. Likewise, the resistance of the sampling spoon to penetration at the bottom of the boring casing is being relied upon as a measure of soil bearing value. There seems to be no foundation to such reliance, except possibly for the clean granular soils, which require no such assumptions since their physical characteristics are well known. Any attempt to determine static stress: strain relationships by dynamic methods must be carefully scrutinized.

A recent paper on the compression stress: strain relation by H. Straub is summarized (*20*) in *Highway Research Abstracts*. In a study of settlements under loadings carried up to the point of shear failure, the ratio is determined to be a function of the state of densification, of the pressure conditions and of the shape and roughness of the grains. Under controlled, arbitrarily specified conditions, the stress: strain ratio can be used as a classification index for loading resistance, just as it is used in the static problems involving all other materials in the solid state.

Determination of soil shear strength in the laboratory has been studied at great length and methods are now closely specified. Correlation of shear angles as measured by the failure lines, in the field, with limiting internal stress values is fairly simple procedure, and can receive further study and standardization. Determination of soil shear in situ, for fairly homogeneous soils is possible with the imbedded rotating surface device of Carlson (*21*).

In glacial or detritus soil deposits, local hard spots in the soil will give misleading results, but in such soils only the minimum test values should be considered. Laboratory tests on such soils are influenced by the same difficulty of getting average samples of non-uniform mixtures.

SUMMARY

This paper is not intended as a presentation of a complete solution of the soil classification and identification problem. Textural classifications, just as color and other classification will remain in use and serve well in their limited fields of application. The ever broadening complexity of combination systems, embodying textural, geological, mineralogical, physical, chemical and strain criteria of stress has caused several authors to question the present method of approach. For example, R. R. Proctor (*22*) recommends a complete break with present methods and limits identification to a few physical test values. The conclusion proposed by E. de Beer, reporter on Section XII of the Rotterdam Conference, concerning the subject of classification of soils, is quite pertinent: "in communicating the experience obtained in a certain case, not only the name of the soil should be given, but also the figures concerning all the properties involved in the experience." The necessary and sufficient properties to be described by figures or by graphs are outlined in this paper and the study is presented as a skeleton for a complete soil identification and classification system for the field of stress determination and engineering control of the materials known as soils.

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