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The relevance of the yield shear strength of plastic clays in the bearing capacity of foundations

Importance de la charge limite au cisaillement a la résistance des argiles plastiques dans la capacité portante des fondations

Jaime Graterol M.
Geotecnica de Venezuela, C.A. Caracas-Venezuela

ABSTRACT
A comparative analysis is made, of the bearing capacity of foundations using the undrained shear strength c_u, $\phi_u = 0$ vs. using the yield shear strength, Sc for saturated normally and over consolidated plastic clays. I bring up the yield shear strength concept in this kind of soil, following the criteria that cohesive clay is a plastic solid and could be expected to exhibit the basic properties of such a material. Finally a criteria is formulated for determination of the bearing capacity of foundations based on the yield shear strength in this kind of soil to keep static equilibrium without experimenting any progressive settlements.

RÉSUMÉ
Une analyse comparative sur la capacite des fondations utilisant le non draine resistance au cisaillement c_u, $\phi = 0$ vs, a ete faite, utilisant la charge limite au cisaillement a la resistance, Sc pour des argiles plastiques satures normalement, et sur consolide et on se base ici sur le concept de la charge limite au cisaillement a la resistance dans ce type de terrain, suivant le critere qui stipule que l´argile cohesif est un plastique solide, dont on s´attendait a ce qu´il montre des caracteristiques de base, d´une telle matiere.
Finalement un critere a ete formule pour determiner la capacite des fondations, base sur la charge limite et la resistance dans ce type de terrain, afin de garder un statique equilibre, sans experimenter d´autres tassements progressifs.

1 INTRODUCTION

35 years ago when I began my first design of earth works and foundations in plastic clays, among all other investigation, I had the opportunity to read the extensive work on shear Resistance of Plastic Clays, it's application in foundation engineering and field observations developed by W.S. Housel, University of Michigan.

2 BASIC CONCEPTS IN HOUSEL´S WORK

2.1 *Shearing Resistance Due to Cohesion*

Shearing resistance due to cohesion or cohesion is that property of soil which provides finite static resistance to tangential displacement through mutual attraction between particles of the mass, characteristic of microscopic and sub-microscopic matter. Shearing resistance due to cohesion is independent of applied normal pressure, a relationship inherent in any material capable of sustaining a permanent constant difference in principal stresses.

2.2 *The Ring Shear Test*

Accepting the definition of cohesion as being independent of normal pressure, the ring shear test procedure was set up to measure the transverse shearing resistance at zero normal pressure. Setting up the test procedure with definitive control of the other factors to be measured, that cohesive clay is a plastic solid and could be expected to exhibit the basic properties of such a material, in Fig. 1 is illustrated the relationship between shearing stress and rate of shearing deformation, in accordance with the long accepted definition of a plastic solid.

With normal pressure eliminated as a variable in test procedure, there remain three variables to be measured: time, shearing stress, and rate of shearing displacement. It follows that a valid relation between the two variables, shearing stress and rate of shearing displacement, can only be obtained by holding the third variable, time, constant.

Typical results from such a transverse shear test are shown in Fig. 2. Figure 2(a) shows a series of time-deformation curves for the selected load increments. The rate of deformation or terminal slopes of the time-deformation curves are then plotted against the respective shearing stresses, defining the two stages of behavior: the first, in which the plotted points represent substantially elastic deformation, and the second, representing the stage of plastic flow, with the rate of deformation directly proportional to the shearing stress in excess of the yield value. This yield value is then determined as the intersection of the two straight lines and represents the static or permanent shearing resistance of the soil, Sc.

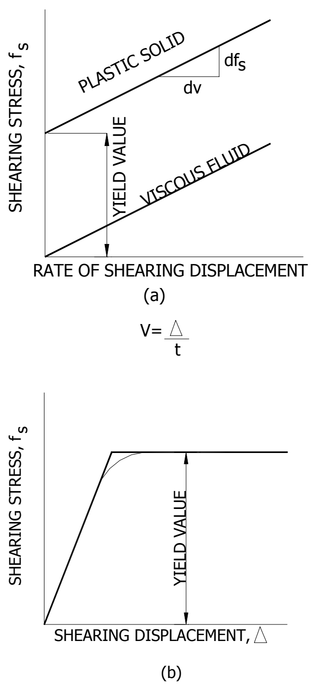

Fig. 1. PROPERTIES OF PLASTIC SOLIDS

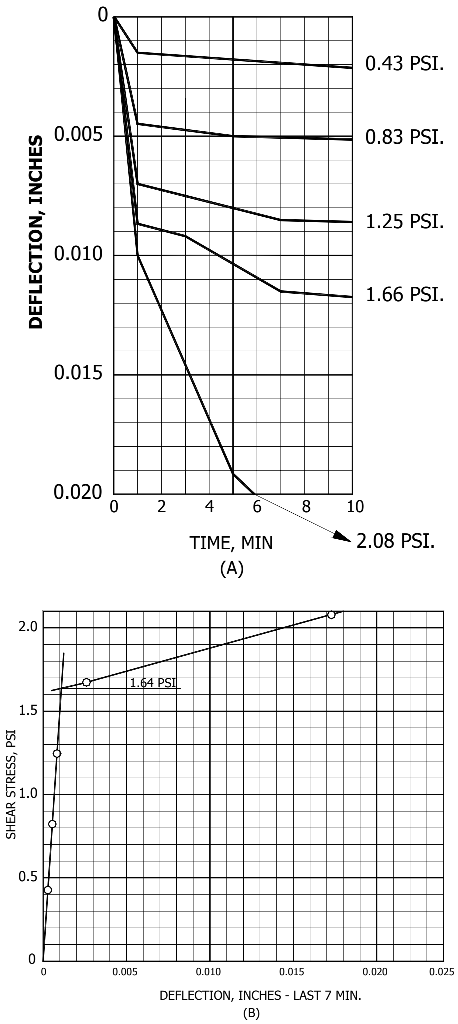

Fig.2 TYPICAL RESULTS FROM TRANSVERSE SHEAR TEST

3 THE UNDRAINED SHEAR STRENGHT ON SATURATED COHESIVE PLASTIC SOILS

These test are carried out on undisturbed samples of clay, as a measure of the existing strength of natural strata, and on remoulded samples when measuring sensitivity or carrying out model test in the laboratory.

The compression strength (i.e. the deviator stress at failure) is found to be independent of the cell pressures.

If the shear strength is expressed as a function of total normal stress by Coulomb's empirical law:

$$\tau_f = c_u + \sigma \tan \phi_u$$

where in terms of total stress:
c_u = denotes apparent cohesion.
ϕ_u =denotes angle of shearing resistance
it follows that, in this particular case,

$$\phi_u = 0$$

$$c_u = 1/2\ (\sigma_1 - \sigma_3)_f$$

The shear strength of the soil, expressed as the apparent cohesion, is used in a stability analysis carried out in terms of total stress, which, for this type of soil, is know as the $\phi_u = 0$ analysis (Skempton, 1.948). Since the value of cu may be obtained directly from the unconfined compression test (where $\sigma_3 = 0$), and from the vane test in the field, it is a simple and economical test, but is often used without regard to the class of stability problem under consideration.

Terzaghi and Peck, both of whom participated in the 1942 Symposium on Earth Pressure and Shearing Resistance of Plastic Clay, used the shearing resistance from unconfined compression test in their investigations which were reported at that time. They had adopted and it has become more or less accepted practice to conduct the unconfined compression test in a 5 min period with load applied to the point of shearing failure or 20 per cent vertical deformation in that period of time. The use of a 5-min time period apparently goes back to the following statement by Terzaghi.

"By loading a great number of nonconfined seamless tube samples (3 1/2 in. long, 1 7/8 in. in diameter) to the point of failure within a time ranging between 2 and 20 min, it was found that, within this range, the time factor is immaterial. Therefore it was decided to run the tests within the shortest time compatible with satisfactory technique. This time was 5-min"

Housel has always referred to this unconfined compression test as a rapid shear test and one which produces a shear value known as the ultimate shearing resistance which, for cohesive clays, has a value of approximately four times the yield value from the ring shear test. These tests have been run in parallel in the University of Michigan Soil Mechanics Laboratory from 1942 to 1.958, some 25,000 comparative test have been conducted. Comparative results in considerable detail were reported in 1956 and the author has run these test from 1974 to the present time 2004 both in terms of individual tests and job averages. The 4:1 ratio first found by Housel was called to the attention of research workers in soil mechanics many times.

A review of current literature indicates that many research workers today quite clearly recognize that rapid rates of loading involve dynamic or temporary resistance, which should be eliminated in arriving at a reliable shear value to be used for design of permanent structures.

Geuze, general reporter at the Third International Conference on Soil Mechanics and Foundation Engineering in 1953, stated as follows, with respect to dynamic resistance encountered in rapid shear test:

"The rate of deformation at increasing shear stresses may have considerable effects on strength… . Results of tests in term of ultimate strength only….. are of little value since design and foundation engineering should be based on permissible stresses derived from the ratio between "stress-deformation-rate of deformation"…. Obtained from test-results".

4 RELATION BETWEEN OVERLOAD RATIOS AND SAFETY FACTORS

Recognition that plastic clays do have a definite yield value that can be reliably measured in accordance with the fundamental concept of plastic solids provides the key to a reliable frame of reference by which the results of laboratory shear tests can be translated into foundation behavior in the field. In fig. 3 the overload ratio based on the yield value is compared with the factor of safety based on the ultimate shearing resistance for the ratio between these two shear values of 1 to 4. In terms of foundation behavior, the significant ranges of shearing resistance have been outlined on the right-hand margin of fig. 3. The limit of static equilibrium is at an overload ratio of 1 or a factory of safety of 4. Progressive displacement is represent by overload ratios ranging from 1 to 4, with equivalent safety factors being the reciprocal of the overload ratio referred to the numerical ratio of 4 or vice versa. Failure or collapse would be represented by overload ratios greater than 4 and safety factors less than 1.

Housel has suggested that for temporary loading conditions such as excavations during the period of construction overload ratios as high as 2.0 or 2.5 may be employed without serious danger of slides. In addition there are other conditions frequently encountered in practice where considerable settlement

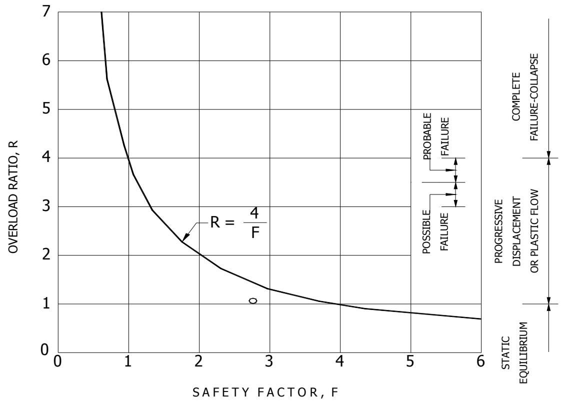

Fig. 3 RELATION BETWEEN OVERLOAD RATIO AND SAFETY FACTOR

may be permitted and where overload ratios as high as 2.0 or 2.5 may also be accepted as calculated risk. Particular reference is made to mass storage of materials such as ore, coal and building materials in which complete flexibility is involved with no rigid or semirigid substructure to be seriously damaged.

5 CONSOLIDATION THEORY

One approach to the problem of evaluating settlement under a loaded area presumes that settlement is due primarily to consolidation of the supporting soil. The consolidation theory which has been vigorously promoted and gained wide acceptance conceives that settlement due to consolidation is caused by squeezing water out of the voids of a saturated soil under the applied pressure. The consolidation theory which postulates that the movement of moisture is caused by pore water pressure or excess hydro-static pressure as distinguished from pressure components originating in shearing resistance due to cohesion, cannot be applied to compressible soils in which the voids are not filled with water.

The experimental procedure followed in applying the consolidation theory is to obtain relatively large undisturbed samples which are brought into the laboratory and subjected to a consolidation test. In this test the sample is placed between porous stones and completely confined in a test cylinder in which it is subjected to applied pressure in sufficient magnitude to squeeze the water out of the sample. These laboratory test are then translated into consolidation settlement under practical conditions by a coefficient of consolidation involving a change in the void ratio of the soil mass and modified by the permeability of the soil in order to obtain predicted settlements under field conditions.

For a number of years our Soil Mechanics Laboratory conducted such consolidation tests but they have been abandoned as part of their routine soil testing procedure due primarily to the fact that settlement predictions based on these tests have frequently proved to be quite unreliable. This experience has also been con-firmed by numerous examples in the engineering literature in which the settlement predictions based upon consolidation tests have failed by wide margins as a prediction of the actual settlement that has been experienced.

The inaccuracy in settlement predictions has occurred in two ways. In the first place, when the applied pressures are substantially less than the ultimate bearing capacity of the soil with respect to displacement settlement experienced in the field has been very much less than that which was predicted from the laboratory consolidation tests. In the second place, when the applied pressure exceeds the ultimate bearing capacity of the soil, progressive settlement under plastic flow generally continues without any noticeable decrease due to the presumed consolidation of the soil. The latter experience is of the greatest practical importance because it illustrates the danger of overemphasis on consolidation as a source of settlement. This has led practicing engineering in many notable cases to ignore the danger of exceeding the ultimate bearing capacity of the soil which has resulted in total mass displacement.

There may be several reasons for the unsatisfactory experience in predicting settlement by the consolidation theory. To begin with, the theory is not applicable to unsaturated soils with unfilled void space characterizing most of the compressible soils encountered in practice. In connection with saturated clays in which the settlement observed has been substantially less than that predicted from consolidation tests, Terzaghi and Peck account for these discrepancies as a secondary time effect due to the lag in the reaction of clay to a change in stress as noted in the following quotations:

"These delays in the reaction of clay to a change in stress, like the secondary time effect and the influence on cv (coefficient of consolidation) of the magnitude of the load increment, cannot be explained by means of the simple mechanical concept on which the theory of consolidation is based. Their characteristics and conditions for occurrence can be investigated only by observation".

"It is obvious that the results of a settlement computation are not even approximately correct unless the assumed hydraulic boundary conditions are in accordance with the drainage conditions in the field. Every continuous sand or silt seam located within a bed of clay acts like a drainage layer and accelerates the consolidation of the clay, whereas lenses of sand and silt have no effect. If the test boring re-cords indicate that a bed of clay contains partings of sand and silt, the engineer is commonly unable to find out whether or not these partings are continuous. In such instances the theory of consolidation can be used only for deter-mining an upper and lower limiting value for the rate of settlement. The real rate remains unknown until it is observed".

These statements touch upon Housel´s primary misgivings as to the practical applicability of the consolidation theory. In his opinion the conditions under which an isolated sample in the laboratory is tested depart so far from the conditions under which the soil mass is loaded in the field that there is little reason to expect that such test would provide a reasonable basis for predicting settlement. Aside from the obvious difficulty of reproducing the actual drain-age conditions in the laboratory, the sample is completely confined in the test cylinder so that there is no opportunity to observe the weakness of the soil with respect to displacement which becomes a controlling factor under actual field conditions.

This is the source of the major weakness in the practical application of the consolidation test which has been referred to above as the second and more important source of inaccuracy in settlement predictions. In summarizing Housel´s position on the consolidation theory it is concluded that this approach does not provide an acceptable basis of designing footings for constant settlement and it is not recommended.

6 CONCLUSION

The degree to which the soil is stressed is reflected in the overload ratio "R". This "R" is obtained by dividing the imposed shearing stress by the static or yield value shearing resistance. When "R" = 1 or less, the stresses are equal to or less than yield value shear resistance and the foundation is in static equilibrium. Experience indicates that overload ratios in the range of 1 to 1.5 involve progressive settlements due to plastic deformation of the bearing clay, usually taken as consolidation settlements, and for values above 1.5 involve significant rates of progressive settlement, and rapid settlement or imminent failure accompanies an overload ratio approaching and/or exceeding 3.0.

In other words, using the untrained shear strength cu, and a factor of safety of 3 when computing the allowable bearing capacity of foundations in plastic clay we are overstressing the clay foundation beyond the yield value with an overload ratio "R" = 1.33>1.0

The information given in this paper is sup-ported by laboratory testing and a historical correlation of the overload ratio "R" for differents foundation conditions, such as:

- Immediate failure of foundations after load-ing, R = 2.0>3.0
- Foundations under progressive deformation, R = 1.0-2.0
- Foundations design in static equilibrium by the undersign during the last 30 years us-ing the yield shear strength, R = R < 1.0.

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