The relevance of the yield shear strength of plastic clays as the boundary between elastic and progressive plastic settlement of foundations

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ABSTRACT
The bearing capacity and settlements of foundation on saturated normally and over consolidated plastic clays are analyze under the concept of the yield shear strength, following the criteria that cohesive clay is a plastic solid and could expected to exhibit the basic properties of such material. A comparative study is made of the elastic and progressive plastic settlements of foundations in this kind of soils following the yield strength concept with the consolidation settlements from the theory of consolidation. Finally the inaccuracy in settlement prediction following the theory of consolidation in this kind of soil is explained and several foundation failures are analyze.

RESUMEN

La capacidad admisible de carga y de asentamientos de fundaciones apoyadas en arcillas plásticas saturadas normalmente consolidadas y preconsolidadas son analizados bajo el concepto de la resistencia al esfuerzo cortante del límite cedente y siguiendo el criterio de que una arcilla cohesiva es un sólido plástico, pudiéndose esperar que desarrolle las mismas propiedades de dicho material. Se hace un estudio comparativo entre los asentamientos elásticos inmediatos y aquellos plásticos progresivos de fundaciones en este tipo de suelos, siguiendo el concepto de la resistencia al límite cedente y los asentamientos por consolidación obtenidos de la Teoría de Consolidación. Finalmente se explica la inexactitud en la predicción de los asentamientos, siguiendo la Teoría de Consolidación en este tipo de suelo y se analizan algunos casos de falla de fundaciones.

1. INTRODUCTION

40 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 il-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.

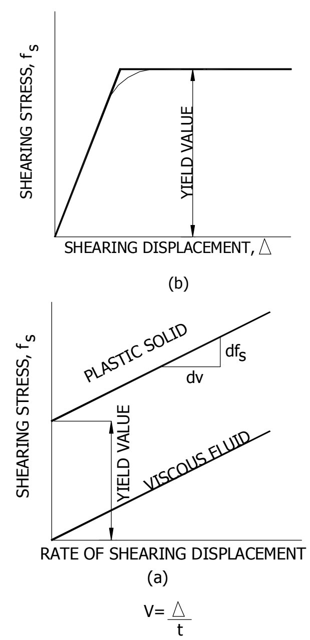


Figure 1. Properties of plastic solids.

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 stage 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.

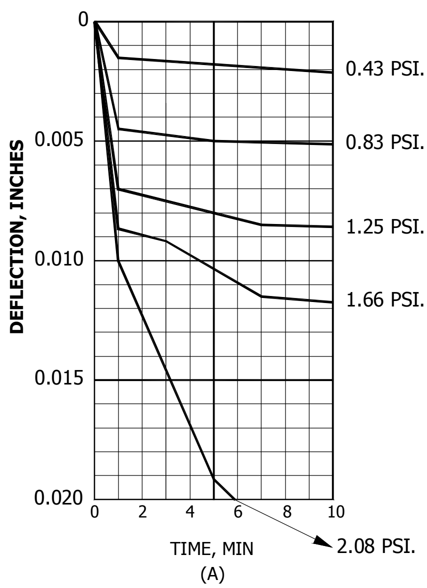
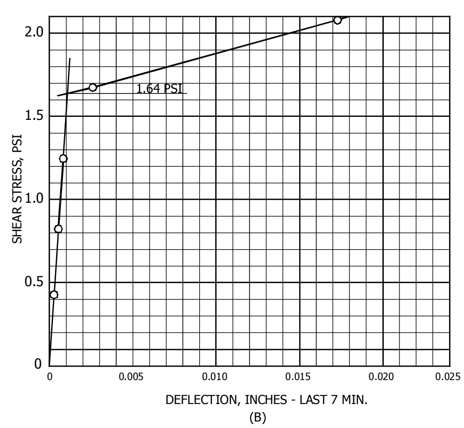


Figure 2. Typical results from transverse shear test.



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:

$$\tau_f = c_u + \sigma \tan \phi_u \qquad (1)$$

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,

$$\phi_u = 0 \qquad (2)$$

$$c_u = \tfrac{1}{2}\,(\sigma_1 - \sigma_3)_f \qquad (3)$$

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 c_u 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 ½ 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 2010 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.

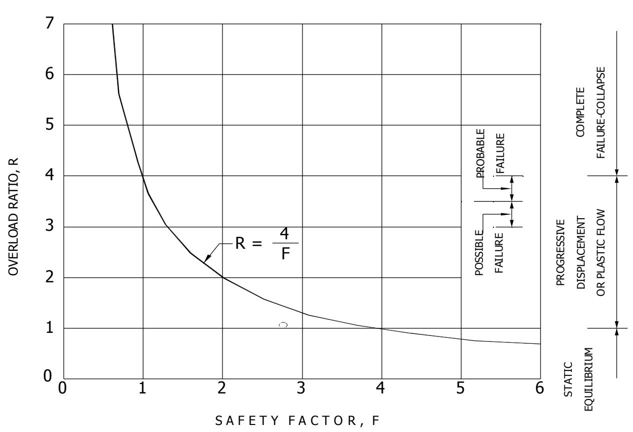


Figure 3. Relation between overload ratio and safety factor.

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 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. REVIEW OF FOUNDATION FAILURES

5.1 Immediate Failure of Foundations after Loading

Transcona Silo Failure. Perhaps the classic example of a catastrophic failure of a shallow foundation is that of the million bushel capacity Transcona grain elevator on the Canadian Prairie, 7 miles N .E. of Winnipeg, Manitoba.

The elevator consisted of two principal structures, the bin house, containing 65 bins, 14 ft diameter by 92 ft high in five rows of 13, carried by a 2 ft thick concrete raft 77 ft wide and 195 ft long at a depth of 12 ft, and the work house, containing the machinery, 70 ft by 95 ft by 180 ft high, also carried on a raft at 12 ft depth.

Construction started in 1911 and was completed in September 1913, when filling with grain was commenced (Fig. 4). On 18th October 1913, 875,000 bushels of grain had been stored and at lunch time on that day the bin house began to tilt, much of the movement took place during the first half hour. (Fig. 5)

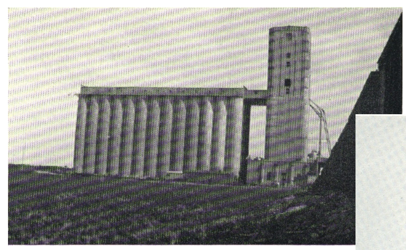


Figure 4. Transcona Silo. Filling with grain. (White, 1.953)

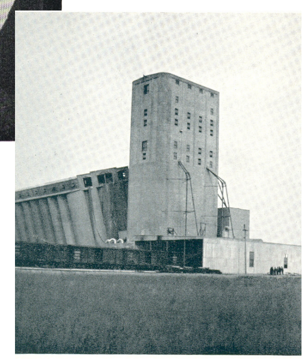


Figure 5. Transcona Silo. Detail of movement after failure showing undamaged workhouse.

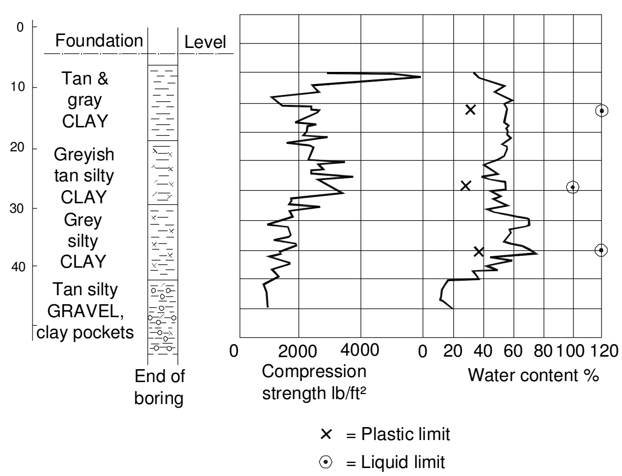


Figure 6 Transcona Silo. Subsoil Profile

Table 1. Summary of calculated values, Transcona Silo; using Skempton's formula, $q_{ult}=c_u.N_c + \gamma D_f$

c_u (ton/m^2)	S_c (ton/m^2)	q_t (ton/m^2)	$q_{ult}=(c_u)(N_c)$ (ton/m^2, □□= 0)	qs_c (ton/m^2)	Safety Factor Fs	Oveload Ratio $R=q_t/qs_c$
5,25	1,31	20,5	28,87	7,22	1,4	2,83

Ncr =5 (1 + 0.2 B/L) (1 +0,2 Df/B)
Ncr =5,5
B =77 ft
L =195 ft
Df =12 ft
c_u: undrained shearing strength of clay bearing stratum
S_c: yield Shear strength of clay bearing stratum
q_t: foundation stress on bearing clay stratum
q_{ult}: ultimate bearing capacity of clay bearing stratum, Skempton's Formula.
qs_c: allowable bearing capacity of clay bearing stratum, yield shear strength criteria.
Safety Factor Skempton´s Formula = 1.4

Overload Ratio:

$R = \frac{qt}{qs_c} = 2.83 \approx 3.0$, Progressive plastic settlements or immediate failure after loading.

<u>Failure of a Bauxite Dumpmp.</u> Newport (reported by Skempton and Golder, 1948)

After relatively rapid tipping, failure occurred at height of 25 feet; the factor of safety by $\phi_u = 0$ analysis was subsequently found to be 1.08, which can be accepted as agreement to within the limit of experimental accuracy.

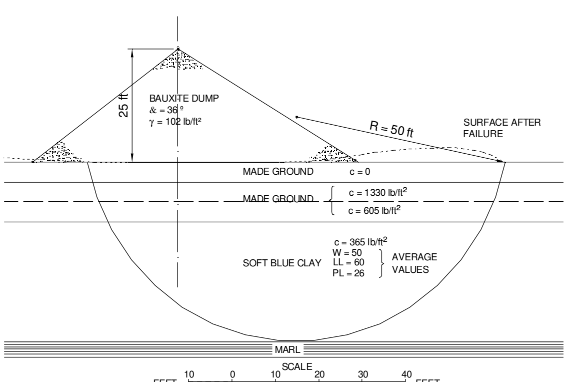


Fig. 7. Failure of a Bauxite Dump at Newport (after Skempton and Golder; 1948)

Table 2. Summary of calculated values, Bauxite Dump, Newport; using Skempton's formula, $q_{ult}=c_u.N_c + \gamma D_f$

c_u (ton/m^2)	S_c (ton/m^2)	q_t (ton/m^2)	$q_{ult} = (c_u)(N_c)$ (ton/m^2, □□= 0)	qs_c (ton/m^2)	Safety Factor Fs	Oveload Ratio $R=q_t/qs_c$
2.58	0.65	12.4	13.25	3.34	1.06	3.71

H = 25 feet = 7.6 m.
L =74" = 22.5 m.
c_u: undrained shearing strength of clay bearing stratum
S_c: yield Shear strength of clay bearing stratum
q_t: foundation stress on bearing clay stratum
q_{ult}: ultimate bearing capacity of clay bearing stratum, Skempton´s Formula.
qs_c: allowable bearing capacity of clay bearing stratum, yield shear strength criteria.
weighed value: C_u = 2.58 ton/m^2
C_1= 6.4 ton/m^2 H = 5´
C_2= 2.9 ton/m^2 H = 5´
C_3= 1.75 ton/m^2 H = 25´

Overload Ratio:
$R = \frac{qt}{qs_c} = 3.71 > 3.0$, Immediate failure after loading

5.2 <u>Foundations Under Progressive Displacement or Plastic Flow</u>

<u>La Previsora Bank, Guayaquil, Ecuador, 1992.</u>
- Reinforced concrete structure, frame's span 6.70 to 9.60 m.
- Plan dimension; length = 59 m; width = 30 m.
- One basement level + 36 floors
- Mat foundation, (two-way beam and slab) resting on 648 precast reinforced concrete driven piles, 0.50 m. width square section and 18.0 m. depth. The piles were driven from level -5.20 (see attachment A, Composite Soil Profile)
- Total building weight = 72,882.00 Ton. including mat foundation.
- Ground water level, -1.20 m.

Table 3. Summary of calculated values, La Previsora Bank, Guayaquil, Ecuador; using Skempton's formula, $q_{ult}=c_u.N_c + \gamma D_f$

Depth (m)	N (blows/foot)	c_u (ton/m^2)		Sc (ton/m^2)	qt (ton/m^2)	q_{ult}= (Cu)(Nc) (ton/m^2, $\phi = 0$)	q_{sc} (ton/m^2)	Conventional theory, Fs	$R = q_t/q_{sc}$
-34	12	7.8	10.4	2.6	20.26	53.46	13.36	2.6	1.5
-37	20	13			18				

c_u: undrained shearing strength of clay bearing stratum (t/m^2)
S_c: yield Shear strength of clay bearing stratum
q_t: foundation stress on bearing clay stratum
q_{ult}: ultimate bearing capacity of clay bearing stratum, Skempton's Formula.
qs_c: allowable bearing capacity of clay bearing stratum, yield shear strength criteria.

Overload Ratio:

$$R = \frac{qt}{qs_c} = 1.5 > 1.0$$, under progressive displacement or plastic flow

End of construction of the piles, February 1992
End of construction of the building, June 1994
Measured settlements began on, August 1992
Calculated consolidation settlements of the deep clay layer at 34.0 m. depth:

- First 34 month: 6.0 to 11.0 cm.
- 27.5 years after construction: 12.0 to 16.0 cm.

Measured Settlements:

- End of first year: 11.0 to 26.0 cm.
- End of second year: 37.0 to 46.0 cm. before finishing construction
- End of the third year: 45.0 to 55.0 cm.

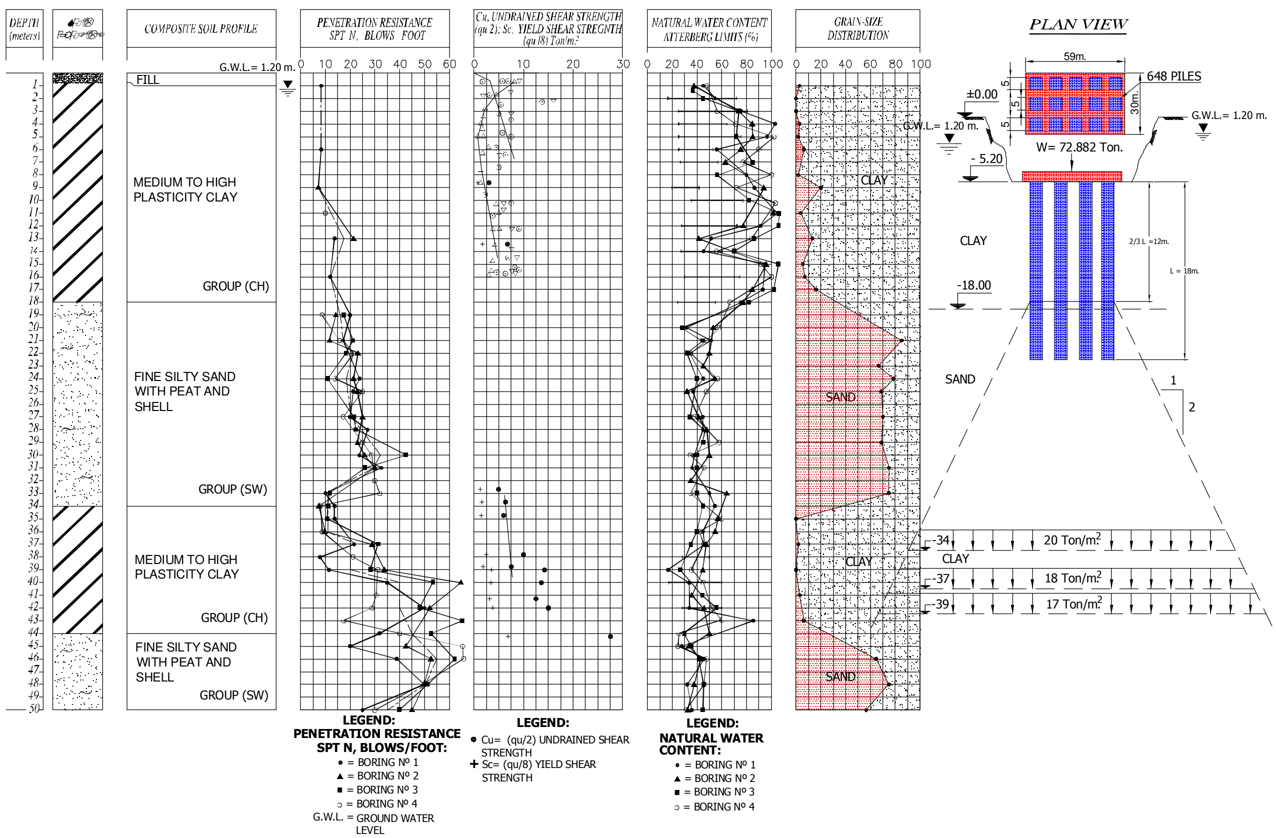


Figure 8. Composite soil profile, La Previsora Bank, Guayaquil, Ecuador.

6. 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 consoli dation 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 hydrostatic 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 confirmed 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 c_v (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 records 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 determining 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 drainage 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.

7. CONCLUSION

Following the review of the foundation failures and the recognition that cohesive soils, saturated clays behave as plastic solids with a definite yield shear strength value we may conclude that there are several types of foundation failures on plastic clays depending of the following conditions:

- If the clay bearing layer is overstress beyond the yield shear strength; with an over load ratio value in between 1.0-1.5 the foundation is under progressive settlements due to plastic flow been **the yield Shear strench the boundary between elastic and progressive plastic settlement of foundation** and a rigid reinforced concrete structure will not tolerate the differential settlements with time under this condition of a bearing capacity failure.
- If the clay bearing layer is overstress beyond the yield shear strength; with an over load ratio value in between 1.5-2.5 the foundation is under progressive settlements due to plastic flow. This condition represent a calculated risk and must be done with full realization of the conse-

quences of progressive settlements and the increasing possibility of rapid progressive settlements, sudden mass movements or a catastrophic failure.

- As Housel has pointed out: "There are other conditions frequently encountered in practice where considerable progressive settlement may be permitted and where overload ratios as high as 2.0 or 2.5 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 semi-rigid substructures to be seriously damaged".

The undersign have design successfully in the last 40 years more than one hundred building foundations on plastic clays under static equilibrium using the yield shear strength criteria with a calculated overload ratio R<1.0

Finally, bearing in mind the importance of this topic, in feel myself forced to recall the following thougths:

1. Professor Arthur CASAGRANDE, "*The structure of clay and its importance in foundation engineering*", April 1932 ("*Contributions to Soil Mechanics 1925-1940*", Published by the Boston Society of Civil Engineers, 1963, pp. 111)
"I have tried to illustrate that the whole problem of building foundations on clay boils down to these *two* simple *principles*: first, *do not disturb the natural structure of the clay; if you do, no human being is able to restore its original strength;* second, *decide on a certain rate of settlements which you do not wish to exceed, and determine that pressure which will cause this rate of settlement; the difference between the building load and the above pressure is the weight of soil which must be removed before erecting the building.*
A definite bearing value of clay dos not exist. As long as engineers are guided by building codes containing definite bearing values for clay, they are consciously guessing without any assurance in their own minds that they are guessing correctly.
The engineer must learn that the kind of questions he asks an expert regarding the properties of a clay underground should not be, "How much load may I put on this soil?" Or, in an apparently more scientific manner, "What is the bearing capacity or the bearing value of this clay?" His question should be, **"How must I design my foundation so that the rate of settlement under the given building load will not exceed certain limits?"**

2. Professor Ralph PECK [1963], " *The first Terzaghi Lecture*", Presented at the American Society of Civil Engineers Annual Meeting and Structural Engineering Conference, San Francisco, California
("*Terzaghi Lectures 1963-1972*" [1974], Published by American Society of Civil Engineers, New York, pp. 3)
"The relation between lateral deformation and loading is studied to ascertain the extent to which the clay behaves elastically, **the possible existence of a threshold stress at which progressive nonrecoverable movements are initiated**, and the influence of the cyclic character of the loading".

3. N.E. SIMONS and B.K. MENZIES. [1977]. "*A Short Course in Foundation Engineering*", Published by Butterworth & Co., USA, pp. 78
"At the present time, laboratory studies alone will not allow accurate settlements predictions to be made. Long term regional studies are vitally necessary to determine in particular:

- Whether in the field, primary consolidation and/or secondary settlements will develop over a long period of time, and
- Whether **a threshold level exists, below which acceptable settlements develop and above which large and potentially dangerous settlements will be experienced".**

4. Professor William S. HOUSEL, Discussion, "*Foundation behavior of iron storage yards*"
("*Terzaghi Lectures 1963-1972*", Published by American Society of Civil Engineers, New York, 1974, pp. 64-65)
"Recognition that cohesive soils such as the saturated clays behave as plastic solids with a definite yield value should do much to clarify an extremely important and much confused phenomenon in the field of soil mechanics.
It is difficult to understand the reluctance of many investigators in soil mechanics practice to accept the applicability of the basic principles of plastic solids to cohesive soils.

It is difficult to understand the failure to recognize that these principles have long been available for engineers to apply to their problems.
The only contribution required to modern soil mechanics was to develop reliable methods for measuring shearing resistance in terms of a definite yield value and to translate the result into foundation behavior in the field. When this is done, there immediately becomes available a definite and reliable frame of reference by which field performance can be evaluated and anticipated".

REFERENCES

Bishop, A.B. and Bjerrum L. [1960]. "*The Relevance of the Triaxial Test to the Solution of Stability Problems*". Norwegian Geo-technical Institute, Publication # 34, Oslo.

Casagrande, A. [1932]. "*The Structure of Clay and its Importance in Foundation Engineering*". Journals of the Boston Society of Civil Engineers, Contributions to Soil Mechanics, 1.925-1.940.

Geuze, E.C.W.A. [1953]. "*Laboratory Investigations, Including Compaction Test, Improvement of Soil Properties, General report Session 2, Proceeding*", *Third Int. Conference on Soil Mechanics and Foundation Engineering*, Vol. II, pp. 313-317, (1.953), (a) Vol. III, pp. 119-122 (1.953).

Housel, W.S. [1.939]. "*Shearing Resistance of Soil its Measurement and Practical Significance*". Presented before the Annual Meeting of the American Society for Testing Materials. Atlantic City, N.J.

Housel, W.S. [1952]. "*Design Memorandum on Bearing Capacity of Spread Footing on Cohesive Soil*", Dpt. Civil Engr. University of Michigan, Ann Arbor, Soil Mechanics Laboratory.

Housel, W.S. [1959]. "*Dynamic and Static Resistance of Cohesive Soils, 1846-1958*", *Symposium on Time Rates of Loading in Soil Testing*, American Soc. of Testing Materials.

Housel, W. S. [1974]. Discussion, "*Foundation Behavior of Iron Storage Yards*", "*First Terzaghi Lecture*", Proceeding, Am. Soc. Civil Engrs., Soil Mech. and Foundation Division.

Hvorsley, J. [1960]. "*Physical Components of the Shear Strength of Saturated Clays*", ASCE, Research Conference an Shear Strength of cohesive soils, Colorado.

Little, A.L. Little [1961] "*Foundations*", Edward Arnold (Publishers) LTD, London.

Peck, R. B. and Raamot, T. [1974]. "*Foundation Behavior of Iron Ore Storage Yards*", "*First Terzaghi Lecture*", Proceeding, Am. Soc. Civil Engrs., Soil Mech. and Foundation Division.

Skempton, A.W. and Bishop, A.W. [1950]. "*The Measurement of Shear Strength of Soil, Geotechnique*", Vol. II, pp. 90-108.

Terzaghi, K. [1925]. "*Modern Conceptions Concerning Foundation Engineering*". Journals of the Boston Society of Civil Engineers, "Contributions to Soil Mechanics, 1.925-1.940.