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A Review of Foundation Failures on Plastic Clays, Following the Yield Shear Strength Concept of a Plastic Solid in This Kind of Soil

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ABSTRACT

Several Foundations Failures are analyze and a comparative study is made, of the bearing capacity of foundations using the undrained shear strength cu, $\phi_u = 0$ 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.

INTRODUCTION

35 years ago when I began my first design of earth works and bearing capacity analysis of foundations on 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.

BASIC CONCEPTS

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.

The Undrained Shear Strength on Saturated Cohesive Plastic Soils

The undrained shear strength test is 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:

$$\tau_f = c_u + \sigma \tan \phi_u \tag{1}$$

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,

$$\phi_u = 0 \tag{2}$$
$$c_u = \tfrac{1}{2} (\sigma_1 - \sigma_3)_f \tag{3}$$

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"

The Yield Shear Strength Concept of a Plastic Solid and 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 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.

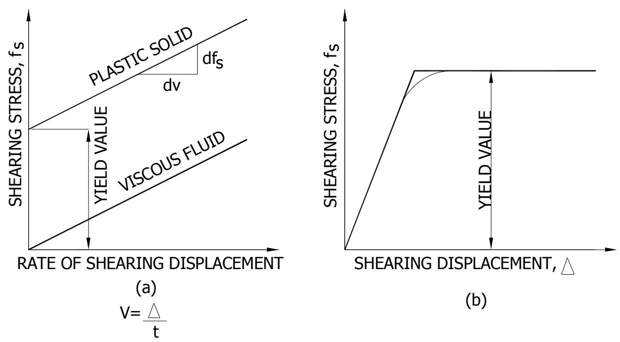


Fig. 1. Properties of plastic solids

With normal pressure eliminated as a variable in the 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. 2(b))

THE UNDRAINED SHEAR STRENGTH VS THE YIELD SHEAR STRENGTH

The shear value known as the ultimate shearing resistance or the undrained shear strength c_u 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.

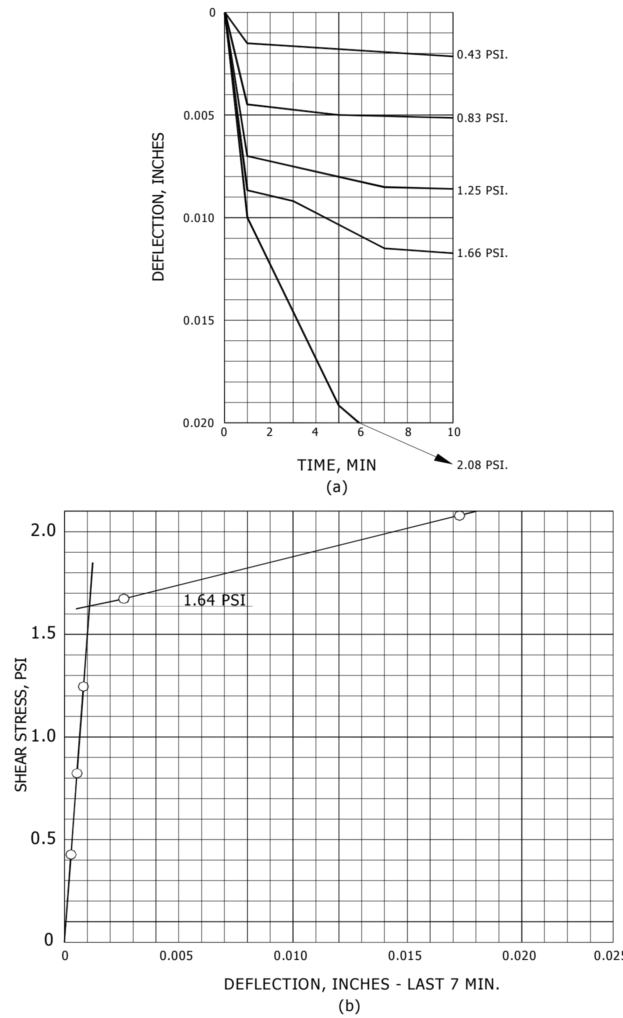


Fig. 2. Typical results from transverse shear test

Comparative results in considerable detail were reported in 1956 and the author has run these tests from 1974 to the present time 2007 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".

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.

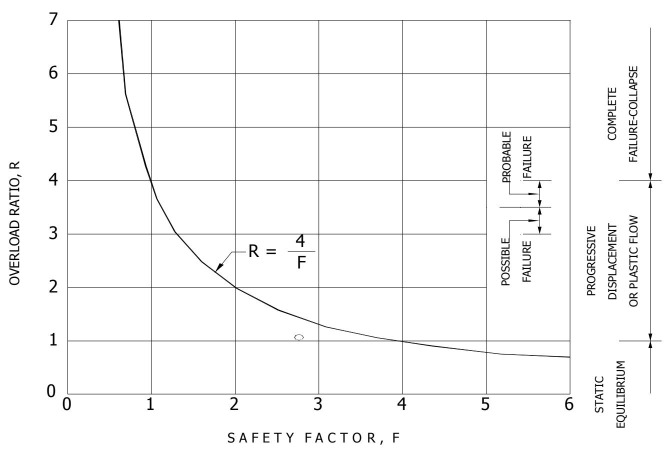


Fig. 3. Relation between overload ratio and safety factor

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 semi-rigid substructure to be seriously damaged.

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 clays, 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 undrained shear strength cu, and a factor of safety of 3 when computing the allowable bearing capacity of foundations in plastic clays we are overstressing the clay foundation beyond the yield value with an overload ratio "R" = 1.33>1.0. We are under progressive displacement or plastic flow.

The information given in this paper is supported by laboratory testing and a historical correlation of the overload ratio "R" for different foundation conditions, as the ones that I'm presenting next for:

- Immediate failure of foundations after loading, R >2.0
- Foundations under progressive deformation, R = 1.0-2.0

REVIEW OF FOUNDATION FAILURES

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)

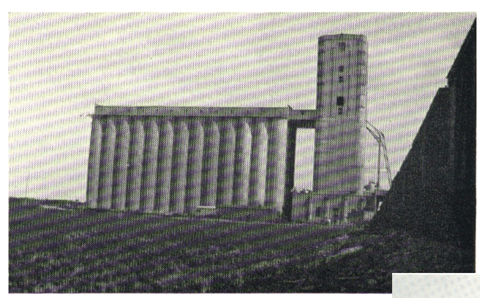


Fig. 4. Transcona Silo. Filling with grain. (White, 1.953)

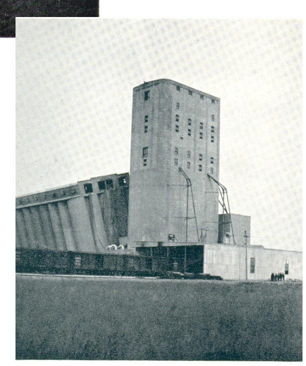


Fig. 5. Transcona Silo. Detail of movement after failure showing undamaged workhouse.

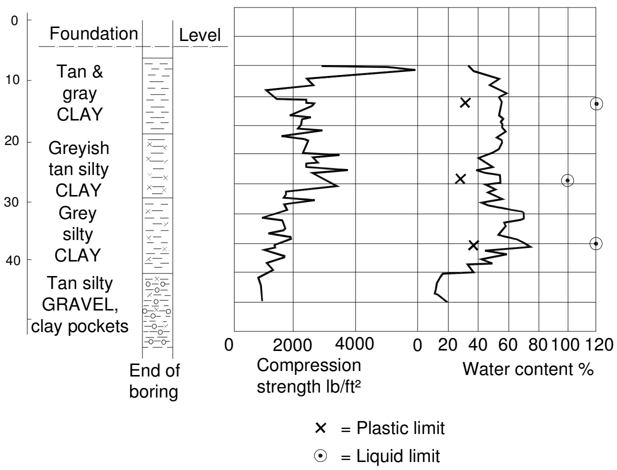


Fig. 6 Transcona Silo. Subsoil Profile

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, ϕ = 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:

$R = \frac{qt}{qs_c} = 2.83 > 2.0$, Immediate failure after loading

Oil Tank Failure, Grangemouth Scotland. Saurin (1949 A) and Nixon (1949 A, B) describe the failure of tanks at Grangemouth, Scotland, and Shellhaven, England. At the time no really satisfactory explanation was forthcoming for the value of bearing capacity observed. The difficulty at both sites was the presence of a stiffer crust; at Grangemouth some 15 ft. and at Shellhaven, 4 ft. in thickness.

In the Grangemouth example (Fig 8) assuming a spreading angle of 45º (Fig.7), using a weighted average shear strength of 330 lb/ft^2 and an Nc of 6.4 corresponding to the value appropriate to a foundation of with B´=150 ft. at a depth of 15 ft. (the estimate depth of the crust), we obtain an apparent nett bearing capacity of 2100 lb/ft^2 giving a factor of safety of 1.7 on the estimates pressure at this depth of 1235 lb/ft^2; and yet the foundation failed.

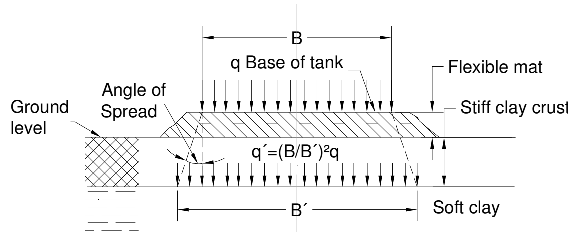


Fig. 7. Oil tank resting on stiff crust over soft clay

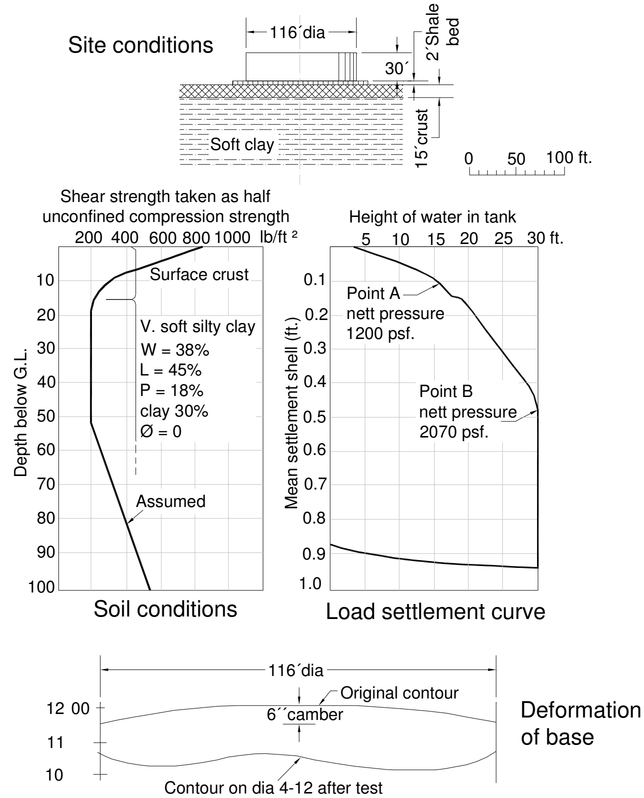


Fig. 8. Grangemouth oil tank

Table 2. Summary of calculated values, Oil Tank, Grangemouth, Scotland; 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 $R=q_t/qs_c$
1.37	0.34	4.56	8.75	2.18	1.9	2.1

Nc = 6.4
B =150 ft
Df =15 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 = 1.7

Overload Ratio:
$R = \frac{qt}{qs_c} = 2.36 > 2.0$, Immediate failure after loading

Oil Tank Failure, Shellhaven, England.
Sellhaven tank (Fig. 9.), the corresponding values are:
- Thickness of crust - 4 ft;
- Weighted average shear strength for depth 2/3B´: 280 lb/ft^2;
- Nett bearing capacity: 1800 lb/ft^2;
- Estimated bearing pressure assuming a 45º spread: 935 lb/ft^2;
- Apparent factor of safety: 1.9

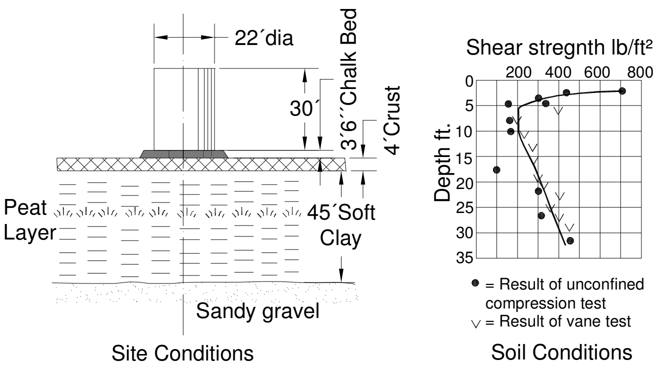


Fig. 9. Shellhaven oil tank

Table 3. Summary of calculated values, Oil Tank, Shellhaven, England; 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, ϕ = 0)	qs_c (ton/m^2)	Safety Factor Fs	Oveload Ratio $R=q_t/qs_c$
1.61	0.4	6.0	10.3	2.54	1.7	2.36

Nc = 6.4

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 = 1.9

Overload Ratio:
$R = \frac{qt}{qs_c} = 2.1 > 2.0$, Immediate failure after loading

Failure of a Bauxite Dump. 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.

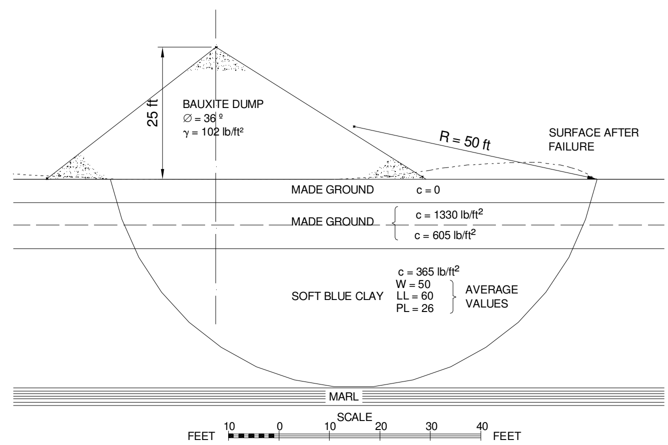


Fig. 10. Failure of a Bauxite Dump at Newport (after Skempton and Golder; 1948)

Table 4. 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, ϕ = 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:

$R = \frac{qt}{qs_c} = 3.71 > 2.0$, Immediate failure after loading

<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 5. 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.

<u>"Isla de Oro" Beach Resort Condominium 1979-1984, Río Chico, Edo. Miranda, Venezuela.</u>

- Reinforced concrete structure, frame's span 7.0 to 8.0 m.
- Plan dimension; three separate buildings of variable height converging into a circulation core of 15 floors (see Fig. 11)
- Isolated foundation on the surface sand layer, level -10.0 m. (see attachment B, Composite Soil Profile)
- Net total pressured applied at -1.0 m.; q_t = 19 Ton/m^2
- Ground water level; -1.50 m.

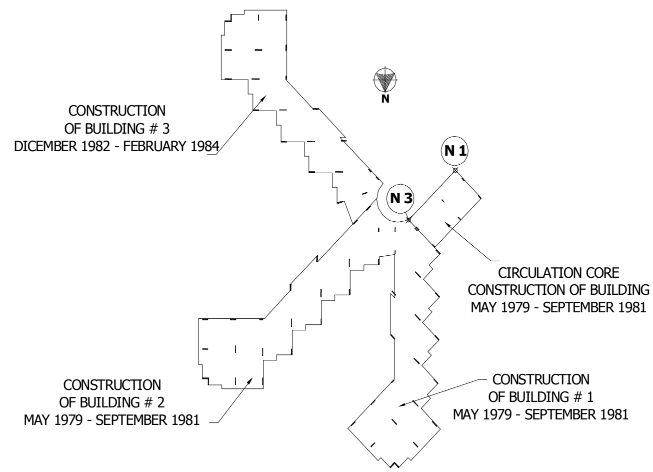


Fig. 11. Plan Drawing and Locations of settlements, points N3 and N1

Table 6. Summary of calculated values, "Isla de Oro" Beach Resort Condominium, Rio Chico, Venezuela; using Skempton's formula, $q_{ult}=c_u.N_c + \gamma D_f$

Depth H (m)	c_u (ton/m^2)	S_c (ton/m^2)	q_t (ton/m^2)	q_{ult}= (c_u)(N_c) (ton/m^2, $\phi = 0$)	qs_c (ton/m^2)	Safety Factor Fs	Oveload Ratio R=q_t/qs_c
8.50-12.0	6.5	1.63	11	33.4	8.38	3.0	1.31
12.50-15.0	3.0	0.75	5.84	15.42	3.85	2.6	1.52

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.

Overload Ratio:

$R_{8.5\text{-}12.0} = \dfrac{qt}{qs_c} = 1.31 > 1.0$, under progressive displacement or plastic flow

$R_{12.5\text{-}15.0} = \dfrac{qt}{qs_c} = 1.52 > 1.0$, under progressive displacement or plastic flow

Construction of buildings # 1 and building # 2, May 1979 to September 1981.
Construction of building # 3, December 1982 to February 1984
See settlement curve vs. time for point N3, north corner of the central core. Total settlement after 68 month ≈ 5.6 years = 57.7 cm. (attachment c, Total Settlement Curve)
Differential settlements between point N3 and point N1 (south corner of the central core) after 5.6 years: 14.5 cm.

<u>Foundation Performance of Tower of Pisa</u>
(James K. MITCHELL, Vitoon VIVARAT, T. William LAMBE "Foundation Performance of Tower of Pisa", Journal of Geotechnical Engineering Division, GT3, 12814, March 1977)

"Construction of foundation of the Tower of Pisa began on August 9, 1173 and reached Cornice 1 (Fig.12) in 1174. When de tower reached a height of three and one-half stories and a load of 9,480 metric tons (1,000 kgf/ton) in 1178 work was stopped……….Construction was not resumed until almost a century later, when the Tower was completed to the eighth floor level and a total load of 13,728 tons during the period 1272-1278. Construction then stopped and did not resume until 1360 when the final story was added and the whole Tower completed in 1370. By the time of the final stage of construction the lean of the Tower was significant, as evidenced by the changed center line direction for eighth floor (Fig. 12)".

"The completed Tower has a maximum base diameter of 19.58 m., a center line of 58.4 m. or a height from the base of the foundation to Cornice of 58.2 m., allowing for the tilt of 5.2º that existed in about 1970. This tilt corresponds to a maximum differential settlement of 1.77 m. The base of the foundation was located at a depth of 3.0 m. below the surrounding ground surface. The total weight of the tower is 14.453 tons (141,640 KN)"

Fig. 12. Locations and Depth of Borings to Explore Subsoils in Vicinity of Tower (Rome,1971).

When the Tower reached a height of three and one-half stories and a load of 9,480 metric tons, (Net stress at base of Tower of 23 Ton/m^2), see Table 7, year 1178; the upper clay layer between 8.0 and 11.0 m. depth (see Fig. 13) had a vertical stress increase of 12.5 Ton/m^2, higher than the ultimate bearing capacity calculated using the yield shear strength;

q_{sa}= Sc*Nc=(S_c/4)*N_c
q_{sc}=1.125*7.2=8.1 Ton/ft^2

a Overload Ratio:
R = q_t/q_{sc} = 12.5 / 8.1 = 1.54 > 1.0, under progressive displacement or plastic flow

and a conventional factor of safety of Fs = 2.6

Following Fig 12 we could see that from year 1178, the Tower foundation is under progressive displacement do to plastic flow of the upper clay layer or an equivalent bearing capacity failure.

Description and significance of ore yards
(Ralph B. PECK and Tonis RAAMOT, "Foundation Behavior of Iron Ore Storage Yards", Terzaghi Lectures 1963-1972, Published by American Society of Civil Engineers, New York, 1974).

"Some of the largest, most heavily loaded masses of soil to be found in the world are undoubtedly those beneath the storage areas for iron ore adjacent to blast furnaces. A modern blast furnace consumes some 1500 tons of iron ore each day.

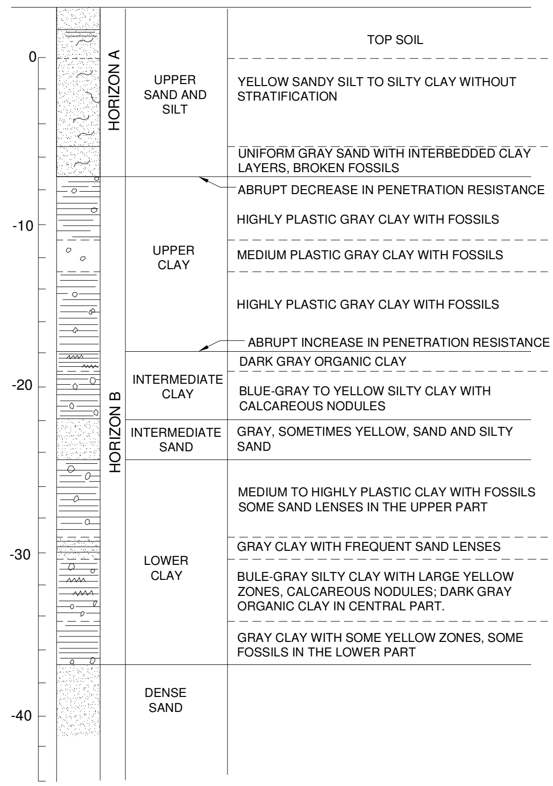


Fig. 13. Soil profile with description of soil types

Table 7. Results of Undrained Bearing Capacity Analysis

Date (year) (1)	Net stress at base of Tower (Ton/ft^2, Average Value) (2)	Average vertical stress increase at top of clay (Ton/ft^2) (3)	Maximum vertical stress increase on clay (Ton/ft^2) (4)	S_{u1} (Upper Clay) (Ton/ft^2) (5)	Yield Shear Strenght, S_{c1} (Upper Clay) (Ton/ft^2) (6)	S_{u2}(Upper Clay) (Ton/ft^2) (7)	Yield Shear Strength, S_{c2} (Lower Clay) (Ton/ft^2) (8)	Nc (9)	Ultimatr bearing capacity using Yield strength (Ton/ft^2) (10)	Factor of Safety Fs (11)	Overload Ratio (10)	Max shear stress increase in clay (Ton/ft^2) (11)
1178	23.0	12.5	-	4.5	1.125	9.7	2.42	7.2	32.4	2.6	1.54 > 1	7.3
1278	42.3	23.0	-	6.6	1.65	9.7	2.42	7.2	47.5	2.1	1.9 > 1	13.5
1975	45.1	24.5	59.0	9.5	2.37	10	2.5	6.2	58.9	2.4[1] 1.0[2]	1.66 > 1	14.4

[1] Against average stress increase on clay
[2] Against maximum stress increase on clay

To assure no interruption in its operation, an ample reserve of ore is needed close at hand. Moreover, along the Great Lakes, which are closed to traffic by ice during almost half the year, each furnace must be provided with enough ore during the shipping season to last until shipping can be resume the next year. To satisfy this requirement, storage areas several hundred feet wide and often on the order of 1000 ft long are customarily filled in the fall to heights of roughly 40 ft. to 45 ft. with ore having a unit weight of about 160 lb/ft^2. If the subsoil contains strata of lay, the corresponding unit load of about 800 psf is likely to produce substantial displacements".

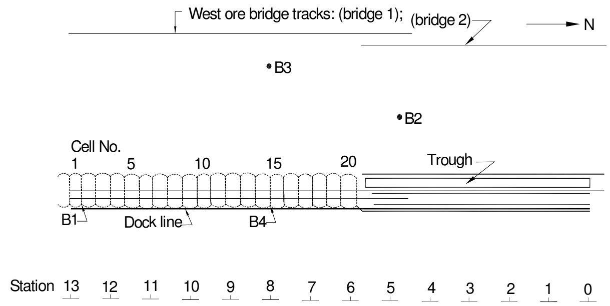
"Plastic clays constitute the subsoil for many of the ore yard in the Great Lakes region and for several on the Atlantic seaboard."

"The most significant feature is the deposit of plastic clay, about 65 ft. thick, with an unconfined compressive strength of about 1.8 ton/ft^2. The deposit was preconsolidated during the glacial epoch by an overburden of deltaic materials, since removed by erosion. The preconsolidation load is estimated to be about 5.0 ton/ft^2 (equivalent to about 60 ft. of ore) above the present overburden pressures. The clay is overlain by about 25 ft. of sand upon which the ore is piled. It is apparent that the 45-ft. timber piles beneath the 1919 and 1943 construction penetrated only a short distance into the clay beneath the sand."

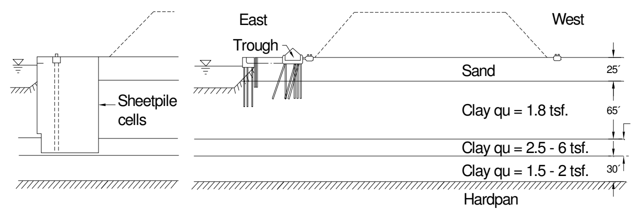
"Each loading season the dock wall advances toward the river. Surveys have shown that the horizontal movement is essentially the same at a given station whether measured at the dock line or at any distance up to at least 80 ft. from the dock; hence, it appears that the block of soil including the dock structure moves or distorts as a unit under the influence of the ore. The general pattern of the movements is represented by Fig. 15 which covers a 6- yr period from 1952 to 1957. It may be observed that major movements, if any, occur suddenly near the end of a loading season. A small percentage of such movements, up to a inch, may be recoverable, but recovery does not necessarily occur upon each unloading. The elastic or recoverable component of the deformation is therefore very small when compared with the total magnitude of ore yard movement".

"During 1955, when the displacements of the ore yard were maximum and when the total load exceeded all previous maxima, relatively frequent observations were made of both movement and the contours of the ore pile. These data are particularly useful in reviewing the behavior of the storage area".
"The maximum movements occurred in the neighborhood of Sta. 4+00. The development of movements with time is shown in Fig.17. Near the end of November the increase in displacement was very abrupt. The increment, on the order of 0.4 ft., obviously took place in such a short time that the total load on the storage area could not have changed by an appreciable percentage. Nevertheless, the distribution of the load in the critical area did change significantly".

"Although there was a large quantity of ore in storage, to heights exceeding 40 ft., at various time between July 11 and October 21, the riverward face of the ore pile had relatively flat slope".



(a) Section



(b) Section

Fig. 14. History and construction details, Yard A

"From similar detailed records, the increase in deflection for each year from 1952 to 1962 has been plotted for points along the entire storage area. The results are shown in Fig.18. The greatest movements occurred in 1955 and 1962; in contrast, almost no movements took place in 1956 or 1959. The total weight of stored material for the same years is shown in Fig. 16".

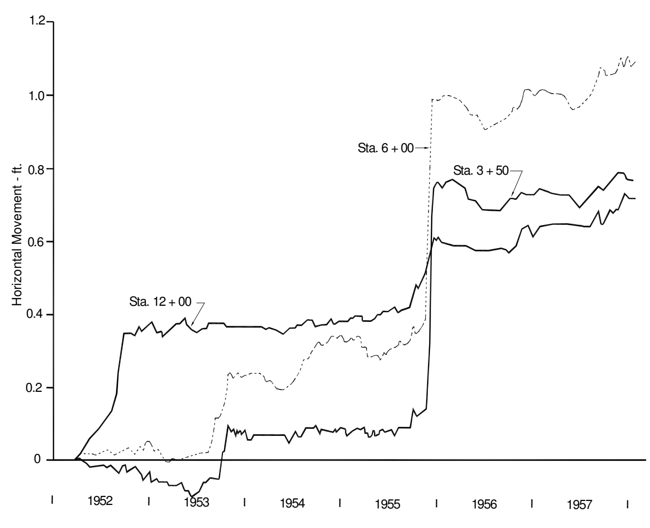


Fig.15. Representative time displacement relations, Dock Wall, Yard A

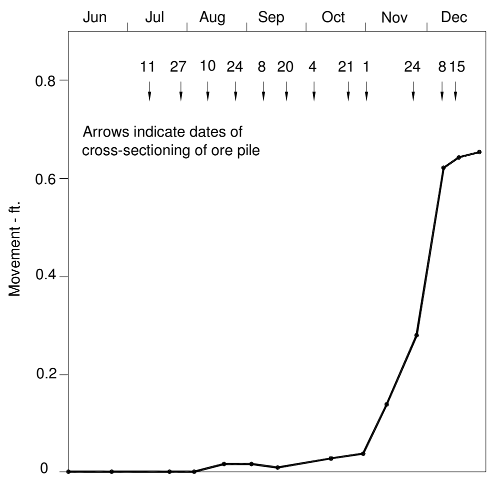


Fig. 16. Total load on Yard A by months

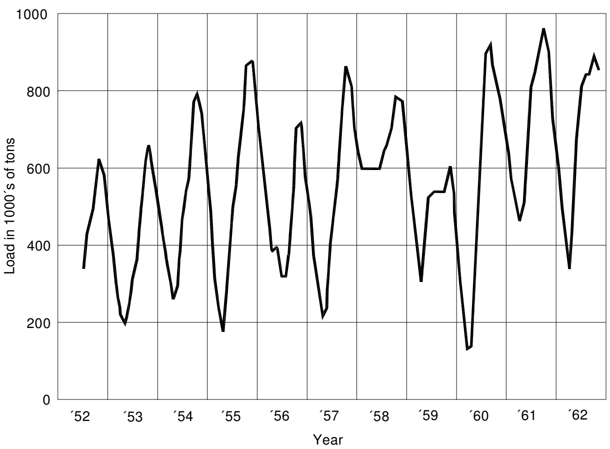


Fig. 17. 1955 Movement at station 4, Yard A

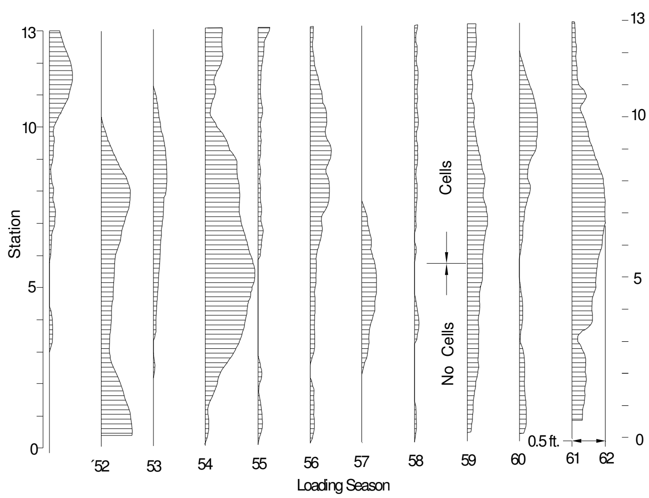


Fig. 18. Annual movements of Yard A

Table 6. Summary of calculated values, Ore Yards; using Skempton's formula, $q_{ult}=c_u.N_c + \gamma D_f$

c_u	S_c (ton/m^2)	q_t^* (ton/m^2)	$q_{ult}=(c_u)(N_c)$ (ton/m^2, $\phi = 0$)	qs_c (ton/m^2)	Safety Factor Fs	Oveload Ratio $R=q_t/qs_c$
8.8	2.22	30.0	45.2	11.4	1.50	2.60

Height of ore above yard level = 40´= 12 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
Safety Factor Skempton's Formula = 1.50

Overload Ratio:
$R = \frac{qt}{qs_c} = 2.60 > 1.0$, under progressive displacement or plastic flow.

CONCLUSIONS

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. Under this condition a rigid reinforced concrete structure will not tolerate the differential settlements with time and this represents 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 consequences 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 with complete flexibility is involved with no rigid or semi-rigid substructures to be seriously damaged".

The undersign have design successfully in the last 30 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, i feel my self 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".

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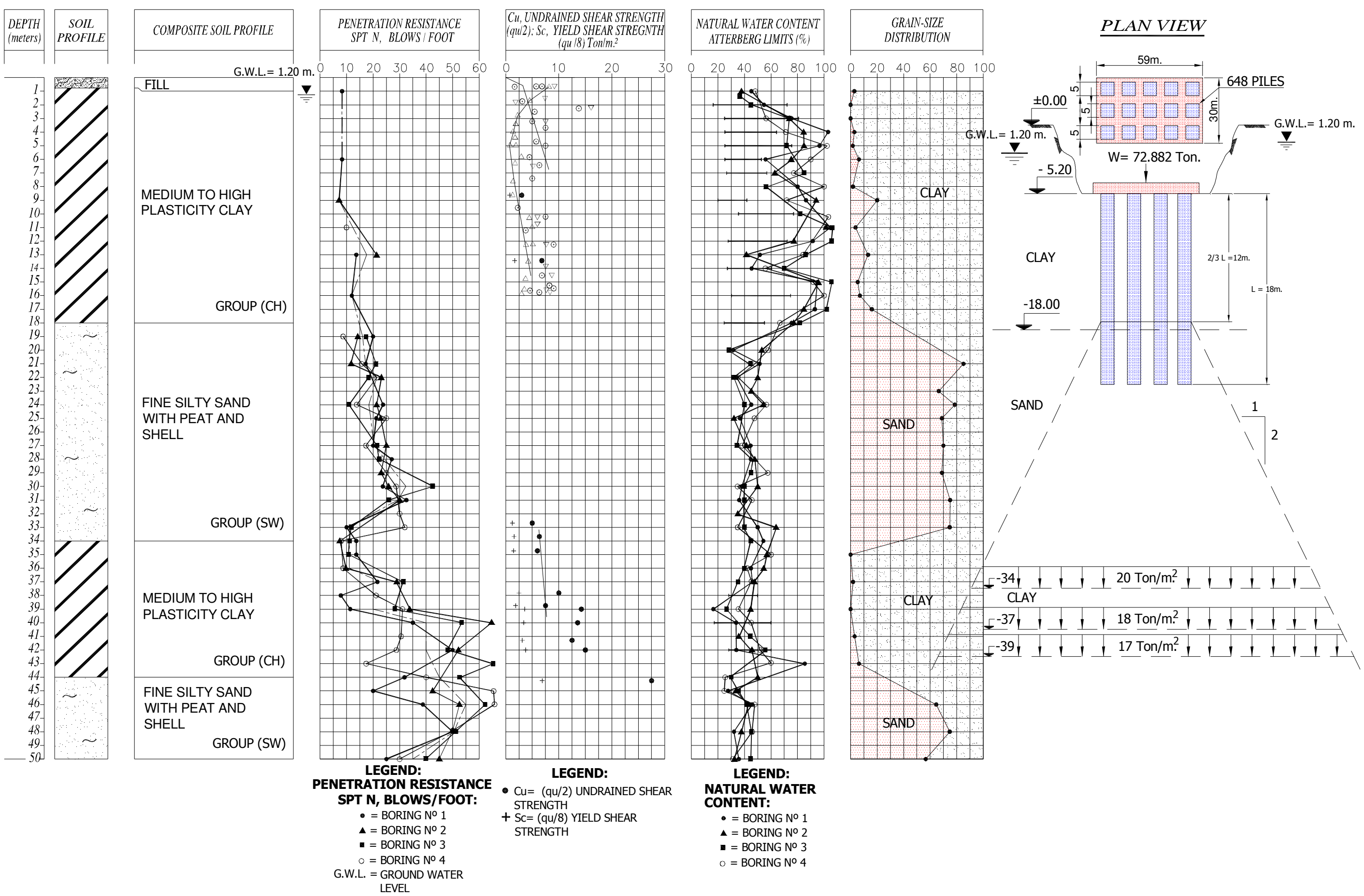
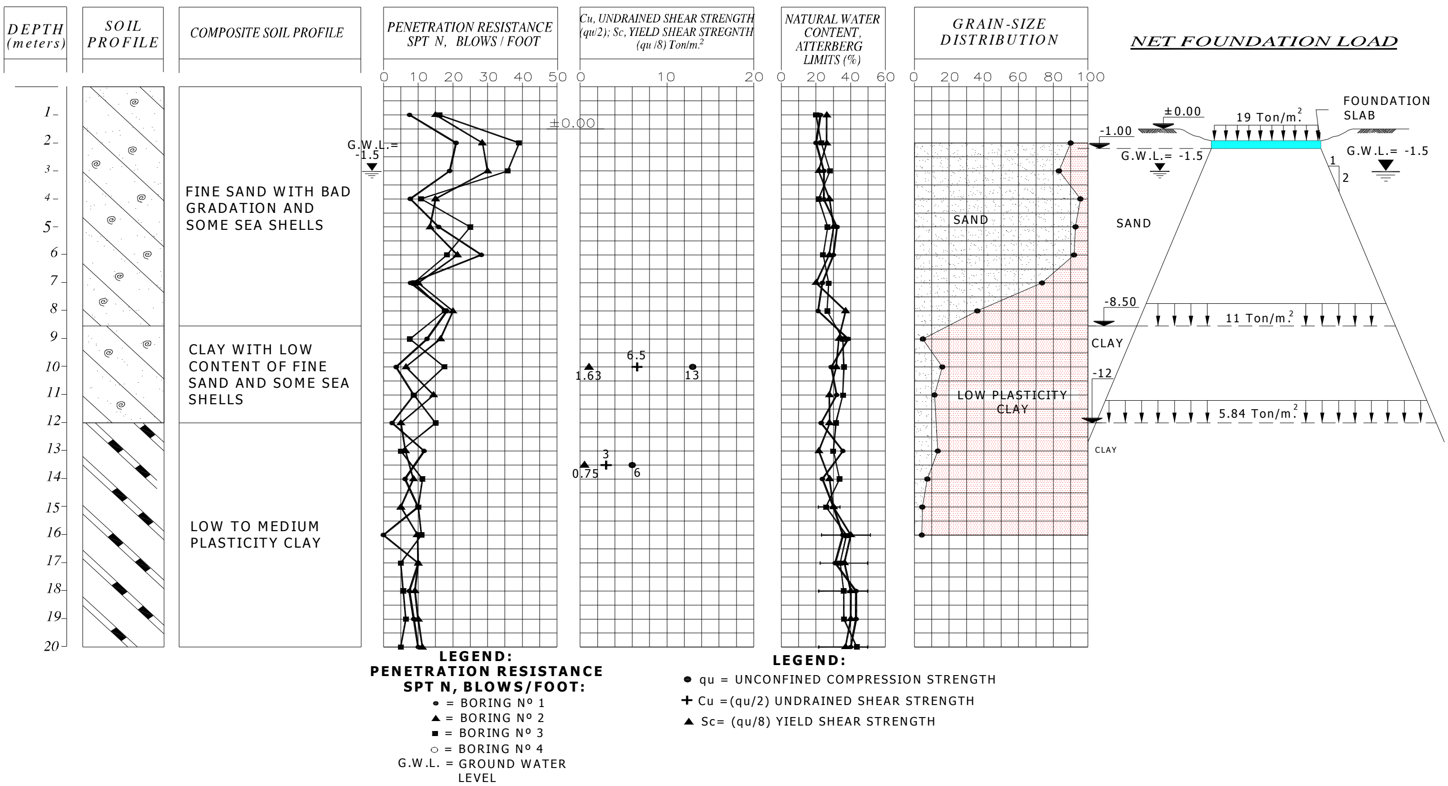
ATTACHMENTS

A. composite soil profile, La Previsora Bank, Guayaquil, Ecuador, 1992



B. COMPOSITE PROFILE. "ISLA DE ORO" BECH RESORT CONDOMINIUM, RIO CHICO, EDO. MIRANDA, VENEZUELA



C. TOTAL SETTLEMENT CURVE, "ISLA DE ORO" BEACH RESORT CONDOMINIUM, RIO CHICO, EDO. MIRANDA, VENEZUELA