AN APPARATUS FOR MEASURING THE STRENGTH OF SOILS IN PLACE

BY

LOUIS FRANÇOIS MÉNARD

Ingenieur Civil, Ecole Nationale des Ponts et Chaussees, (France)

THESIS

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INTRODUCTION

Introduction.

In the last decades, particular emphasis has been stressed on the necessity of a simple method to compute the bearing capacity of a foundation as a function of the settlement. To solve this problem, the foundation engineer needs to use an elasto-plastic theory and to test the elastic properties of the soil.

Consequently, an extensive program of tests has been conducted with a new apparatus, called "Pressiometer". Then, it was possible to study the behaviour of the "in-situ" soil when it was submitted to a field of stresses. The variables beeing not only the cohesion and the angle friction, but also the modulus of elasticity and the degree of saturation.

The program was very broad, and soils such as glacial clay, fluvial clay, compacted clay, glacial till, loess and sand were tested.

It is felt investigation of soil resistance with this method has improved our knowledge of the behaviour of actual foundation structures: it is proved that the elastic properties of the soil are very important and should be measured and taken into account for the study of a foundation project.

2) Object and scope.

The general purpose of this investigation was to explore the behaviour and strength of the soil, when it is submitted to a special field of stresses.

The stress distribution is produced by the following process: a device is lowered into a bore hole, to the desired depth; a fluid is forced into the device in order to apply an uniform lateral stress on the wall of the bore hole. The diameter of the hole increases according to the quantity of the liquid poured. The strain is plotted versus the stress on the diagram.

From the diagram an estimate of the index properties of the soil can be made at the considered depth, exactly as a load test permits an estimation of the bearing capacity of the first feet of soil.

This new method of subsurface investigation would not seem very interesting at first sight, because of the concurrence of well known apparatus, such as the Vane test for clays and the standard penetration for sands. However, we strongly recomend it because of the three following reasons:

1) A theorical interpretation of the curve "strain versus stress" gives immediately the values of the cohesion, the friction angle and the modulus of elasticity.

2) In order to apply on the wall of the bored hole a cylindrical stress distribution, an inexpensive and ~~ready~~ very easy to use device called pressiometer is available.

3) The soil in the vicinity of the apparatus, is submitted to a stress distribution, which is nearly the same as the distribution applied under a point bearing pile.

Actually it is possible to notice on the plot "settlement versus loading" of a point bearing pile some of the characteristics of the pressiometer diagram.

An elementary theory of the pressiometer and its application to test in the field are introduced in this paper.

At the end of this study, it will be possible to compare the results and to draw the following conclusions:

a) The pressiometer is a very precise method of subsurface exploration.

b) The bearing capacity increases with the modulus of elasticity of the soil.

3) Summary.

The scope of this thesis is to:

-describe briefly the equipment used in the field.

-give some developments of the main elasto-plastic theory.

-introduce the method used for analysing the diagrams "strain versus stress".

-show the influence of the elastic properties on the bearing capacity of the soil.

-describe and give the results of tests carried out on clays, tills, loess and sands.

4) Acknowledgments.

This projet was carried out in the Soil Mechanics Laboratory of the Engineering experiment station at the University of Illinois.

General direction for the investigation was given by Dr.R.B.Peck, Research Professor of Soill Mechanics.

Professor J.Kerisel, of "Ecole Nationale des Ponts et Chaussées" of Paris, supervised the work in its early stage, up to September 1955.

Appreeition is expressed to M.Y.Lacroix, research assistant in Civil Engineering for his cooperation in the realisation of the test program.

5) Notation.

Nr	The horizontal radial stress.
Ns	The horizontal circumferential stress.
	All normal and total stresses.
Nz	The constant vertical stress.
r	Distance of appoint to the axis of the hole.
ρ	The initial radius of the hole.
Rp	The outer diameter of the zone of plastic equilibrium.
U	The axial displacement of the wall of the bore hole.
P_0	The natural horizontal radial stress on the limit cylinder of plastic equilibrium.
p	Applied pressure.
pl	Ultimate pressure.

p_o	Pressure at the beginning of the plastic phase.
E	Modulus of elasticity.
σ	Coefficient of Poisson.
C	Cohesion.
$\mathcal{R}$	Radius of a Mohr circle.
$\mathcal{R}_o$	Radius of a particular Mohr circle.
V	Volume of liquid in the central cell, corresponding to the axial displacement U.
ε	Base of natural logarithm.

DESCRIPTION OF THE APPARATUS.

1) Introduction.

The first prototype was built in Paris by the "Ecole Nationale des Ponts et Chaussées", and was brought over in the United States after some tests in the West of France.

In November 1955 very important experiments were made in the clays of Chicago to verify the validity of the main theory and to obtain very precise data in order to build another prototype.

In February 1956, with the help of the University of Illinois, a second type of apparatus was constructed with the following characteristics: diameter 2 inches, a single pressure gauge and automatic levelling of the pressure in the three cells. In the meantime, a very good solution was given to the problem to find out very resistant rubber sleeves.

As it was not required to test the soil at large depth, and that it was expensive to drill deep holes, a very cheap device was specially constructed and used for most of the tests described in this paper. There is no difficulty to improve the device in order to carry out experiments at greater depth.

2) Description.

The device lowered into the hole to the desired depth consists of a metal tube with a rubber membrane tightly fixed around it.

Since the length of the cell is constant, an increase in the average diameter of the rubber membrane corresponds to an increase in volume of the cell.

Two tubes are fitted into the cell, one supplying fluid under pressure and the other measuring the pressure and draining the apparatus after the end of a serie of experiments.

The effect of the strength of the rubber membrane has a negligeable action on the results.

The water is injected into the cell by means of compressed air device, connected to the top of the apparatus. The displacement of the wall of the bore hole corresponds to the increase of volume of the cell and is measured by the variation of the water level in the upper part of the device.

A pressure gauge indicates at all times the pressure in the cell. The data can be recorded by an automatic device; this is very important to measure very slow or very fast increase in pressure.

To keep the principal cell from expanding parallel to its axis, and mainly to get a nearly cylindrical stress distribution in the vicinity of the measuring cell, two outer cells are fitted to the first one.

The additional membranes can be attached to the same cylinder or fixed on separated ones, which are then bolted together.

As a matter of course, the data are derived from the main cell.

Fig(1) shows the actual design of the metallic body of the prototype number two.

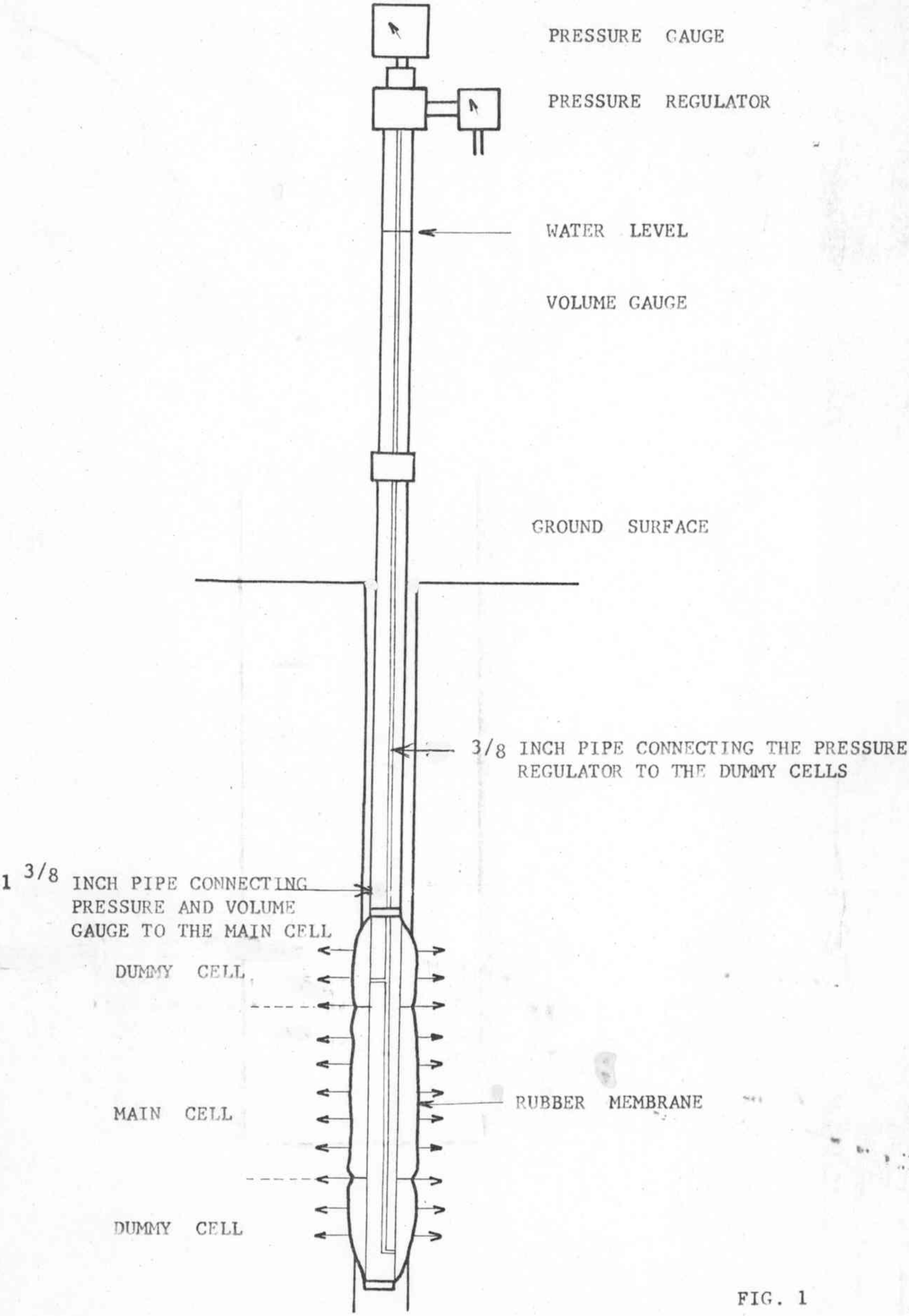

PRESSURE GAUGE
PRESSURE REGULATOR
WATER LEVEL
VOLUME GAUGE
GROUND SURFACE
3/8 INCH PIPE CONNECTING THE PRESSURE REGULATOR TO THE DUMMY CELLS
1 3/8 INCH PIPE CONNECTING PRESSURE AND VOLUME GAUGE TO THE MAIN CELL
DUMMY CELL
RUBBER MEMBRANE
MAIN CELL
DUMMY CELL

FIG. 1

The rubber membranes are not designed on this figure. All cylinders are bolted together with a tube connecting the pressure gauge to the main cell. The outer cells are directly connected with small tubes to the upper part of the apparatus. In this way the same pressure is obtained int the three cells; however, the outer cells are injected only with air and there is no volume interaction between the main cell and the outer ones.

THEORY.

An elementary application of the elasto-plastic theory is exposed in this paper.

Hypothesis.

1) The Mohr's concept of the stress conditions for failure is justified only if tthe cohesion of the clay subject to investigation is a constant of the material; however all plastic material has a shearing resistance that varies with the velocity at which the shearing strain occurs. The following results are strictly valid only if a certain speed of shear is maintained all through the experiment. We assume that for a given velocity of shear, the shearing resistance of a clay is constant.

2) For most of the soils the shearing resistance is a function of the strain. On the curve " strain versus stress" of a not remolded soil, the shearing resistance is maximum for

a given value of strain and decreases down to an assymptotic limit for high values of strain (Fig 2b)

For these reasons it is difficult to maintain the Coulomb's concept of a cohesion and a friction angle. For instance the resistance of loess material or of pretty much compacted sand is much larger for smaller strain than for larger strain. In account of these facts, the rough application of Coulomb's concept may be in some cases somewhat confusing.

3) The initial modulus of elasticity is defined as the ratio between the stress and the corresponding strain, at the beginning of the elastic range. Although no measure has ever been made, this modulus of elasticity may change with the rate of strain. But practically, these variations are without influence and we will suppose that the initial modulus of elasticity of a not disturbed soil is constant.

Theory.

If a pressure p is applied on the wall of a bore-hole, a stress distribution of revolution is resulting in the soil. As three cells are used, the distribution is quite cylindrical all around the central cell and we have only a two-dimension problem to handle.

By comparison with the triaxal apparatus, which has various aspects quite similar to the pressiometer, the pressiometer tests are quick consolidated tests, with an amount of consoli-

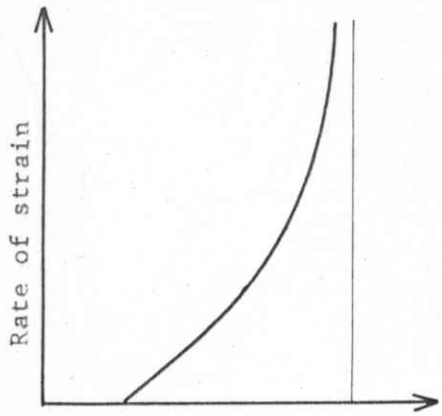


Fig.2a

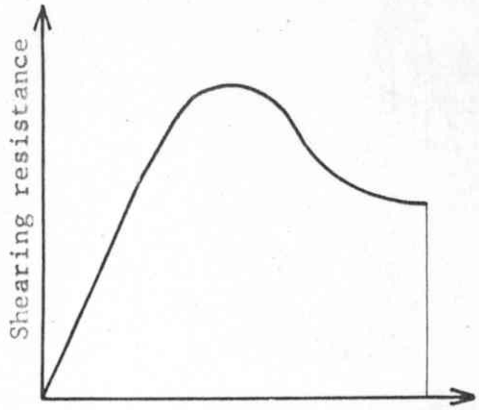


Fig.2b

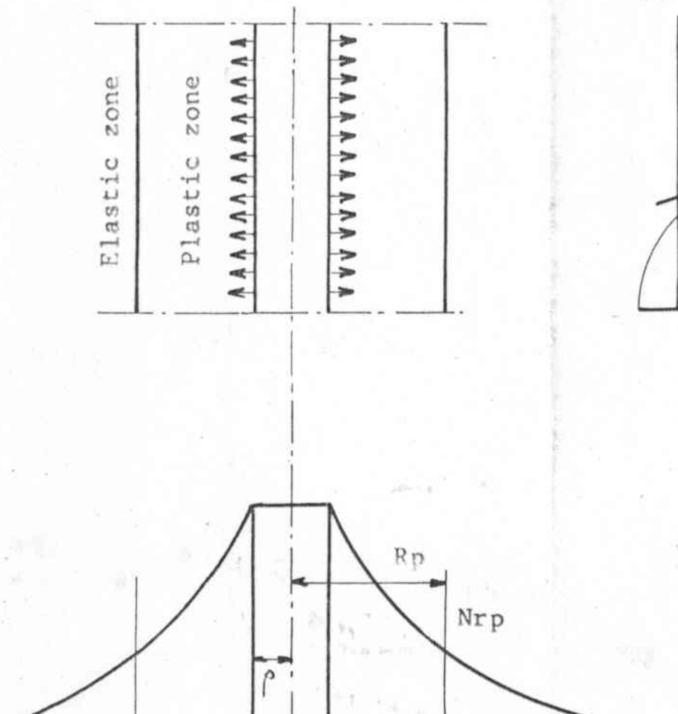


Fig.2c Distribution of stresses in plastic and elastic zones.

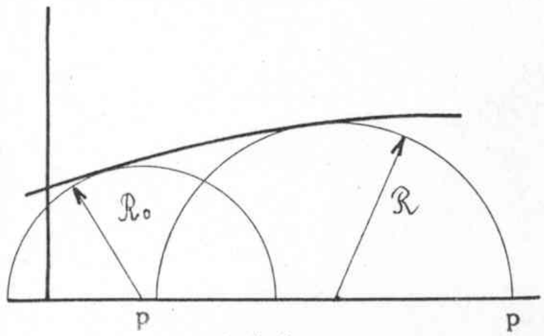


Fig.2d Mohr's Concept.

Fig.2

dation corresponding to the actual one existing in the field.

As the test is carried in the soil in situ, the operator has not to be worried by the disturbance, caused during the sampling of the testing processes.

An elementary theory will now be introduced; it is called "elementary" because of a lot of refinements, specially in the elastic phase, might be used without special difficulty. But practically these refinements are not needed; consequently the so-called elementary theory has always been used with success for the analysis of the test results.

Two states of equilibrium have to be considered. In the plastic zone, the closest to the cell, the soil is strongly sheared under high pressure and the theory of plasticity has to be used. Outside this zone, the stresses are smaller and a so-called theory of elasticity has to be used. Some people often question the elastic theory, but in this particular case the experiments show that the theory may be used with great success.

Hence the theory of plasticity will be used to compute the magnitude and the distribution of stresses inside the plastic zone, limited by a cylinder of radius Rp and the so-called theory of elasticity will be used to compute the stresses and strains outside this cylinder in the elastic zone. (Fig. 2c)

Assuming the magnitude of the horizontal radial stress is known on the limit cylinder of plastic equilibrium, it is possible to compute the axial stress and strain at any point taken outside the limit cylinder; the theory is very often used in structure to compute the resistance of piles, and in Civil Engeenering, to verify the stability of tunnels.

At the boundary between the zones of plastic and elastic equilibrium, the displacement is:

(1) $$U_{RP} = \frac{1+\sigma}{E}\left(N_{RP} - p_o\right) R_P$$

and the magnitude of the principal stresses is given by:

(2) $N_R = N_{RP}$ (3) $N_s = 2p_o - N_{RP}$

We know that p_o is the natural horizontal stress in the soil at the depth of the test. It is often assumed that there is a relationship between this horizontal stress p_o and the vertical stress at the same point; we do question strongly this statement because the surconsolidation or drying or compacting effects may be very important and it is felt that there is no relationship at all between the natural horizontal stress in the soil and the corresponding weight of the soil above the considered point.

There is no difficulty to see on the formula (2) that the stress N_s is positive in the natural state and is decreasing as the pressure is applied; as a matter of fact there is a good chance that this part of the soil failed by traction rather than by shear; this result is very easy to show on the pressiometer diagrams.

In the plastic zone, the stresses are immediately computed from the two following equations:

(4) $N_r - N_s = 2\mathcal{R}$ (Mohr's concept)

(5) $N_r - N_s = -r\frac{dN_r}{dr}$ (Equilibrium equation of an element of soil adjoining a

cylindrical section having an arbitrary radius r)

By combining these two equations, it is possible to determine the stress function which satisfies the boundary conditions of the problem.

(6) $$dN_r = -\frac{dr}{r} \times 2\mathcal{R}$$

-In the special case where the soil is assumed to have a constant shearing resistance c, the equation (6) is immediately resolved:

(7) $$N_r = p - 2c \log \frac{r}{\rho}$$

(8) $$N_s = p - 2c\left(1 + \log \frac{r}{\rho}\right)$$

log: natural logarithm

-In the general case, the shearing resistance is a function of the percentage of strain and of the axial stress; in this theory, we do not assume anything on the magnitude of the radius R of Mohr's circle.

The stress distribution is determined by the equations:

(9) $$\int_{N_r}^{p} \frac{dN_r}{2\mathcal{R}} = \log \frac{r}{\rho}$$ (10) $$N_s = N_r - 2\mathcal{R}$$

The contact stresses between the elastic and plastic zone must verify the conditions for the plastic equilibrium of the soil located inside limit cylinder and the conditions for the elastic equilibrium of the soil located beyond the boundary. Hence, we obtain the value of the stress Nrp and the value of the radius Rp of the limit cylinder.

The solution needs to satisfy the equations (2) and (3) which are valid ~~IF stresses~~ for elastic material, and the equations (4) and (5) which are valid if stresses exceed the yield point.

Combining these equations together, we obtain:

$$(11) \qquad N_r = N_{rp} = \mathcal{R} + p_0 \qquad\qquad N_s = -\mathcal{R} + p_0$$

The radius Rp of the cylinder of transition is determined by the relations:

$$(12) \qquad R_p = \rho \times \varepsilon^{\frac{p - p_0 - c}{2c}} \qquad \text{for clays}$$

$$(13) \qquad R_p = \rho \times \varepsilon^{f(\mathcal{R} + p_0)} \qquad \text{where } f(\mathcal{R} + p_0) = \int_{\mathcal{R} + p_0}^{p} \frac{dN_r}{2\mathcal{R}}$$

The distribution of stresses either in the plastic phase or in the elastic phase is well determined; in the second part of the theory the strain distribution and specially the increase in diameter of the bore hole is computed.

The strain distribution is different according to percentage of saturation of the soil. If the voids of soil are completely filled with water, the material is incompressible; if the soil is not completely saturated, it is compressible and a special theory has to be used in this case.

1) <u>Incompressible soil</u>.

This case is encountered in most of the clays, and in general with slowly permeable soils.

Urp is the axial displacement of the soil located on the cylinder of transition and U is the correlative displacement of the wall of the bore hole.

Since the soil is incompressible, the volumes (1) and (2) represented on the Fig.3a

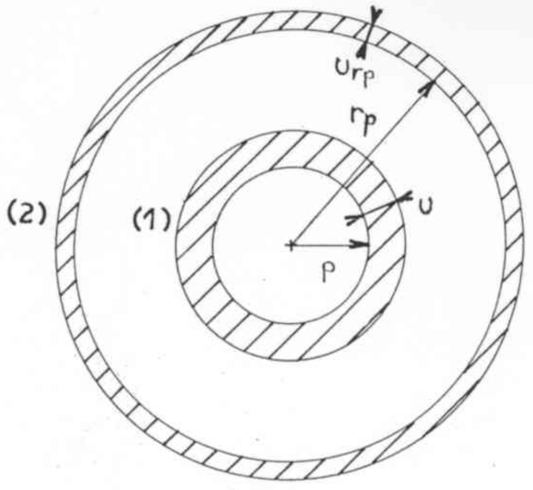


Fig.3a

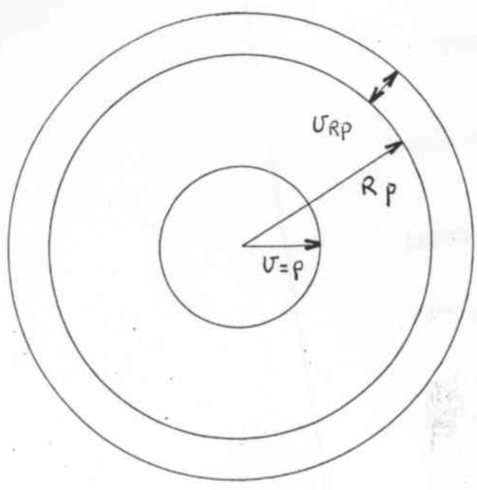
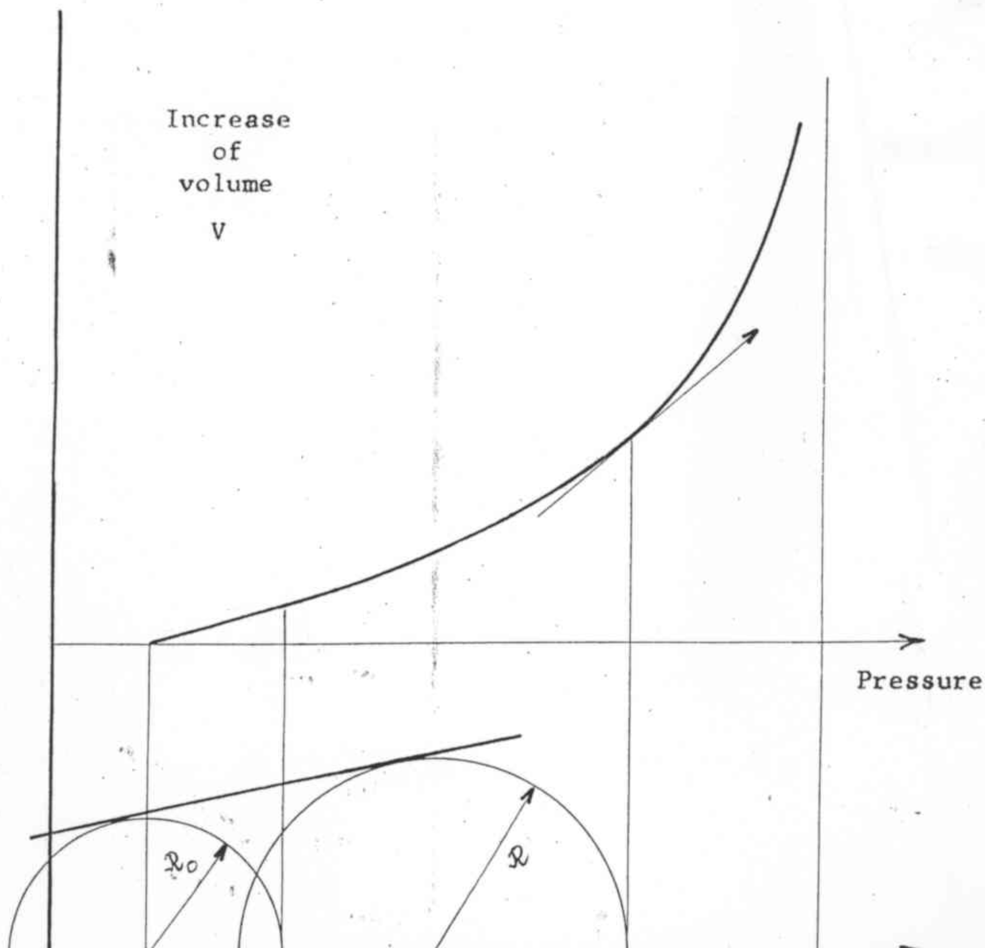


Fig.3b



CORRELATION BETWEEN THE TEST CURVE

AND THE MOHR'S DIAGRAM

Fig.3c

have equal magnitudes and we can write:

$$(14) \qquad \rho \times U + \frac{\bar{U}^2}{2} = R_p \times U_{R_p} + \frac{\bar{U}_{R_p}^2}{2}$$

a) If the quantities U and U_{R_p} are very small, compared to the quantities ρ and Rp, the equation (14) might be replaced by the following relation:

$$(15) \qquad U = \frac{R_p}{\rho} \times U_{R_p}$$

If we substitute the values of Rp and Urp in this relation, we obtain:

$$(16) \qquad U = \rho \times \frac{1+\sigma}{E} \times \mathcal{R}_o \times \varepsilon^{2f(\mathcal{R}+p_o)} \quad \text{where} \quad f(\mathcal{R}+p_o) = \int_{\mathcal{R}+p_o}^{p} \frac{dN_r}{\mathcal{R}}$$

For the clays with constant shearing resistance, the relation (16) becomes:

$$(17) \qquad U = \rho \times \frac{1+\sigma}{E} \times c \times \varepsilon^{\frac{p - p_o - c}{c}}$$

From this equation, we can deduce that the axial displacement of the wall is:

1 Proportional to the initial radius of the bore hole.

2 Indirectly proportional to the modulus of elasticity E.

3 Increasing exponentialy with the applied pressure, in the case of a constant shearing resistance C.

In fact, it is more convenient to use the formulae (16) and (17) in the differential form:

$$(18) \qquad \frac{dU}{U} = \frac{dp}{\mathcal{R}}$$

This very simple relation is fundamental in the application of the theory to the analysis of the pressiometer diagram. As ~~a matter~~ this relation is obtained from the conditions that must be satisfied for plastic equilibrium, it is rigorous.

The result of the above equation has to be considered only during the plastic range; during the elastic phase the relationship between the applied pressure and the deformation of the drill hole is:

(19) $$U = \frac{1+\sigma}{E} \cdot (p - p_o) \times \rho$$

b) When the range of the deformation becomes quite similar to the primitive diameter of the hole, the equation (15) is inaccurate, if we assume that the initial diameter of the drill hole is small and that the deformations are large, we obtain the following relation:

(20) $$\frac{U^2}{2} = R_p \times U_{R_p} \qquad \text{(Fig.3)}$$

Introducing the value of Rp and Urp in this relation, we obtain:

(21) $$\frac{U^2}{2} = \frac{1+\sigma}{E} \mathcal{R}_o \times U^2 \times \varepsilon^{2f(\mathcal{R}+p_o)} \qquad \text{where } f(\mathcal{R}+p_o) = \int_{\mathcal{R}+p_o}^{p} \frac{dN_r}{\mathcal{R}}$$

This relation is indetermined and U is increasing indifinitely when the pressure reaches the limit value p_ℓ given by the equation:

(22) $$\int_{p_o+\mathcal{R}}^{p_l} \frac{dp}{\mathcal{R}} = \operatorname{Log} \frac{E}{2(1+\sigma)\mathcal{R}_o}$$

For clays of constant cohesion, we obtain the formula:

(23) $$p_\ell = p_o + c\left[1 + \operatorname{Log} \frac{E}{2c(1+\sigma)}\right]$$

We can see from the equation (22), that the magnitude of the net pressure $p_\ell - p_o$ is independant of the depth of the test.

This relation is very important, and its verification was considered as one of the main point of the test program. We will see in the application of the theory to the analysis of the diagram, that this relation is used specially to verify the results given by some main formulae already exposed in the beginning of the theory.

2) Compressible soil.

In this case the mathematical approach of the problem is a little more complex, because the formula (14) is no more valid and the decreasing of the volume due to the compressibility of the plastic zone has to be taken into account.

Only the main results are exposed in this paper.

Similarly to the relation (18), the equation (24) is fundamental:

$$\frac{dV + \frac{(1-\sigma)\rho}{E} \times dp}{V + \frac{(1-\sigma)\rho}{E}(p-p_0)} = \frac{dp}{\mathcal{R}} \tag{24}$$

Corresponding to the relations (22) and (23), we obtain the equations:

$$\int_{p_0+\mathcal{R}}^{p_\ell} \frac{dp}{\mathcal{R}} = \mathrm{Log}\,\frac{E}{4\mathcal{R}_0} \tag{25}$$

$$p_\ell = p_0 + c\left[1 + \mathrm{Log}\,\frac{E}{4c}\right] \tag{26}$$

APPLICATION OF THE THEORICAL RESULTS

TO THE ANALYSIS OF THE PRESSIOMETER DIAGRAMS

From the diagram "strain versus stress" recorded during the test, the values of the cohesion, the friction angle and the modulus of elasticity of the soil are obtained with a very good precision. The theorical interpretation of the diagram is a direct application of the elasto-plastic theory already exposed.

Three phases have to be considered during the test: an elastic phase, a plastic phase, and an ultimate phase. To each of these phases, corresponds a part of the diagram (Fig.3)

1) Elastic phase. $p_o < p < p_o + \mathcal{R}$

When the applied pressure increases from p_o to $p_o+\mathcal{R}$, there is not yet any plastic failure in the soil around the cell, and the curve is a straight line even at the end of the phase.

There is a direct relationship between the slope of the straight line and the value of the modulus of elasticity:

$$(27) \qquad \frac{E}{1+\sigma} = K \frac{dp}{dV}$$

K is a characteristic of the cell and V is the variation of volume in cm3 which corresponds to the displacement U.

For a cell of 2" diameter and 8" height, K=785.

In clays, the limits of the elastic phase are p_o, ~~the limits of the elastic phase are~~ p_o, the natural horizontal stress, and p_o+c.

2) Plastic phase. $p_o+\mathcal{R} < p < p_\ell$

If the shearing resistance of the soil is constant, and if there is no failure in tr traction, the representative curve is an exponential one.

At each point of the diagram corresponds one Mohr's circle with the relations;

a) for incompressible soil

$$\frac{dU}{U} = \frac{dp}{\mathcal{R}} \qquad \overline{oc} = p - \mathcal{R}$$

b) for compressible soil:

$$\frac{dU + \frac{(1-\sigma)\rho}{E}\,dp}{U + \frac{(1-\sigma)\rho}{E}(p-p_o)} = \frac{dp}{\mathcal{R}} \qquad \overline{oc} = p - \mathcal{R}$$

In fact it is more convenient to use this relation integrated between two close values p1 and p2.

For the incompressible soil, for instance, the formula is $\mathcal{R}_{p_1}^{p_2} = \dfrac{p_2 - p_1}{LU_2 - LU_1}$

where $\mathcal{R}_{p_1}^{p_2}$ is the mean value of $\mathcal{R}$ between the pressure p_1 and p_2.

Before beginning the computations, the value of p_ℓ has to be determined; generally this point is situated at the beginning of the straight line; an error on the appreciation of the natural pressure introduces a very small error in the computations of the shearing resistance. The use of rebound curve or the relaxation method are more accurate, but these refinements are not needed in the practical tests.

3) Ultimate phase.

When the pressure reaches the limit value p_ℓ , the diameter of the drill hole increases very rapidly; whatever large is the displacement, the soil cannot sustain a pressure larger than the limit value p_ℓ .

The theory shows that the value of limit pressure is given by the following relations:

$$p\ell = p_o + C\left[1 + L\,\frac{E}{2(1+\sigma)C}\right] \longrightarrow \text{for the saturated clays of resistance C.}$$

$$p\ell = p_o + C\left[1 + L\,\frac{E}{4C}\right] \longrightarrow \left\{\text{for the compressible clays of constant shearing resistance C.}\right.$$

$$\int_{p_o+\mathcal{R}}^{p\ell} \frac{dp}{\mathcal{R}} = L\,\frac{E}{4C} \longrightarrow \text{for the compressible soils.}$$

$$\int_{p_o+\mathcal{R}}^{p\ell} \frac{dp}{\mathcal{R}} = L\,\frac{E}{2(1+\sigma)C} \longrightarrow \text{for the incompressible soils}$$

It is to be noticed that to have a good agreement between the results of the plastic phase and the ultimate phase, it is recommended to keep constant the rate of strain during the whole test; this has been practically realized without any difficulty with the prototype used for the experiments.

VERIFICATION OF THE THEORY

ON THE ULTIMATE PRESSURE

One of the most important theorical results is the analysis of the ultimate resistance of the soil, when it is submitted to an increasing pressure.

The theory shows that two important cases have to be distinguished: incompressible soils and compressible soils.

1) Incompressible soils.

Most of the incompressible soils encountered during the tests were saturated clays.

We know that it is not absoluralynecessary to assume that the shearing resistance of the clay is constant; but in fact the shearing resistance of the clay is roughly constant, it is a good simplification to assume that it is equal to the cohesion C.

In this case, we know from the theory that the ultimate pressure that the soil can sustain is:

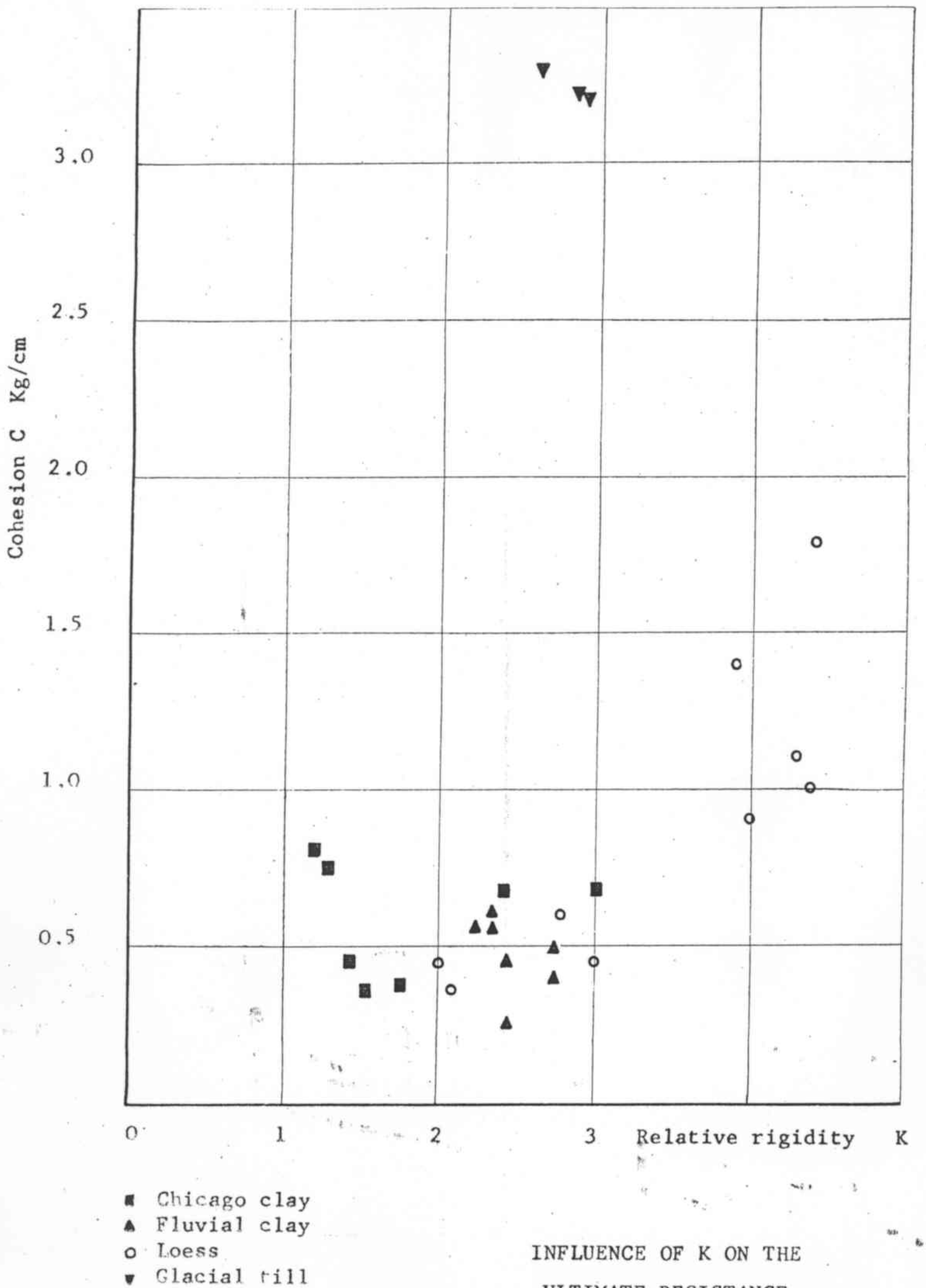
$$p_\ell = p_0 + C\left[1 + L\,\frac{E}{(1+\sigma)\,2C}\right]$$

$p_\ell - p_0$ will be called p_ℓ^* net ultimate pressure

$$p_\ell^* = C\left[1 + L\,\frac{E}{(1+\sigma)\,2C}\right]$$

We will introduce a new characteristic of the soil: the relative rigidity K; by definition the relative rigidity K is equal to $L\,\frac{E}{(1+\sigma)2C}$; it ranges from 1 to 4 for the soils which have been tested with the pressiometer; it is small for soils which have a high

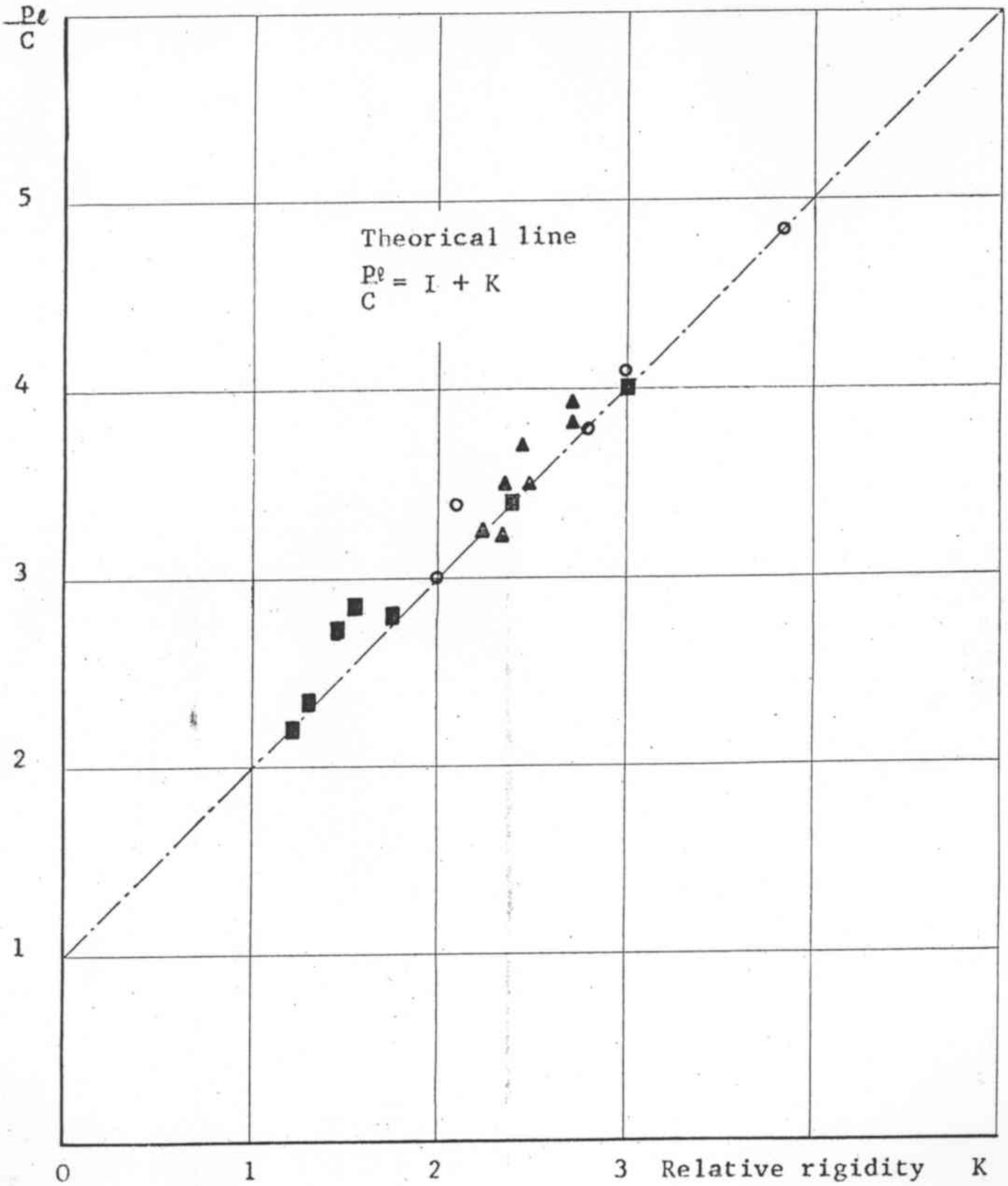
TEST RESULTS



INFLUENCE OF K ON THE
ULTIMATE RESISTANCE

Fig.4

TEST RESULTS



■ Chicago clay
▲ Fluvial clay
○ Champaign clay

COMPARISON OF THE THEORY AND EXPERIMENTS DATA ABOUT THE LIMIT PRESSURE.

Fig. 5

Saturated clays.

SATI ATED CLAYS

COMPA ED RESULTS.

Test Number	Soil type	E	C	E/C	1 + K	$\frac{P_\ell^x}{C}$
3	Fluvial clay	15	.5	30	3.75	3.8
4	" "	6	.27	22	3.45	3.7
5	" "	11	.54	20	3.55	3.5
6	" "	12	.6	20	3.35	3.2
7	" "	12.5	.41	30	3.75	3.9
8	" "	11.0	.45	22	3.45	3.45
9	" "	13	.37	35	3.9	3.65
9 bis	" "	10.5	.57	18.5	3.25	3.25
10	Chicago clay	4	.444	9.1	2.55	2.7
11	remolded by the	3.5	.35	10	2.65	2.85
12	rollin of earth	5	.72	7	2.3	2.85
13	moving equipment	5	.77	6.5	2.2	2.2
34	Fluvail clay	17	.53	32	3.80	3.80
36	" "	16	.43	37	4.00	4.10
37	" "	6	.43	14	3.00	3.00
39	" "	5	.32	15.5	3.10	3.40
42	Chicago clay	27	.68	40	4.00	4.00
45	" "	15	.68	22	3.40	3.40
44	" "	43	.37	11.5	2.75	2.80
43	" "	16.5	.63	27	3.6	3.5
46	" "	14.0	.58	24	3.5	3.45

Fig.6

cohesion but a low modulus of elasticity; it is large for soils with a very high rigidity, compared to the value of the cohesion.

The fig.4 is the result of a tentative of classification of the clays or clayey materials which have been tested with the pressiometer. The cohesion of these soils ranges from 0.4Kg/cm^2 to 3.5 Kg/cm^2 and the relative rigidity from 1 to 4.

The clays of Chicago have a very small relative rigidity, specially the clays encountered in Congress Express-way; it is to be explained in the following of this paper that some earth slides and troubles occured during the construction of the embankements of this Express-way.

On the other side, the loess, which consists of small grains cemented together, has a very high relative rigidity.

The tills of the Wesley Foundation in Urbana, have a high cohesion for a rigidity of 3.

The figure 5 is very important because it is pertaining to one of the most precise verificationof the theory of the pressiometer.

We have represented the ratio $\frac{P_l^*}{C}$ in function of the relative rigidity.

The ratio $\frac{P_l^*}{C}$ of the net ultimate pressure over the cohesion, is increasing with the r relative rigidity.

For a relative rigidity having a value of 1, the ratio $\frac{P_l^*}{C}$ is 2, but for a relative rigidity of 4 the ratio is 4. So the variations are very important.

In the same figure, we have drawn the theorical relationship between K and the ratio

$\frac{p_\ell^*}{C}$ $$\frac{p_\ell^*}{C} = 1 + L \frac{E}{(1+\sigma) 2C} = 1 + K$$

It appears that the correlation is good.

The conclusion of this study would be:

That the relative rigidity of the soil, and in general the elastic properties have to be taken into account to study the stability of the soil.

That the data given by the pressiometer are very precise and that there is a correlation between the theory and the experiments.

2) Compressible soils.

The compressible soils which have been tested are loess, glacial sand (Mahomet, Ill.) and a compacted clay (Champaign, Ill.)

The limit pressure is smaller for a compressible soil than for an incompressible one.

The limit pressure is given by the following equation for an unsaturated clay of constant shearing resistance C:

$$p_\ell = p_o + C\left[1 + L \frac{E}{4C}\right]$$

But and generally, unsaturated soils do not have a constant shearing resistance and the following equation applies in most cases:

$$\int_{p_o + c}^{p_\ell} \frac{dp}{\mathcal{R}} = L \frac{E}{4C}$$

where $\operatorname{Log} \frac{E}{4C}$ is the relative rigidity.

Fig. 7 shows the value of the quantity $1 + \int_{p_0+R}^{p_l} \frac{dp}{R}$ as a function of the relative rigidity.

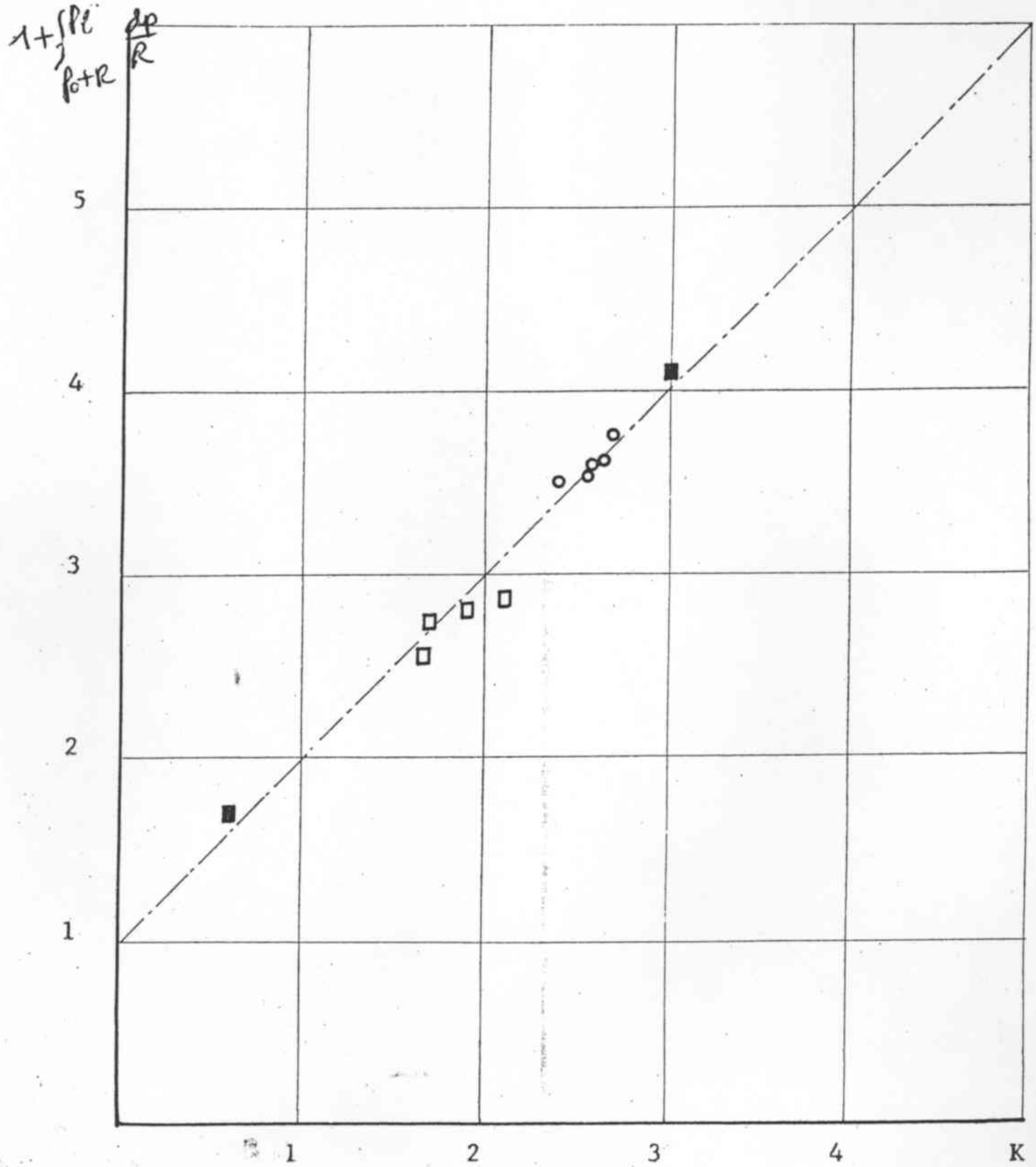
There is a good agreement between the results of the tests and the theory.

It is to be noticed that the ultimate resistance of the till tested in the Wesley Foundation, have not always been reached. Actually, some tests were not carried out

[illegible]

deeply enough, a heave appeared before the usual lateral movement.

TEST RESULTS



COMPARISON OF THE THEORY
AND EXPERIMENTS DATA ABOUT
THE LIMIT PRESSURE

UNSATURATED SOILS

Fig.7

DESCRIPTION OF THREE DIFFERENT TESTS MADE

IN THE FIELD

1.CHICAGO CLAYS.-

The city of Chicago lays on a glacial clay which has been thoroughfully described by the director of this thesis. The two tests series described below were carried out at Congress Expressway and at the site of the Inland Steel Building.

Congress Expressway.

On November 10, 1956, a first test was performed with prototype N°1 at the location of the Congress Expressway construction; more precisely at the intersection of Congress parway and Aberdeen Avenue. The test was made 8'-6" below the ground surface in the medium yellow dessicated clay.

The area had been submitted to a slide, the pressiometer test showed that the modulus of elasticity of the soil was very small, this could explain the movement of the slope.

Some of the tests were made in March 1956, a few hundred feet west of the first test but in the bottom of the excavation. It has been possible to study the modulus of elasticity, the cohesion and the limit pressure as a function of the depth. It can be seen that these characteristics obey to very similar laws. The small values of the first strata can be explained by the swelling of the clay due to the excavation and by the remolding action of heavy equipment.

Inland Steel Building site.

In june 1956 a serie of tests was completed on the site of the new Inland Steel Building at Deaborn and Monroe, right in the "Loop" of Chicago.

The aim of investigation was to measure the remolding action due to the driving of H st steel piles. The tests were made at El-20 (C.C.D.) after that excavtion had made appear the top of the piles driven by mean of a follower.

The tests were made at 1,2, and 3 feet from the pile. The experimental plots (Fig.) gives the increase of volume of the cell versus the pressure.

From the tests, it is seen that the soil around the pile was remolded. The vibrations due to the driving destroy the structure of the clay, though some thixotropic phenomena restore the structure after some time, we see on the table that the elasticity modulus and the stearing strength are smaller than their initial values.

Test number	Distance from the H pile	Relative depth	Modulus of elasticity	Average cohesion	Limit pressure	Theorical limit pressure
44	1ft.	3'4"	6Kg/cm²	0.37Kg/cm²	1.1Kg/cm²	1.05Kg/cm²
45	2	3'4"	27	0.68	2.6	2.6
46	2	1'6"	21	0.58	2.3	2.35
42	3.5	3'4"	41	0.68	3.1	3.1
43	3.5	1'6"	25	0.63	2.6	2.5

The relative depth is taken from the bottom of the excavation which was 20 13 33' below the steel level.

The experimental and theorical limit pressures can be seen on the right end of table . Though the results justify a more complete investigation, we stress the fact that the theory and tests agree completely.

Let consider the vertical plan of the H pilesaxis which passes by the axis of the three bore holes. The different values of the modulus of elasticity and of the cohesion are shown by the lines.

It appears that the cohesion decreases when we go closer to the pile or the bottom of the exxavation. The remolding action due to the pile driving and to the excavation would cause a decrease of the cohesion of the order of 40%.

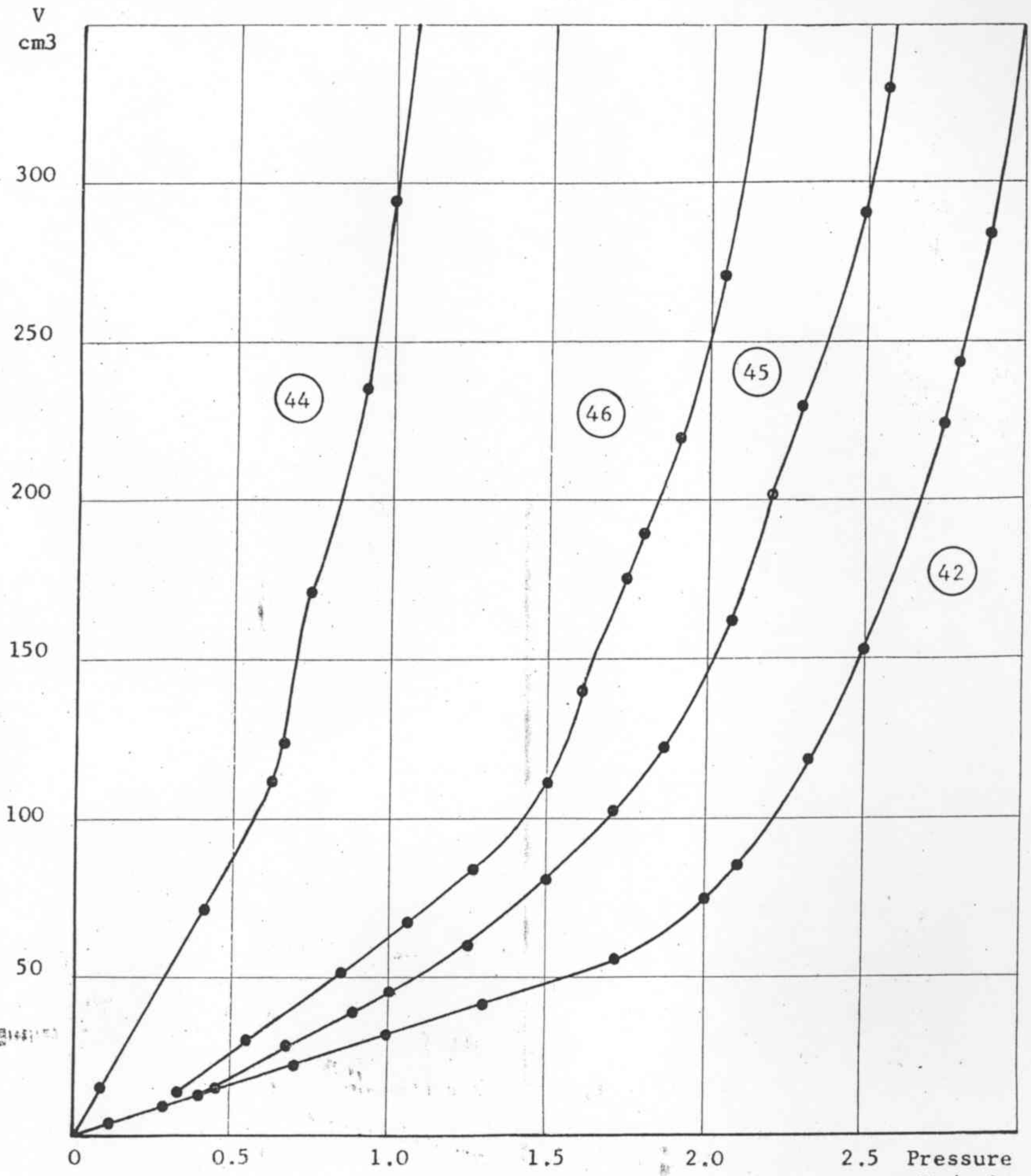
The modulus of elasticity varies much more than the cohesion. It varies from $41 Kg/cm^2$ for the clay in the natural state to 6 Kg/cm2 in the remolded state.

The water content decreases of 1% , which checks the general theory according which a process of consolidation occurs in the remolded clay.

Finally the variation of the limit pressure as a function of the distance to the pile is represented.

It can be concluded that the pressiometric tests preformed in connection with the Inland Steel Building have shown that the mecanical properties of a soil can be deeply altered in account of the construction procedure. The vibrations due to the pile driving and the decompression due to the excavation seem to produce an important remolding

TEST RESULTS

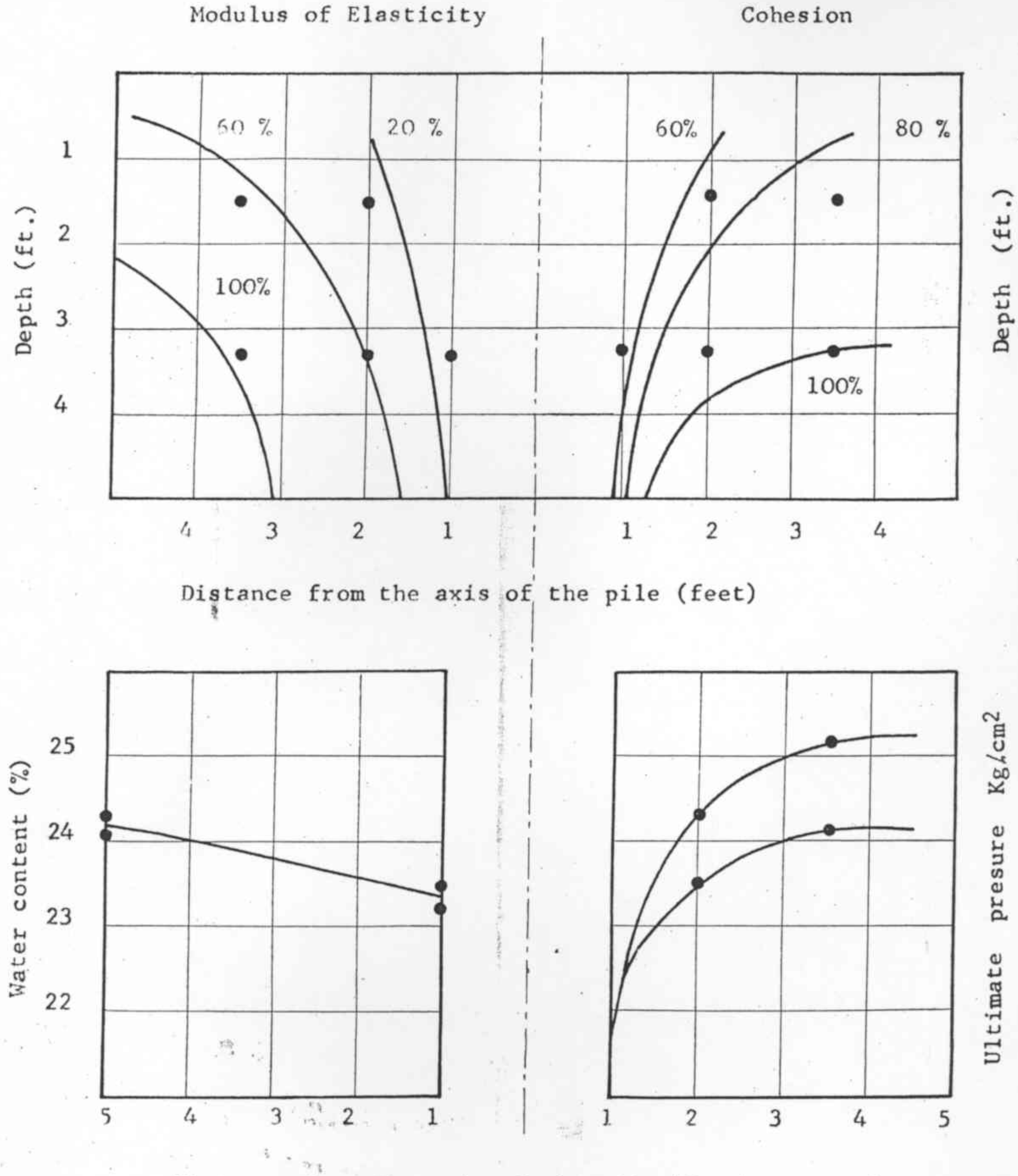


CHICAGO CLAY

(INLAND STEEL BUILDING)

Fig.8

Variation of :



CHICAGO CLAY

(INLAND STEEL BUILDING)

Fig.9

action which will stay several months. Though the essential aim of the tests were to check the pressiometer theory, they gave the possibility to notice and measure with a great precision a phenomenon thought very delicate.

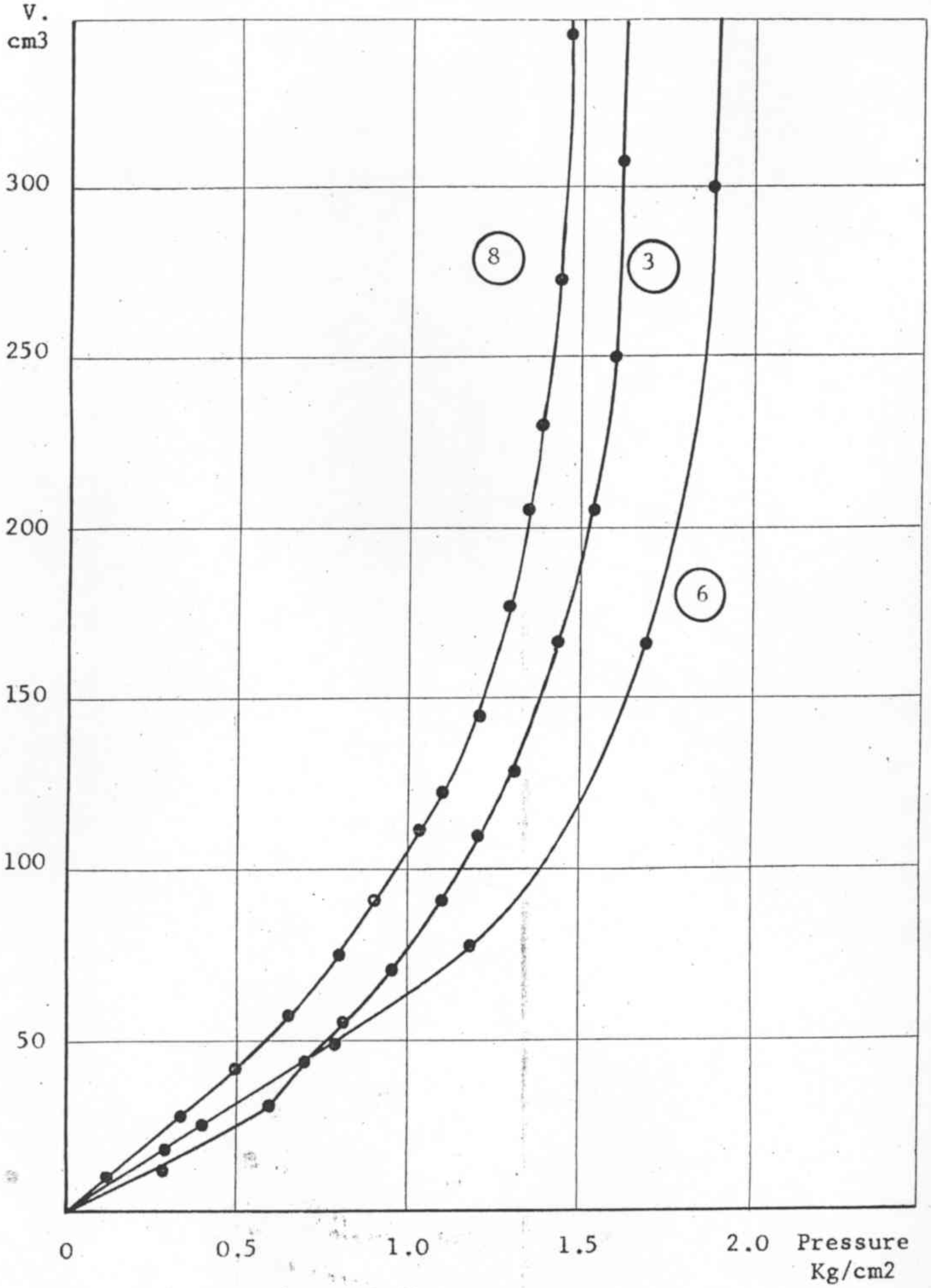
11.FLUVIAL CLAYS (CHAMPAIGN).

A serie of tests have been performed South of Talbot Laboratory, a few feet North of Boyard creek. Several hand borings gave the following results: the top 3 feet are constitued of a sandy clayed silt; from 3 to 5 feet a fluvial grey clay with some pebbles lays above a loose gravelly sand.

On November 23, 1955, the first test was made with the 6" diameter apparatus imported from France. It is worthwhile to remark that the other tests made with the 2" apparatus check perfectly the results given by the 6" apparatus. This shows that the results are iindependant of the diameter of the apparatus what was already demonstrated by the theory.

Some diagrams are represented on Fig.10. All characteristics of the typical curve can be seen on the plot; the straight line of the elastic phase, the small bump at the beginning of the plastic phase, the exponential of the plastic phase, and the limit,pressure. It should beenoticed that the small bump characterizes the tensil resistance of the soil; during the theorical stage it was assumed, as it is generally done, that the tensil strength of the soil had absolutely no influence in the computation of the stability of

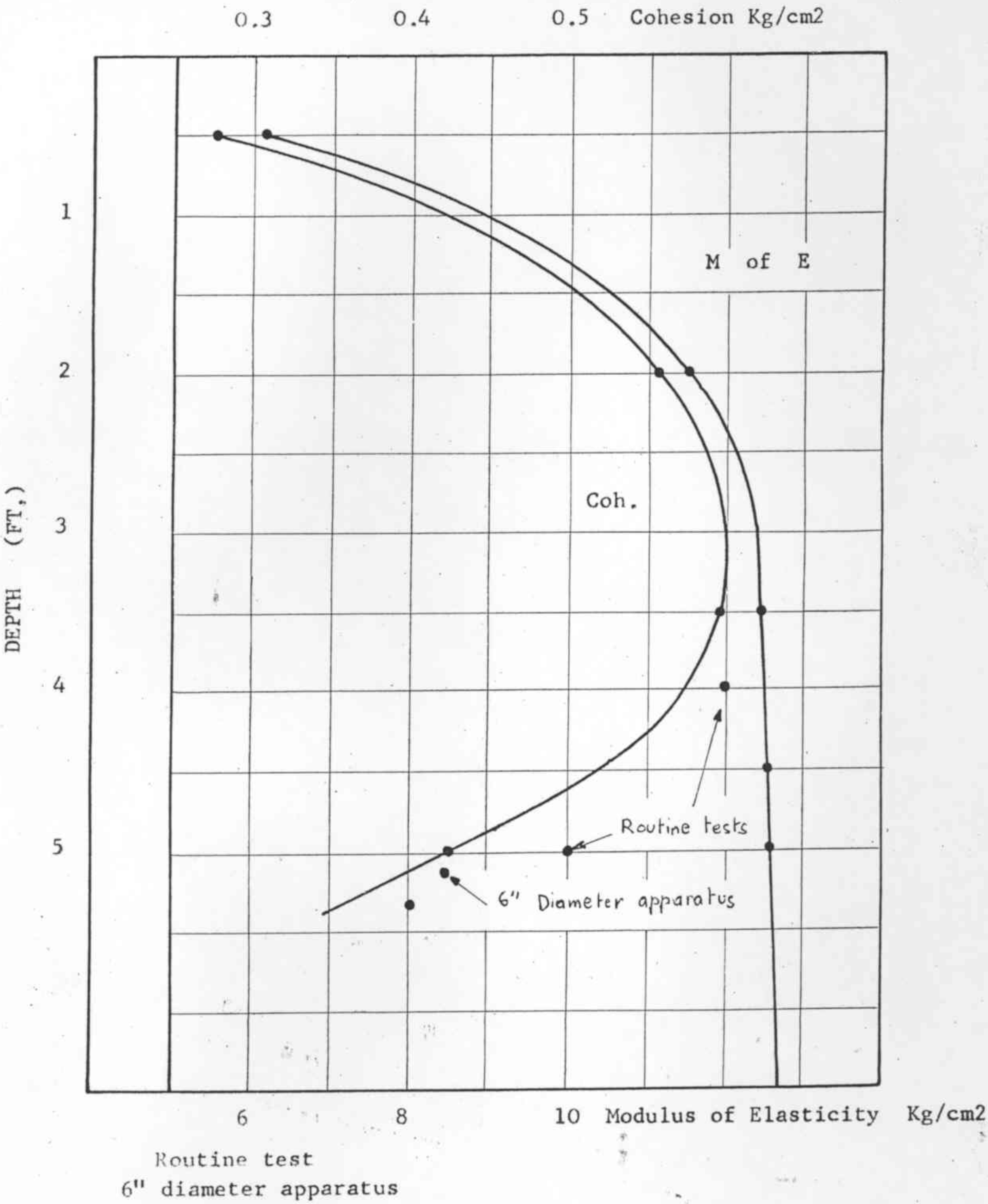
TEST RESULTS



FLUVIAL CLAY

(CHAMPAIGN)

Fig.10



FLUVIAL CLAY

(CHAMPAIGN)

Fig.11

the soil; the results of the tests showed that this assumption was not right and that in several cases a tensile failure occured in the soil. The tensile strength is measured with the same method that the shearing resistance. In general the shearing strength of a soil is approximately half the value of the cohesion.

We have plotted, Fig.11, the value of the cohesion and the modulus of elasticity as a function of the depth. The increase rate is approximately the same for both; however at the bottom of the clay layer, the cohesion is decreasing while the modulus of elasticity stays constant.

A great number of routine tests have been made, they have checked the tests performed with the pressiometer. They prouved a good correlation between the theorical and experimental results.

III. CLAYS. EISNER WAREHOUSE.

On the request of Dr.D.U.Deere, Consultant Engineer, a serie of tests were carried out on the site of the new Warehouse, of Eisner Co. Champaign.

The tests results are shown on Fig.(12) and Fig.(13), where some characteristics of the clay have been plotted versus the depth. It appears that the modulus of elasticity decreases more rapidly than the cohesion from the ground line to the water table. Below the water table, the rate of variation is approximatly the same

It should be noticed once more, that the modulus of elasticity is a very sensitive characteristic of the soil. While it ranges from 23 to 8 Kg/cm² the cohesion ranges from .9 Kg/cm to .65 Kg/cm²

Settlement, due to the first loading, is closely related to the modulus of elasticity. Variation of the late is expected to produce differential settlements.

Unconfined compression test performed on samples coming from the site gave the same rate of variation for the cohesion.

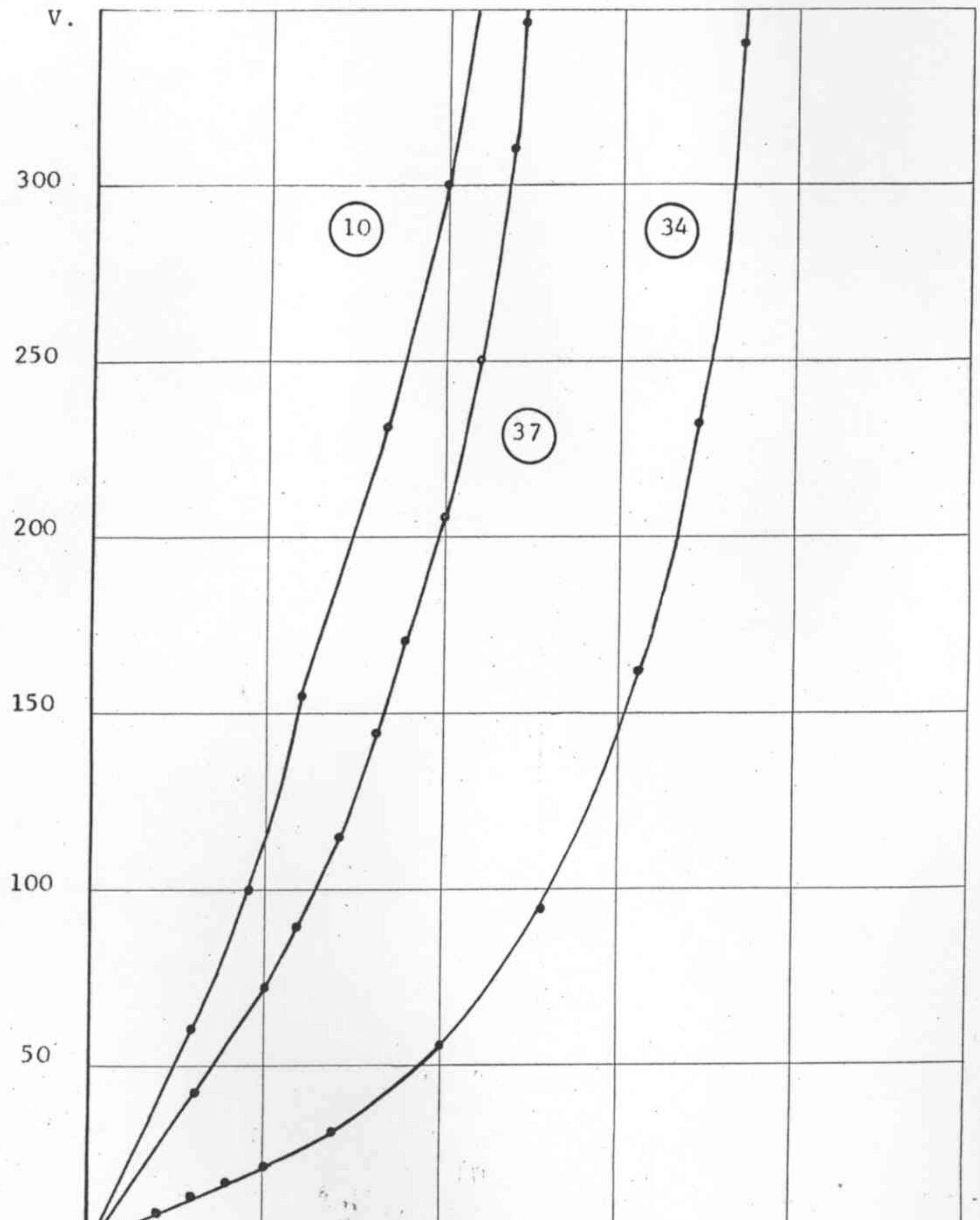
As usual, the relation between the soil characteristics and the limit resistance is always well verified.

Depth	C.Pressiometer	C(U.C.T.)	Modulus of elasticity
1'-6" Natural soil	.55 Kg/cm²	.60 Kg/cm²	25 Kg/cm²
1'-6" " "	.45	.65	23
2'-6" " "	.42	.55	9
4'-0" " "	.40	.45	8
5'-0" " "	.32		7
2'-0" Compacted clay	1.8	1.8	125

U.C.T.: unconfined compressive test(Routine test).

TESTS RESULTS.

TEST RESULTS

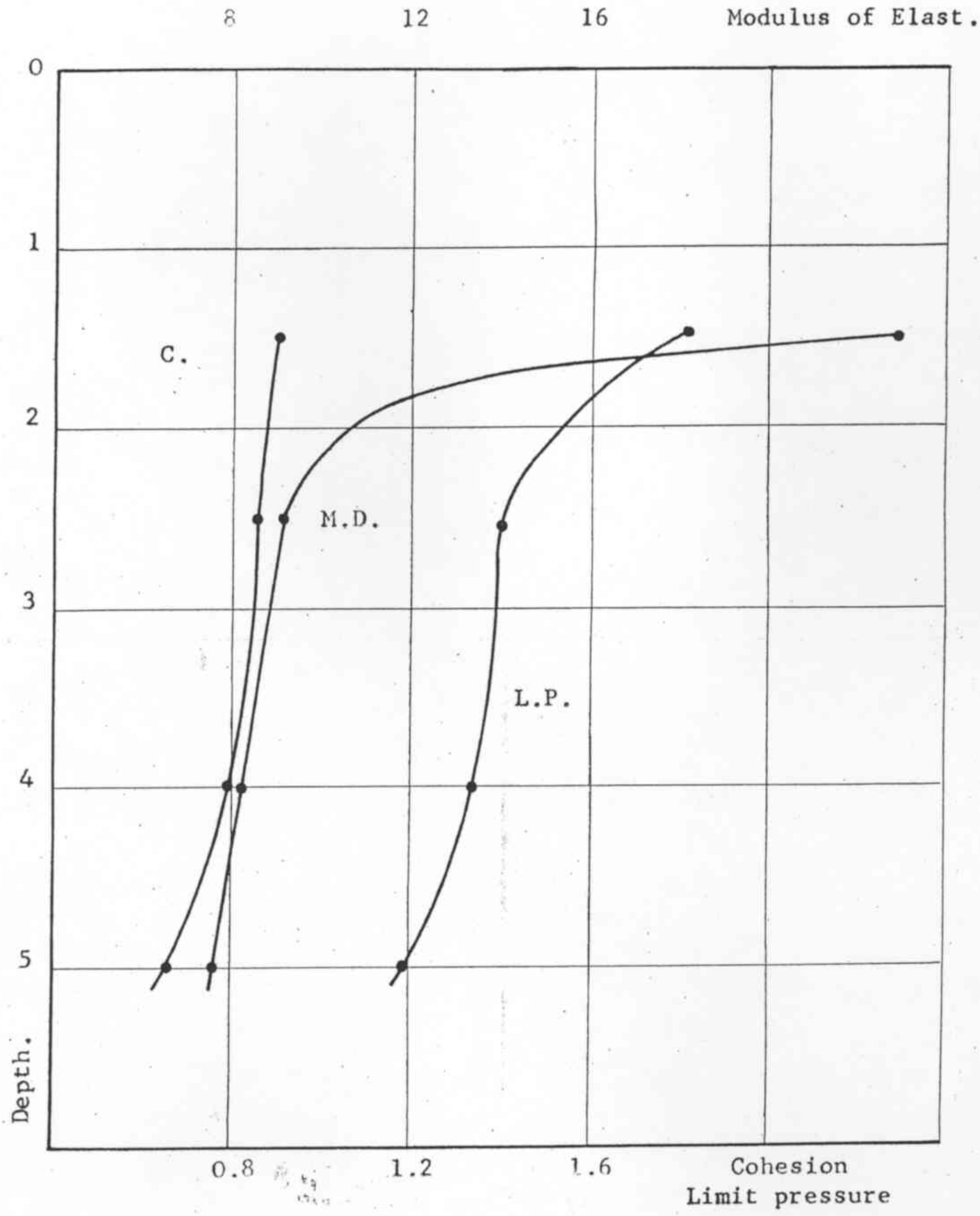


SATURATED CLAYS

EISNER WAREHOUSE

Fig.12

TEST RESULTS



SATURATED CLAYS

EISNER WAREHOUSE

Fig.13

TESTS IN THE LOESS DEPOSIT

A séries of Pressiometer tests have been carried out in the loessial areas of the Mississipi Rivers in Western Illinois. Loess is a wind deposited soil covering vast areas of North America. It is composed of uniform silt size particles, bonded together with relatively small fractions of clay. Two dependable tests of the compressibility of the loess are beleived to be load test and the standard consolidation test. At a certain pressure, designated as the "critical load" a break occurs in the compressibility curve: this break indicates an internal collapse of the structure of the loess. If the pressure exceeds the critical load, the soil becomes very compressible and the settlements are very large.

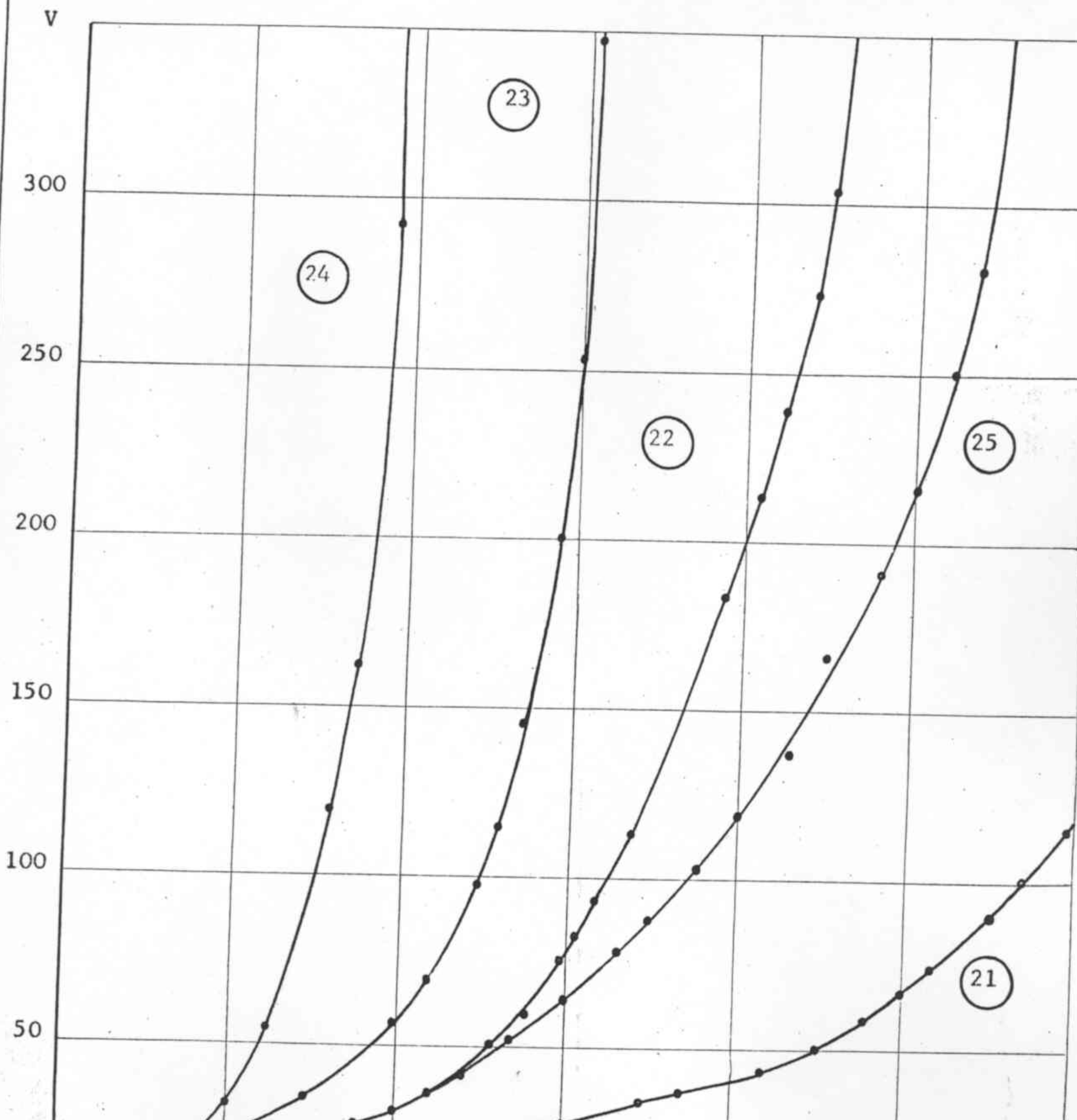
It may be worthwhile to note that the relative rigidity of the loess is very high compared to the other soils tested with the pressiometer.

At the site of the tests some standard load tests have been carried out and one attempts to correlate the results of the load tests with the results of the pressiometer: but as the loess is not regarded as a soil of remarkably constant and uniform properties, the correlation is made only with the average test results.

Modulus of elasticity

On figure 14 are plotted the pressiometric diagram of some of the tests run at the depth of 5 feet. The average modulus of elasticity given by the slope of the curve (21-22-25) during the elastic phase is 94 Kg/cm2. The load tests yield an average value of E = 100 Kg/cm2 (mean value of three tests). Though the results would justify a more complete investigation, it is apparent that the value of the modulus of elasticity given by both tests (load and pressiometer tests) agree well.

TEST RESULTS



LOESS DEPOSITS

Fig.14

Cohesion and friction angle.

These tests give the writer the opportunity to introduce more elaborate formulae to compute the radius of Mohr's circle. The routine formula are valid for reasonable strain, but as soon as the strains are higher than 20 % the following formulae should be used:

for compressible soil : $$\frac{d\overline{U}}{\overline{U}} \left(1 - \frac{\overline{U}}{\frac{p}{2} + \overline{U}}\right) = \frac{dp}{R}$$

for incompressible soil : $$\frac{dU}{U} \left(1 - \frac{U}{\frac{p}{2} + U}\right) = \frac{dp}{R}$$

where $$d\overline{U} = dU + \frac{dp}{E} \quad (1 - \sigma)$$

$$\overline{U} = U + \frac{p - p_o}{E} \quad (1 - \sigma)$$

It should be noticed that $\frac{dP}{du} \times U$ is a well known geometrical characteristic of the curve $U = F(p)$ this result is intensively used to compute the radius of the Mohr's circle.

$$R = \left(\frac{dp}{dU} \times U\right) \left(1 + \frac{U}{\frac{p}{2} + U}\right)$$

Furthermore, for very precise tests it is recommended to take in account the influence of the diameter increase velocity. For a rapid test of less than 2 minutes the shearing resistance in clay may be 30 % higher and in loess 20 % higher. This has been proved by recent pressiometer tests, not described in this thesis and some experience with the unconfined test ant the rotating auger.

vane test?

Figure 15 shows two representative Mohr's diagrams relative to the tests N° 22 and N° 33, 5 feet 1/2 deep. The diagram 29 shows a gentle curvature of the intrinseque curve and the equivalent c and φ are c = .7t/sqft and $\varphi = 31°$. But the diagram 24 shows a very sharp break of the intrinseque curve. Up to .1t/sqft which corresponds to the natural lateral pressure the soil behaves as solid without friction angle, for a higher pressure the soil behaves as a solid without appreciable cohesion; the equivalent c and φ are c = .5t/sqft and $\varphi = 33°$.

The mean value for c and φ which is the most representative of the shear resistance of the loess at the site of the tests may be taken as c = .5t/sqft and $\varphi = 33°$; and in taking in account the influence of the speed of the test the computed value are c = .7t/sqft and $\varphi = 33°$.

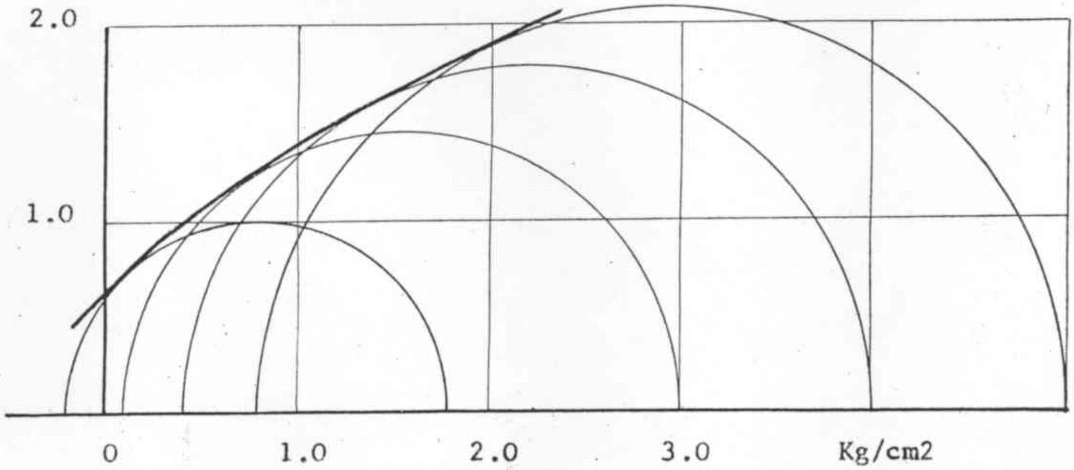
The loading tests show a large scattering for the value of the ultimate load. Furthermore the value of the ultimate load is difficult to choose on the load settlement diagram. In the very crude approximation the mean value of .6t/sqft will be chosen.

A theoretical estimate of the bearing capacity of the plate can be made on the assumption of c = .6t/sqft and $\varphi = 27°$. If different formulae may be used, there is a large scattering in the results: a reasonable value of .7t/sqft may be chosen

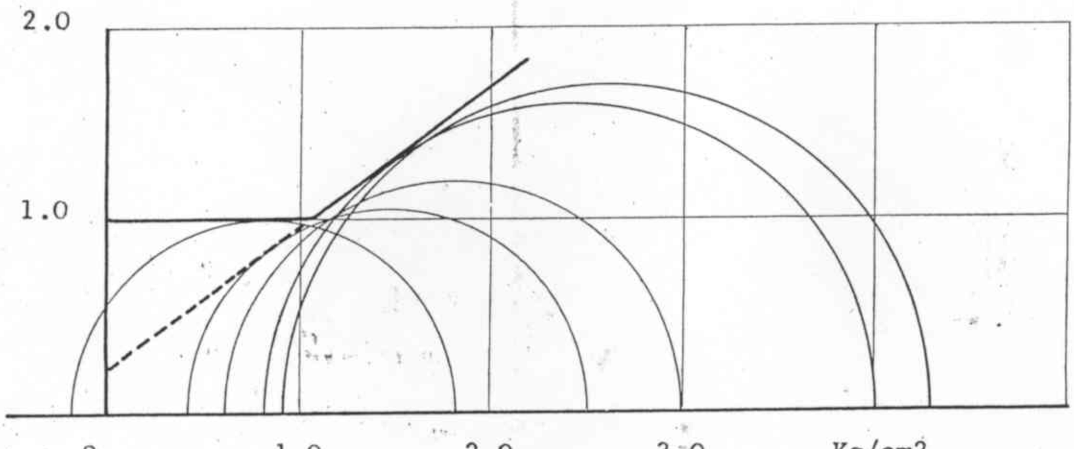
Though the theoretical estimate of the bearing capacity is very crude and though there is a large scattering in the tests results, there is a better agreement than expected between the bearing capacity theory and the tests.

Though a more complete investigation is required to compare the load test and the pressiometer tests it is apparent that a good agreement between these tests in the field may be hoped.

PRESSIOMETER RESULTS



MOHR'S DIAGRAM (25)



MOHR'S DIAGRAM (23)

Fig.15

CONCLUSION

The investigations on the pressiometer are not completed. Though numérous tests are made every day for a commercial purpose, the writer goes on to carry out some research tests in order to compare the results with the routine tests in the laboratory and more especially in the field. Some research studies are still required to have a better understanding of the unloading curve and to give a simple method to compute the natural lateral pressure. Furthermore recent tests have shown that it was possible to run "slow drained tests", if the speed of loading was slow enough. The time required for a test in this case averages one hour, it increases somewhat for very impervious clays.

Now that more then five hundred tests have been made in the field and analysed, one may conclude :

1 - The pressiometer test are very cheap and time saving, consequently the field of work of the foundation engineer might be enlarged, especially in Europe. This explains the success of the pressiometer for small jobs.

2 - The tests seem very reliable, several tests at the same place give the same results whatever the size of the equipment.

3 - The theory bases on the ultimate pressure has been very well checked wether the soil is compressible or saturated

4 - The pressiometer tests and routine tests give approximately the same results or show the same relationship as exists between the results of routine tests with vane tests.

5 - The pressiometer tests give the cohesion and the friction angle of the soil, whatever the soil is, clayey, silty, sandy or gravelly.

6 - The modulus of elasticity given by the pressiometer compared very well with the results of the routine tests. Furthermore as we found recently a very simple relationship between the modulus of elasticity and the compressibility of the clayey soils, it is possible to compute the settlement of any foundation with values yelded by the pressiometer tests.

As it is cheap and time saving, one of the main fields of work of the pressiometer is the compaction control of the soil, especially for highways and earth dams

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