AN APPARATUS FOR MEASURING THE STRENGTH OF SOILS IN PLACE

by

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THESIS

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INTRODUCTION

1) Generalities.

Most of the theories of Soil Mechanics assume the compressibility of the soil has no influence on the bearing capacity of a foundation. However Terzaghi, on the basis of compressibility, has divided bearing capacity problems into two categories and considered two cases of shear, local shear and general shear. This subdivision of the problem into two types of shear gives estimates of much practical value but is an arbitrary one. A more rigorous approach requires the use of an elasto-plastic theory.

Consequently an extensive program of tests has been conducted with a new apparatus, called the " pressiometer", to investigate the behavior of the soil in situ when submitted to a field of stresses, as a function of the cohesion, of the friction angle and in particular of the compressibility.

A broad program of tests has been carried out on glacial, fluvial and compacted clays, on glacial tills, loess deposits, and sands.

It is felt that the investigation of soil by this method has improved our knowledge of the behavior of actual foundations and has shown that the compressibility of the soil should be taken into account in any theory of the ultimate bearing capacity of a foundation.

2) Object and scope.

The general purpose of this investigation was to explore the behavior 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 bored hole, to the desired depth; a fluid is forced into the device in order to apply an uniform lateral stress to the wall of the hole. The diameter of the hole increases according to the quantity of the liquid injected. 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 foot of the soil.

This new method of subsurface investigation might not have appeared very interesting at first sight, because of the existence of well know apparatus, such as the vane test for clays and the standard penetration test for sands. However, we were strongly urged to use it on account of the following reasons:

1) A theoretical interpretation of the curve " strain versus stress" gives immediately reliable 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 very simple device called the pressiometer is available.

An élementary 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 investigation.

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 diagram "strain versus stress"
- show the influence of the elastic properties on the bearing capacity of a soil;
- describe and give the results of tests carried out on clays, tills loess and sands.

4) Acknowledgments.

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

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

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

Appreciation is expressed to Mr. Y. Lacroix, research assistant in Civil Engineering, for his cooperation in the performance of the test program.

5) Notation.

N_r Horizontal radial stress.

N_s Horizontal circumferehtial stress

) All normal and total stresses

N_z Constant vertical stress.

r Distance of a point to the axis of the hole.

ρ Initial radius of the hole

R_p Outer diameter of the zone of plastic equilibrium.

U Axial displacement of the wall of the bored hole.

p_o Natural horizontal radial stress on the limit cylinder of plastic equilibrium

p applied pressure.

p. ultimate pressure

p_e pressure at the beginning of the plastic phase.

E modulus of elasticity.

σ coefficient of Poisson.

C cohesion

R radius of a Mohr circle

R_o radius of a particular Mohr circle.

V volume of liquid in the central cell, corresponding to the axial displacement U.

e base of natural logarithm.

DESCRIPTION OF THE APPARATUS.

1) Introduction.

The first prototype was built in Paris at the "Ecole Nationale des Ponts et Chaussees", and was brought over to the United States after some tests in the west of France.

In November 1955, experiments were made in the clays of Chicago to verify the validity of the main theory and to obtain 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 3 cells. Moreover, after considerable investigation, sufficiently resistant rubber sleeves were found commercially.

As it was not required to test the soil at large depth, and since it was expensive to drill deep holes, a very cheap device was especially constructed and used for most of the tests described in this paper. No difficulty occurs in the following, to modify the device in order to carry out experiments at greater depth.

2) Description.

The device lowered into the hole to the 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 series of experiments.

The strength of the rubber membrane has no effect on the results.

The water is injected into the cell by means of air pumps, or small compressed air bottles connected to the top of the apparatus. The displacement of the wall of the bored 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 membranes on the outer cells can be attached to the same cylinder or fixed on separated ones, which are 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 prototype number 2. The rubber membranes are not shown 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 in the three cells; however, the outer cells are filled only with air and there is no volume interaction between the main cell and the outer ones.

PRESSIOMETER APPARATUS

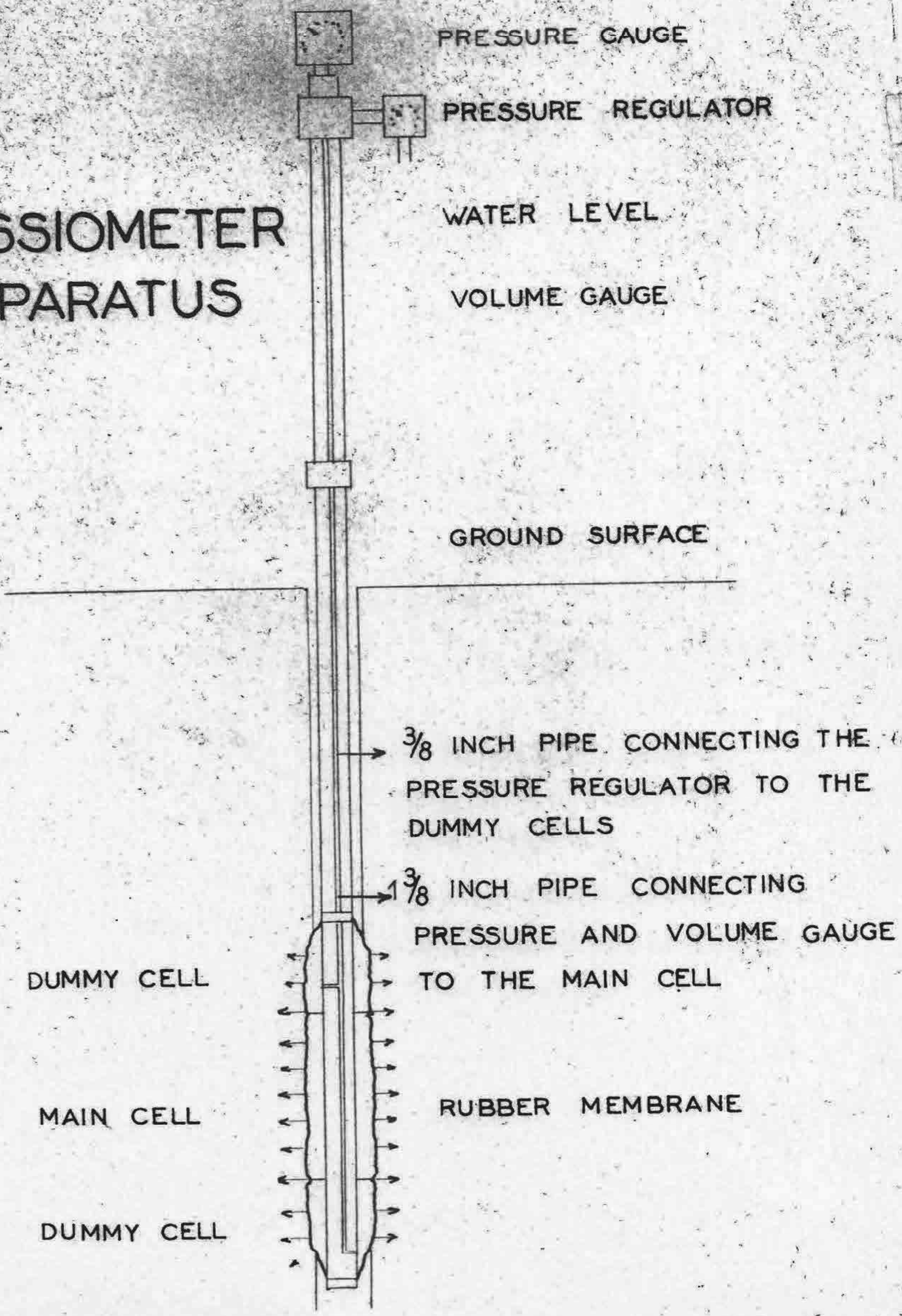


FIG 1

THEORY

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

Hypothesis.

1) Mohr's concept of the stress conditions for failure is justified only if the 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 roughly maintained all through the experiment. We assume that for a given velocity of shear, the cohesion of a clay is constant.

2) For most 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 asymptotic limit for high values of strain(Fig.2b)

For these reasons it is difficult to maintain Coulomb's concept of a cohesion and a friction angle. For instance the resistance of loessial material or of highly compacted materiel (sand) is much larger for smaller strain than for larger strain. On account of these facts, the application of Coulomb's concept is 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. This soil characteristic is measured with great

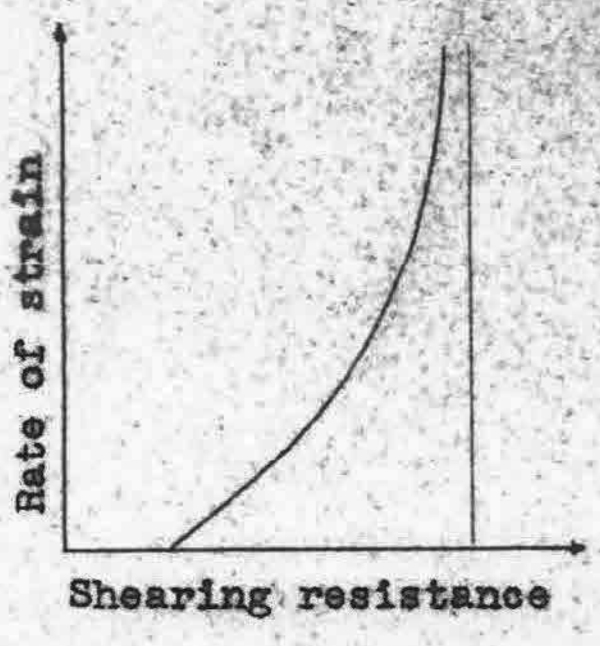


Fig.2a

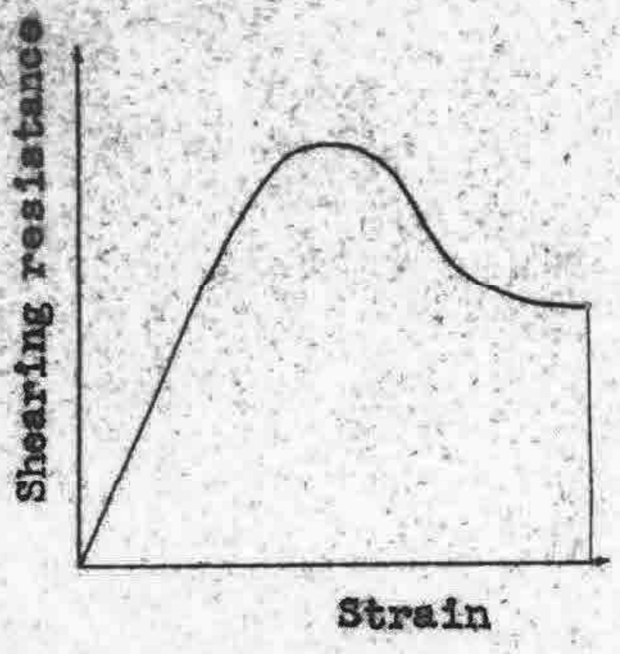


Fig.2b

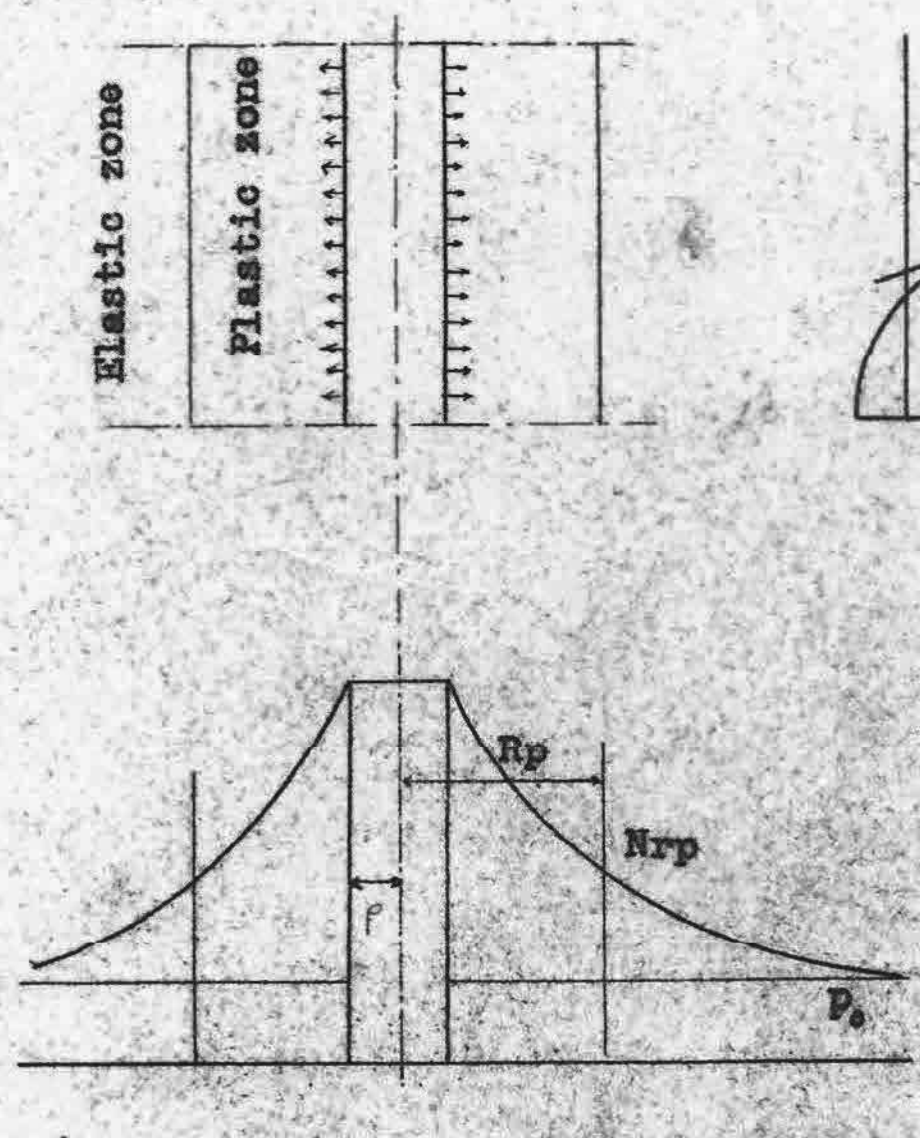


Fig.2c Distibution of stresses in plastic and elastic zones.

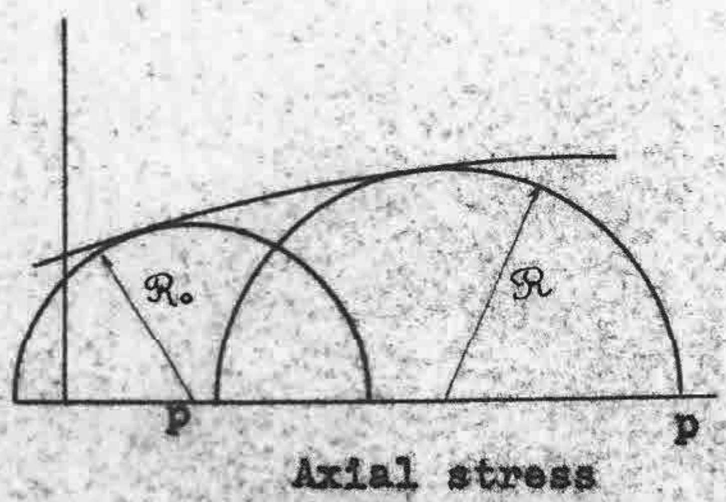


Fig.2d Mohr's Concept.

Fig.2

precision during the tests.

Theory.

If a pressure p is applied on the wall of a bored hole, a stress distribution of revolution is produced 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 triaxial apparatus, which has various aspects quite similar to the pressiometer, the pressiometer tests may be consolidated quick tests, or slow tests.

An elementary theory will be now introduced; it is called "elementary" because several refinements, espcially in the elastic phase, might be used without special difficulty. But practically these refinements are not important, and the so-called elementary theory has been found satisfactory for the analysis of the test results.

Two states of equilibrium have to be considered. In the plastic zone, next 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. Use of the elastic theory is often questioned, but in the particular case the experiments show that the theory may be applied ~~with great success.~~

Hence, the theory of plasticity will be used to compute the magnitude and the distribution 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 that we know the magnitude of the horizontal radial stress Nrp 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 to compute the résistance of pipes.

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

(1) $U_{R_p} = \frac{1+\mu}{E} (N_{R_p} - P_o) R_p$

and the magnitude of the principal stresses is given by:

(2) $N_R = N_{R_p}$

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

Zone limite d'equilibre

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; this statement is questioned because over consolidation 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.

According to formula (2), 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 fails by tension 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) $N_r - N_s = 2 R$ (Mohr's concept)

(5) $N_r - N_s = -r \dfrac{d N_r}{dr}$ (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) $d N_r = - \dfrac{dr}{r} \times 2 R$;

[C = R]

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

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

(8) $N_s = p - 2 c \left(1 + \log \dfrac{r}{p} \right)$ intégration entre N_r et P.

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, no assumption is made regarding the magnitude of the radius R of Mohr's circle.

The stress distribution is determined by the equations:

(9) $\displaystyle\int_{N_r}^{P} \frac{d N_r}{2 R} = \log \frac{r}{p}$

(10) $N_s = N_r - 2 R$

The contact stresses between the elastic and plastic zone must verify the conditions for the plastic equilibrium of the soil located inside the 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 equations (2) and (3) which are valid for elastic material, and equations (4) and (5) which are valid if stresses exceed the yield point.

Combining these equations, we obtain :

(11) $N_r = N_{R_p} = R + p_o$ $\qquad N_s = R + p_o$

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

(12) $R_p = \rho\, e^{\frac{p-p_o-c}{2c}}$ for clays

(13) $R_p = \rho\, e^{\varphi(R + p_o)}$ where $\varphi(R + p_o) = \int_{R+p_o}^{p} \frac{d\,N_r}{2\,R}$

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 train distribution and specially the increase in diameter of the bored hole is computed.

The strain distribution is different according to the percentage of saturation of the soil. If the voids of the 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) Incompressible soil.

This case is encountered in satured clays, and in general with slowly permeable soils, when the pressiometer test is a "consolidated quick test".

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

Since the soil is incompressible, the volumes (1) and (2) represented on Fig.3a have equal magnitudes and we can write:

(14) $\rho \times U_p + \frac{U_p^2}{2} = R_p \times U_{R_p} + \frac{U_{R_p}^2}{2}$

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

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

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

$$(16) \quad U = \rho \times \frac{1+\sigma}{E} \times R_o \times \varepsilon^{2f(R+p_o)} \quad \text{where} \quad f(R+p_o) = \int_{R+p_o}^{p} \frac{d\,N_r}{2R}$$

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

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

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

1) Proportional to the initial radius of the bored hole.

2) Indirectly proportional to the modulus of elasticity E

3) Increasing exponentially 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:

$$(18) \quad \boxed{\frac{dU}{U} = \frac{dp}{R}}$$

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

The result of the above equation is 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:

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

b) When the magnitude of the deformation approaches the original diameter of the hole, equation(15) is inaccurate. If we assume that the initial diameter of the drill hole is small and that deformation are large, we obtain the following relation :

$$(20) \quad \frac{U^2}{2} = R_p \times U_{R_p}$$

(Fig. 3)

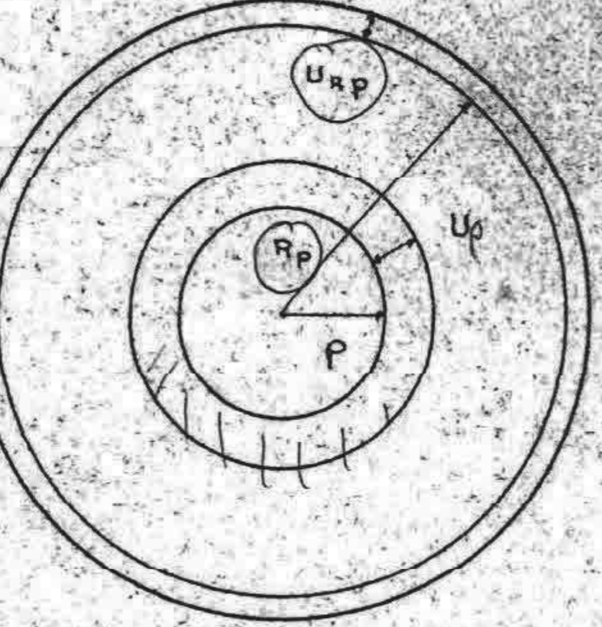


Fig.3a

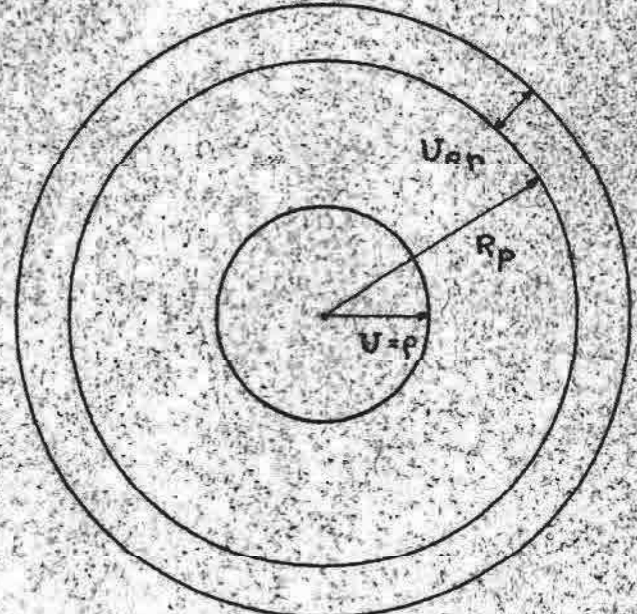
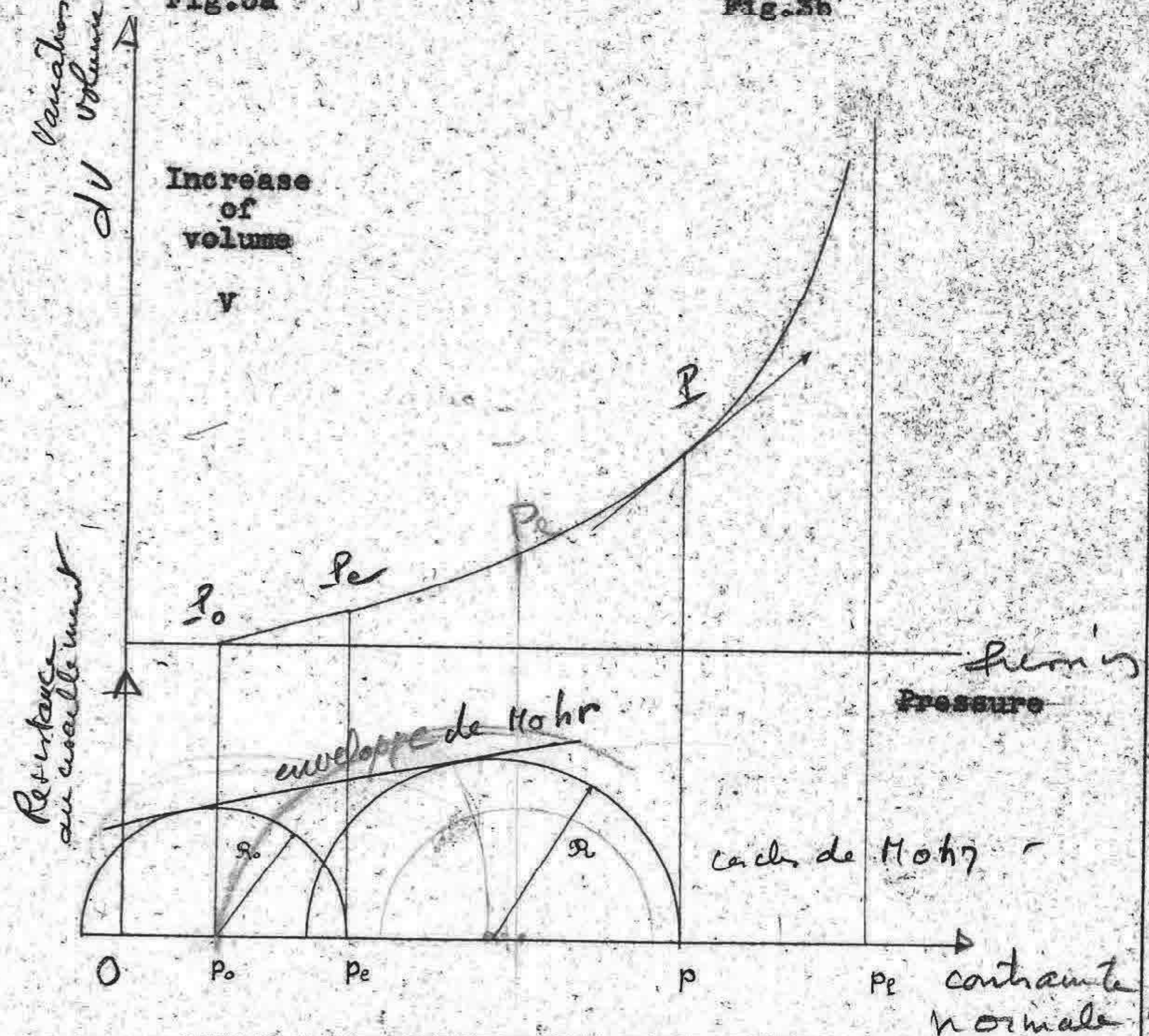


Fig.3b



CORRELATION BETWEEN THE TEST CURVE

AND THE MOHR'S DIAGRAM

Fig.3c

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

$$(21)\quad \frac{U^2}{2} = \frac{1+\sigma}{E} R_0 \times U^2 x \varepsilon\ 2f\ (R+p_0) \text{ where } f(R+p_0) = \int_{R+p_0}^{p} \frac{d\ N_r}{R}$$

This relation is indeterminate and U increases indifinitely when the pressure reaches the limit value p_1 given by the equation:

$$(22)\quad \int_{p_0+R}^{p_1} \frac{dp}{R} = \text{Log} \frac{E}{2(1+\sigma) R_0}$$

For clays of constant cohesion, we obtain the formula:

$$(23)\quad p_1 = p_0 + c \left\{ 1 + \text{Log} \frac{E}{2(1+\sigma)c} \right\}$$

We can see from equation (22) that the magnitude of the net pressure $p_1 - p_0$ is independent 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 theary to the analysis of the diagram, that this relation is used especially to verify the results given by some of the main formulae already stated in the beginning of the theory.

2) Compressible soil .

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

Only the main results are pointed out in this paper.

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

$$(24)\quad \frac{dU + \frac{(1-\sigma)}{E}\rho\ dp}{U\ \frac{(1-\sigma)}{E}\rho\ (p-p_0)} = \frac{dp}{R}$$

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

$$(25)\quad \int_{p_0+R}^{p_1} \frac{dp}{R} = \text{Log} \frac{E}{4 + R_0}$$

$$(26)\quad p_1 = p_0 + c \left\{ 1 + \text{Log} \frac{E}{4c} \right\}$$

APPLICATION OF THE THEORETICAL 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 good precision. The theoretical interpration of the diagram is a direct application of the elasto-plastic theory already explained.

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 (Fig 3c) a part of the diagram.

1) Elastic phase. $p_o < p < p_e$

When the applied pressure increases from p_o to p_e , 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 no direct relationship between the slope of the straight line and the value of the modulus of elasticity:

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

K is a characteristic of the cell and V is the variation of volume in cm^3 which corresponds to the displacement U of the wall.

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

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

2) Plastic phase. $p_e > p > p_e$

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

At each point of the diagram there is a corresponding Mohr's circle with the relations:

a) for incompressible soil: $\frac{dU}{U} = \frac{dp}{R}$ $\overline{OC} = p-R$

b) for compressible soil:

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

In fact it is more convenient to use this relation integrated between two close values p_1 and p_2.

For the incompressible soil, for instance, the formula is:

$$R_{p_1}^{p_2} = \frac{p_2 - p_1}{\text{Log } U_2 - \text{Log } U_1}$$

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

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

3) Ultimate phase.

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

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

$$p_l = p_o + c\left(1 + \text{Log}\frac{E}{2(1+\sigma)c}\right)$$ for satured clays of constant shearing resistance C.

$$p_\ell = p_0 + c\left\{1 + \mathrm{Log}\,\frac{E}{4c}\right\}$$ for compressible clays of constant shearing resistance C.

$$\int_{p_0+R}^{p_\ell} \frac{dp}{R} = \mathrm{Log}\,\frac{E}{2(1+\sigma)c_q}$$ for incompressible soil.

$$\int_{p_0+R}^{p_\ell} \frac{dp}{R} = \mathrm{Log}\,\frac{E}{4c}$$ for compressible soil.

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 that the rate of strain be kept constant during the hole test; this has been found possible without any difficulty with the prototype used for the experiments.

VERIFICATION OF THE THEORY
ON THE ULTIMATE PRESSURE

One of the most important theoretical 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.

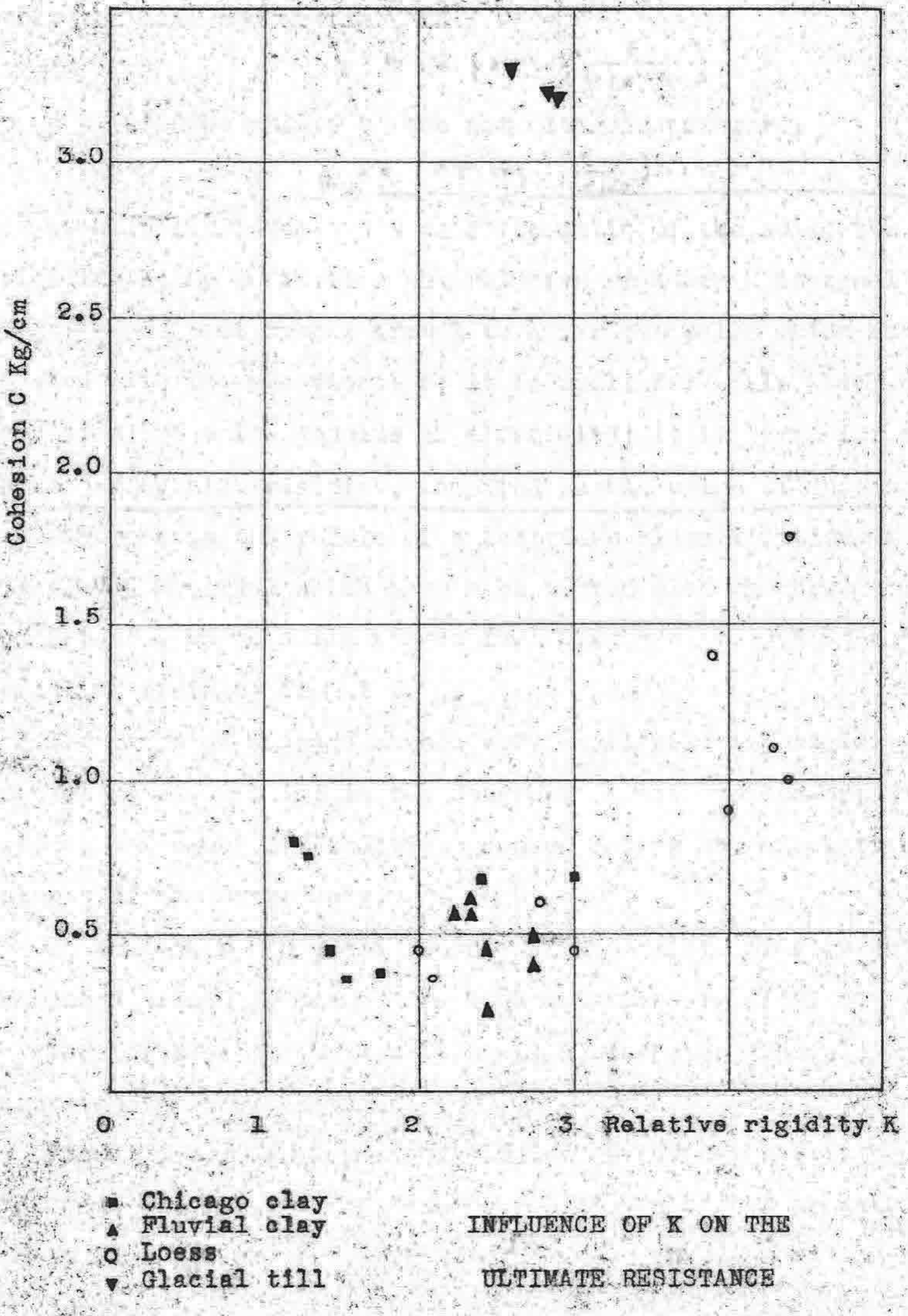
1) <u>Incompressible soils.</u>

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

Although it is not necessary to assume that the shearing resistance of the clay is constant we will first investigate the soils which have no friction angle.

In this case, we know from the theory that the ultimate pressure

TEST RESULTS



INFLUENCE OF K ON THE

ULTIMATE RESISTANCE

Fig.4

that the soil can sustain, is :

$$p_\ell = p_0 + c \left\{ 1 + \text{Log} \frac{E}{(1+\sigma)2c} \right\}$$

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

$$p_\ell^* = c \left(1 + \text{Log} \frac{E}{(1+\sigma)2c} \right)$$

We will introduce a new characteristic of the soil: the relative rigidity K; by definition the relative rigidity K is equal to Log $\frac{E}{(1+\sigma)2c}$; 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 cohesion but a low modulus of elasticity; it is large for soils which have a very high rigidity, compared to the value of the cohesion.

Figure 4 is the result of a tentative classification of the clays or clayey material which have been tested with the pressiometer; the cohesion of these soils ranges from 0.4 Kg/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, especially the clays encountered in the Congress Street Expressway; several earth slides and other difficulties occured during the excavation of the slopes of the Expressway.

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

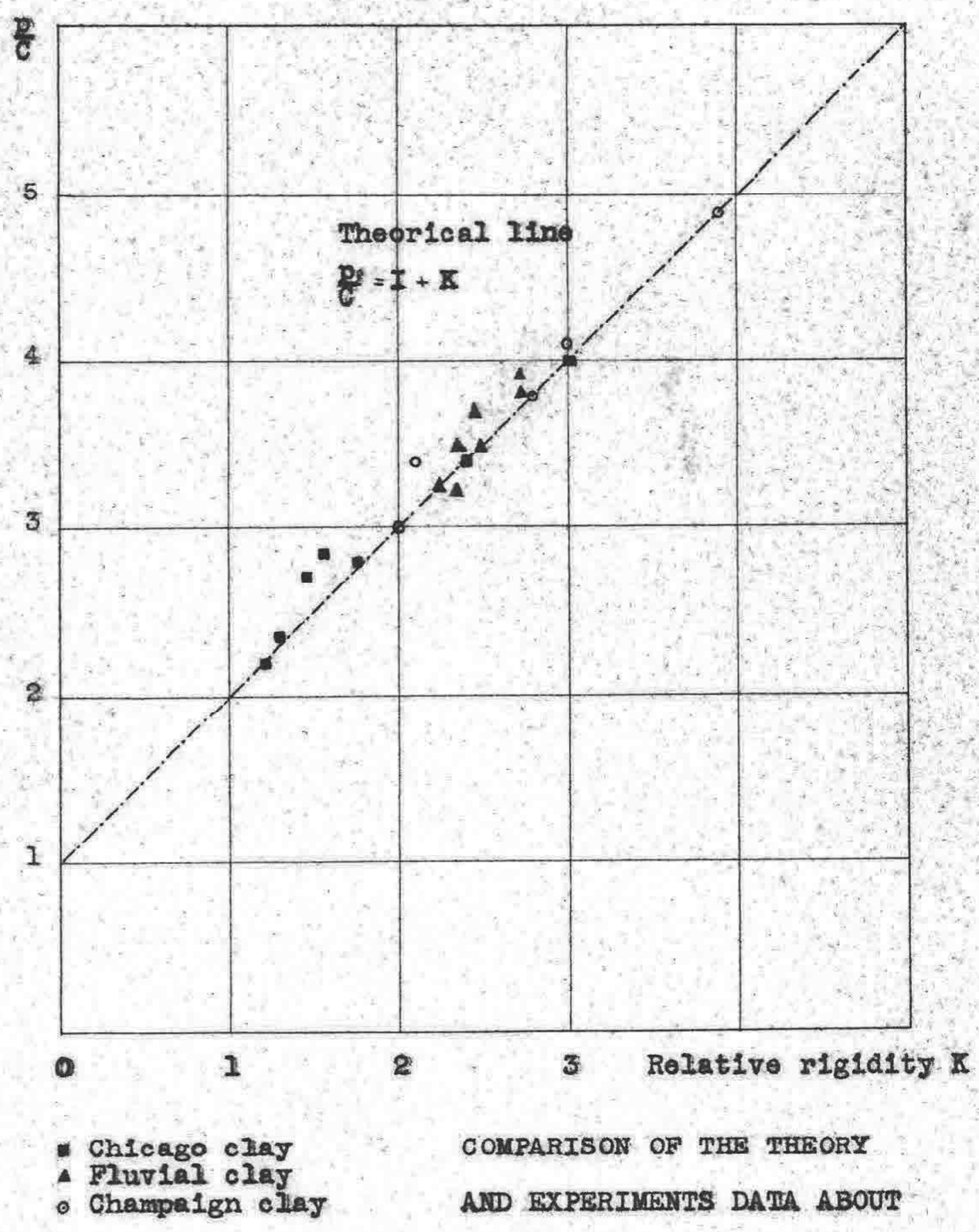
The tills of the Wesley Foundation, in Urbana, have a very high cohesion for a rigidity of 3.

Figure 5 is very important because it gives the results of one of the most precise verification of the theory of the pressiometer.

We have represented the ratio $\frac{p_\ell^*}{c}$ as a function of the relative rigidity.

The ratio $\frac{p_\ell^*}{c}$ of the net ultimate pressure to the cohesion increases with the relative rigidity.

TEST RESULTS



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

Fig.5

Saturated clays

SATURATED CLAYS

COMPARED RESULTS

Test Number	Soil type		E	C	E/C	I + K	$\frac{P_1}{C}$
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.	.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 re-		4	.444	9.1	2.55	2.7
11	molded by the		3.5	.35	10	2.65	2.85
12	rolling of earth		5	.72	7	2.3	2.85
13	moving equipment		5	.77	6.5	2.2	2.2
34	Fluvial 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

For a relative rigidity having a value of 1, the ratio $\frac{p_e^*}{c}$ is 2, but for a relative rigidity of 4 the ratio is 5. So the variations are very important.

In the same figure, we have drawn the theoretical relationship between K and the ratio $\frac{p_e^*}{c}$

$$\frac{p_e^*}{c} = 1 + \mathrm{Log}\frac{E}{(1+\sigma)2c} = 1 + K$$

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 in 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 of equal rigidity.

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

$$p_e^* = \cdot \left(1 + \mathrm{Log}\frac{E}{4c}\right)$$

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

$$\int^{p_e} \frac{dp}{R} = \mathrm{Log}\frac{E}{4c}$$

Where Log $\frac{E}{4c}$ is the relative rigidity.

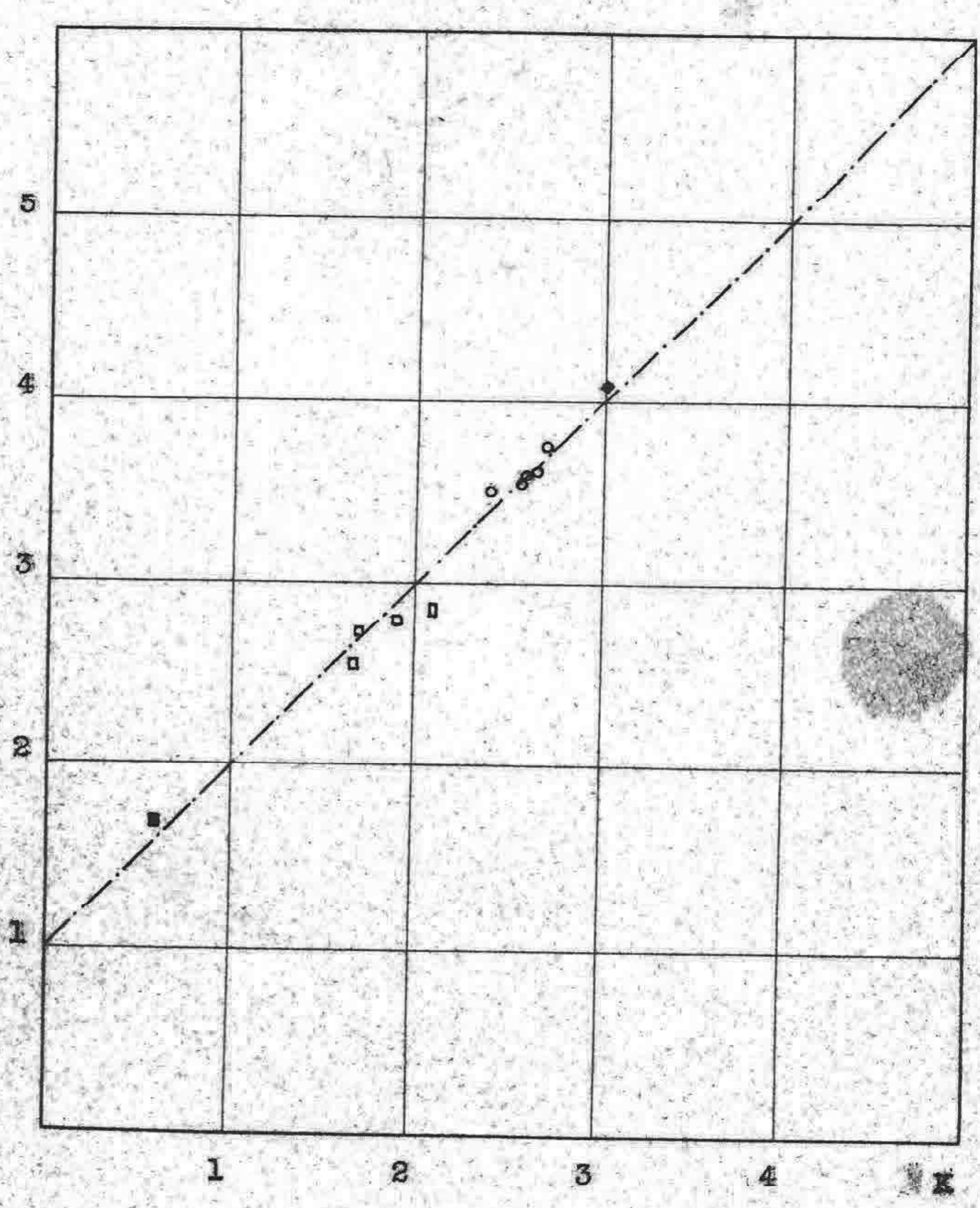
Fig. 7 shows the value of the quantity $1 + \int_{p_o+c}^{p_e} \frac{dp}{R}$ as a function of the relative rigidity.

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, was not always reached.

Actually, some tests were not carried out deeply enough, and a heave of the topsoil 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 THE DIFFERENT TESTS MADE IN THE FIELD.

1. CHICAGO CLAYS

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

Congress Street 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 Street and Aberdeen Avenue. The test was made 8'-6" below the ground surface in the medium yellow dessicated clay.

The area had experienced a slide; the pressiometer test showed that the modulus of elasticity of the soil was very small.

Further tests were made in March 1956, a few hundred feet west of the first test but in the bottom of the excavation. It was 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 very similar laws. The small values for 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, in the "Loop" of Chicago.

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

The tests were made 1, 2, and 3 feet from the pile. The plots in Fig.8 give the increase of volume of the cell versus the pressure.

From the tests, it is seen that the soil around the pile was remolded. According to the following table, the elasticity modulus and the shearing strength are smaller than their initial values.

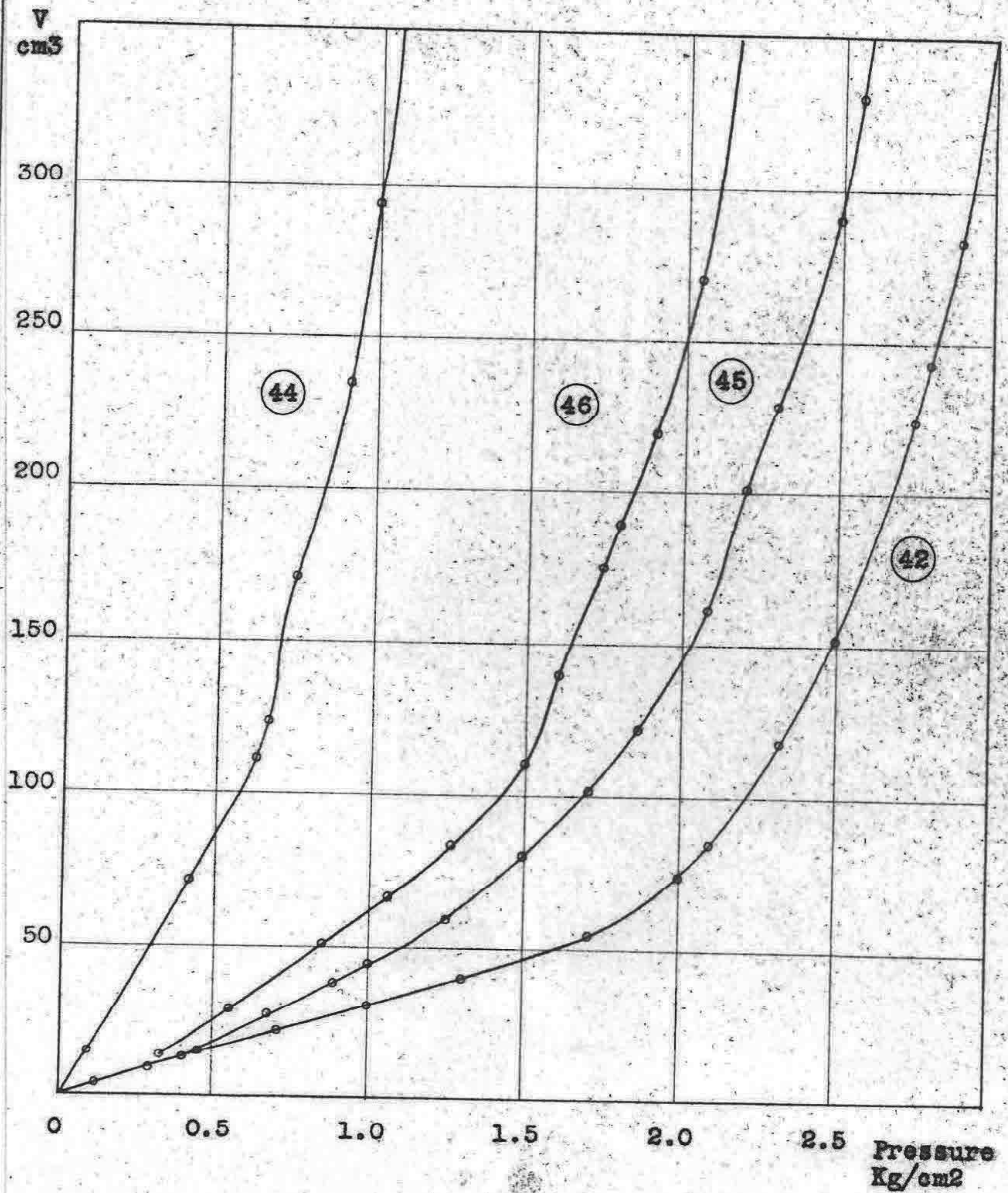
The depth is taken from the bottom of the excavation which was 33' below the street level.

The experimental and theoretical limit pressures can be seen on the right side of table 6. Though the results justify a more complete investigation, it is apparent that the theory and tests agree well.

Test number	Distance from the H pile ft	Depth ft	Modulus of elasticity Kg/cm²	Average cohesion Kg/cm²	Limit pressure Kg/cm²	Theoretical limit pressure Kg/cm²
44	1	3' 4"	6	0.37	1.1	1.05
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

Figure 9 shows the different values of the modulus of elasticity and of the cohesion at various distances from the axis of the H-pile.

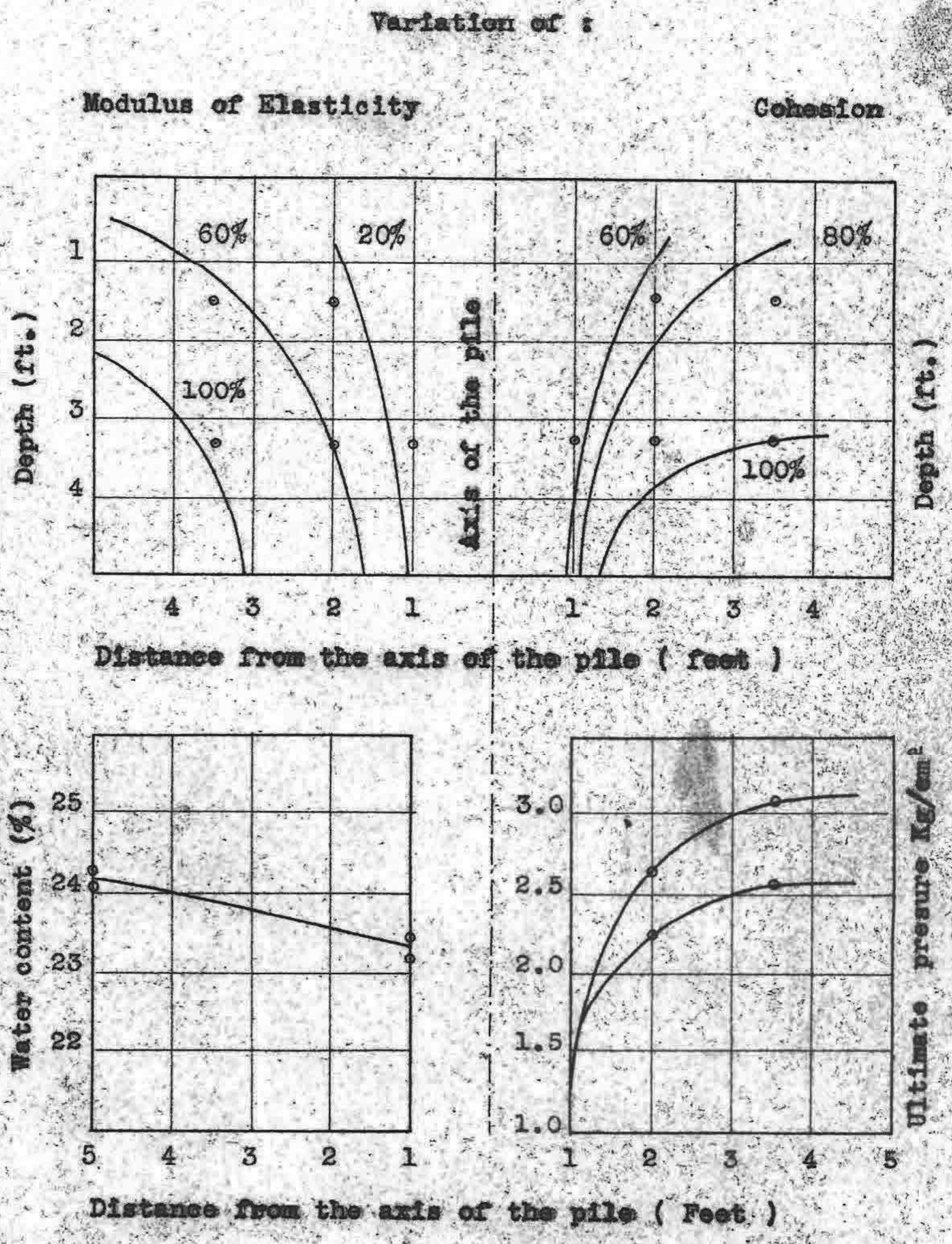
TEST RESULTS



CHICAGO CLAY

(INLAND STEEL BUILDING)

Fig.8

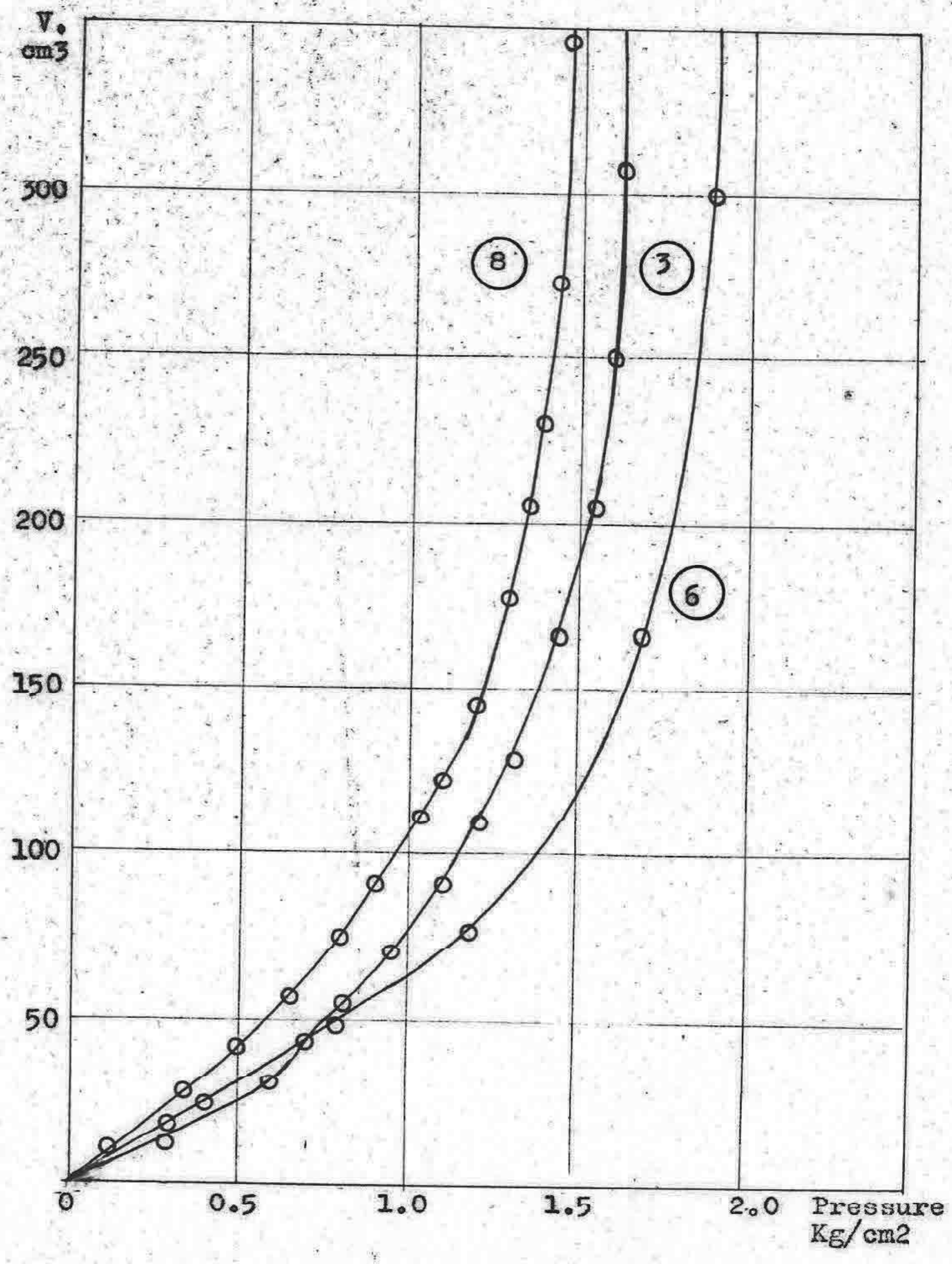


CHICAGO CLAY

(INLAND STEEL BUILDING)

Fig.9

TEST RESULTS



FLUVIAL CLAY

(CHAMPAIGN)

Fig.10

It appears that the cohesion decreases when we go closer to the pile or the bottom of the excavation. The remolding action due to the pile driving and to the excavation caused 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/cm² for the clay in the natural state, to 6 kg/cm² in the remolded state.

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 performed in connection with the Inland Steel Building have shown that the mechanical properties of a soil can be altered in account of the construction procedure. The pile driving and the excavation seem to produce a measurable remolding action. Tough the essential aim of the tests was to check the pressiometer theory, they indicated the possibility to observing with precision a phenomenon thought very difficult to study.

11. FLUVIAL CLAYS (CHAMPAIGN)

A serie of tests has been performed south of Talbot Laboratory, a few feet north of Boneyard creek. Several hand borings gave the following results: the top 3 feet consist of a sandy clayey silt; from 3 to 7 feet, a fluvial grey clay with some pebbles lies 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 the results given by the 6" apparatus, as 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 character of the plastic phase, and the limit pressure. It should be noticed that the small bump characterizes the tensile resistance of the soil; during the theoretical development it was assumed, as it is generally done, that the tensile strength of the soil had no influence in the computation of the stability of the soil; the results of the tests showed that this assumption was not correct, and that several cases of tensile failure occured in the field.

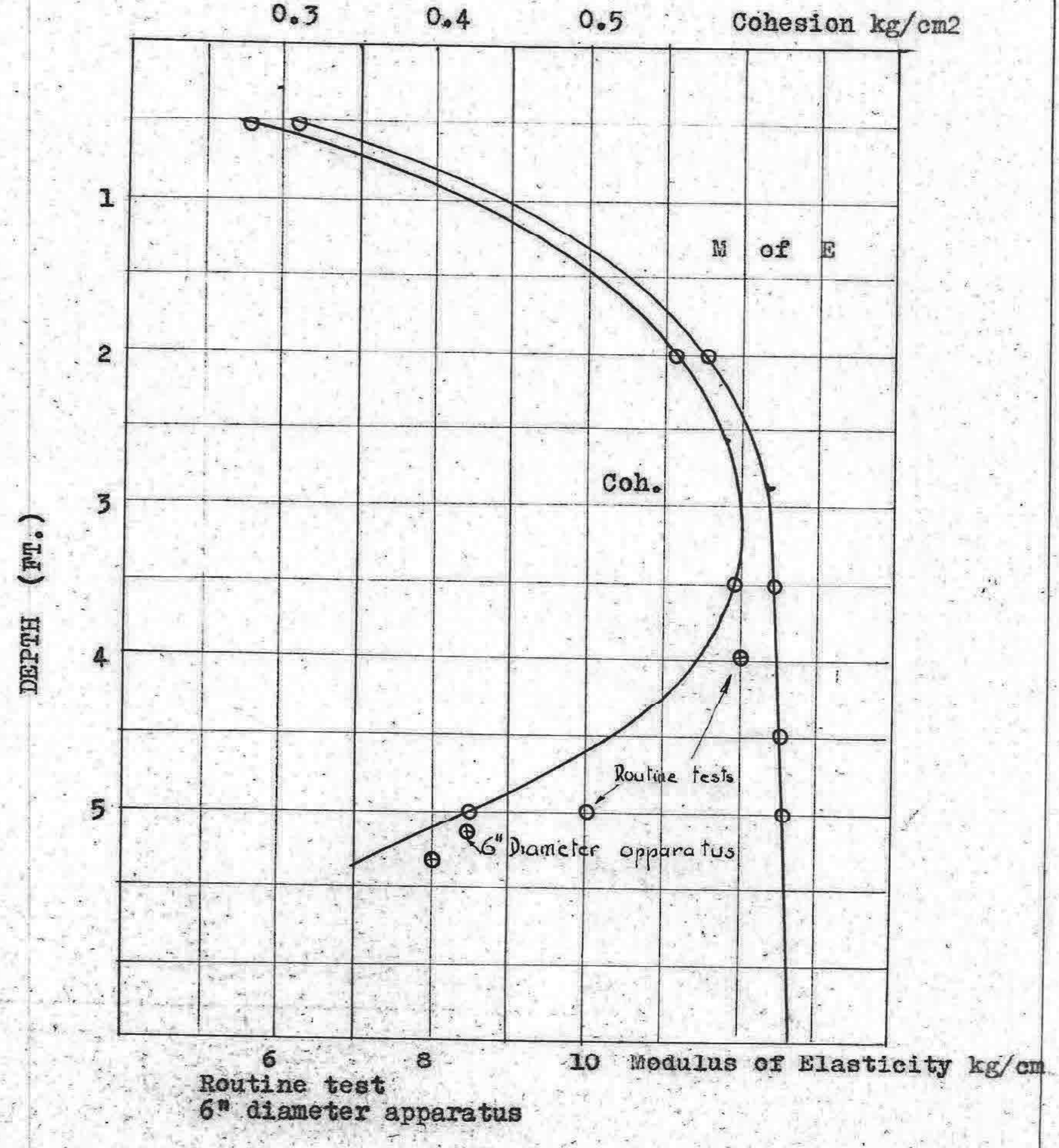
We have plotted, Fig. 11, the values of the cohesion and the modulus of elasticity as a function of the depth. The rate of increase is approximately the same for both; however, at the bottom of the clay layer, the cohesion decreases while the modulus of elasticity stays constant.

A great number of routine tests have been made. They have checked the tests performed with the pressiometer. They have indicated a good correlation between the theoretical and experimental results.

III. CLAYS, EISNER WAREHOUSE.

A series of tests was carried out on the site of the new Warehouse of the Eisner Co, Champaign.

The 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, the rate of variation is

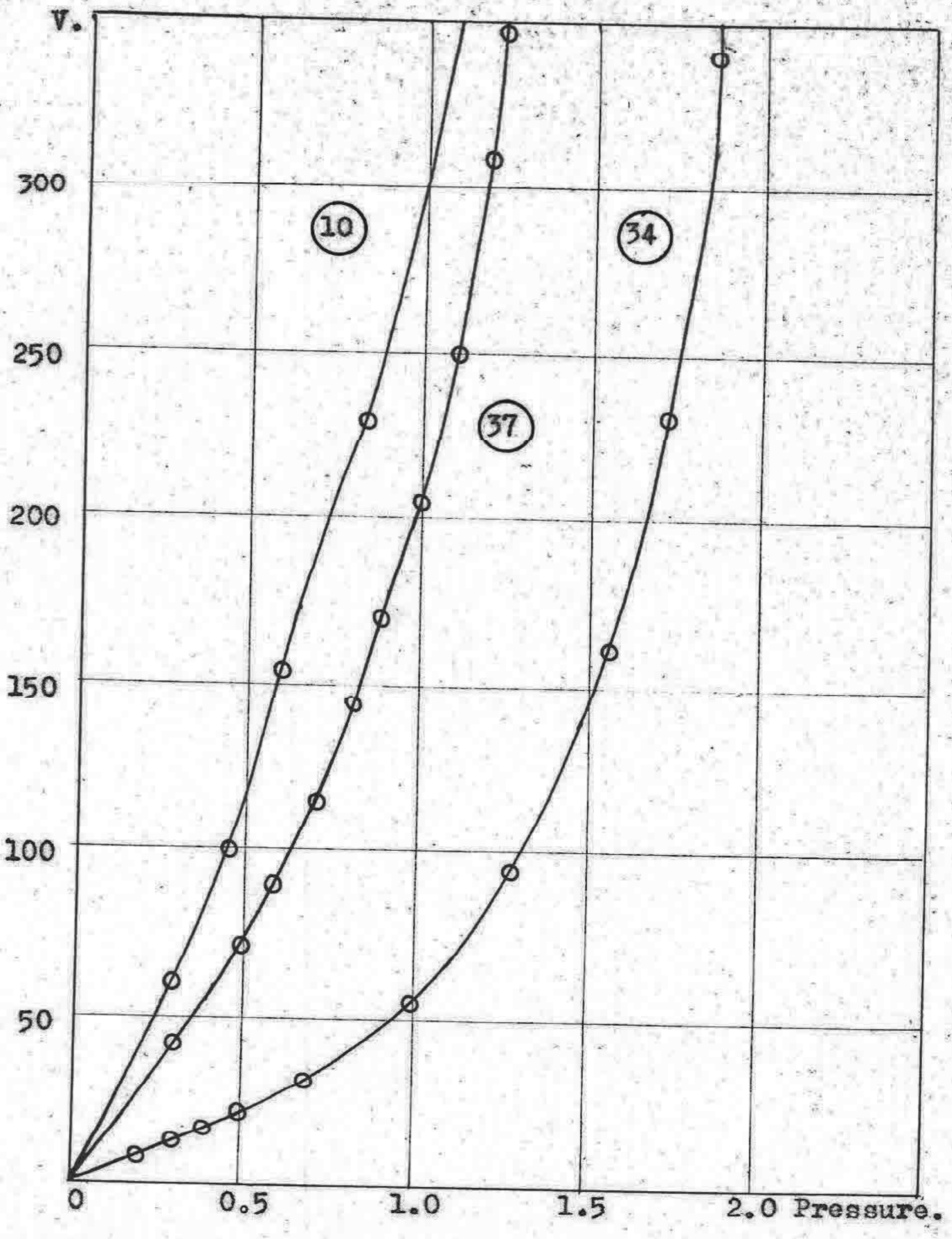


FLUVIAL CLAY

(CHAMPAIGN)

Fig. 11

TEST RESULTS

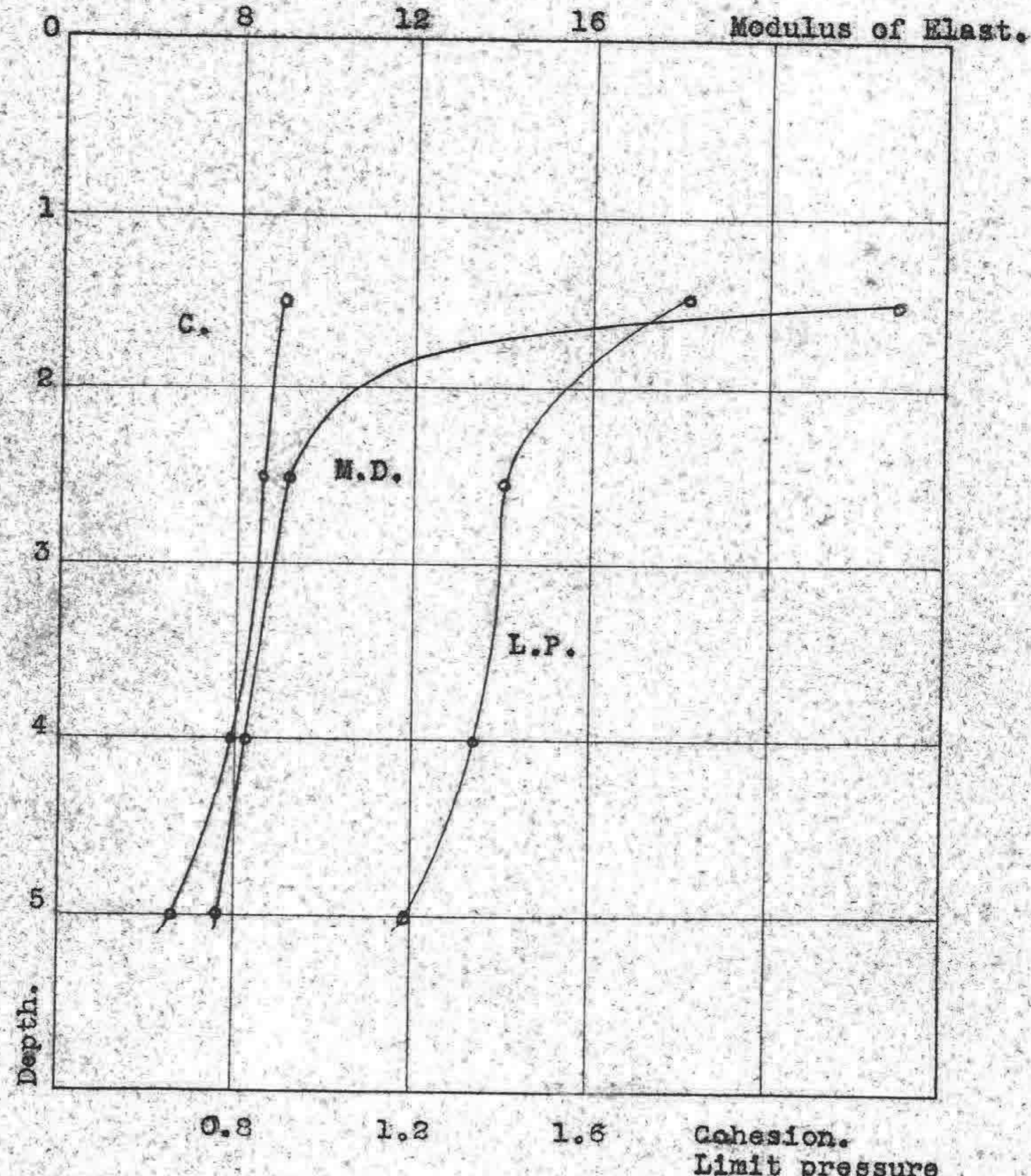


SATURATED CLAYS

EISNER WAREHOUSE

Fig.12

TEST RESULTS.



SATURATED CLAYS.

EISNER WAREH OUSE

Fig.13

approximately 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 kg/cm^2 to 8 kg/ cm^2 the cohesion ranges from 9 Kg/cm^2 to 65 Kg/cm^2

Settlement, due to the loading, is closely related to the modulus of elasticity. Variation of the latter is likely to produce differential settlements.

Some unconfined compression tests 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 was well verified.

Test results

Depth		C. Pressiometer Kg/cm^2	C (U.C.T.) Kg/cm^2	Modulus of elasticity Kg/cm^2
1'-6"	Natural soil	.55	.60	25
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	112.5

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

TESTS IN THE LOESS DEPOSIT

A serie 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 the 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.

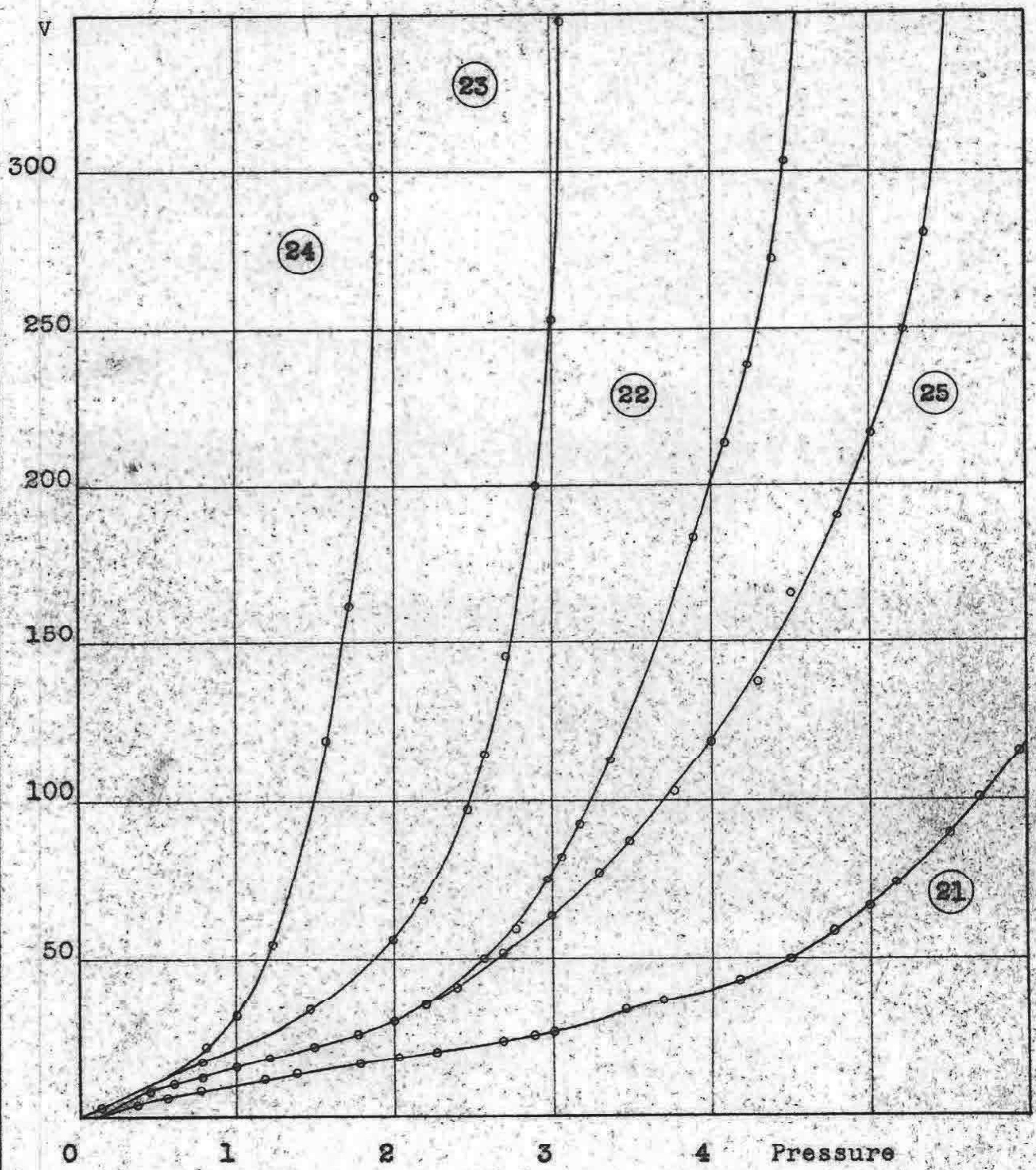
It may be worthwhile to note that the relative rigidity of the loess is very high compared to the other soils tested with the pressiometer.

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 diagrams 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/cm^2. The load tests yield an average value of E= 100 kg/cm^2 (mean value of three tests). Tough the results would justify a more

TEST RESULTS



LOESS DEPOSITS

Fig.14

complete investigation, it is apparent that the value of the modulus of elasticity given by both tests (load and pressiometer tests) agree well.

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: $\frac{d\bar{U}}{\bar{U}}\left(1-\frac{\bar{U}}{\frac{\rho}{2}+\bar{U}}\right)=\frac{dp}{R}$

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

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

$\bar{U}=U+\frac{p-p_0}{E}\rho(1-\sigma)$

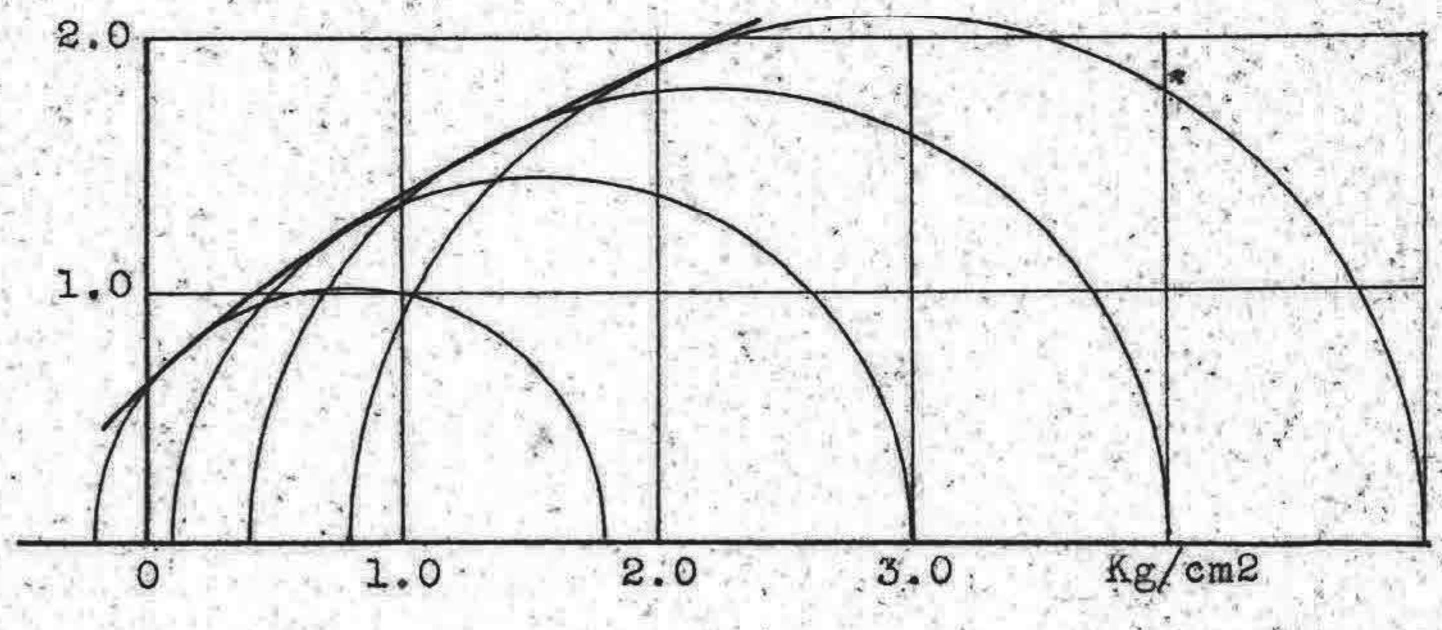
I should be noticed that $\frac{dp}{dU}\times U$ is a well know geometrical characteristic of the curve $U=f(p)$ this result is intensively used to compute the radius of the Mohr's circle.

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

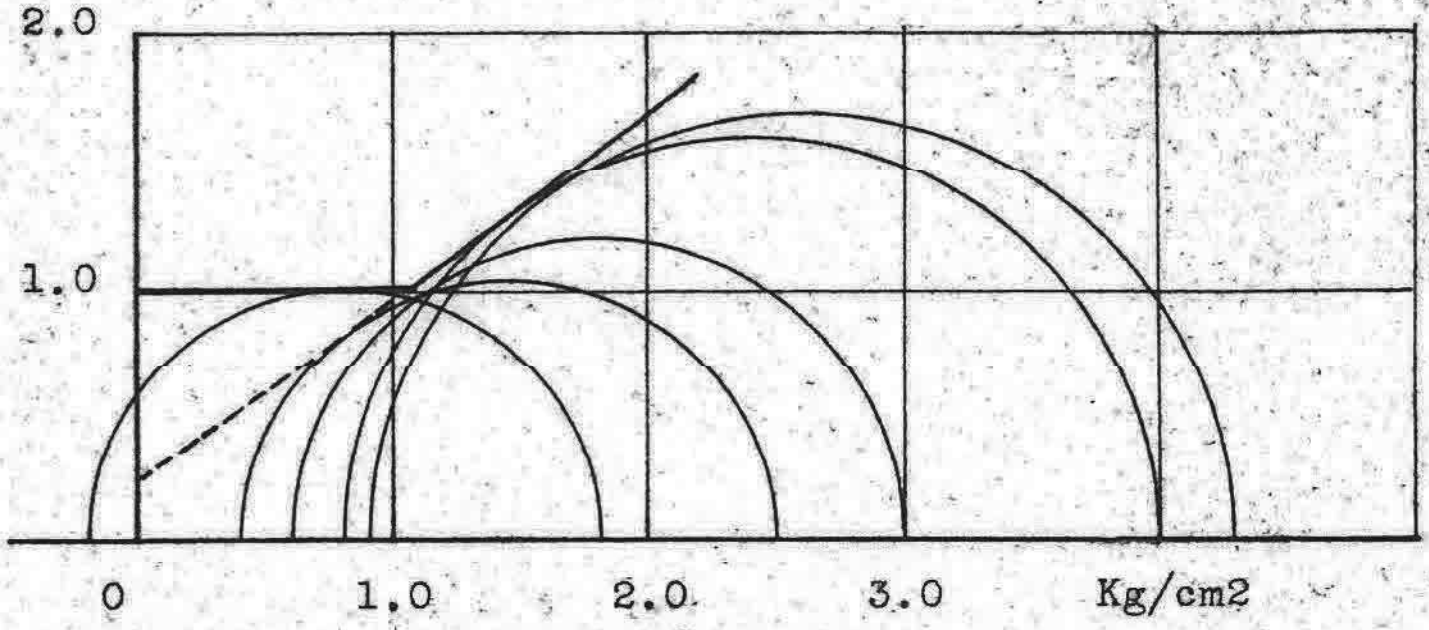
Furthermore, for very precise tests it is recommended to take in account the influence of the diameter increase velocity. For a rapid test of less than 2 minutes the shearing resistance in clay may be 30% higher and in loess 20% higher. This has been proved by recent pressiometer tests, not described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

Figure 15 shows two representative Mohr's diagrams relative to the tests N°22 and N°23, 5 feet 1/2 deep. The diagram 29 shows a gentle curvature of the intrinseque curve and the equivalent c and φ are c = .7 T/$\square$' and φ = 31°. But the diagram 24 shows a very sharp break of the intrinseque curve. Up to 1 T/$\square$'; which corresponds to the natural lateral pressure the soil behaves as solid without friction

PRESSIOMETER RESULTS



MOHR'S DIAGRAM (25)



MOHR'S DIAGRAM (23)

Fig 15

angle; for a higher pressure the soil behaves as a solid without appreciable cohesion; the equivalent c and φ are $c = 5\ T/m^2$ 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 $c = 5\ T/m^2$ and $\varphi = 33°$; and in taking in account the influence of the speed of the test the computed value are $c = 7\ T/m^2$ 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 $6\ T/m^2$ will be chosen.

A theoretical estimate of the bearing capacity of the plate can be made on the assumption of $c = 6\ T/m^2$ and $\varphi = 27°$. If different formulae may be used, there is a large scattering in the results: a reasonable value of $7\ T/m^2$ 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.

CONCLUSION.

The investigations on the pressiometer are not completed. Though numerous 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 showed 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 than five hundred tests have been made in the field and analysed, on 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 based 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 dayey soils, it is possible to compute the settlement of any foundation with values yielded 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, espcially for highways and earth dams.

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