MAIN SESSION 1
I-e SESSION PLENIERE
ПЕРВОЕ ПЛЕНАРНОЕ ЗАСЕДАНИЕ

UP-TO-DATE METHODS OF INVESTIGATING THE STRENGTH AND DEFORMABILITY OF SOILS (LABORATORY AND FIELD TESTING OF SOILS FOR THEIR STRENGTH, DEFORMATIVE AND RHEOLOGICAL PROPERTIES)

Chairman: Prof.L.Šuklje, Yugoslavia; General Reporter: Prof.T.W.Lambe, U S A

Participants: C.P.Wroth (U.K.), N.Janbu (Norway), Yu.G.Trofimenkov (USSR), L.Menard (France), B.Broms (Sweden), J.D.Nelson (Thailand), J.B.Burland (U.K.), R.J.Marsal (Mexico), W.H.Craig (U.K.), M.N.Goldstein (USSR), J.M.Duncan (USA), G.Gudehus (W.Germany), K.Zlatarev (Bulgaria), M.Y.Pevzner (USSR), A.Z.Kryzhanovsky (USSR), Z.G.Ter-Martirosjan (USSR), A.W.Bishop (U.K.), G.G.Meyerhof (Canada),

Chairman: Prof.L.Šuklje, Yugoslavia

Ladies and gentlemen,

I have the honour to open the first Main Session of the 8th International Conference on Soil Mechanics and Foundation Engineering.

I am very pleased to have, by my side, as the Vice-Chairman of the Session, Professor C.Meyerhof from Canada, a man of wide international reputation.

As you know, our General Reporter is Professor Lambe from the famous Massachusets Institute of Technology, and I am sure that everybody agrees with me that the Organizing Committee could hardly find, for the subjects of our Session, a more competent expert than this year Rankine-Lecturer.

The delegate of the Organizing Committee at the First Session is Prof.S.S.Vyalov, a distinguished scientist, well known though his books and papers on frozen soils and viscous soil behaviour.

The presedency of the Session will kindly be assisted by Mr.V.P.Petrukhin as the scientific Secretary of the Session Mr.Petrukhin is an outstanding representative of the younger generation of the soil mechanics science in the Soviet Union whose rich tradition we admire remembering with esteem the names of Gersevanov, Florin, Denissov, Sokokovsky and many others.

Our Session is devoted to laboratory and field testing of soils for their strength and deformability. If the strength is considered as the stress state at which the strain speeds become critical, the investigation of the stress-strain-time relationships for soils, i.e. of their rheological properties appears the fundamental subject of our today's discussions. Owing to the two- or three-phase composition of soil media, their permeability intervenes as a furhter basic factor determining the soil reactions to loads. The determination of rheological and filtration parameters yields physical data for predicting stress- and strain states in soils and earth structures, and for establishing criteria governing the acceptability of predicted and observed displacements and their speeds.

The refinement of experimental techniques has contributed to important increase of our knowledge of rheological relationships for soils. The simultaneous brilliant development of electronic computers and numerical methods for solving stress- and strain problems has opened large possibilities for considering complex rheological relationships at arbitrary boundary conditions. Nevertheless, our possibilities remain restricted. The limitations are due to the disturbance of soils when testing their deformability, to the imperfections of experimental devices and to the amount of experimental work needed to establish numerous sets of parameters that determine the deformability of layered soils and earth structures of a heterogeneous profile. These parameters depend not only on the mineralogical composition of soils, on their grain-size distribution, particle arrangement, initial porosity and saturation degree, but also on the stress- and strain history and on loading interval and speed. Further limitations are due to basic assumptions of the theories used to the amount of programming and programme-testing when the boundary conditions are complicated, and last but not least, to the capacity of our computers.

Consequently, simplifications in geotechnical computations cannot be avoided. They have to be governed by our knowledge of the effects of single influencing factors as found by isolated analyses at simple boundary conditions, further by comparing simplified and rigorous solutions with measurements and observations made on structures, in foundation soils and on natural slopes. The degree of simplifying experimental and calculation methods and procedures depends on the nature of the problem to be solved, on the means that are at our disposal, on the preliminary or final character of decisions to be taken, on the importance of our decisions etc. When preparing appropriate investigation and calculation schemes serving for quick decisions of the practicing engineer, we have to define limitations and conditions under which they can be applied. Outside these restrictions, the geotechnical expert must search for the most appropriate methods to determine soil properties and predict the displacements, their speeds and safety, by choosing, evaluating, eliminating and combining different possibilities which arise from previous experimental and theoretical investigations, and considering experience obtained by observations and measurements on structures and in nature. The complexity and combinatory character of solutions are the typical features of our profession.

The papers published subsequent to the foregoing International Conference and those which appeared among the Proceedings of the present Conference, reflect important advancement in laboratory equipment, in establishing rheological relationships for soils and in improving field devices which facilitate quick decisions in everyday engineering practice. Professor Lambe, our general reporter, has prepared an interesting review of the work done. Following the suggestions by the Organizing Committee he selected, for our today's Session the theme: Soil Parameters for Predicting Deformations and Stability. He requested that the members taking part in discussion should present only material of immediate or near-term value to the practicing engineer. Among six questions and topics that Professor Lambe had prepared for discussion, the Organizing Committee has recommended to restrict our attention to subjects concerning: (a) in situ test devices, (b) predictive techniques based on in situ measurements of soil parameters, (c) laboratory model tests, especially the centrifuge test, and (d) use of probability and decision theory in practical soil engineering problems.

The schedule of our Session will be accorded with the programme of the Conference as established by the Organizing Committee. I am sorry however, to let you know that the Past-President of our Society Prof. Casagrande does not participate at the Conference and that we shall not have the pleasure to listen to his comments concerning the topics of our discussion.

Thus, I am proposing to start with the discussion of the topics selected by the General Reporter immediately after his introductory speech. After four contributions we shall have an intermission of 15 minutes.

Following the instructions of the Organizing Committee given in the Bulletin No.2 34 participants of the Conference have submitted the texts for discussion. 22 of those texts are related to the selected topics. I am sorry to say that the time we have at our disposal does not permit the oral presentation of all these contributions. Professor Lambe and myself have chosen, among them, 16 debators whose comments and ideas have been considered to complete at best the General Report and to elucidate the way to further advancement in determining soil parameters which are used in predicting deformations and safety of soils and earth structures. You can find their names on the blackboard. Each debator will be allotted up to five minutes. This time limitation is rigorous. The discussion will be followed by the concluding remarks of the General Reporter and the closing speech of the Vice-Chairman Prof. G.G. Meyerhof.

General Reporter: Prof. T.W. Lambe, USA
Prof. Lambe's State-of-the-Art Report appears on p.3 of the vol.III.

Chairman Prof. L. Šuklje.
Thank you very much Prof. Lambe. We greatly appreciate your report. We will now hear the prepared discussion. I would like to invite Dr. Wroth to give us his discussion.

Dr. C.P. Wroth (United Kingdom)

In Situ Measurement of Soil Properties

My contribution is in response to the invitation from the General Reporter to describe our attempts at the University of Cambridge to measure the properties of soils in the undisturbed in situ condition. The work has been carried out by Dr. Hughes and has led to the development of a special instrument which is described in our paper to this conference. We have concentrated our efforts on causing the minimum possible disturbance to the soil by introducing the instrument into the ground by a self-drilling action.

A schematic diagram of our instrument appears as Fig.1 of our paper (No.1/75 p.488). The instrument is in principle very similar to the autoforeuse pressuremeter developed independently in France by M. Jezequel and his colleagues, which is described in their paper to this conference. The apparatus consists basically of a hollow cylinder with a sharp lower cutting edge, which is jacked very slowly into the ground. At the same time, soil is excavated by a rotating cutter inside the cutting head, and the resulting pieces of soil are washed up to the ground surface by water which circulates down the centre of the shaft and up the annular space between the shaft and the interior of the walls of the cylinder. Radiographic tests in the laboratory confirm that the disturbance of the soil around the instrument is very small indeed - perhaps 1/2% radial strain, or even less.

A conventional pressuremeter test can be conducted on the surrounding soil by inflating the rubber membrane, and measuring the applied pressure. The displacement of the membrane is accurately measured by mechanical feelers which are spring loaded and follow the movement of the membrane. The bending of the leaf springs is monitored by electrical resistance strain gauges. A pore-pressure gauge is fitted to the membrane and records the pore-water-pressure in the deformed soil.

The results of an undrained test on soft clay at Ellingsrud, in Norway, are shown in Fig.1. The upper diagram relates the applied pressure with the radial strain of the membrane.

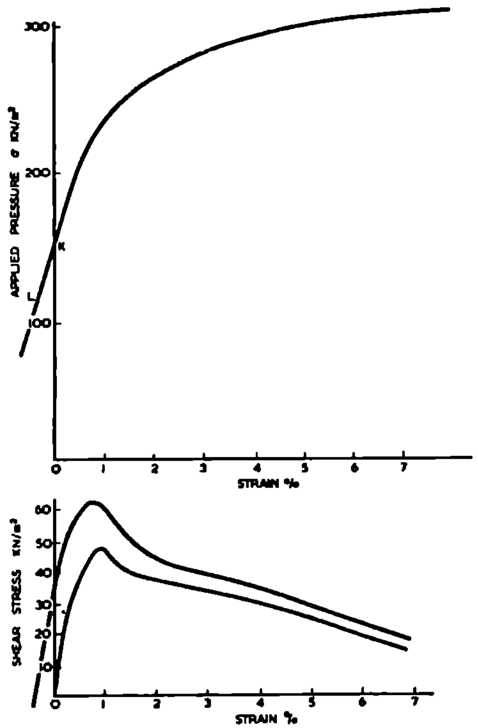

Fig.1 Results of an Undrained Test on Soft Clay at Ellingsrud, Norway.

Note that the point K at which movement of the membrane is first recorded indicates the total lateral pressure,from which a value of the coefficient of earth pressure at rest K_o can be estimated. The lower diagram shows the shear stress-shear strain curve for the clay that can be uniquely derived from the upper curve,due to the analysis presented by Palmer (1972).It must be emphasised that it is possible to follow completely the <u>unloading</u> part of the stress-strain curve,and obtain some idea of the soil's sensitivity and residual strength.The derived stress-strain curve is sensitive to the choice of datum for the strain-axis: the lower curve corresponds to point K and the upper curve to point L.This variation is discussed in more detail by Wroth and Hughes (1973).

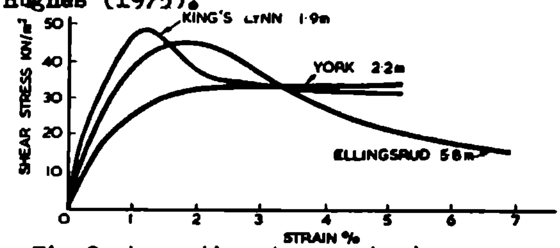

Fig.2 shows the stress-strain curves obtained for tests at three different sites of soft clays:two in England and the one in Norway already mentioned.The diagram shows the variety of shape of curve depending on the particular clay tested.The curve can indicate both a value of a shear modulus G pre-peak,and the degree of sensitivity S_t of the clay.

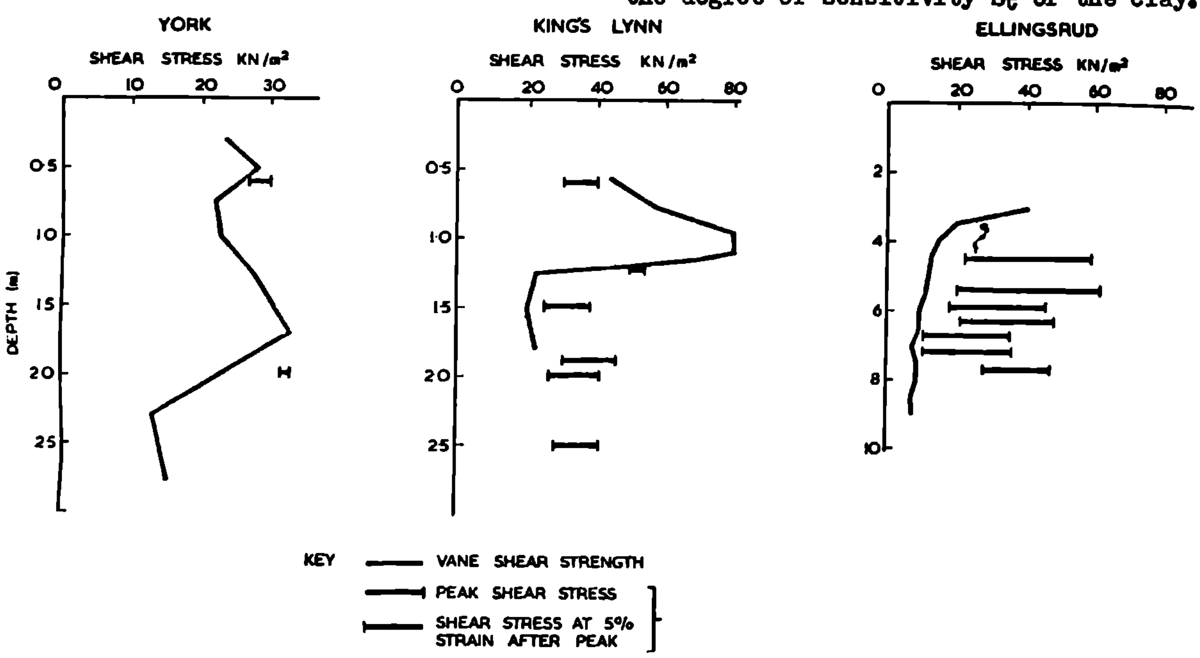

Fig.3 Comparison of Undrained Shear Strengths for 3 Soft Clays as Measured by Pressuremeter and Vane Tests.

Fig.3. provides a comparison of the shear strengths c_u measured with the pressuremeter and those measured with a conventional vane shear tests for the 3 sites already mentioned. The continuous lines represent the results of the vane shear tests; the individual bands (parallel to the axis of shear stress) show the range of shear strength recorded by the pressure-

meter from peak value (at the right hand end) to that at 5% strain (at the left hand end). It should be noted that the final values after 5% strain are reasonably close but generally greater than the vane test results. In contrast the peak values are much greater than the vane test results, particularly for the more sensitive clays.

From the pore-pressure measurements the results of the undrained test can be interpreted in terms of effective stress parameters, and lead to a value of the angle of shearing resistance ϕ'.

It is suggested therefore that the instrument will give a much better indication not only of the strength of soils, but also of their stiffness and deformation parameters. Estimates of the following can be derived: K_o, G, c_u, S_t and ϕ'.

There are possibilities for measuring other soil parameters in situ by similar techniques as will be described at Specialty Session No.5 by Baguelin, Jezequel and LeMehaute and I warmly congratulate them on these achievements.

References

PALMER A.C.(1972) Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Geotechnique 22, 451-457.

WROTH C.P. and HUGHES J.M.O.(1973) Discussion on above paper by Palmer. Geotechnique 23, 284-287.

Chairman Prof.L.Šuklje

Thank you very much Dr.Wroth. I would like to ask Mr.Janbu to present us with his prepared discussion.

Mr. Nilmar Janbu /Norway/

I am pleased to be able to respond to the General Reporter's Written request for delivering a short discussion on his topic No.1: "Which in situ test devices offer the most promise for yielding reliable soil parameters for predicting stability and deformation?"

Initially, I would like to state that I agree with the General Reporter's strong emphasize on the initimate relationship between methods and data, and with his general philosophy for the predictive process as a whole.

Required parameters

Consequently, the soil parameters to be determined by in situ measurements are those defined by the basic equations of the methods used for predicting settlements and stability, see Fig.1.

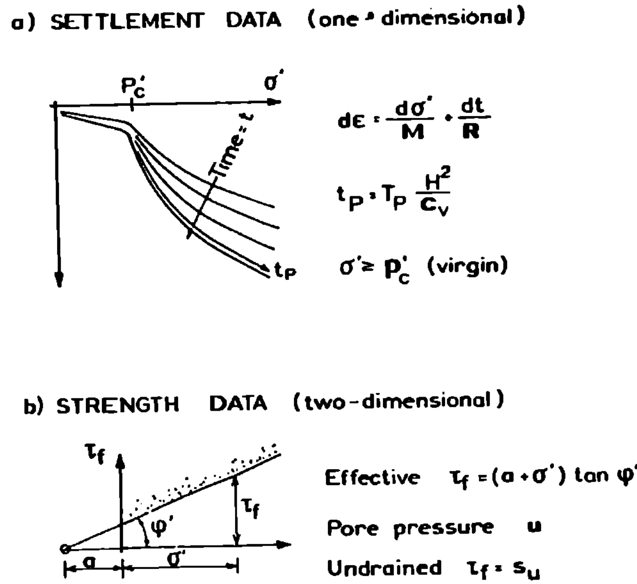

Fig.1. Required settlement and strength parameters

For instance, most settlement predictions are based on one-dimensional analyses, in which case strain (ε) is a function of stress (σ') and time (t) for each soil element. The pertinent parameters are therefore included in the basic $\varepsilon - \sigma' - t$ relationship, expressed in total differential form

$$d\varepsilon = \frac{d\sigma'}{M} + \frac{dt}{R} \qquad (1)$$

where

$M = \partial\sigma'/\partial\varepsilon$ Stress resistance (modulus)
$R = \partial t/\partial\varepsilon$ Time resistance ("secondary")

Both parameters, M or R, depend on the preconsolidation pressure p_c'. Finally, time rate predictions are usually based on the coefficient of consolidation c_v.

Altogether, four soil parameters are hence required for a complete one-dimensional settlement analysis, namely:

$$\left.\begin{array}{l} \text{Modulus, } M \\ \text{Preconsolidation press., } p_c' \\ \text{Coeff. of consolidation, } c_v \\ \text{Creep resistance, } R \end{array}\right\} \qquad 1a$$

Especially, for the estimate of initial settlements in clay, the undrained values M_u and R_u may be required.

Stability predictions are usually based on two-dimensional analysis, using the strength concepts of a Coulombian material, Fig.1(b). In terms of effective stress the required parameters are:

$$\left.\begin{array}{l} \text{Friction, } \tan\phi' \\ \text{Attraction, } a \text{ (or } a\tan\phi\text{)} \\ \text{Pore pressure, } u \end{array}\right\} \qquad (2)$$

Especially, for short term stability analysis of saturated, fine-grained soil one often use one strength parameter only, namely the undrained shear strength, s_u.

In situ devices

Some of the best known devices for in situ tests to determine settlement and strength parameters are illustrated in principle in Fig.2.

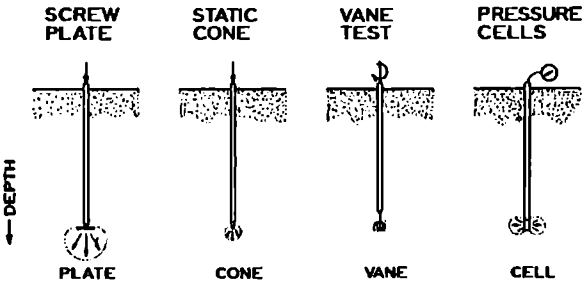

Fig.2. Devices for in situ measurements of settlement and stability parameters

The main devices are:

(a) Plates
(b) Cones (or points)
(c) Vanes (or blades)
(d) Pressure cells

The vane test (c) is strictly speaking limited to undrained, short term stability condition for isotropic, saturated clays of low plasticity and low sensitivity. Owing to this limited validity, no comments are required here.

Pressure cells (d) of various kinds, used for in situ pore pressure measurements, are so well-known that no comments are needed here.

The special cell test developed by Menard (c) yields horizontal stress-deformation pattern, from which one may obtain information pertaining to settlements assuming elastic, isotropic conditions. To my knowledge, however, no unique two-dimensional strength data on effective stress basis has yet been obtained. The speaker has no personal experience with the application of the Menard pressiometer.

My discussion will therefore be focused on the plate and the cone tests, and particularly on their applicability for effective stress parameter determinations.

Plate tests

By means of screw plates one can carry out load tests at various depths and thereby obtain parameter profiles.

A settlement parameter profile is shown in principle in Fig.3. For details in design of the field compressometer, the interpretation procedure, and the data obtained, see the article by Janbu and Senneset in Session 1 of this Conference, Vol.1.1 pp.191-198.

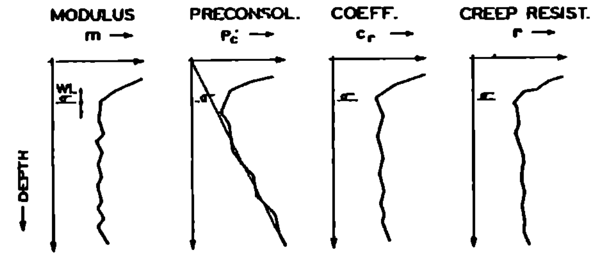

Fig.3 Typical in situ settlement parameter profiles obtained by screw plate tests (field compressometer) in homogeneous sediments along the shores of the Trondheim fjord

The field compressometer has hitherto been used for silt, sand, gravel, overconsolidated clay, and silty clay. So far the maximum penetration depth of the present instrument is 27 meters into fine sand.

Failure loads obtained by plate tests performed at various depths may also yield effective shear strength parameters (Janbu 73). When shear strength is expressed in terms of attraction and friction the net ultimate bearing capacity $q_{un}=q_u-p'$ at a depth of overburden p' is equal to

$$q_{un}=N_{qn}(p'+a) \qquad (3)$$

Hence, by plotting q_{un} as function of p' one obtains the attraction a as the intercept on the p'-axis while the slope N_{qn} determines $\tan\phi'$ from the theoretical and experimental solution of $N_{qn}=f(\tan\phi')$.

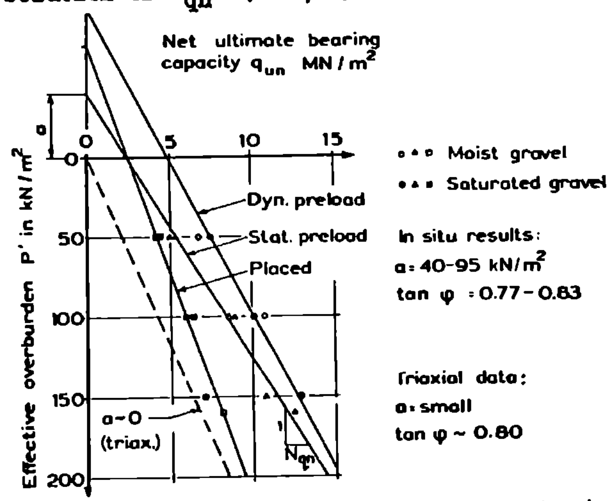

Fig. 4 Strength parameters a and tanϕ obtained from (drained) in situ plate load tests.

An example is shown in Fig.4 which is extracted from a paper by Senneset and Janbu (1973). The plate load tests were carried out by Eresund (1972) on gravel placed in a field station, and tested at the placement condition, the statically preloaded, and the dynamically preloaded condition. Drained triaxial test data are also available for comparison.

In short, dynamic preloading with very little decrease in porosity leads to increased attraction (or cohesion) while friction remains roughly the same. Improved particle interlocking probably accounts for increased attraction. The overriding question for future research may seem to be the durability and reliability of the measured in situ attraction for actual design.

Static cone tests

If the ultimate net point resistance obtained by means of drained in situ static cone tests at various depths are plotted as function of effective overburden, one can use the principle expressed by Eq.(3) to obtain attraction and friction. An example is shown in Fig.5, published by Senneset and Janbu (73), and based on tests performed by Eresund (72).

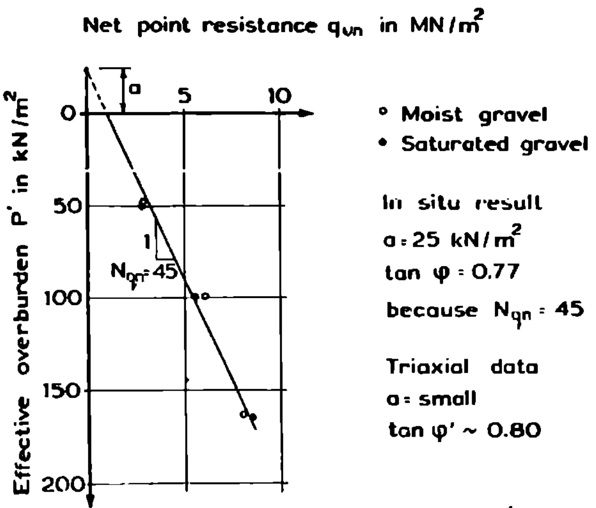

Fig.5 Strength parameters a and tan ϕ obtained from (drained) in situ static cone resistance.

With the improved interpretation principles drained static point resistance tests may now seem considerably more promising for in situ determination of soil parameters, particularly for shear strength parameters on effective stress basis. To clarify the applicability of the principle for various soil sediments systematic research are encouraged.

Summary

In summarizing, in situ load tests on plates and cones, properly used and properly interpreted, have already yielded a number of useful in situ soil parameter profiles for different soil layers. The crew plate has so far been very successful in sandy sediments.

However, the speaker is firmly convinced that internationally we can rapidly improve our skill in obtaining much more reliable in situ parameter profiles for stability and settlement predictions. The first step consists of learning more about the true action of plates and points being loaded at the interior of various soil sediments. Simultaneously we must improve the design of the instruments using these devices, and above all we must learn to interpret the field measurements within exactly the same frame of reference as we use in our predictive process. At least, this is the policy we will follow in our own efforts in the years to come. Thank you for your kind attention.

References

ERESUND, S. (1972): "Sättningar hos cirkulära stela fundament pa friksjonsjord. Inverkan av dynamisk förbelastning". Dr.-thesis, CTH, Göteborg

JANBU, N. (1973): "Shear strength and stability of soils". The NGF-lecture 1973, Oslo

SENNESET, K. and JANBU, N. (1973): "Feltkompresometer og trykksondering for bestemmelse av styrkeparametre". Trycksonderingsdag, Stockholm

Chairman Prof. L. Suklje
Thank you very much Mr. Janbu. Now I would like to call on Mr. Trofimenkov Ju.G. Will he come up, please?

Mr. Trofimenkov Ju.G. (USSR)

This discussion refers to the first subject proposed by the General Reporter: the in-situ determination of reliable soil parameters for predicting stability.

During recent years pressuremeter tests are being widely carried out for determining the deformation modulus of soils in-situ. The straight part of the graph, u=f(p), obtained by the pressuremeter test is used for the determination of the modulus of deformation of soil. With the increase of pressure the zone of plastic deformation around the borehole is formed and the relationship between pressure and deformation becomes nonlinear.

The use of the nonlinear part of the graph u=f(p) makes it possible to determine very easily shear strength parameters φ and c (Trofimenkov, 1973). This gives way to a wider use of pressuremeter tests. These tests and vane tests can compliment each other nicely as vane tests are employed more particularly for soft clay and permit no separate determination of φ and c.

The problem of the expansion of axially symmetrical cavity was solved by Ruppeneit K.V., Bronstein M.I. (USSR) and Vesic A. (USA). For soils with cohesion only this problem was solved earlier by Menard L.

Soil deformation versus pressure, u=f(p) after taking into account pressure losses on the expansion of the pressuremeter, takes the form shown in Fig.1.

There is the following relationship between pressure and soil deformation developed by Ruppeneit and Bronstein:

$$U = \frac{1+M}{E} r_0 \sin\varphi \; (P_0 + c\,\mathrm{ctg}\,\varphi)$$

$$\left[\frac{P + c\,\mathrm{ctg}\,\varphi}{(1+\sin\varphi)\,(P_0 + c\,\mathrm{ctg}\,\varphi)}\right]^{\frac{1+\sin\varphi}{\sin\varphi}} \qquad (I)$$

If the function "u" is divided by its derivative we shall obtain.:

$$\frac{U(P)}{U'(P)} = \frac{P\sin\varphi + c\cos\varphi}{1+\sin\varphi} \qquad (2)$$

The right part of the expression (2) is the half difference of principle stresses, σ_r and σ_θ at the borehole and hence is equal to the radius of Mohr's circle:

$$\frac{\sigma_r - \sigma_\theta}{2} = \frac{p\sin\varphi + c\cos\varphi}{1+\sin\varphi} = R \qquad (3)$$

where R- radius of Mohr's circle.

As evident from the expressions (2) and (3) the ratio of the function "u" to its derivative is equal to the radius of Mohr's circle. At the same time it is known that the ratio of any function to its derivative is equal to subtangent. Thus the radius of Morh's circle is equal to subtangent.

Hence, we obtain a simple method of drawing Mohr's circles: we draw the tangent to the point in the grapn corresponding to pressure, P_I. The intersection of the tangent and the axis p gives the point that determines

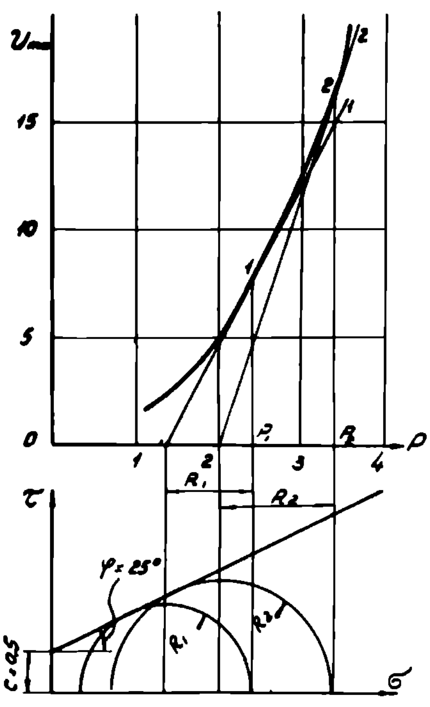

Fig.1. Determination of φ and c on the graph of pressuremeter test.

the subtangent, i.e. the radius of Mohr's circle for pressure P_I. The radius of Mohr's circle for other pressures (p_2, p_3) is determined in the same way. Knowing radii (R_I, R_2) we can draw the Mohr's circles and by drawing the tangent to them we obtain as usual the angle of internal friction and cohesion. The use of the described method has shown that determined in such a manner φ and C are very close to those obtained by the usual laboratory methods on undisturbed samples.

It should be emphasized that by use of pressuremeter it is possible to run both drained and undrained tests.

Thus, the most important characteristics of soils for practical use: the modulus of deformation, the angle of internal friction and cohesion can be determined on the data of one pressuremeter test.

In conclusion it should be noted that it need not oppose field tests to laboratory tests- they supplement each other, especially in designing of complex structures.

Reference:

TROFIMENKOV Ju.G. The Practical Method of the Determination of φ and C from the Pressuremeter Test Data. "Osnov. Fund.MechGrunt.", 1973, N 3.

Chairman Prof. L.Suklje

Thank you very much Mr.Trofimenkov. Mr. Menard will you continue?

Mr.Menard Louis (France)

Following the various publications relating to the pressuremeter, as well as the friendly reminder of President Ganichev on the utilization of this technique within the USSR, I feel it is my duty, some fifteen years after the original presentation at the London conference, to stress the main points of this Technique.

1) This Technique cannot be properly analysed except within the framework of a more general theory of soil deformation which involves the three basic moduli:

The compression Modulus E^+
The Tension Modulus E^-
The shear modulus G

The theory of elasticity being a simplified case of this general theory.

2) This general theory can now be utilized for the computation of settlement with the help of a program based on the finits elements theory but adapted to analyse elementary cubes dx,dz,dy, as in any conventional analysis.

3) This analysis yields strength and settlement values identical to those obtained previously through experimental Testing and which were published some years ago.

4) Pressuremeter results are very closely tied to the shear modulus G and to the shear resistance of the material in the in-situ state of stress conditions.

The shear modulus, which practically can be measured only by the pressuremeter,plays a predominant role in the deformation of a soil beneath a foundation when the ratio of the width of foundation to the thickness of compressible soil is less than 1.

5) The actual mechanical caracteristics of the founding material as well as the modulus and shear strength are dependant on the strain and the amount of remolding; this requires very strict definition and therefore any worth-while correlation between the measuring instrument and the performance of the actual foundation requires that the in-situ conditions of remolding and strain be identical in both cases; as these conditions vary with the types of foundations (driven piles,bored piles,spread footings) it is evident that the techniques in the placing of the pressuremeter must vary also.

6) Further Technological development of the pressuremeter for practical applications can be justified only if a parrallel effort is made in the analysis and general theories related to the deformation and strength of soils.

We should pay particular attention to the main problem of remolding which in most cases is a problem of liquefaction.

Liquefaction occurs readily in materials with a weak structure (loose sands,silts, clays) and results in a partial or even complete destruction of it's fabric when subjected to rapid alternate loadings.

This is why it occurs frequently at the points of driven piles, split spoon samplers and penetrometers when in this case the values of $N_c = \frac{\text{Rn}}{c}$ vary between 2 and 3 instead of 10 and 15.

Liquefaction also occurs under large foundations such as spuds of petroleum jack-up barges, oil tanks, or even motorway em - bankments when these have been raised too quickly; it results in a large increase in settlements.

It appears again,but in a beneficial way, during the process of Dynamic Consolidation a New Technique which we have developed for the improvement of founding materials.

It has now become indispensable to measure the parameters controlling this phenomena and to achieve this purpose we have retained the pressuremeter tests with rapid cyclic loadings.

Chairman Prof. L.Šuklje

Thank you very much Mr.Menard

Ladies and gentlemen, it is time for 20 minutes intermission.

Chairman Prof.L.Šuklje

I would like to call on Prof.B.Broms of Swedish Geotechnical Institute. Will he come up please?

STABILITY OF EMBANKMENTS AND EXCAVATIONS IN SOFT CLAYS. Bengt Broms (Sweden)

The following comments are concerned with the stability of embankments and excavations in soft clays.

Bjerrum (1972) has shown on the basis of published data that the stability of embankments on soft clays is overestimated if the analysis is based on the undrained shear strength evaluated from field vane tests and the plasticity index or the liquid limit of the soil is high. In Fig. 1 the nominal factor of safety at failure for the cases analyzed by Bjerrum has been plotted as a function of the liquid limit of the soil. It can be seen that the nominal factor of safety increases with increasing liquid limit of the soil.

This increase can probably at least partly be attributed to rate effects. In Fig. 2 is shown the vane strength as determined at two different rotation rates, 60 degrees/ min and 0.06 degrees/min. These tests were carried out in a soft clay which close to the surface was organic. At the bottom the clay was warved. It can be seen from this figure that the liquid limit of the clay close to the surface was high and that it decreased with depth. The measured shear strength decreased with decreasing rotation rate. At a depth of 2.0 m where the liquid limit of the soil was 170 the measured shear strength at a rotation rate of 0.06 degrees/ min was only 60% of that at 60 degrees/min. The corresponding shear strength at 10.0 m depth where the liquid limit of the soil was about 80 was approximately 80% of that at 60 degrees/min (Wiesel, 1973).

John Olsson, the Secretary of the Geotechnical Committee of the Swedish State Railways recognized already in 1930 from analysis of slope failures that the undrained shearing strength as evaluated by, for example, the fall-cone test is overestimated when the liquid limit of the soil is high. He suggested that the undrained shear strength should be reduced when the liquid limit of the clay exceeds 70, as illustrated in Fig. 1 (Osterman, 1960). The reduction coefficients proposed by Olsson have resulted in the stepped curve shown in the same figure and in Table 1.

Table 1.

Reduction of shear strength as measured by vane of fall-cone tests (SGI).

Liquid limit w_L	Reduction coefficient
< 80	1.0
80-100	0.9
100-120	0.8
120-150	0.7
150-180	0.6
>180	0.5

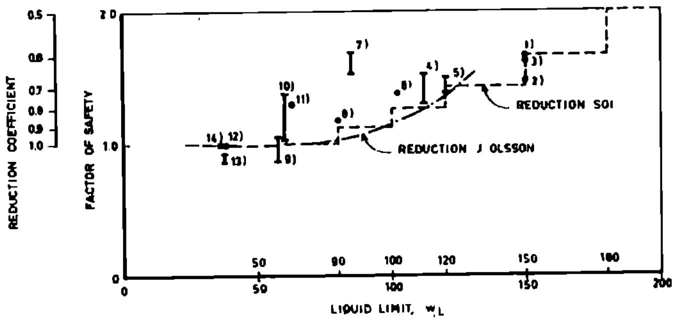

Fig. 1. Reduction of undrained shear strength of clays

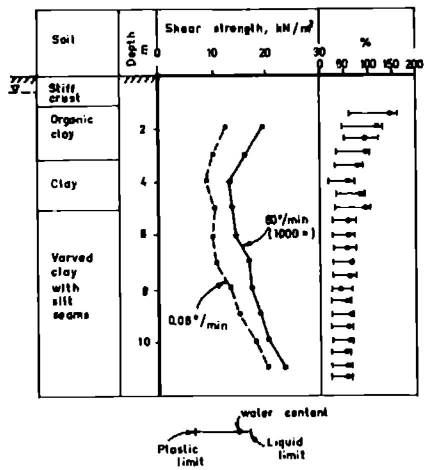

Fig. 2. Rate effects as determined by field vane tests (after Wiesel, 1973)

A reduction has been used in Sweden since about 1950 in the evaluation of the results from field vane and fall-cone tests.

R E F E R E N C E S:

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Chairman Prof.L.Šuklje

Thank you very much Prof. Broms for your interesting discussion,

Now we shall listen to Mr Nelson from Asian Institute of Technology. Mr Nelson, will you , please.

Mr J.D.Nelson , Thailand

Professor Lambe has devoted considerable attention to the problems associated with embankments on soft ground and has commended the contribution made recently by Bjerrum (1972) in correlating the factor of safety of an embankment at failure with the plasticity index of the foundation clay. Bjerrum suggested that the theoretical factor of safety was often greater than unity because of the combined effects of rate of shear, anisotropy and progressive failure. I wish to confine my remarks to this correlation, which is shown in Fig.IV-3 of Professor Lambe's report, and to comment on its application to Soft Bangkok Clay.

Figure 1 shows some data additional to that presented by Bjerrum (1972). The dark points represent the theoretical factors of safety at failure for five embankments on Soft Bangkok Clay. These new data points can be seen to fit Bjerrum's straight line correlation fairly well. It is of some interest to note the considerable effect on the calculated safety factor of the shearing resistance of the embankment material itself. This data was available to me only for the Rangsit and Bang Bo embankments. When the strength of the Rangsit embankment is neglected, the factor of safety at failure is calculated to be close to unity, which is well below the proposed correlation line. The Bang Bo embankment on the other hand gives a factor of safety which falls on the correlation line when the embankment strength is neglected and well above it when the strength is taken into account. It might be significant that the Rangsit and Nonthaburi embankments were

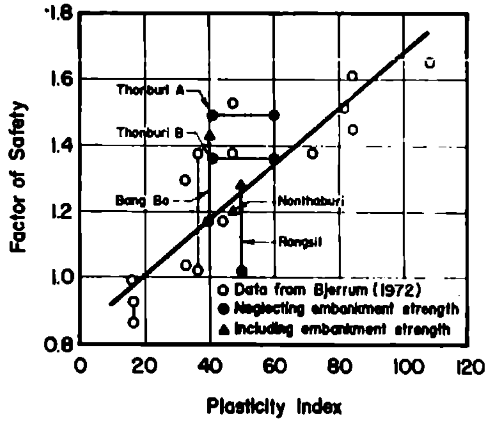

Fig.1 — New data from embankment failures on Soft Bangkok Clay compared with the data of Bjerrum (1972)

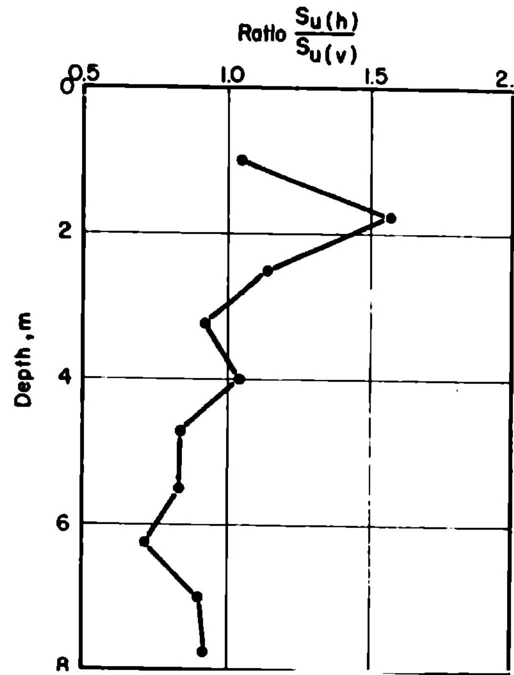

Fig.2. Ratios of horizontal to vertical undrained shear strength of Soft Bangkok Clay at Rangsit

constructed of clay while the other three were of sand.

Data published by Eide and Holmberg (1972) for the Soft Bangkok Clay from Suracha show an appreciable effect of strain rate on the undrained strength measured in the triaxial test. Bjerrum's correlation, however, is based on strengths measured by the field vane, and a few tests conducted with a vane borer recently by the Asian Institute of Technology indicated only a very small effect of strain rate. These tests were carried out not too far from Suracha at a location where the clay has a natural water content of 140%, a liquid limit of 130%, a plasticity index of 70%, an undrained shear strength of about 1.2 ton/m^2 and a sensitivity of ten.

Appreciable anisotropy in respect of strength is known to exist in the Bangkok Clay. Figure 2 shows the ratios of undrained strengths on the horizontal and vertical planes as measured by direct shear tests on samples from Rangsit. $S_{u(h)}/S_{u(v)}$ is greater than 1.0 only to a depth of 3m because of vertical fissures caused by surface weathering (Moh et al, 1969). At greater depths, this ratio is considerably less than 1.0, as is common for normally consolidated clays. The commonly used field vane has a height/diameter ratio of 2.0 and measures almost exclusively the strength on a vertical plane (Cadling and Odenstad,1950). This fact alone might be sufficient to account for theoretical factors of safety at failure which are well in excess of unity. It would be interesting to see the theoretical factors of safety recalculated on all the embankments represented on Fig.1 to take account of strength anisotropy.

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Chairman Prof.L.Šuklje

Thank you Mr Nelson Mr.Burland,please.

Mr.J.B.Burland (England)

In recent years we at the Building Research Station have become increasingly aware of the limitations of the routine approach to site investigations and design methods based on the examination and testing of small borehole samples. These limitations have become apparent as a result of our concentrated efforts to observe and measure the performance of full scale structures in the field. Most of our work has been concerned with structures founded on or in the stiff, fissured deposits of the Triassic, Jurassic, Cretaceous and Eocene systems which cover much of the densely populated middle and south east regions of England. We have been startled by the very poor agreement between the in-situ properties of the ground obtained by back analysis of the observed full-scale behaviour and the properties measured in the laboratory.

An example is the full-scale loading test that we carried out on the Chalk at Mundford in 1967 in which we made very careful measurements of settlements at various depths beneath and around the loaded area (Ward, Burland and Gallois, 1968). When the observations were analysed we found that the in-situ stiffness of the chalk is between 20 and 100 times greater than measurements based on laboratory tests and small diameter plate loading tests. Subsequently very careful large diameter plate tests at various depths were carried out and very good agreement with the full-scale results was obtained (Burland and Lord, 1969; Burland, Sills and Gibson, 1973). Already this work has resulted in the saving of many thousands of pounds by eliminating piles from most foundations on Chalk.

Another example is in the London Clay where the back analysis of heave measurements and retaining wall movements has given stiffnesses which are five times greater than very careful laboratory tests and up to twenty times greater than routine laboratory tests (Cole and Burland, 1972).

It has become evident that many stiff fissured clays and soft rocks are extremely sensitive to sample disturbance and laboratory determined properties are very unreliable. In an effort to improve our methods of prediction we have been concentrating on the development of equipment and techniques for carrying out large diameter plate loading tests at the bottom of shafts. The equipment we have developed (Fig.1) allows a test to be completely set up within an hour or less of boring the shaft (Marsland, 1971a). In this way softening of the ground beneath the plate is kept to a minimum and the costly time element of this type of test has been reduced substantially. Up to three tests a day can be carried out. The measured stiffnesses are within a factor of two of the stiffnesses obtained by back analysing full-scale observations (Marsland, 1971b) Moreover, very consistent values of shear strength are obtained often approaching the lower limit of the very wide scatter of laboratory strengths.

A recent development (Marsland and Eason, 1973) has been to measure the settlement at various depths beneath the plate as shown in Fig.2 (Marsland and Eason, 1973). These measurements permit the determination of

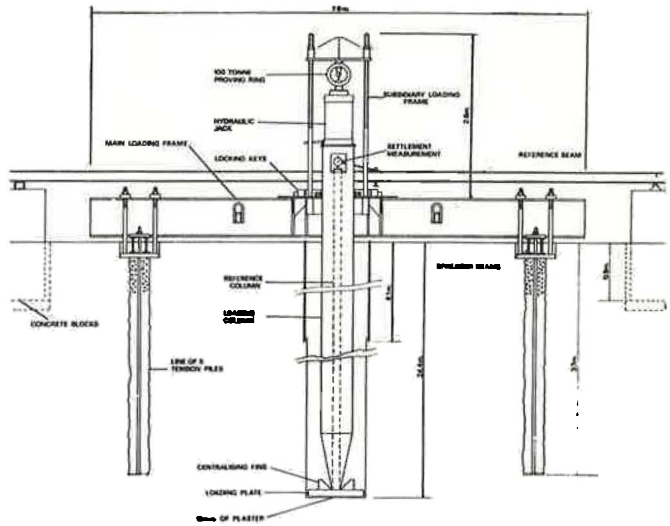

Fig.1.

stiffness in the almost totally undistrubed ground remote from the plate and agreement with the full-scale values is excellent. Also the stiffnes of ground immediately beneath the plate which has undergone stress release, can be measured and is found to be substantially lower than the undisturbed material lower down. Measurements of this type will throw light on the softening effects of stress release as at the base of excavations or during sampling. The equipment is now being developed for use in water filled shafts.

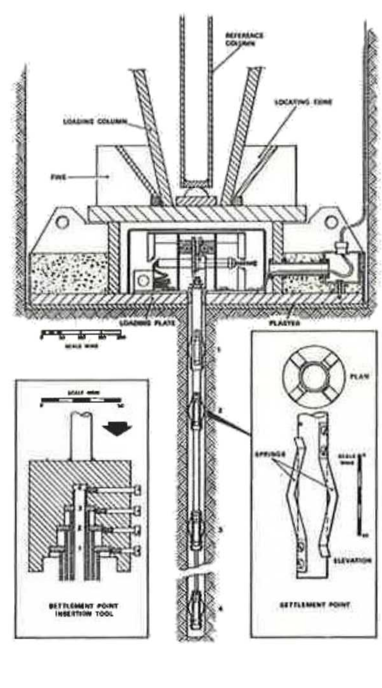

Fig.2.

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Chairman Prof. Šuklje.

Thank you very much Mr. Burland for your discussion. Mr. Marsal will you continue?

Mr. Raul J. Marsal (Mexico)

As pointed out by the General Reporter, the field situation, mechanisms of failure, methods of analysis and strength parameters are unseparable components of a predictive process. For overconsolidated clays in a natural slope, the geological history of the formation is of paramount importance, particularly in relation to the presence of fissures and fractures caused by tectonic forces, slides or other. The attached table contains a simplified classification of these materials. Case 1, designated intact soils, includes from thin layers to thick deposits of overconsolidated clays that are neither fissured nor crossed by fractures; most of clayey soils with a small overconsolidation ratio belong to this group. Case 2 refers to clay deposits that are from moderately to intensely fissured, due to shear forces developed close to a fault or by unloading. Finally, case 3 comprises those overconsolidated clayey formations that are affected by fractures of tectonic origin or slip surfaces induced by old slides.

In case 1, one can often secure undisturbed samples and determine the strength parameters at the laboratory. Sampling and laboratory work become difficult, if not impossible, in case 2; therefore, assessment of the relevant soil conditions by field investigations (tests and observations of natural slopes) are necessary. A careful geological exploration is needed in case 3, to determine the existence and distribution of fractures or surfaces of shear failure; however, it is usually problematic to identify these fractures by standard sampling operations. Test pits or large diameter boreholes may be required to detect the above discontinuities of the mass and recover samples of the material filling them, since the thickness involved can be of the order of millimeters.

The determination of strength parameters in case 1 is normally done by means of strain-controlled, triaxial compression or simple shear tests. Through drained tests one obtains parameters $\bar{c}_p$ and $\bar{\phi}_p$ (peak values); for determining the angle of residual friction $\bar{\phi}_r$ large deformations are required, which can be achieved by repeated direct shear or rotating shear. With the same type of tests and thoroughly remolding the clay, the parameter $\bar{\phi}_s$

OVERCONSOLIDATED CLAYS

Case	(1)	(2)	(3)
Type	INTACT	FISSURED	FRACTURED
Samples	Undisturbed	Sampling difficult	Location and sampling of weaknesses problematic
Tests	Drained triaxial or simple shear	Field-drained, direct or rotating shear	Drained direct or rotating shear along weak planes, or with remolded sample of filling
Parameters (field conditions)	$\bar{c}_p$ and $\bar{\phi}_p$, at peak (small strains) $\bar{\phi}_s$, remolded (moderate strains) $\bar{\phi}_r$, residual (large strains)	$\bar{\phi}_r$, residual (all problems)	$\bar{\phi}_r$, residual (mechanism controlled by weaknesses)

is obtained. The selection among the above parameters depends on the problem on hand. If small strains during construction and afterwards are anticipated, $\bar{c}_p$ and $\bar{\phi}_p$ are introduced in the design of the structure, e.g., rigid walls for a deep excavation; expansion due to unloading could be most important in this type of problem. Based on considerations of the stiffness and the brittleness of the clay ($\bar{\phi}_p/\bar{\phi}_r$ and $\bar{\phi}_s/\bar{\phi}_r$ ratios), expansivity, effects of weathering, variation of water pressures, etc., as well as the characteristics of the structure proper, one should expect moderate strains; then, the stability analisis is performed assuming null cohesion and a linear strength envelope with an angle of inclination $\bar{\phi}_s$. This apply to a cut similar to that shown in Fig V-1 of the General Report. In this example, the factor of safety varies with time from a maximun value estimated with parameters $\bar{c}_p$ and $\bar{\phi}_p$ to a lower one computed with $\bar{\phi}_s$, as deformation of the soil develops. The residual strength given by $\bar{\phi}_r$ is relevant to problems which may involve large strains of the mass. The terms small, moderate or large deformations are intimately related to the stress-strain behavior of the soil as illustrated by Figs V-2 and 3 of the General Report.

To find out the shear strength of a fissured overconsolidated clay (case 2), large scale field tests are required. These tests have been performed in severals projects by means of direct shear apparatus or rotating shear devices, and they are carried out slowly (strain-controlled) so as to have complete dissipation of the induced pore pressure throughout the loading process. The strength envelopes are usually straight lines passing through the origin, provided that large enough displacements are imposed to the soil. The angles of inclination of these envelopes are the residual values $\bar{\phi}_r$ and, since they are the minimun ones, one should not expect a further decrease of the factor of safety with time.

The mechanism of failure in case 3 is generally related to the location and orientation of the weak surfaces. Therefore, it is necessary to determine the shear strength of the clay contained in the fractures or slip surfaces. When undisturbed samples that include these weaknesses are obtained, drained shear tests (direct or rotating) along the fracture or slip plane should be performed in the laboratory. Sometimes sampling is difficult, but remolded specimens of the material in the weak surfaces can be secured; then, drained-rotating or direct shear tests are run to determine the parameter $\bar{\phi}_r$. In other instances, neither type of sampling is feasible and the engineer must evaluate the shear strength based on field evidence and past experience.

The above considerations discard the possibility of a straight forward procedure of analysis; sound engineering judgement is thus essential when dealing with overconsolidated clays.

Chairman Prof.Šuklje L.

Thank you very much Prof.Marsal. We greatly appreciated your report; we will now have a short break.

Recess

Chairman Prof.Šuklje.

We will now hear the prepared discussion of Dr. Craig.

Dr.W.H.Craig (England)

The General Reporter has asked for discussion on the utility of centrifuge model tests for determining parameters for prediction. He has pointed to a need to determine both 'parameters' and 'mechanisms'. The two are inter-related and the choice of parameter depends upon the selection of a mechanism.

The centrifuge at Simon Engineering Laboratories in Manchester described by Rowe (1972) is currently being used to determine the mechanisms of deformation and failure of a range of structures including:

a) Circular Diaphragm Walls (Fig.1)
b) Flexible Sheet Pile Walls (Fig.2)
c) Earth Dams
d) Foundations for Oil Rigs in the North Sea
e) Excavations in natural soils

Figure 3 shows a simplified model of a vertical trench excavation in a large block sample of fissured Lower Lias clay obtained from site following a field failure previously reported by Rowe. This model was brought rapidly to failure under an increasing Centrifugal acceleration and reproduced at an acceleration closely approximating the field scalo a behaviour similar to that of the prototype, where a narrow column of soil adjacent to the face became detached from the mass and fell forwards into the excavation. The trench was designed to stand open for a short period on the basis of particular shear strengths and an implicit mechanism of rotational shear failure. The actual failure both in the model and in the field, took place by a mechanism now attributed to brittle fracture followed by a buckling failure of the detached soil column- such a mechanism was not visualised at the design stage.

Given that a model can indicate the mechanisms of behaviour, it is possible to use the results from that model either directly, to make predictions for the particular prototype modelled, or more generally, to select and determine those parameters which will fit methods of analysis for that model in order to extrapolate predictions which should be accurate for a field prototype which may differ slightly from the one modelled

Models tested of new reservoir embankments have yielded information (Fig.4) on the shape and position of critical slip surfaces and the distribution of pore pressures around these surfaces, allowing selection of parameters which fit particular methods of slope stability limit analysis or finite element deformation analysis.

For successful application of the centrifuge technique models must be sufficiently large to include representative geological features of natural strata, to allow the smaller critical features of the prototype to be modelled reliably and to allow sufficient time for realistic pore pressure changes to be monitored. Models up to 2 tonnes mass have now been tested at S.E.L. and while the cost of natural samples included in models of this size can be relatively high, the cost-benefits to the designer may be greater than those from extensive programmes of more conventional laboratory testing.

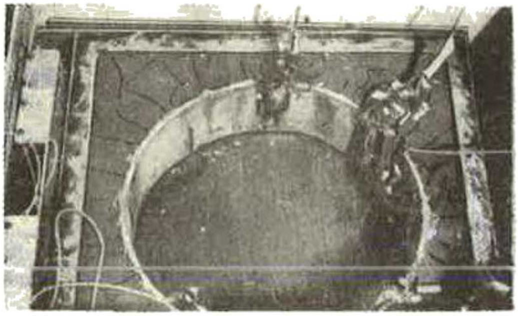

Fig.1. Circular diaphragm wall (1m square)

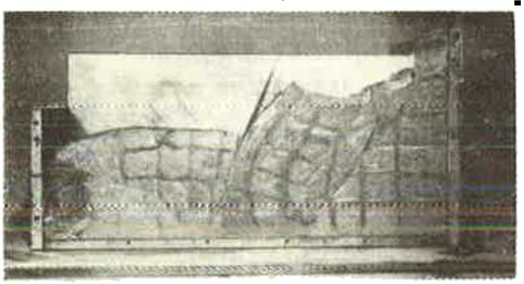

Fig.2. Anchored flexible wall after tie rod failure. (model length 0.65m)

Fig.3. Excavation in stiff fissured clay (model width 0.67m)

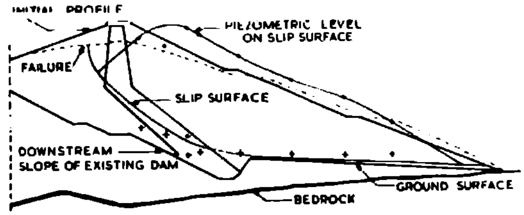

Fig.4. Model of Proposed Raising of an existing embankment (model length 1.7m, scale 1:100)

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Chairman Prof. Šuklje.

Thank you very much for your discussion I would like to call on Prof.Goldstein of the USSR. Will he come up, please?

Prof. Goldstein M.N. (USSR)

As is known the original for a mathematical model is not the immediate nature but its physical model. From experience follows that the main source of data which are necessary for the development of physical models are fullscale experiments. Our congress shows that the quantity of such experiments is steadily declining. That may be explained but not justified by their complexity and expensivness. But without them soil mechanics will inevitably come to the deadlock, as there will appear more and more theories which are too far from the real geological conditions and from the genuine character of the base-structure interaction. It seams to me that now some gap is being outlined between the soil mechanics methods and the practice of engineering design.

This may be seen for example from the appearance of articles, which express some disappointment in soil mechanics. Even our discussion is divided into two parts: one-theoretical and the other- practical designs. This gap is also manifest in the accuracy of the settlement and stability design being only slightly increased in the recent 20-30 years.

Strictly speaking this increase is due mainly to empirical methods as for example the design of the settlement of large foundations, as shown in an exellent work by prof. Yegorov. So the first and most important conclusion that can be drawn at our congress is, in my opinion, the necessity of an intensive development of fullscale experimentation.

Now about the necessity of dividing present soil mechanics into proper soil mechanics and the mechanics of dispersed systems. All pure theoretical investigations, having no immediate implementation in practice must be referred to the mechanics of dispersed systems which we consider as the science of the fundamental cycle in distinction of soil mechanics as an applied science.

From this point of view the applied soil mechanics should be considered as the mechanics of foundations and soil structures, while the mechanics of dispersed systems must become a parallel to the mechanics of continuous media. The best principle method of soil mechanics development permitting to link the latter with its base- engineering geology is the so called method of conditional design created by Gersevanov and published in his distinguished, not aging, work "The application of mathematical logic to structure design". This method permits us to use even very simple models, but only if they satisfy two main requirements: first, taking into account the most essential factors of foundation work and, secondly, placing the structure in less favourable conditions than in reality.

The results obtained are compared with the observation and test data and are corrected by empirical coefficients or factors of safety, included in the codes.

The settlement design called the "active layer method" fully comply with this idea. This method is to my mind the best of all the existing for settlement design.

Some think the appearance of computers requires the transition to more complicated design methods. It is not so. Not the complication but the simplification of mathematical models with the simultaneous improvement of physical ones is the call of modern time. The rapid propagation of the finite elements method is a convincing proof of this.

The wide accepted model of stress distribution borrowed from the theory of elasticity has already proved its invalidity for discrete media. As it seems to me the problem of stress distribution is the problem number one in soil mechanics.

The investigations conducted in our laboratory by A.G.Dorfman show the wide prospects opened by using the variational methods both in soil mechanics and in the mechanics of dispersed systems. This trend deserves intensive development.

Chairman Prof. Šuklje.

Thank you very much Prof.Goldstein.

We will hear Prof.J.M.Dunkan from University of California, Berkeley.

Prof.Dunkan J.M. (USA)

Mr.Chairman, Professor Lambe, ladies and gentlemen:

On the topic of modelling, I would like to address myself to the question of how an engineer should determine parameters for use in mathematical models. In particular, how should an engineer determine the strength for designing excavations in clay?

I am very glad to have an opportunity to comment on the question of how one should determine the strength for predicting the stability and movements of a supported excavation in clay. I think, in fact, that the answer to this question is the same as the answer to another question: "How do you teach a crocodile to shake hands?" The answer is "Very carefully".

Professor Lambe has pointed out clearly in his report that we cannot separate the data we use from our method for using it. It is not possible to say how one should go about determining the shear strength of clay for excavation bracing without saying what procedure will be used to design the bracing.

If one uses an empirical method for estimating movements and strut loads, such as the method developed by Professor Peck, it is of course only appropriate to determine the strength in the same way as it was determined in the first place. In the case of Peck's rules the strength should be determined by the unconfined compression test.

The last contributions of Dr.Bjerrum of Norway emphasized the fact that the strength of clay measured in laboratory tests or in vane shear tests is not necessarily the same as the strength which can be mobilized in the ground. The strength measured in laboratory tests or vane shear tests should be corrected for the effects of rate-of-loading and anisotropy before they can be applied to practical problems. The magnitude of the required correction must ultimately be determined by comparing predicted and calculated behavior. Therefore our methods are all partly empirical because we try to increase the accuracy of our predictions by making corrections to our data based on experience.

When we use a new method, for example the finite element method, we don't know in the beginning what corrections we should make. Such a method should not be used by itself for design until the necessary experience has been accumulated. In the case of compacted sand, experience so far shows that no large corrections are needed. In the case of clays, however, the first indications are that rather large corrections are required in some cases. We have therefore to analyze a sufficient number of cases to determine what corrections we should make before any new method is used by itself for design.

Chairman Prof. L.Šuklje

Thank you very much Prof.Duncan for your discussion. We will hear now Prof. G.Gudehus.

Prof. G.Gudehus (West Germany)

A few points can be raised concerning the topics No.3,5 and 6 proposed by the general reporter.

1. Strength and deformability of soils are characterized by hardening and softening. In clayey soils this is not only plastic, but also viscous. Examples are given in a forthcoming paper (to be published in "Der Bauingenieur", 1973) two of which are briefly described here. Hardening: A normally consolidated soil be loaded in such a manner that a short-term failure does not occur; deformations develop with compression and decreasing rate, i.e.viscosity increases; the amount of deformations is relevant. Softening: An excavation in overconsolidated clay (with or without support) be initially stable; in the course of time expansive deformations may occur; cohesion and viscosity diminish and the rate of deformation increases;

progressive failure can be reached; the factor of long-term stability is relevant.

2) In centrifuge tests the time-scale laws for shear viscosity and for pore pressure dissipation (Terzaghi's law) cannot be satisfied simultaneously. However, from the reports on centrifuge tests it is known that the time lapse up to failure in some clay models obeyed Terzaghi's law. This means that at least in some cases dissipation is the dominant factor for time-dependance which is very promising for the centrifuge technique. It also means that "cohesion" of overconsolidated soils is mainly due to suction and can vanish in the course of time.

3. Concerning finite element methods three comments can be made:

a)As yet no allowance was made for viscous hardening or softening.

b) Some solutions allowing for strain-softening (contribution by Lee and Lo to this conference,e.g) are generally not kinematically correct. In the "initial-stress" iteration the flow role (including dilatancy) is not allowed for. If the obtained solutions are statically admissible these are not necessarily on the safe side as the lower bound theorem of plasticity does not hold.

c) As yet no allowance was made for the difference between critical and residual angles of friction.

Chairman Prof. Šuklje.

Thank you very much Mr.Gudehus for your discussion.I would like to ask Dr.Zlatarev to present us with his prepared discussion.

Dr. Zlatarev K.A. (Bulgaria)

Soils,considered as a dispersed system,represent in fact a completely suitable field for the application of mathematical statistics and of the recently developed new trends of modern mathematics, such as the theory of random functions, of random fields.

The efficiency and future possibilities of these subjects to supply a more reliable and most economic appraisal of the building properties of soils are doubtless to our opinion.

In Bulgaria these sections of mathematics are applied for the solution of the following problems in the field of soil mechanics and engineering geology:

1. Determination of the minimum necessary number of samples and of laboratory tests.

On the basis of our studies published in 1964 in the Transactions of the Geological Institute of the Bulgarian Academy of Sciences and in the Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering /1/, as well as on the basis of normed probability and accuracy, depending on the class of the structure and the stage of study the necessary minimum number of samples is established. Thus corresponding tables are set up, for example in the "Provisional Instruction for the Type and Amount of Geological Investigations in Engineering Geological and Hydrogeological Studies" drawn up in the Study and Design Institute Vodproekt. The Minimum number of samples is obligatory for the laboratory tests of the concrete aggregates, of soils, of stone for embankments and inverse filters.

2. Determination of calculation data of soil mechanics indices for the solution of geotechnical problems.

An essential part of the "Soil Design for Embankments and Compaction Works" which we have introduced in the practice of the Study and Design Institute Vodproekt and the other consulting engineering institutes, is the section for calculation data. These are determined in compliance with the instructions and codes of practice /e.g.2/ by applying the methods of mathematical statistics (confidence interval, coefficient of non-uniformity, adjustment by the method of least squares,etc.)

3. Establishment of correlation between soil mechanical properties.

The correlation of some rapidly and easily determined indices (such as I_p, W_{opt}, etc.) with others which are more difficult to evaluate (e.g.compression, shearing strength, collapse,etc.) is widely applied.

4. Application of the theory of variability to the methods of engineering geological studies.

With a view to reducing the investigation time and increasing the reliability of the engineering geological forecasts,new more rational and scientifically based methods are being introduced also in Bulgaria by using corresponding means for their realization. On the basis of the statistical analysis of the variability of the engineering geological properties of soils the working hypothesis is modified and improved.

Concluding we should say that our long practice clearly shows that we are on the right trail.

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Chairman Prof. L.Šuklje

Thank you Dr.Zlatarev. Mr.Pevzner, please.

Mr.Pevzner M.Y. (USSR)

"EFFICIENCY AND PROMISES OF IDEAS OF PROBABILITY AND MATHEMATICAL STATISTICS THEORY FOR ESTIMATING SOIL PROPERTIES"

At present there exists a wide range of methods for estimating soil mechanical properties with similar accurancy. In such a situation the choice of procedure for treatment and estimating the data is finding increasing importance and Respectable General Reporter Professor Lamb has undoubtedly put under discussion a very urgent problem.

It should be noted that in soil mechanics both Probability and Statistical methods are not widely-used for treatment and estimating the results. Investigators are usually aimed to getting practically reliable data through the use of the theory of safety limits, calculate a variation coefficient and solve some other elementary problems, the limit of safety being defined purely subjectively. Experience of large-scaled engineering and geological research on building sites and mineral deposits, however, has necessitated a general technique of statistical estimation and treatment of soil and rock characteristics. Mathematical aspects of Probability theory and mathematical statistics are known to be developed rather carefully. But simultaneously, there exist reasons for excessive attention to mathematical exercises and thus the accuracy of the results obtained will be much higher than that of the original data. Therefore it's very important to clarify the range of methods for effective solving particular practical tasks.

We consider that studies of soil properties
1068 involve problems of 2 classes. In class 1 can be subclassed. Subclass A presumes operations over mono-parameter statistical totality: establishing the law of distribution in the selection, checking the hypothesis of classifying a selection as a particular general totality, calculating the mean value of the characteristics obtained, eleminating irregular magnitudes and so on. An in subclass B we analyse multi-parameter statistical totality with a view to establishing co-relationship among the characteristics. The general scheme of solving tasks of class 2 involves 3-staged checking statistical hypotheses: Stage 1-checking the hypothesis of random distribution of the characteristics in the selection tested. An alternative-evidence of trend. Stage 2-checking the hypothesis of stationary changeability. An alternative- unstationary changeability. Stage 3-checking the hypothesis of ergodic model of a stochastic process. An alternative - an unergodic model.

We have every reason to omit discussing the technique of solving these problems because the necessary information can be obtained from suitable papers. A particular question on the accurancy of the characteristics necessitates considering peculiarities, importance and useful life of an engineering construction. Recommendations on this subject available are not well-grounded.

Wide use of Probability and statistical methods will undoubtadely result in more careful studying dependence of soil Properties on natural condition and evidently improve accuracy and reliability of the results obtained.

Chairman Šuklje, L.

Thank you very much Dr.Pevzner.
We will now hear Mr. Kryzhanovsky.

Mr.A.L.Kryzhanovsky (USSR)

ON THE PROBLEM OF DETERMINATION OF SOIL STRENGTH

In the USSR, as well as in many laboratories around the world the testing equipment has recently found a wide application, in which three-four components of stress or strain condition are imposed in a soil sample.

The results of strength studies of a large variety of soils using such equipment clearly show that the parameters of such notorious strength theories as the More and Mises-Schleicher-Botkin theories largely depend on the ratio between the stress or strain components. In other words, the above mentioned strength theories, which have found a wide application in the engineering practice, are not confirmed by the experimental data because they do not comply with the basic requirement of the stability of the parameters obtained for different types of stress-strain condition. It should be noted that the difference in the parameters amounts to 150% and greater for certain soils.

The problem of soil strength studies is complicated in view of the test data obtained which revealed an important role of the loading history (loading path).The loading history is determined by the sequence of variation of the stress components,rate of their variation,as well as by the rotation of the principal stress axes relative to their original position in the course of the pre-ultimate deformation stage.It should be noted that new equations for ultimate equilibrium state were contemplated in the USSR to cover soil strength with similar loading path which are supported by the experimental data(reported by M.V.Malyshev, Geniev, G.M.Lomize and the author). The structure of these equations does not, however, reflect the role of the loading path. The problem of the effect of the loading path on strength is very important, scarcely studied, and there is not yet analytical generalization of this problem. The efforts are to be applied to the study of the equation for ultimate equilibrium state so as to reveal the explicit dependence of its parameters on the loading path.

The above-mentioned strength regulations cannot be observed by using conventional equipment, such as shear test apparatus or triaxial apparatus. On the other hand, the opportunities offered by the computer methods of calculations make it possible even now to reveal at least the effect of the type of three-dimensional stress condition on soil strength. Therefore, it is absolutely necessary to introduce the equipment incorporating independent stress control feature into the practice of determination of soil strength in developing critical and large-scale engineering projects.

A number of studies (cf.the works reported by S.S.Vjalov, M.N.Goldstein, Ju.K.Zaretskij) revealed the mechanism of the structure formation in soil in the course of plastic deformation. In this regard the destruction of anisotropic sample should be considered. Therefore, it appears improper to consider the angle of internal friction, which remains unchanged for different stress conditions, to be one of the strength characteristics. This conclusion restricts the validity of the data obtained in studying strength with the employment of shear test apparatus as applied for their extension to cover the general case of the three-dimensional stress condition. The field of application of the strength characteristics obtained in a shear test apparatus should be limited to those calculation methods, where the slip surface shape is preliminary assumed.

To summarize, it should be emphasized that the abovementioned peculiarities of soil strength, when taken into account in solving the engineering problems within the limits of the first ultimate equilibrium state, not only give the quantitative amendment of calculation results, but in a number of cases the qualitative differences are obtained as compared to the conventional calculation methods. Thus, the computer calculations conducted at the Chair of Soil Mechanics of the Moscow Civil Engineering Institute revealed the formation of the ultimate equilibrium zones at the apex of an "elastic ore" with a uniformly distributed band-shaped loading of a sand base, and the formation of several ultimate equilibrium zones in a composite dam erected from local materials.

Chairman Prof.Šuklje,L.

Thank you Mr. Kryzhanovsky for your discussion Mr.Ter-Martirosjan will you please let us have your discussion?

Mr. Z.G.Ter-Martirosjan (USSR)

COMPRESSION-EXTENSION AS A METHOD OF COHESIVE SOIL TEST

A method of cohesive soil test under the condition of compression-extension* and a device for it have been worked out and introduced into research practice of the above-mentioned institutes by the authors of this report since 1968 and are successfully used both under field and laboratory conditions. At present 1500 tests (including more than 700 tests under field conditions) have been carried out due to easiness and reliability of the method.

The method allows to create in a sample under test complex state of tension which includes compressive and tensile stress, and that is of interest to the engineer and research practice.

The principle of the method is: a special form sample (basing bobbinlike) is placed into a squeezing chamber, and is subjected to hydrostatic head on lateral surfaces through a flexible rubber casing. Apparently that simultaneous compression and extension forces spring in the sample under such conditions.

Compression stress value is equal to the inside chamber pressure, tension stress value along the drum is defined depending on cross-section area ratio of the middle part and the face plane of the sample. The main tension and compression stress ratio is defined from the equation $\sigma_3 : \sigma_{1,2} = (S_1 - S_2) : S_2$ where $\sigma_{1,2}$ – stress on the lateral surfaces, equal to the inside chamber pressure, σ_3 – tension stress, S_1 and S_2 – cross-section areas of the middle part and the face plane of the sample accordingly. The state of tension corresponds to Naday-Lode parameter, equal to +1.

Note that with the certain cross-section area ratio (3; 1,5; 1) there are stressed states which correspond to such cases when the first, the second and the third stress tensor in variants accordingly, equal to 0, but with the cross-section area ratio, equal to 2 the so-called pure shear is observed, and that is the case when $\sigma_1 = \sigma_2 = -\sigma_3$.

This way created stressed states in the sample are of importance for strain and strength properties test of cohesive soil under complex state of tension conditions. The method permits also to investigate cohesive soil under pure tensile condition isolating the middle part of the sample from the flexible casing effect with the help of hard cylinder.

Now the authors of the report have worked out some modifications of the method which permit to test the bobbinlike samples with inner cylindrical hole and thus to get stressed states with wide range variations of parameter, $-1 \leq \lambda \leq +1$.

The compression-extension method permits to plot Mohr's circles of stress and strain envelope in the field of the tensile stress using the results of soil structural tests which allow to get the true cohesion value when $\sigma_n = 0$, and the value of friction angle at the origin of coordinates without the approximation of the envelope.

Chairman Prof.Šuklje L.

Thank you Mr.Ter-Martirosjan.

*Author's certificate No 323705.Filed, July 5, 1968

We will now hear Prof.Bishop of England Imperial College of Science and Technology, Department of Civil Engineering.

Prof. A.W. Bishop (England)

The General Reporter has asked me to contribute to this Session,and, in particular, to comment on topic 3. This is the question 'Do laboratory model tests,especially the centrifuge test,<u>have utility for determining parameters</u> for predicting prototype performance?'

In their paper on centrifugal models Bolton,English,Hind & Schofield (1973) (Vol 1-1) indicate that much of the value of centrifugal testing lies in its complete independence from mathematical or computer models. They further say that 'the three-dimensional geometry of the boundaries creates a problem of analysis that is outside the present bounds of theoretical solution'.

If this view is accepted,then the model, and the centrifuge model in particular, may predict prototype performance,but it can not be used to evaluate the basic parameters such as deformation modulus,strength in terms of total or effective stress,and coefficient of consolidation. These would require a detailed knowledge of the stress pattern, either from direct observation, which is difficult and subject to cell factor errors, or from a theoretical prediction based on mathematical analysis , which Professor Schofield wishes to avoid.

In the case of plane strain I do not share Professor Schofield's pessimism about the value of mathematical analysis, particularly the finite element and finite difference methods. Indeed, centrifuge modelling of real situations presents difficulties, some of which are described by Bassett in his paper in Vol 2-2, which may mean that prototype performance is better predicted by finite element analysis based on parameters measured in the laboratory on undisturbed samples with proper regard to stress-history, stress-path, stress-system, rate of testing and anisotropy and on parameters from special field tests, as described by Dr.Burland. The role of the centrifuge would then, in my view, be limited to checking the error in any such predictive method by applying it to such idealised cases as can be modelled in the centrifuge without difficulty. Such checks are necessary since current laboratory methods give little information on the effect of the controlled rotation of the axes of principal stress.

The limitations (referred to also by Professor Goldstein in his discussion) of applying centrifugal modelling to real situations relate to the following problems:

1. The difficulty of obtaining and handling large undisturbed samples (discussed also by Dr.Burland).
2. The difficulty of re-creating the actual in-situ stresses,particularly in heavily overconsolidated soils and in all soils having a dried and weathered upper layer.
3. The scale effect arising from the macrostructure of fissured clays.
4. The heterogeneous nature of real soils, which means that no single block of soil can be taken as representative of even an apparently uniform stratum (for example, even on the Boston Blue Clay, Lambe (1971) reported settlements beneath a uniform trial embankment of 0.94 metres, 1.34 metres and 1.74 metres at 30 metre intervals along the centre line).
5. Time effects due to creep or secondary consolidation, which can not be accelerated in a model, and which according to the General Report to Session 4 by Bjerrum, play a major role in soil strength, delayed failure and long-term deformation.
6. Problems of progressive failure, which depend on the absolute displacement between two sliding blocks rather than on average shear strain; these displacements being of a magnitude which can not be obtained in a model without gross geometrical distortion. Energy considerations discussed by Palmer and Rice (1972) also mean that the mechanism of progressive failure can not be accurately reproduced in small models.
7. Any change in profile, such as the making of a cut or the construction of an embankment, should be made under the full scaled-up gravitational acceleration or else the strains and lateral stresses, in particular within the embankment may be very misleading.

The role of the model test may therefore be more restricted than is sometimes supposed. It may be used with a caution about progressive failure for checking mechanisms in complex cases, and may be used for checking quantitative predictions in relatively idealised cases.

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Palerm A.C. & Rice, J.R.(1973). The growth of slip surfaces in the progressive failure of overconsolidated clay. Proc.Roy.Soc., No. 1591, p.527.

Lambe, T.W., D'Appolonia, D.J., Karlsrud, K. & Kirby, R.C. The performance of the foundation under a high embankment. MIT Report R71-22, 1971.

Chairman Prof. L.Šuklje

Thank you very much Prof.Bishop for your contribution. And the last will be Mr.Baguelin from France. Mr.Baguelin , will you please.

Mr.Baguelin (France)

Avec le pressiometre autoforeur developpe en France par J.F.Jezequel et ses collaborateurs (Baguelin, Jezequel, Le Mee, Le Mehaute, 1972, a,b,c), diverses proprietes des sols peuvent etre etudiees in-situ. Outre la resistance au cisaillement non drainee, voici quelques exemples: la sensibilite des sols, le remaniement, la vitesse de consolidation.

La Fil.1 montre le pressiometre autoforeur. Sur cette photo, il est equipe d'une cellule de mesure de la pression interstitielle.

La Fig.2 montre une courbe d'expansion non drainee menee jusqu'a une deformation de 30%. La courbe de resistance au cisaillement qui s'en deduit met en evidence une sensibilite du sol de l'ordre de 1,5.

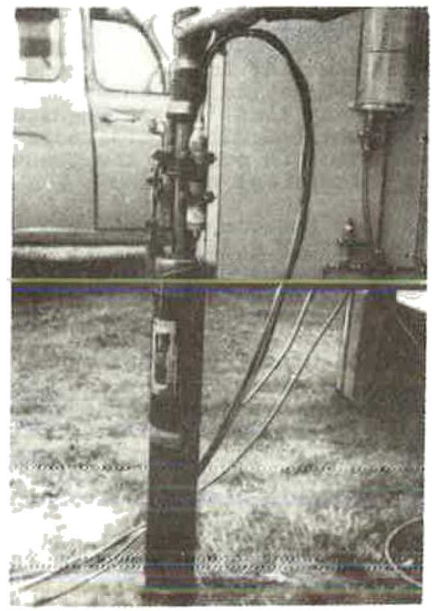
Le remaniement du sol peut etre etudie grace a une bague d'epaisseur 1 mm montee sur la trousse coupante. On peut voir sur la Fig.3 la comparaison avec les courbes du sol intact: courbes pressiometriques et courbes de cisaillement non draine. Dans ce cas, mais ce n'est pas general, la pression initiale n'est pas alteree; elle est le plus souvent accrue par le remaniement. La valeur de pic de la resistance au cisaillement est peu alteree, mais les deformations sont accrues.

Une mesure en place de la vitesse de consolidation des sols peut etre effectuee de la maniere suivante (Fig.4).

1°/ L'expansion est effectuee, du type non draine, jusqu'a une valeur predeterminee, par exemple ici, 0,94%. Durant cette phase, on mesure la pression interstitielle qui se developpe au bord de la sonde. Grace a la theorie de l'expansion cylindrique non drainee, on en deduit le champ des pressions interstitielles initiales.

2°/ La deformation est bloquee. On mesure alors la chute de la pression interstitielle au bord de la sonde. Une interpretation simplifiee, d'apres le travail de Carslow et Jaeger (1959), permet de deduire un coefficient de consolidation radiale moyen, qui, dans ce cas, vaut respectivement 1.10^{-2} et 2.10^{-2} cm^2/sec.

Enfin nous avons developpe d'autres applications, le piezometre autoforeur en particulier. Ces resultats seront presentes en Session Speciale no.5.



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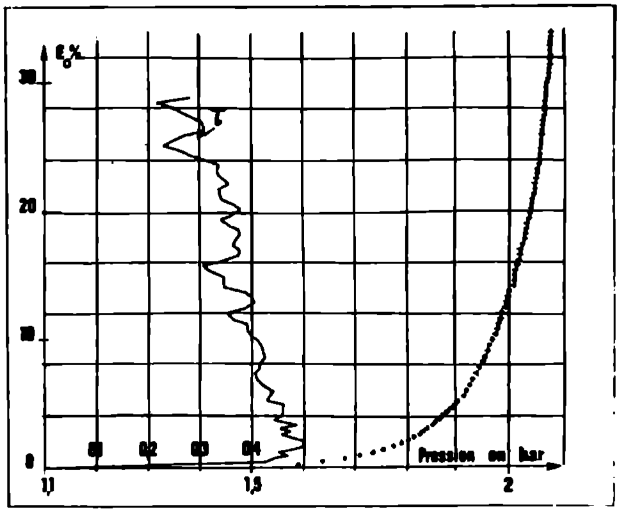
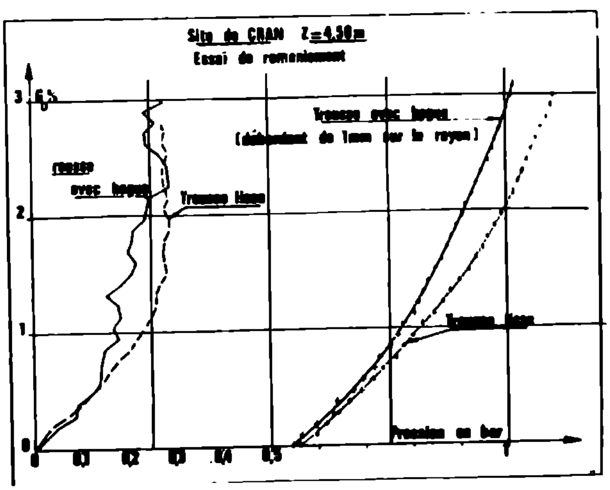
a) F.BAGUELIN,J.F.JEZEQUEL,E. LE MEE, A.LE MEHAUTE (1972): "L'essai d'expansion cylindrique et la loi effort-deformation des sols purement coherents",VIe Congres International de Rheologie,Lyon,4-8 sept. 72,tome IV, p.239.

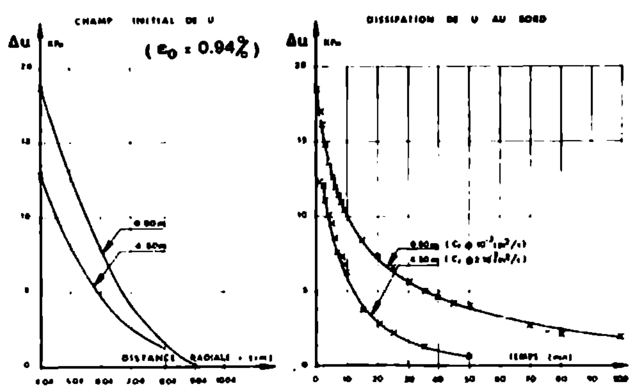
b) F.BAGUELIN,J.F.JEZEQUEL,E.LE MEE,A.Le MEHAUTE (1972): "Expansion de sondes cylindriques dans les sols coherents". Bulletin des Laboratoires des Ponts et Chaussees n°61, sept-oct. 1972, pp.189-202.

c) F.BAGUELIN,J.F.JEZEQUEL, E.LE MEE, A.LE MEHAUTE (1972): "Expansion of cylindrical probes in cohesive soils", Journal of the Soil Mechanics and Fondations Division. Proceedings of the American Society of Civil Engineers. Vol.98,No SM11,Proc.Paper 9377.Nov.72.pp.1129-1142.





MESURE DU COEFFICIENT DE CONSOLIDATION



Chairman Prof. L.Šuklje

Thank you very much Mr.Baguelin for your contribution.

Coming to the end of this session I want to pass the word to Prof.Meyerhof to give the closing speech on the work of our session

Prof. Dr.G.G.Meyerhof (Canada)

As Vice-Chairman of this first Main Session of the Conference, I have been asked to give the closing speech and summarize the main conclusions of our discussion. The subject of this session on the investigation of the strength and deformation proper... of soils in the laboratory and field is still a most important one in soil mechanics and foundation engineering. For planning, design, construction and operation of civil engineering works a knowledge of the in-situ or field behaviour of large soil masses under load is required. Since even elaborate laboratory tests on large undisturbed soil samples can at best only approximate the field conditions, in-situ tests are often preferable. However, field tests are also inevitably simplified and have a margin of uncertainty because they generally give properties of soil masses of limited extent under short-time loading, which may differ considerably from the behaviour of large soil masses under long-term loads of engineering structures in practice.

The topics for discussion of this Session dealt with four main areas: in-situ test devices, predictive techniques based on field measurements, laboratory model tests and probability approaches to soil engineering problems. Regarding the discussions on field tests and their evaluation, it should be remembered that the initial in-situ stresses in the horizontal and vertical directions are often required in addition to the stress-strain-time and strength relationships measured under the applied test loading. Further, pressuremeter measurements give only "horizontal" soil parameters, while cone penetration and plate loading tests are governed mainly by "vertical" soil properties. Moreover, all these devices inform us only about soil behaviour under axial symmetric loading, and the corresponding properties may differ considerably from the plane strain case of many soil engineering problems.

The discussion on laboratory model tests, especially the centrifuge test, showed that they can give useful information about soil parameters and the mechanism of soil failure under load for the design of prototypes. While tests on small models yield results of a qualitative nature only, even large scale laboratory model tests cannot fully represent the actual geology, initial in-situ stresses, construction operations, time effects and other important factors. Accordingly, the correlation of such model tests with the behaviour of fullscale structures by performance observations during and after construction is essential and cannot be overemphasized.

Finally, the discussions on probability and decision theories indicated that these methods can assist us by delineating margins of error and showing some approaches to the solution of simplified soil engineering problems. However, such analyses are only useful if the statistical parameters of the soil properties at the particular site are known in some detail. It should always be remembered that the successful and economical solution of soil engineering problems is both an art and a science. Our semi-empirical methods of prediction based on analyses and the results of soil tests require mature engineering judgement, sound experience and some good luck!

In conclusion, I should like to express our warmest thanks to the General Reporter, Professor T.W.Lambe, for his stimulating General Report and his detailed review of 77 papers, which account for about one-third of all papers submitted to this Conference. I should also like to thank the Organizing Committee of this Conference and our Russian colleagues for the excellent arrangements for this Session and their kind hospitality. Thank you all for your interesting contributions to the discussion. We look forward to seeing you tomorrow.

Спасибо и до свидания. До завтра.

The first Session is closed.

Written contributions

H.JOSSEAUME et B.MANDAGARAN (France)

Les tassements, mesures sous un remblai construit sur une couche d'argile organique molle (vase) et caracterise par un coefficient de securite assez faible, sont generalement tres superieurs aux tassements calcules. Ceci est du essentiellement aux deplacements lateraux du sol de fondation, dont le calcul classique ne tient compte que partiellement (lors du calcul du tassement immediat).

Pour apprecier l'influence de ces deplacements lateraux, une etude du tassement des vases, basee sur des essais triaxiaux a chargement controle, a ete entreprise au L.C.P.C. La methode utilisee proche de celle preconisee par KERISEL(1) tente de reproduire en laboratoire les conditions de chargement du sol en place. Elle consiste a appliquer a des eprouvettes prelevees dans la couche a etudier et reconsolidees dans l'appareil triaxial sous les contraintes en place (contrainte verticale σ_0 contrainte horizontale $K_0\sigma_0$) les supplements de contraintes $\Delta\sigma_1$ et $\Delta\sigma_3$ apportes par le remblai au niveau de prelevement, $\Delta\sigma_1$ et $\Delta\sigma_3$ etant calcules sur l'axe de l'ouvrage

(1) J.KERISEL, M.QUATRE-Tassements sous les fondations. Methode de prevision a partir de l'appareil triaxial. Annales des Ponts et Chaussees- n° 3- mai-juin 1966

On donne ci-apres des resultats obtenus lors de l'etude du tassement immediat.

L'etude a ete faite pour diverses hauteurs d'un remblai large de 70m a sa base et reposant sur une couche de vase normalement consolidee d'epaisseur 16m. Les caracteristiques moyennes de cette vase etaient les suivantes:

Limite de liquidite	: w_L=78
Indice de plasticite	: I_P=42
Teneur en eau	: w =70
Teneur en matiere organique	: 5%
Teneur en CO_3Ca	: 40%
Cohesion non drainee	: C_u=27,5 kPa

$\Delta\sigma_1$ et $\Delta\sigma_3$ ont ete calcules pour des hauteurs de remblai correspondant a des valeurs du coefficient de securite comprises entre F=1,8 et F=1,20. Le calcul a ete fait en elasticite et tient compte d'un substratum rugueux a la base de la couche compressible dont le coefficient de Poisson a ete pris egal a ν =0,5.

On a constate une rupture plus ou moins rapide des eprouvettes essayees (entre un jour et plusieurs jours), sauf dans le cas ou F=1,8. Ces ruptures se sont produites malgre que le deviateur $\sigma_1 - \sigma_3$ ait ete inferieur a deux fois la cohesion non drainee C_u mesuree a partir d'essais triaxiaux UU effectues sur des eprouvettes reconsolidees aux contraintes en place (vitesse de deformation: 3,5% par heure). Ces observations ont conduit a etudier le comportement non draine d'eprouvettes de vase soumises a des contraintes constantes σ_1 et σ_3 correspondant a differentes valeurs des parametres

$n = \dfrac{\sigma_1 - \sigma_3}{2\,C_u}$ et $K = \dfrac{\sigma_3}{\sigma_1}$. L'etude a montre l'existence d'un seuil de fluage correspondant pour le sol considere a la valeur n_f=0,75. En effet, en deça de cette valeur, la deformation verticale de l'eprouvette se stabilise dans le temps (fig.1), alors qu'au dela la deformation s'amplifie et le sol se rompt (fig.2). Les ruptures qui se produisent pour des valeurs de n superieures a n_f=0,75 ont pu etre interpretees en contraintes effectives (fig.3).

L'etude de la deformation d'eprouvettes caracterisees par un niveau de cisaillement inferieur a 0,75 met en evidence une forte decroissance du module non draine E_u dans le temps (fig.4). On remarque en particulier que, une minute apres l'application des contraintes, E_u est egal a environ la moitie du module non draine mesure dans l'essai UU classique pour $\dfrac{\sigma_1 - \sigma_3}{2\,C_u}$ = 0,66. Au bout de deux semaines (20 000 mn), la valeur de E_u n'est plus que de l'ordre du dixieme de la valeur obtenue dans l'essai UU.

En conclusion, il apparait que, pour les valeurs usuelles du coefficient de securite, une zone plastique se developpe dans la fondation d'un remblai pour peu que la couche compressible ait une certaine epaisseur. La prevision du tassement immediat doit alors etre faite par un calcul tenant compte d'un comportement elastoplastique du sol compressible. Pour le choix de la valeur de la resistance au cisaillement definissant le seuil de plasticite, il est necessaire de tenir compte du phenomene de rupture par "fluage" qui se produit pour des contraintes de cisaillement bien inferieures a C_u. Le choix du module non draine caracterisant le comportement de la zone elastique devra egalement tenir compte de la forte decroissance de E_u apres chargement du sol.

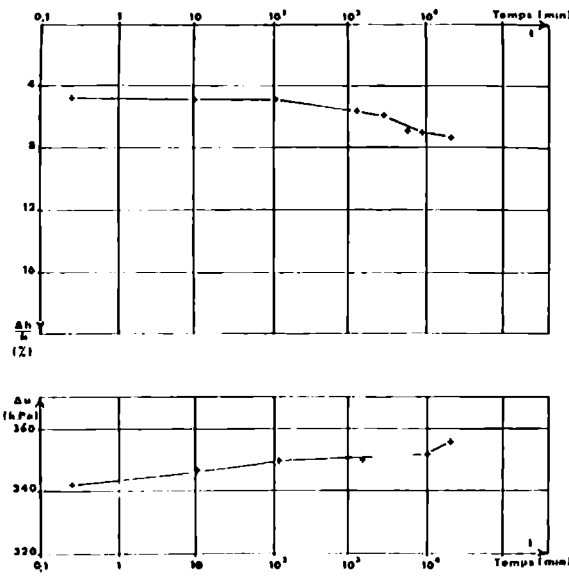


Fig.1. Variations de la deformation et de la pression interstitielle en fonction du temps au cours d'un essai de chargement triaxial non draine pour

$n = \dfrac{\sigma_1 - \sigma_3}{2\,C_u}$ = 0,56

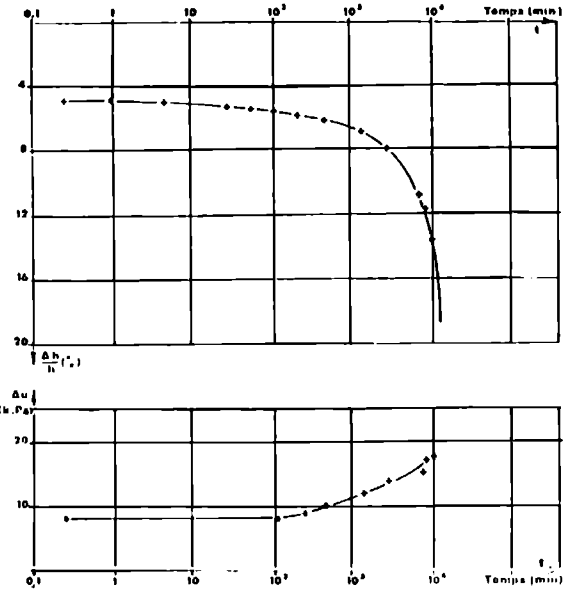


Fig.2. Variations de la deformation et de la pression interstitielle en fonction du temps au cours d'un essai de chargement triaxial non draine pour $n=\frac{\sigma_1-\sigma_3}{2C_u}=0,76$

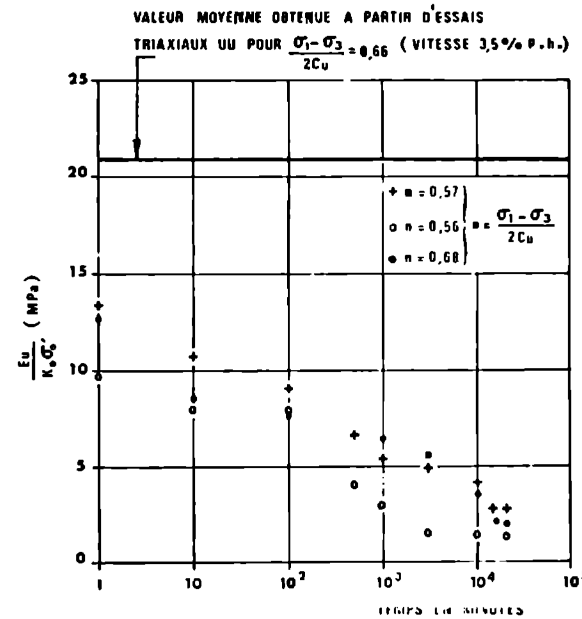
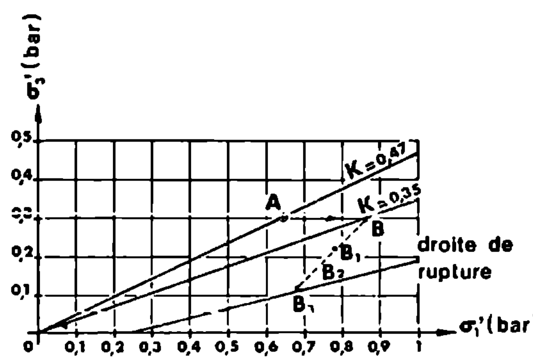


Fig.4 – Variations du module non drainé réduit en fonction du temps.



A ETAT DE CONTRAINTES INITIALES

B ETAT DE CONTRAINTES TOTALES FINALES

B_1 ETAT DE CONTRAINTES EFFECTIVES APRES 2 HEURES

B_2 ETAT DE CONTRAINTES EFFECTIVES APRES 3 JOURS

B_3 ETAT DE CONTRAINTES EFFECTIVES APRES 20 JOURS

Fig.3 – Chemin de contraintes obtenu dans un essai de chargement triaxial non drainé

Lumir Pruška (Czechoslovakia)

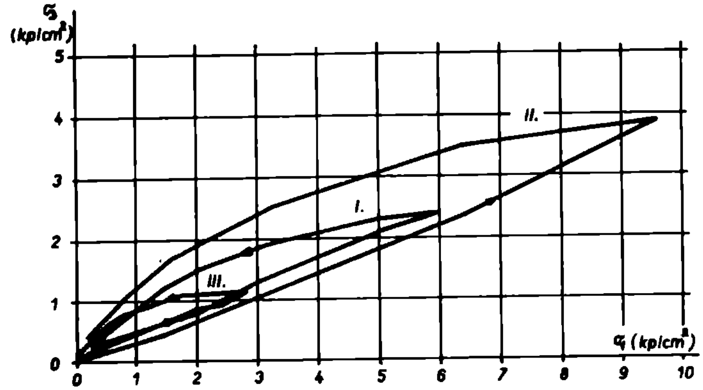
4668

Effect of Initial Stress on the Stress-Strain Relation

1. In loose soils they manifest themselves so that the relationship $\sigma_3'=f(\sigma_1')$ is given when the pressure is at rest and the controlling stress σ_1' varies from zero to a certain maximum $\sigma_1'{}_{max}$ and again back to zero, by the well-known closed hysteresis loop. The hysteresis loop has been ascertained and proved experimentally. Fig.1 shows the hysteresis loops measured by three authors in different countries (Fyodorov, Malyshev 1959, Plehm 1965 Mach 1970) on samples of sand having almost the same mechanical properties. The specimens, too, were similar, cylindrical, with a 2:1 height-to-diameter ratio.

2. Under other boundary conditions, for example, for a cube-shaped specimen (Kjellman 1936) the hysteresis loop, though not the same, is of a similar shape. An analysis of this case leads to the conclusion (Pruška 1973) that the unloading branch of the hysteresis loop is given by the equation

$$\sigma_3' = \frac{K_{o1}\cdot\sigma_1'{}_{max}}{\sigma_1'(1-K_{o1}^2)+K_{o1}^2\cdot\sigma_1'{}_{max}}\cdot\sigma_1' \quad (1)$$



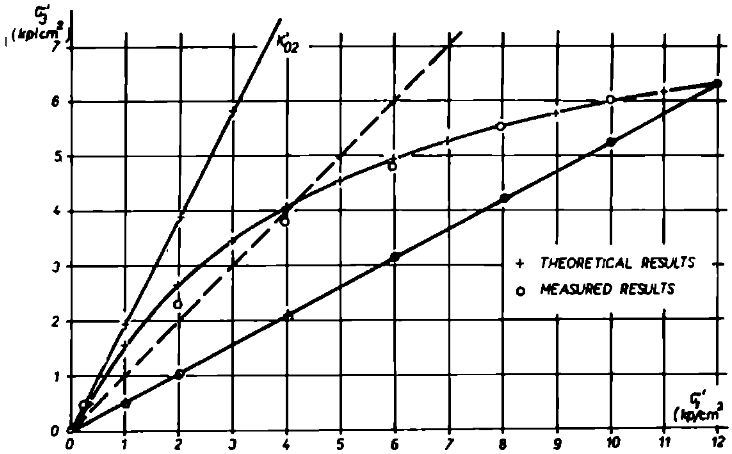
HYSTERESIS LOOPS OF SAND AS MEASURED BY FJODOROV AND MALYSHEV (I.), PLEHM (II.) AND MACH (III.)

FIG. 1

where $K_{o1} = tn\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)$ (Pruška 1972)

$\sigma'_{1\,max}$ - maximum overconsolidation stress.

A comparison of the experimentally ascertained hysteresis loop with the theoretically deduced one is shown in Fig.2.



HYSTERESIS LOOP OF SAND IN A CUBIC SAMPLE AS MEASURED BY KJELLMAN 1936(o) AND CALCULATED BY AUTHOR(+)

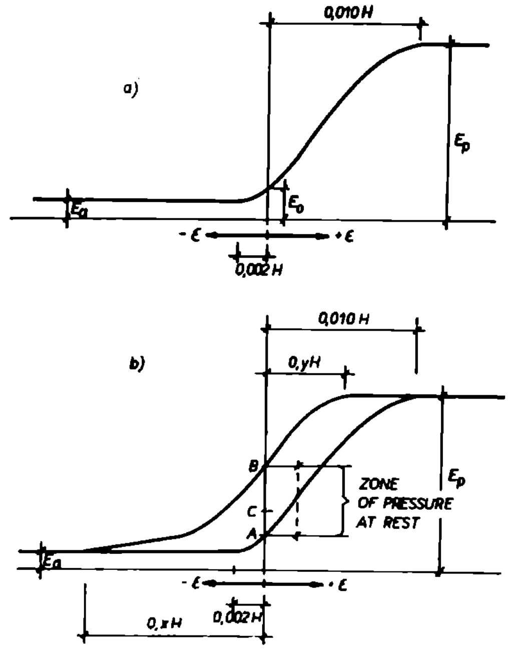
FIG. 2

As equation (1) suggests, the coefficient of pressure at rest is a function of stress history. At the first overconsolidation stress $\sigma'_{1\,max}$ and the subsequent first unloading to stress σ'_1 the coefficient of pressure at rest is defined by the equation

$$K_o = \frac{K_{o1} \cdot Pr}{1-K_{o1}^2(1-Pr)} \qquad (2)$$

where $Pr = \sigma'_{1\,max}/\sigma'_1$ - overconsolidation ratio

3. The dependence of the limiting stresses in soils, of the active and passive pressures, is frequently stated on the basis of Terzaghi's studies by help of the well-known curve (Fig.3a) indicating that in the modelling of soil pressures on a vertical wall the active pressure



RELATION OF PRESSURE AT REST TO THE ACTIVE AND PASSIVE EARTH PRESSURE. FIG. a THE TRADITIONAL DEFINITION, FIG. b THE NEW EXTENDED DEFINITION

FIG 3

arises at a support displacement from the original position of about 0.002 H. The passive pressure originates at a displacement greater by a factor of about 10.

Measurements made at "Centre Experimental de Recherches et d'Etudes des Batiment et des Travaux Public-Paris" (Tcheng, Iseux 1972) have proved this dependence to be not unique, for the passive pressure arises also under other deformations and the displacements mobilizing the passive pressure are not merely a function of the supporting wall height.

4. Let us introduce the relationships set forth in paragraphs 1 and 2 into the problems outlined in paragraph 3 in accordance with Fig.3a in which the initial state of stress at zero strains denotes the pressure at rest defined by a single value, E_o. Since the pressure at rest is not a constant (Pruška 1972), it is necessary to consider rather than a single value of the pressure at rest- a whole set of pressures at rest lying in the interval bounded by the lower and the upper coefficient of pressure at rest, K_{o1}, and K_{o2}.

What the introduction of this procedure into the schematic diagram in fig.3a means, is that for zero strain there applies the line segment AB (Fig.3b) expressing the various stresses from which one can set out when examining the origin of the active or passive pressure. It then follows from the continuity of strains that there exist various ways-given by the initial conditions-of reaching the limit stress and therefore, that it is necessary to introduce in place of a single curve according to Fig.3a, some unknown, not closely defined zone according to Fig.3b. What we know of this zone are points A and B and, basing on Terzaghi's tests, probably also the lower limit given by the results of studies of Terzaghi and his school.

5. The limit values of the pressure at rest are functions of the angle of internal friction. For $\emptyset'=0$ the two values become identical and that is why they are represented by a single point C in fig.3b. It follows from this statement that the magnitude of displacements and hence also the whole zone of transition from the pressure at rest to the active or passive pressure depend equally on the angle of internal friction.

The problems of cohesion have not been considered so far.

6. The aim of this contribution is to point out the necessity of following the initial state of stress and the stress history in soil tests. Respecting this reality leads to surprising conclusions as demonstrated, for example, by Broms (1972) in his studies of soil pressures on supporting walls.

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J.FEDA (Czechoslovakia)

The aim of the theme of this Session-"up-to-date methods of investigating the strength and deformability of soils"- is to provide soil mechanicians with the most reliable constitutive relations of soils. The reliable stress-strain relations, especially if intended to be extrapolated beyond the experimental range, seem to me to require a reasonable structural interpretation, i.e. their understanding on the basis of the mechanism of structural changes. Such a task is certainly a difficult one and whatever help is most invited. Such a help may yield, according to my opinion, some disciplines related to soil mechanics. They deal with materials like coal, ore, metallic powders, grain, kaolin etc. which may be expected to behave similarly like soils. All these materials form therefore different parts of the synthetic mechanics of particulate matter.

Fig. 1 presents the position of the mecha-

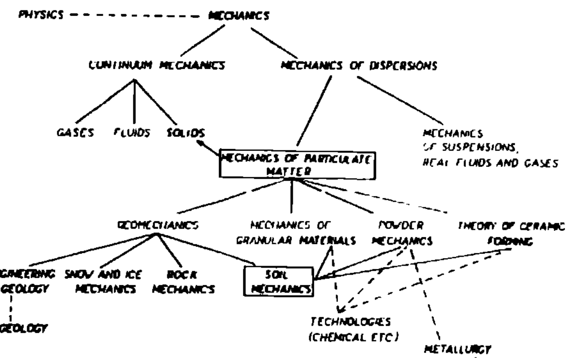

Fig.1. The position and subdivision of the mechanics of particulate matter

nics of particulate matter in the frame of the general mechanics. Soil mechanics treating dry or wet cohesive and cohesionless soils at different stress levels is doubtless the most general of all the branches of the mechanics of particulate matter. Its concepts therefore are understandingly most fruitful for those branches. This contribution however aims toward an opposite question: how the related subjects may contribute to a more profound understanding of the mechanical behaviour of soils. In the following I will present some examples to prove that they may.

One of the present problems of soil mechanics is the behaviour of cohesionless materials like rockfill undergoing grain crushing at the engineering stress level. Fig.2 (Feda, 1971) suggests the effect of the axial strain (at the constant stress level) on the extent of grain crushing of a residual granitic sand triaxially tested. This effect may be explained by the assumption of "grinding" the sand grains. The application of the theory of grinding of the ceramics may therefore prove to be useful when analysing the structural changes at large displacements.

Another soil mechanics problem is the analysis of reversible deformations relevant for the definition of the yield surface. Some interesting knowledge may be deduced from the electrical resistance measurements of electrically conducting powders. Fig.3 shows one of Kantorowicz's (1932) tests-graphite powder

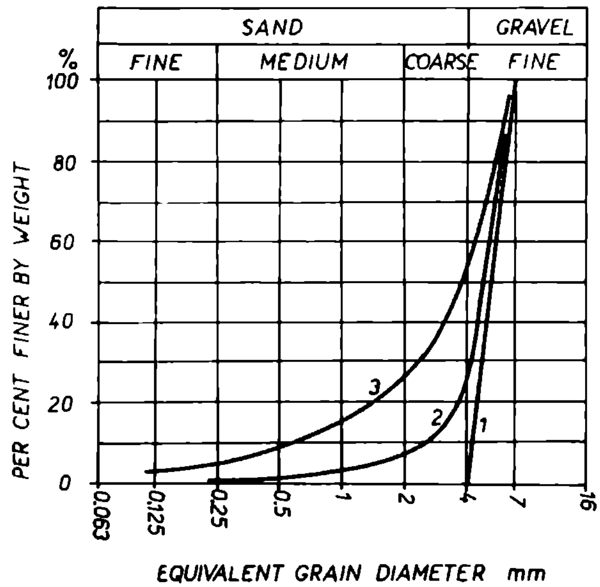

Fig.2. Grain crushing of triaxially tested sand (Feda,1971): 1-original granulometric curve,2-the granulometric curve at the cell pressure σ_r=5 kg/cm^2 and axial strain ε_a=7,25%, 3-the granulometric curve at σ_r=5 kg/cm^2 and ε_a= 24%.

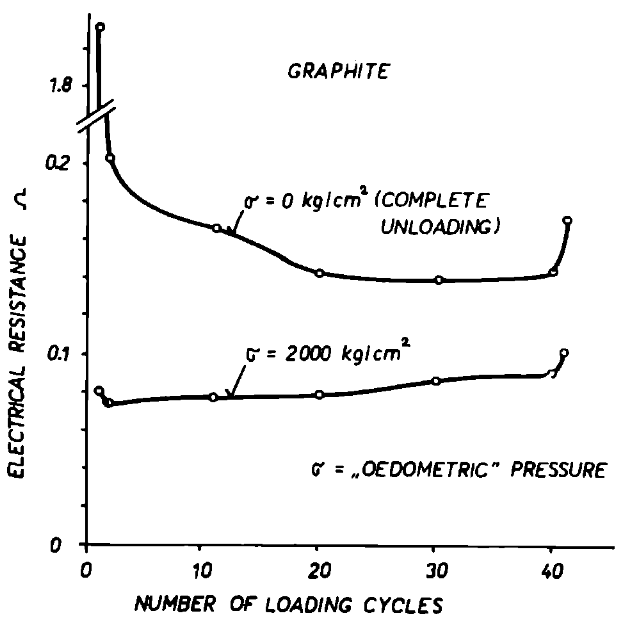

Fig.3. The measurement of the electrical resistance of the pulverized graphite (particle size 9 to 15 μm) at the cyclic one-dimensional loading (Kantorowicz, 1932)

was cyclically loaded and unloaded (cyclic pressure range 0 to 2000 kg/cm^2) at one-dimensional (oedometric) deformation. At first densification takes place (as proved by the decrease of the electrical resistance). Gradual increase of residual stresses at the higher cycles leads to the disturbance (loosening) of the structure both at the complete unloading (when this structural destruction was sometimes accompanied "mit deutlich hörbarer Knacks") and at reloading. These and similar tests indicate the possibility of studying the effect of residual stresses which are in soil mechanics still rather underestimated.

The theory of pressing,both one-dimensional and hydrostatical is highly developed in powder metalurgy and ceramics. Fig.4 (Balšin, 1972) presents an example of one-axial

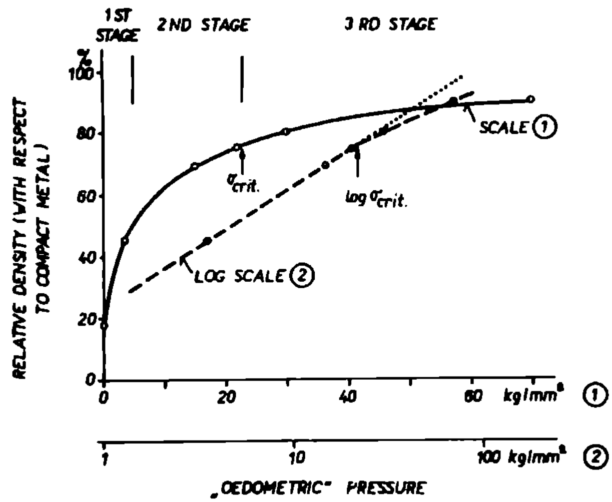

Fig.4. One-dimensional compression of the electrolytically pulverized copper (Balšin, 1972).

(oedometric) compression of the electrolytically pulverized copper. Terzaghi's semilogarithmic law is valid for the first and second loading stage, its compressibility coefficient changes in the third stage. This follows from the compression curve transformed to the semilogarithmic scale. Compression through the first and second stages results from the mutual displacement of grains and the local deformation of their contacts. In the third stage the contacts ceased to move and a total deformation of grains takes place.

This compression process allows an appropriate definition of the particulate matter: it is a material whose defomations result from the individual grain (or structural unit) displacements. One may well imagine materials which get into a particulate state during a deformation process (owing to breakage of their original structure) and those or other materials which after leaving some working range cease . to bear the nature of a particulate matter. Such state (structural) changes exert doubtless a strong influence on the mechanical behaviour (constitutive relations) of soils.

The number of such examples may be increased but perhaps those presented may suffice to illustrate the proposed thesis:considering some contemporary problems of soil mechanics from a standpoint of a more general mechanics of particulate matter is useful not only from the philosophical(synthetic)view but as well for its practical value.

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A.P.Alexiev (Bulgaria)

The fact that most of the soils represent solid-liquid-gaseous systems is wellknown, yet not recognized really in mechanical terms. As stated by McClintock and Argon (1966),the mechanical behaviour of materials from 10^{-4}m down to 10^{-7}m grain size is virtually an unknown land,and the middle part of this range is laid beyond the scope both of the metallurgy and materials science,and fracture mecanics, on Tetelman-McEvily's sheme (1967). Since almost all clayey and silty soils fall in this range it is quite important to find a basic general characteristic for them, differing from those of the other soils. So it is assumed that the basic structural feature, from a mechanical point of view, of clayey and silty soils is the highly discrete (dispersed) distribution of the two fluid phases too. Expressed in quantitative terms,the mechanical (or strength) characteristic mentioned is determined by the following ratio:

$$\frac{(W_g/W_l)^{\alpha(t)}}{(W_l/W_s)^{\beta(t)}} = \text{Temperature} \qquad (1)$$

where W_g, W_l, and W_s are mean volumetric gaseous, liquid, and solid contents of soil phases, respectively,on the "fracture surface", and $-1 \leq \alpha(t) \leq 1$, $-1 \geq \beta(t) \geq 1$ are composite time functions determining the fracture surface development and topography in dependance on solid particle size and shape, solid-liquid-gas phenomena,etc. It was shown on this basis (Alexiev,1970) that only four different types of structural mechanical behaviour are possible for three-phase dispersed systems (soils), and an experimental method for study them is described in brief here, in another place (Alexiev, 1973).

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B.LADANYI /Canada/

Since the last international conference an increasing interest has been shown in several parts of the world in extending the testing potential of the Menard Pressuremeter and in enlarging the scope of soil properties that can be measured by such an instrument.

Thus,as an example,in 1972,three independent papers appeared almost simultaneously in three different countries, describing how the ordinary short-term pressuremeter information can be used for finding the whole undrained stress-strain curve,and the corresponding vector curve for a saturated clay.

In addition to this,two papers present at this Conference show some modified designs of the instrument and consider also problem of a draine pressuremeter test in clay /Wroth and Hughes/, and the pore pressures that are generated by such a test /Baquelin, Jezequel and Le Mehaute/. A third paper,also presented at this conference /Bronstein, Mikheev, Kuppenheit and Shvets/ gives an improved method for determining the elastic anisotropy of the soil.

On my part, I would like to mention here that the field of application of the pressuremeter has recently been extended to cover also the problem of in-situ measurement of creep properties of ice-saturated frozen soils and a paper about it was presented by myself and Gh.Johnston at the recent 2nd Int.Conference on Permafrost in Yakutsk.

For that purpose, a stage-loaded test,with about 15 minutes per stage,has been found convenient. The test information is then processed in a manner similar to that used in the creep testing of metals within the primary creep domain. As a result, each such pressuremeter creep test yields a general creep equation, as well as a time-dependent strength equation of the tested frozen soil.

The method covers up to now only the deviatoric creep problem, which is sufficient for ice-rich frozen soils. Its use in unfrozen saturated clays would require the knowledge of the generated pore pressure variation, in order to be able to the volume creep from the total measured creep deformations. The method for pore pressure measurement in a pressuremeter test described by Baquelin et al. gives hope that this goal may soon be achieved.

Dr.R.H.G.PARRY /England/

Conventional stability analysis and centrifuge model tests have been made on river walls about 4m high which have slipped as a result of dredging 1m or so from the river bed. The purpose of the dredging was to increase flood flow.

The conventional analyses were made using effective stress c', Ø' values measured in the laboratory and pore pressures measured in the field. These analyses showed the stability to be marginal, but the value of F was very sensitive to c' which is difficult to measure to say 2 to 3 kN/m²

Centrifuge models were made to 1/46 scale.

a) with kaolin consolidated to the same undrained shear strength as field soil
b) with natural undisturbed field soil.

Dredging and drawdown were simulated.

The kaolin model failed very much like the field case with a distinct failure surface.

The natural soil model did not show a distinct failure, but because careful observations were made of deformations by photographing surface markers it was possible to distinguish a zone of high shear deformation. Without such careful observations the centrifuge tests might have been misleading. From the observations the boundary of the highly deformed zone was delineated and using this as the failure surface further conventional analyses made. These led to a lower calculated factor of safety than previously.

A.G.Anagnostopoulos /Greece/

SYNOPSIS

Several penetration tests were made in Patras soft silty clay in order to extend the interpretation of cone penetration resistance for evaluating the undrained shear strength of this material. Results are presented in relation to the undrained shear strength of the soil penetrated.

Based on these tests a tentative relationship between cone penetration resistance (q_c) and undrained shear strength (C_u) of Patras silty clay is proposed.

1. GENERAL REMARKS

In order to evaluate static penetration tests (using a Dutch cone penetrometer) in respect to the estimation of the shear strength characteristics of soils, various authors suggested formulas for the correlation between static cone penetration resistance q_c and shear strength of the penetrated soil; some of the given formulas correlate for undrained (quick) conditions (φ=0) the cohesion C_u to the static cone resistance q_c.

From the existing literature some correlations between q_c and C_u are given herebelow:

a) Begeman : $C_u = \frac{q_c}{14}$ for clayey soils

b) Thomas : $C_u = \frac{q_c}{18}$ for London clay

c) Sanglerat : $C_u = \frac{q_c}{15}$ for Annecy soft clays

d) Meigh and Corbett : $C_u = \frac{q_c}{16}$ for Arabian Golf soft clays

Taking under consideration the above formulas and in order to derive C_u from q_c, a theoretical approach was attempted, according Jaky's and Meyerhof's computation methods for a deep foundation, with a base in a shape of equilateral triangle. This took place during the geotechnical investigation in the construction field for a multi storey building at the seashore of the city of Patras, located in the NW part of Peloponnesus (Greece).

The geotechnical investigation consisted of bore holes drilled to depths of 35-50 meters. Near the bore holes static penetra-

Figure 1.

tion tests have been carried out, with a Gouda engine driven hydraulic deep sounding apparatus. A regular cone has been used, with an apex angle of 60° and a section area of 10 cm². The used driving speed was 2cm/sec. Undisturbed samples have been taken from the desirable depths by means of a thin wall sampler. Soil classification and characteristics have been determined from laboratory tests. The undrained shear strength (C_u) has been determined from unconfined compression tests and some quick undrained triaxial test on a consolidated sample at the geostatic pressure.

The soil profile consists geologicaly of recent alluvial deposits (fine to medium grained sands with silt and clay). A representative description of the soil formation is as follows.

0-5m Random man made fill including gravel, sand and stones
5-10m Gray silty fine sand (ML) containing localy fine gravel
10-35m Gray inorganic silty clay of medium plasticity (CL), soft to medium stiff

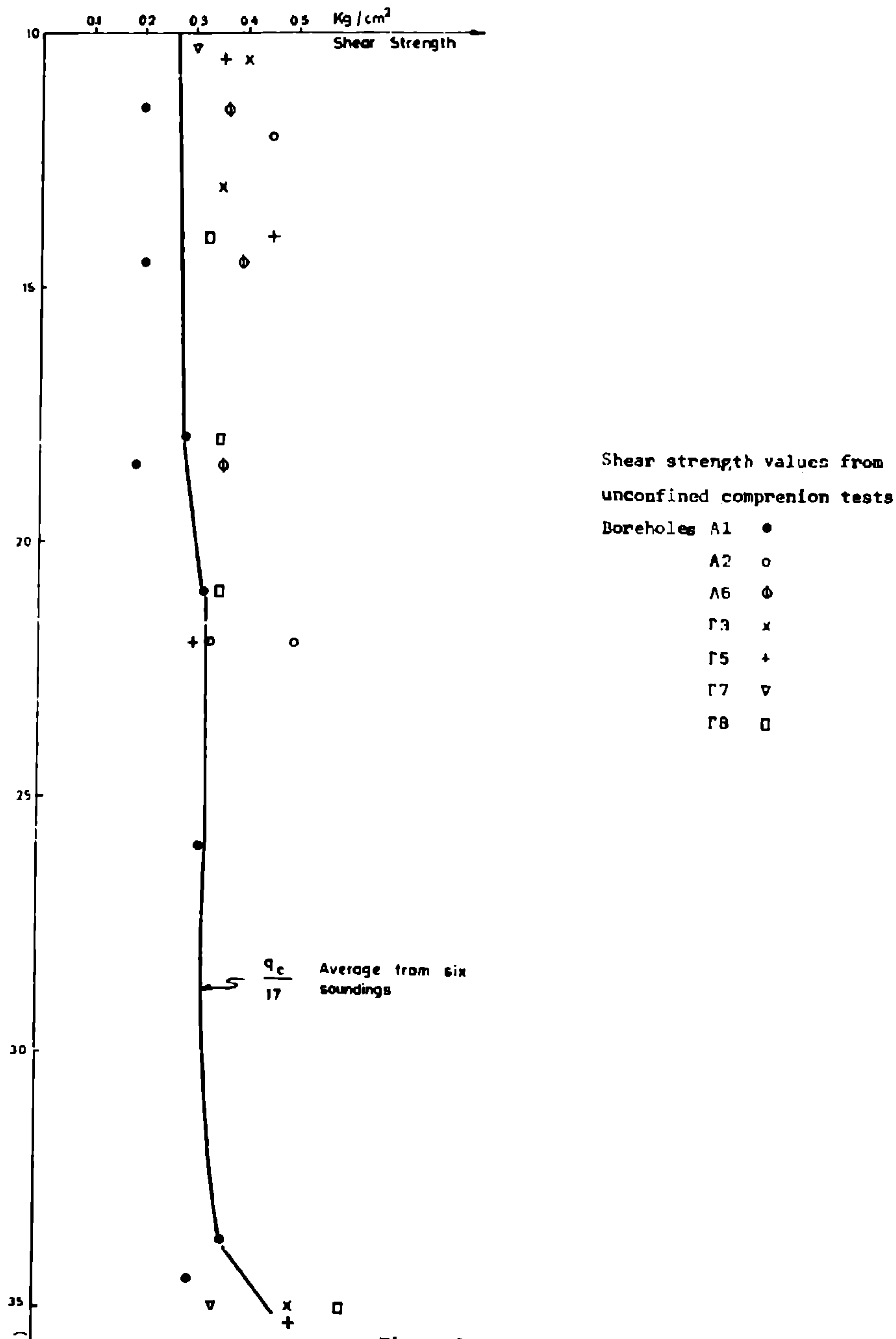

Figure 3.-

Correlation of static cone resistance with undrained shear strength

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From the depth of 35m down to the final explored depth of 50m the soil becomes more and more stiff. The characteristics of the strata under consideration (10-35m) are given below.

Percentage of Clay D < 0,002mm	Passing through sieve No 200	W	W_L	I_p	S
40%	96%	30%	35%	18%	1,5-3

2. CORRELATION BETWEEN q_c-C_u

In order to correlate the undrained shearing strength C_u with the static cone resistance q_c, the material is considered to be purely cohesive ($\varphi = 0$) and the failure conditions of a deep strip foundation with a smooth shaft and a triangular equilateral base are examined.

Following the assumptions of Jaky and Meyerhof the failure conditions are considered as in figure 2.

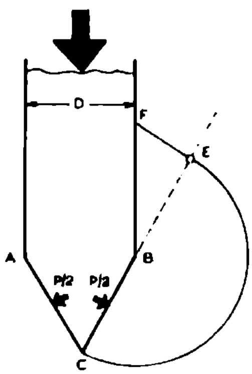

Fig.2.

Taking into account the width of the strip foundation equal to D, diameter of the cone point, the development of the failure line L can be derived:

$$L = \pi D + D \tan 30^{\circ} = D(\pi + 0,58)$$

From the equilibrium condition of the driving to the resisting moments around the point B we obtain:

Driving moment: $M_D = \frac{P}{2} \cdot \frac{D}{2} = \frac{PD}{4}$ but $P = q_c . D$ and

$$M_D = q_c \cdot \frac{D^2}{4} \qquad (a)$$

Resisting moment: $M_R = C_u . D . (\pi + 0,58) = 3,72 C_u D^2$ (b)

equating (a) and (b):

$$q_c = 3,72 \times 4 \times C_u = 14,88\ C_u \qquad (c)$$

The above relation is valid for strip foundation. Because of the circular cross-section of the cone base, it is necessary to convert the derived relation for the circular case. In this respect it is considered to be a good approximation the correlation given by Meyerhof for the coefficient N_c for both circular and strip foundation. This leeds to the figure:

$$\frac{Nc,\ circular}{Nc,\ strip} = \frac{9,34}{8,28} = 1,13 \qquad (d)$$

According to the above equation (c) is converted for the circular case:

$$q_{circ} = 1,13\ q_{strip} = 1,13 \times 14,88\ Cu \cong 17\ C_u \qquad (e)$$

This relation expresses the dependance of the cone resistance q_c with the undrained shear strength C_u. The contribution of the overlying soil loads has not been taken into account due to the rather moderate depths and the compensating influence of the weight of the rods.

3. TESTS RESULTS

According to the relation (e) the undrained shear strength of the soil has been calculated from the penetrometer readings and is plotted on figure 3 together with the results of laboratory shear strength tests on undisturbed samples from different depths.

It can be seen from figure 3 that the linear relationship between cone penetration resistance and shear strength is in good agreement with the values determined from uncofined compression tests, for the explored depth.

The scatter of results is acceptable.

In consequence the author considers that a relationship of $q_c = 17\ C_u$ is applicable for the examined case and does not differ considerably from the proposals of other authors.

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Sven Hansbo (Sweden)

Determination of the creep strength of high-plastic, normally consolidated clays

The empirical reduction factors μ_R with reference to shear strength of high-plastic clays, if based on liquid limit as in Sweden or on plasticity index as recently suggested by Bjerrum, might yield results on the unsafe side. This has been shown by me in an article delivered to the Nordic geotechnical meeting in Trondheim in August 1972 (Hansbo, 1973). Therefore it seems necessary to determine by some proper testing technique what I would call the creep strength τ_{cr}.

One method, suggested by Singh & Mitchell, is to make triaxial tests with constant deviatoric stresses at different stress levels below failure (as determined by conventional tests) but this is time-consuming and it is difficult to find by this method the exact value of creep strength. An alternative method, e.g. used at the Norwegian Geotechnical Institute, is to make triaxial tests with different axial strain rates. Although this is a much quicker method it is still time-consuming and requires (as the previous method) many samples with "identical" geotechnical properties. Both these methods are thus less usable from a practical point of view.

A third method, which seems very promising, is to use pressuremeter tests. Thus the creep strength seems to correspond to the yield pressure p_{fl} (pression fluage), determined with Menard's pressuremeter. If the modulus of elasticity of the clay is assumed equal to 250 τ_{cr} to 500 τ_{cr} (an estimate that is based on loading tests), then the creep strength can be calculated from

$$\frac{\tau_{cr}}{p_{fl}-p_o} = 5,4 \text{ to } 6.1$$

where p_o = horizontal in-situ pressure in the soil at the level in question

The creep strength determined in this way has agreed well with both plate loading tests and triaxial creep tests.

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Dipl.eng. Beikoff, (Bulgaria)

DETERMINATION THE MODULES OF GENERAL DEFORMATION BY DYNAMIC PENETROMETERS. CORRELATING DEPENDANCES

In the course of several years specialists have been working on determination the deformation modules of different soils through calculations of measured settlings.

The modules calculated in this way are compared to the determined modules by means of a settlement plate /5000 cm²/. On the other hand the experiment with the settlement plate is difficult to carry out.

For that reason as an initial step was established the correlating dependance between the deformation modules, obtained by means of a settlement plate and the cone resistance $C_{k,d}$, which is defined much easier by means of state penetrometer.

The linear correlating dependance, established by means of n=50 parallel experiments /correlating coefficient r=0,753/ :

$$E_o = 4\, C_{k,d} - 40 \qquad (1)$$

In the process of work we have searched for an easier method of defining the module of general deformation and we began working by means of a dynamic penetrometer. The mass of the striking part is 20±0,2kg, the falling height is 25±0,25cm and the solid rods diameter is 22mm. Then we determine the number of strokes necessary for penetrating of the terminal 10 cm-N_{10}. To avoid the effect of the critical depth we use the data obtained under it and to exclude the effect of rods length we don't take data from depth more than 6-7 meters. All this means that the obtained correlating dependances $C_{k,d}=fN_{10}$/apply to similar cases. For the values of N_{10} with different effects it is necessary first to make the corresponding corrections.

We found out the following dependances:

1/ Sand with gravel

$$C_{k,d} = 5N_{10} + 10,5 \qquad (2)$$

/n=50, r= 0,945/

2/ Sand

$$C_{k,d} = 4N_{10} + 7 \qquad (3)$$

/n=50, r=0,793/

3/ Clayey sand

$$C_{k,d} = 4,2N_{10} - 10 \qquad (4)$$

/n=78, r= 0,908/

4/ Sandy clay

$$C_{k,d} = 2,7N_{10} - 12 \qquad (5)$$

/n=50, r= 0,817/

5/ Clay

$$C_{k,d} = 2,4N_{10} - 2,2 \qquad (6)$$

/n=56, r=0,952)

By means of the values for $C_{k,d}$ obtained in this way we could define the module of general deformation by formula (1)

The modules of deformation as well as the angle of inner friction respectively the angle of cutting strength and other qualities of soil could be calculated using also the existing dependances of Buisman- de Beer, Sanglerat and others.

Jezequel J.(France)

Une nouvelle methode pour l'etude des sols: l'auto-forage

Au congres de Madrid aussi que dans notre communication a la presente seance,nous vous avons presente le pressiometre auto-foreur.

Principe

Un carottier a pression,portant lateralement la sonde pressiometrique,est introduite par verinage dans le sol.

Au fur et a mesure de la penetration,la carotte est detruite grace a un outil desagregateur actionne en rotation depuis la surface.

Simultanement,les sediments sont remontes a la surface grace a une injection de fluide sous pression.

Arrive a la cote d'essai voulue,la penetration est stopee et,apres une procedure speciale permettant de connaitre la pression naturelle des taux,on peut proceder a l'essai d'expansion.

Cette methode permet de tester un sol vierge et son rendement dans des sols meubles est bien superieur a celui de la methode classique.

On peut aussi determiner la resistance au cisaillement non draine jusqu'a des deformations de 30 a 40%.

L'appareil presente est cependant limite a des profondeurs de penetration de 20 metres environ: la rotation des tiges de forage pouvant sectionner des fils de commande du pressiometre.

Pour palier cet inconvenient nous avons construit un nouvel appareil dit "pressiometre auto-foreur-hydraulique". Immediatement au dessus de la sonde est place un moteur hydraulique entrainant l'outil en rotation.

C'est ainsi possible d'atteindre des profondeures de 100 metres et plus,la mise en place etant egalement de meilleure qualite.

Sur le meme principe d'autoforage nous avons egalement construit de nombreux autres appareils:

- la sonde frottante

Elle permet de mesurer le frottement lateral mobilise lors du verinage et ce-ci sur un sol vierge puisque l'effet de pointe n'existe pas.

- le piezometre autoforeur

Il permet de mesurer la permeabilite k et le coefficient de consolidation C_v en place.

Outre le dispositif d'autoforage, l'appareil comporte:

- une partie filtrante bornee par deux cellules de garde gonflables.
- un cache utilisable pour le dessae rage et pour la penetration dans le sol au dessus de la nappe.

On a ainsi pu mettre en evidence l'influence tres importante du remaniement sur la permeabilite et le coefficient de consilidation: la mise en place de l'appareil sans autoforage a montre par exemple une chute d'une puissance de 10 sur la permeabilite et une chute plus grande encore pour le coefficient de consolidation.

Conclusions.

L'autoforage est une methode nouvelle pour l'etude des sols au meme titre que des carottages ou des essais de paroie de forages.

Elle permet seule d'etudier le sol dans son etat naturel en lui apportant le minimum de perturbation.

Son domaine d'avenir est certainement tres grand en particulier pour l'etude des sols en sites aquatiques.