Session 7

Soil Exploration and Sampling

Recconaissance du Sol et Prise d'Echantillons

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Co-Chairman	D. Mohan (India)
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Panelists	E. Alonso (Spain), A. Anagnostopoulos (Greece), R.K. Bhandari (India), M. Tammirinne (Finland), F. Tavenas (Canada), A. van Wambeke (Belgium)

V.F.B. de Mello, Chairman

INTRODUCTION

Ladies and gentlemen. We shall begin promptly and maintain a tight schedule; "Soil Exploration and Sampling" covers a vast field, at the root of all our technology. The selected contributors at the head table are well--known and duly identified by name plates. My thanks to each and all of them on behalf of the audience.

Do we intend to rehash the succession of recent significant efforts? For instance:

1. 4th ASIAN CONF., 1971, BANGKOK. SPECIALTY SESSION
   QUALITY IN SOIL SAMPLING
   → INFLUENCE OF SAMPLE DISTURBANCE ON SOIL PROPERTIES
   → MEANS OF AVOIDING OR ALLOWING FOR SAMPLE DISTURBANCE
   → (CHANGE TO: MINIMIZING AND; MY SUGGESTION)
   IMPORTANT CONCLUSION: "THERE IS A NEED FOR DEFINITIONS* OF THE QUALITY OF SAMPLES"
   *(MY SUGGESTION: UNIFY TO SINGLE DEFINITION AFTER COMPARISONS OF SEVERAL)
2. 6th SOUTHEAEST ASIAN CONF., 1980, TAIPEI, STATE-OF-THE-ART REPORT (S. OHYA.) "CURRENT PRACTICE OF SITE INVESTIGATION"
3. OTHER EFFORTS, e.g. < ISSMFE SUB-COMMITTEE / STOCKHOLM '81

Kerisel's biographical portrait of Albert Caquot includes a statement of impact to what I emphasize as "the professional conscience" of geotechnical engineers. Pag. 62". Il juge dangereuse la référence au precédent parce qu'il n'y a jamais deux ensembles de conditions identiques, ensuite parce qu'elle paralyse l'imagination et la creativité". I must emphasize the conflict between the needs for some standardizations (temporary, changing by increments) and the recognition of the inexistence of any two cases comfortably alike, but principally the freezing of imagination (inexorably implicit in subsoil exploration) and creativity (essence of engineering). I have recently discussed (Bangkok, Dec. 1980, Seminar on High Dams) the concepts implicit in (A) below.

Exploration, Sampling and Testing are a means to our Engineering ends. As is well recognized, Engineering implies a need for DOMINANT DECISIONS DESPITE DOUBTS: our Designs must guarantee dominance, because of the investments and risks supported. In subsoil and earthwork we are harassed by many doubts, such as listed above (B):-

In careful appraisal of the very fruitful contributions to this Session, I have felt the importance of emphasizing some general points that have not merited mention, because of the compactness of papers.

Firstly, the absolute need for information or minimum indications on GEOLOGIC BACKGROUND. In analysing the papers

A. PRACTICE / PRECEDENTS / PRINCIPLES / PROBLEMS, and / PRUDENCE } IN GEOTECHNICAL ENGINEERING

B.

1. WHAT PARAMETERS < "INDEX" / "FUNDAMENTAL?" / "COMPLEX, LUMPED?" / FOR MODEL ⇌ PROTOTYPE ?
2. FIELD + IN SITU } VS. ? AND/OR ? { SAMPLING + LABORATORY
3. EXTENSIVE + STATISTICAL } VS. ? AND/OR ? { INTENSIVE, SOPHISTICATED
4. PARTICULARIZED TEST, PROPERTY } VS. ? AND/OR ? { GENERAL "PERSONALITY" → PARTICULAR
5. DISPERSIONS → HOW FAR ACCEPTABLE ?
   → BEYOND CERTAIN LIMIT, TOTALLY IRRELEVANT TO ENGINEERING DECISION "UMBRELLA SOLUTION"

submitted to this session two points stand out: firstly how very few give any indication or reference to the geologic background of the site investigation; secondly, the influence of a "school of thought" responsible for the few that do. We must create the persistent habit: we are but an instant in the "stress-strain-time-weathering--lithification-continuous-discontinous" process of geologic history. It is natural that historically soil mechanics should have begun by investigations of the (assumed) homogeneous masses and average behaviors. However, we know that such a mental model is seldom adjustable to reality or to our processes of cognizance of reality (C).

C. GEOLOGIC BACKGROUND

1. MACRO-SCALE BEHIND REGIONAL AND SPECIFIC PROJECT
   e.g. FISSURED CLAYS DUE TO HIGH OCR CLAYS / FISSURED BY DRYING SHRINKAGE }*
   * K_0' VERY DIFFERENT, AND VARYING WITH DEPTH
2. SEARCH FOR DISCONTINUITY
   2.1- IN OPTIMIZED PLANNING OF INVESTIGATION
   a- OUR FIRST COGNIZANCE OF THE CONTINUUM DERIVES FROM ITS LIMITS, THE DISCONTINUITY
   b- ANY MISS AT THE DISCONTINUITY AUTOMATICALLY IDENTIFIES AND DESCRIBES THE CONTINUUM (e.g. YOUNGSTER'S GAME OF NAVAL BATTLE)
   2.2- MANY IMPORTANT ENGINEERING BEHAVIORS ARE CONDITIONED NOT BY AVERAGES BUT BY THE EXTREMES, THE DISCONTINUITY, THE WEAKEST LINK
3. HISTORIC INFUENCES
   HYSTERESIS IN BEHAVIORS IS INEVITABLE REALITY.
   TIME SCALE. GEOLOGIST MUST QUANTIFY HIS BEST ESTIMATE FOR US
   e.g. LEONARDS - BJERRUM QUASI CONSOLIDATION, etc.

Secondly, it must be recognized that any result of engineering investigation is (and always will be) "nominal". There is no culmination of knowledge and/or perfection; even if there were, in theory, such a feat, in practice the limitations of time, money etc. will exclude it. Therefore we must always employ a judicious balance of extremes between alternates shown in D below:-

All this hinges on CLOSING THE (NOMINAL) EXPERIENCE CYCLE (cf. TOKYO 1977, Vol.3, p. 365), AND RE-CYCLE. Are we making our very chance of closing the experience cycle too fluid, because each effort to improve any link automatically upsets the chain? As quality of samples and tests has rapidly improved in advanced areas, the dispersion of such quality across geography and time has greatly widened. Meanwhile engineering solutions (equipments, treatments) frequently tend to dispense with refined investigation? (schematized in E):-

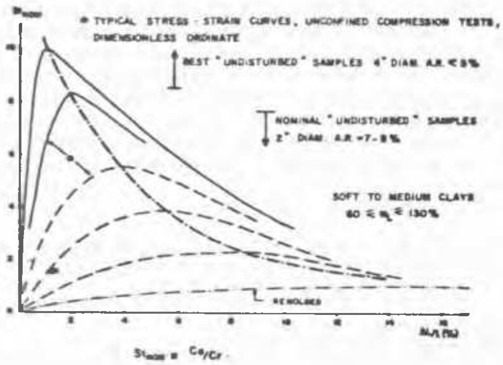
D.

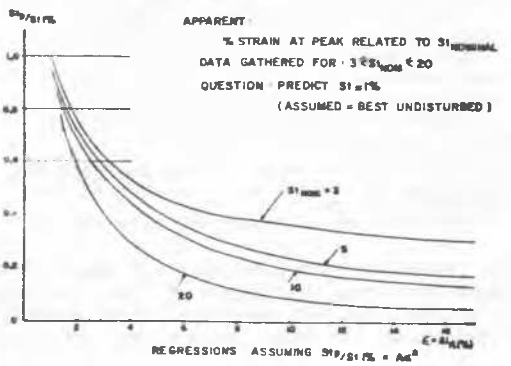
1. STANDARDIZATION FOR REFERENCE.
   MAXIMIZED MULTIPLE PROFILING. (COMPLEX PERSONALITY OF SOIL, MANY UNKNOWNS NEED MANY SIMULTANEOUS EQUATIONS, JUDICIOUS)
2. INVESTIGATION OF WHAT IS, (INDEPENDENT OF)
   THE PRACTICAL WHY AND WHAT FOR.
3. SOLUTION OF SPECIFIC PROBLEM. (BY PRESCRIPTION, PREDICTION OF WHAT WILL NOT HAPPEN).
   KEEPING ALIGHT CULTURAL FLAME, ITERATIVE IMPROVING ADJUSTMENT FACTOR ⟶ 1.00.
   AIM AT STARS OF PREDICTION OF WHAT WILL HAPPEN.

E.

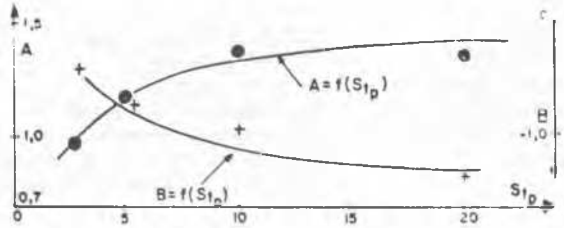
QUALITY OF SAMPLES AND TESTS 1980 1970 1960 1950 1940 ? ? 1940 1960 1980 CONSTRUCTION EQUIPMENT "CAPACITY TO DISPENSE" WITH SOIL

Thirdly the enthusiasm has hitherto been of discovering and describing additional methods of detecting sample disturbance. Quality of sampling has never, to my knowledge, been specified quantifiedly: when indicated, it has never been by END-PRODUCT SPECIFICATIONS but merely by METHOD SPECIFICATIONS which is insufficient in so delicate a problem. The traditional index of Sensitivity St (always a Partial Sensitivity S_{tp}) on undrained strengths and stress-strain curves raises many questions.



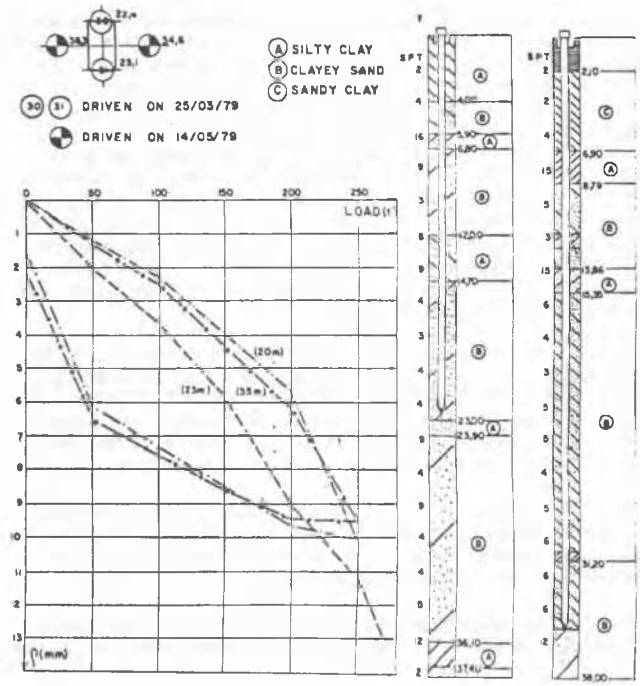


S.S. Bromham (1971, loc. cit. p.68) defines an index based on remolding effects of oedometer tests (Schmertmann ASCE Proc. Oct. 1953, Trans. 1955 p.1201), which will also be found subject to questions and imprecisions. Sophisticated methods based on electron-microscopy etc. have no quantified indices. Never has any correlation been attempted between even the first two routine test indices.



Around 1953-56 I had opportunities of a significant volume of sampling and testing of shelby samples of foundation clays, and obviously noticed the relationship between % strain at failure and the degree of disturbance. The test data (see Fig. below) were analysed statistically, and thereupon used repeatedly to estimate a presumed "perfectly undisturbed" specimen's behavior as corresponding to a failure peak at 1% strain. Candid hypotheses, but they served, and may yet serve, as hints of working hypotheses.

Before turning over the floor to the General Reporter and the debates, I succumb to the urge of giving you just one example of the gross frustrations for which we must be prepared. The following figure summarizes just one example of a big project with consistent unexpected driving of precast centrifuged concrete piles. A Kobe-Diesel K35 was used in alluvia in which all our experience would indicate very good correlation of pile lengths vs. SPT. Yet at distances as close as 3m c. to c. penetrations to "refusal" varied as much as between 20-24m vs. 36-49m. The classical influences of driving sequence, geologic erraticities, time of set, etc. were meticulously examined and disproved. The climax of the surprise came when an architectural change of column was resolved by driving two ad-



ditional piles. The original two had penetrated 22m, the final two an average of 36m. Load tests on all piles were close to identically excellent. Erratic penetrability is of concern and cost. Possibly one part of the question concerns the difference between penetrability and need of penetration (for given load capacity). We often force penetrations much farther than needed. One part, however, resides in a disproportionate influence of partial obstructions near the point, in absorbing driving energy.

G. Stefanoff, Co-Reporter

ADDRESS OF SELECTED TECHNICAL QUESTIONS

We can be satisfied, that after the great discoveries of Karl v. Terzaghi and his collaborators and successors, no major construction works are carried out any more since the middle of this century without extensive prior soil investigations. Therefore it is quite natural that one of the 12 sections of the Xth Conference is on "Soil exploration and sampling".

After the competent introduction of our Chairman Prof. de Mello and the detailed and excellent Report by Dr. H. Mori, my task to address the Selected Technical Questions is made very easy. I shall limit myself to some short comments. Although the number of the papers presented in the Section, namely 38, does not express directly the importance of the questions under discussion, nevertheless in 6 of the 12 sections of the Conference less papers have been submitted, that is our number is above the average. Naturally this number depends on many factors. The papers could be very few, if regional or other conferences dealing with similar problems had been held recently, and reversely, they could prevail if there had been no opportunity to present them on any recent forum. In our case we should take into account the 7th European Conference in Brighton, the International Symposium of Soil Sampling in Singapore, both held in 1979, as well as some other regional and national meetings. This is the reason why not all the topics presented initially have been dealt with in the papers submitted. The examination of the papers indicates, that in spite of the constant progress of science and practice of soil exploration and sampling, some very important problems remain unresolved. Thus for example we can note with regret that although we carry out offshore soil investigations and should be ready to take samples even from other planets, we still do not possess reliable methods for taking "undisturbed" samples from non-cohesive soils under water and from cohesive soils in very soft and liquid consistency; in fact we are not able to take truly undisturbed samples at all. However, we hope that during our discussion we shall learn more about the questions raised, than we know at present.

So let us recall the main topics of our discussion. During the preparation of his General Report Dr. Mori had proposed a greater number, but for lack of papers, some had to be dropped. Only some of those mentioned in the General Report remained.

I ask the panelists, the invited experts and the participants from the Floor for the sake of saving time to abide by the following 3 Technical Questions proposed by the Organizing Committee and the topics proposed by the officers of our Session:

Technical question No 1

IN-SITU MEASUREMENT OF DEFORMATION AND STRENGTH OF SOILS

(1) Interpretation of Pressuremeter Tests for Normally Consolidated Clays.

Technical question No 2

QUALITY OF PARAMETERS

(1) Influence of Sample Disturbance on the Undrained Stress-Strain Relationship of Soils Under Static or Dynamic Load.

(2) Interpretation of the Standard Penetration Test N-value.

(3) Relevance of Undrained Shear Strength Obtained from in-situ Tests.

Technical question No 3

NEW TECHNIQUES OF SITE INVESTIGATION

(1) Advantages and Limitations of the Cone Penetration Test with Pore Pressure Measurement.

(2) Subsurface Interface Radar, Acoustic Emission Techniques, Dilatometer, Non-Sampling, etc.

Finally it is my pleasure to stress again some of the outstanding features of the General Report:

In contrast to some other sections, no state-of-the-art report was required in our Section. In fact, however, the written Report of Dr. Mori analyses and comments in such a manner the papers and relates them to other important 68 papers indicated in the List of References enclosed in the General Report, that this excellent General Report besomes at the same time a first-class state-of-the-art report, giving a solid basis for a fruitful discussion. I myself fully support the conclusions drawn by Dr. Mori.

D. Mohan, Co-Chairman

OPENING REMARKS

The chairman and the general reporter have already emphasised the importance of proper soil exploration and sampling. It is a fact that little attention is paid to both these aspects in most projects on geotechnical engineering. Both the tasks are carried out in a casual manner and it is a common experience that when actual soil strata presents a picture which is different to that mentioned in the soil report. Proper education and training in this field is therefore very important, from the soil engineer down to the foreman supervising a boring and sampling operation. In this respect I would like to commend the work of the ISSMFE subcommittee on soil sampling which under the chairmanship of Dr. H. Mori, the general reporter for this session, has recently published an excellent "International manual for the sampling of soft cohesive soils". I hope the committee would soon take up work on cohesionless soils also. I also hope Mr. S Wilson's subcommittee on "Site Investigations" also will submit their report soon.

The technical session, although named "Soil exploration and sampling" is also going to discuss in-situ test methods, e.g., pressuremeter test and the vane shear test. One of the panelists, Dr. R.K. Bhandari (India) will be describing the new soil exploration technique. I hope the panelists and various speakers who will follow them will critically examine and make an overall assessment of various equipment and test methods which will make all of us wise in the proper use of these equipment and subsequent interpretation of the test data.

Now, to the program for the afternoon session. The technical questions have already been presented by the co-reporter Prof. G. Stefanoff. They will be taken up by panel members, one by one, and after each panelist has made his contribution I propose that 2-3 speakers from the floor, who have either been invited to the discussion or who have sent in their names on that particular subject. Thereafter, time permitting, I would like to allow very short informal discussion from the floor. We have 20-25 minutes for each question and I request you to be brief in your presentation

A. van Wambeke, Panelist

Being in charge of the presentation and discussion of technical question No. 1:

"Interpretation of the pressuremeter test for normally consolidated clays", I would first of all like to welcome the pressuremeter specialists with us today and bring to your attention the absence of two of their members: the late Louis Menard who supplied the initial impulsion, which we still benefit from, and Jean François Jezequel who through illness is unable to attend but to whom we send our sincerest wishes for a rapid recovery.

Fifteen years ago it was difficult to talk about the pressuremeter in academic circles but this situation has considerably changed to the point where the technique is now a highly respected in situ testing method. The early work by Menard sparked off various inventions such as the self boring pressuremeter designed by the Ponts et Chaussées (France) and the Camkometer of Cambridge University which differ from the Menard Pressuremeter in both the Moduls operandi and the Interpretation of the test.

I believe that we have now reached the point where a comparison is possible between the two types of pressuremeter in order to find their similarities and specific caracteristiques. As is the case with all in-situ testing techniques, the personal experience of the operator is of relative importance. Since no perfect system exists also the use of each type of equipment must be pointed out. In fact there are two different objectives which may be looked for. Sometimes the goal is a scientific one in the form of a theoretical study or a concrete problem which may undergo a mathematical approach depending on the scope and limiting conditions of the study. However, in the majority of cases the project is essentially of a practical nature and the solution must be rapidly found with sufficient security and economic considerations, the scientific preoccupations taking second place: this is a problem of dimensioning. Here the only criterion acceptable is in fact the prediction of the reality.

It is within this frame work that I would set out these questions with a view to continuing the discussion along these lines:

1 - How important is the problem of soil remoulding which occurs mainly during insertion of the probe, and in fact may it be considered negligible for self boring pressuremeter?

2 - Are the results influenced due to the absence of guard cells on the self boring pressuremeters?

3 - Is the practical use of the self boring pressuremeter based on a direct and specific experimental control?

4 - Is there a well defined relationship between the conventional limit pressure (Menard) and the 20% limit pressure P_{20} calculated from the self boring probe expansion curve on one hand and the pressuremeter Modulus Em with the shear moduli Gp2 and Gp5 determined from the self boring pressuremeter expansion curve on the other.

F. Baguelin (Oral discussion)

PRESSIOMETETRE AUTOFOREUR ET PRESSIOMETRE MENARD
Self-Boring and Menard Pressuremeters

1. PERTURBATION DU SOL A LA MISE EN PLACE (TABLEAU I)

PERTURBATION (o) nulle (+) faible (++) importante (+++) très importante	PAF.76 Autoforage	PRESSIOMETRE MENARD forage préalable	 battage direct
1 Mouvement du fond du forage, entraînant déplacement latéral du sol à tester.	(+) l'enlèvement du sol intérieur est compensé par l'effort de fonçage à la base de la sonde.	(++) - les outils de forage appliquent des efforts variables. - le fond est déchargé en fin de forage.	(+++) - le sol est poinçonné et refoulé latéralement.
2 Maintien de la paroi.	(o) paroi maintenue	(++) paroi déchargée en fin de forage.	(++) paroi fouettée par le tube lanterné.
3 Cisaillement vertical de la paroi.	(+) limité au contact, car la sonde est bien rectiligne et le fonçage sans à-coup.	(+++) - va-et-vient répétés des outils et tiges de forage, de la sonde. - calibre variable de ceux-ci.	(+++) - cisaillement par à-coups. - calibrage médiocre du corps de sonde.
4 Pollution par l'eau du forage.	(o)	(++) ou (+++) suivant les sols	(o)

En conclusion, les perturbations apportées par l'autoforage sont sans commune mesure avec celles des mises en place traditionnelles du pressiomètre MENARD (ou des autres essais in-situ).

2. VALEURS D'ESSAI CARACTERISTIQUES

Pour les 2 cas extrêmes de sol : argile molle, et sable compact, avec mise en place du pressiomètre MENARD dans un forage préalable, la comparaison des paramètres p_{20} et G_{p2}, G_{p5} pour le PAF, p_l et G_M pour la sonde MENARD, et accessoirement de p_o et p_{oM} s'établit ainsi :

argile molle : $p_l \simeq p_{20}$

$$\begin{cases} G_M \div G_{p5} \div G_{p2} \\ 1 \div 5 \div 10 \end{cases}$$

$$p_{oM} > p_o$$

sable compact : $p_l \simeq 1,5\ p_{20}$

$$G_M \underset{(<)}{\simeq} G_{p2} \underset{(<)}{\simeq} G_{p5}$$

$$p_{oM} > p_o$$

Dans l'essai MENARD, la phase initiale est fortement affectée par les perturbations dues à la mise en place, de telle sorte que la pression initiale p_{oM} et le module G_M n'ont pas de valeur intrinsèque, sauf pour les sols compacts. La pression limite, p_l, d'ailleurs définie conventionnellement, est moins sensible à ces perturbations.

Pour le PAF, les paramètres p_o, G_{p2}, G_{p5} peuvent être regardés comme intrinsèques, p_{20} est conventionnel.

3. DOMAINE D'EMPLOI

Les essais au PAF peuvent contribuer à l'étude des paramètres fondamentaux des sols : pression p_o, modules, courbe de cisaillement,
mais peuvent aussi être utilisés,comme l'essai MENARD, pour les calculs de fondations, par le biais de corrélations directes entre paramètres caractéristiques : les coefficients de passage diffèrent bien évidemment, cf. Réf. 1 et 2.

Cependant, contrairement à l'essai MENARD, réalisable dans presque tous les terrains, l'autoforage est limité aux sol contenant peu ou pas de graves. Il est donc réservé aux reconnaissances à prédominance de sols fins (jusqu'aux sables inclus) ; en étude courante de fondations, il est intéressant quand de telles formations sont épaisses ou pour des conditions particulières (sites aquatiques).

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2. "L'autoforage", S. AMAR, F. BAGUELIN, R. FRANK et J-F JEZEQUEL, Revue Travaux, février 1981, pp. 63-76.

M. Jamiolkowski and R. Lancelotta (Oral discussion)

DISIGN PARAMETERS FOR CLAYS FROM TESTS WITH THE SELF-BORING PRESSUREMETER

Referring to the topics of discussion suggested by panelist Prof. Wambeke we should like to point out that, as is shown in the paper by Battaglio et al. presented to this conference, even in the case of an excellent expansion curve the derived values of undrained strength c_u and stiffness E_u generally tend to be higher than those derived from high-quality laboratory and field vane tests. According to the writers a high-quality expansion should satisfy all of the following requirements:

1° There is no inflection point at the beginning of the curve.

2° The measured initial stress p_o is very close to σ_{ho} (in situ total horizontal stress).

3° The pore pressure u measured at the cavity-soil interface before the start of the expansion test is very close to the hydrostatic pressure, u_o, in the surrounding soil mass.

4° The computed pressuremeter modulus, E_{up}, decreases monotonically as p increases.

The experience gained from the analysis of ≅ 200 self-boring pressuremeter (SBP) tests carried out in six different saturated clays permits the following tentative conclusions.

As regards undrained shear strength:

1° When computing c_u from SBP tests,presently available interpretation methods lead to substantially the same results.

2° Generally, c_u derived from good-quality SBP tests is higher than c_u derived from laboratory and other field tests.

3° This phenomenon is particularly pronounced in the case of a low L/D ratio where the expanded cavity is of near-spherical or elliptical shape.

4° Stress-strain curves from SBP tests show much lower strains at failure than those observed in laboratory tests.

As regards undrained stiffness:

1° E_{u50} (i.e. undrained deformability modulus at a stress level equal to half the maximum shear stress) derived from good-quality SBP tests significantly exceeds the value which results from laboratory tests performed on high-quality undisturbed specimens.

2° E_{u50} generally does not agree with the "pressuremeter modulus" E_{up} computed directly on the expansion curve for the same strain level.

When comparing laboratory and SBP test results one implicitly assumes that the results from the SBP tests should fall within the limits given by the laboratory plane-strain compression (PSC-CK_oU) and the direct simple shear tests (DSS-CK_oU), see Ladd et al. 1979 and Levadoux and Baligh (1980).

The writers apologize that when formulating the discussion topics the panelists omitted the very important problem of determining the existing in situ total horizontal stress, σ_{ho}, during the SBP test. The writers' experience in this respect is based on the experimental evidence shown in Table 1 and may be summarized as follows:

1° When dealing with good-quality expansion tests (without inflection point) the initial pressure, p_o, is a good indicator of the existing in situ total horizontal stress, σ_{ho}.

2° The existing correction procedures (Marsland and Randolph (1977), Denby (1978), etc.), when applied to good-quality tests, do not improve significantly the reliability of the predicted σ_{ho} value.

TABLE 1

Initial pressure p_o measured in the SBP test compared to the best estimate of the existing in situ total horizontal stress σ_{ho}

(Soft Clays, see Battaglio et al. (1981))

S I T E		EQUIPMENT	L / D	TEST YEAR	p_o / σ_{ho}
PORTO TOLLE	(Italy)	PAFSOR	2	1975 - 1976	0.85 ± 0.12
PORTO TOLLE	(Italy)	PAFSOR	2	1979	1.05 ± 0.12
PORTO TOLLE	(Italy)	PAFSOR	4	1979	1.12 ± 0.15
BANDAR ABBAS	(Iran)	PAFSOR	2	1978	0.97 ± 0.15
GUASTICCE	(Italy)	PAFSOR	2	1977	0.82 ± 0.14
TRIESTE	(Italy)	PAFSOR	4 2	1979	1.29 ± 0.18
DRAMMEN	(Norway)	CAMKOMETER	6	1979	0.99 ± 0.06
ONSÖY	(Norway)	CAMKOMETER	6	1979	1.00 ± 0.04

3° Greater reliability may be obtained taking into account factors like:
- self-boring process;
- relaxation time;
- flexibility of the measuring system;
- drift of the electronic equipment.

Based on their four-years experience with the SBP the writers believe that at present the following final remarks may be made on the use of the probe in soft saturated clays:

1° The SBP test is very promising for the assessment of the in situ initial total horizontal stress. However, there is a need for the improvement of the read-out equipment and the optimization of the probe installation procedure.

2° The test may eventually give the complete undrained stress-strain curve of the examined soil. The in situ evaluation of the undrained stiffness is of great practical interest. A better understanding is necessary of the influence of the disturbance caused by the probe insertion on the derived stress-strain curve. Also, further theoretical research is required on the soil model to be used for the interpretation of test results.

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M. Gambin (Oral discussion)

There are more and more talks about pressuremeter. This should not bring displeasure to the promotors. However, we consider that this attraction cannot be natural and dangerous because it is based on several misconceptions.

1 - The theory of the Menard Pressuremeter is based on similarity between the soil response under footing or pile loading:

- somewhat similar stress distribution,
- some progressive loading,
- similar disturbance of soil:
  - for bored piles and pressuremeter probe inserted in a bore hole
  - for driven pile and driven probe
  - for shallow footing resting on the heaving bottom of an excavation and probe in a bore-hole.

2 - The major problem which occurs during foundation design is due to soil disturbance created by the excavation procedure or the pile driving and not soil disturbance due to pressuremeter probe placement, especially because soil disturbance due to foundation construction becomes more and more important with the progress of the foundation techniques.

3 - The only arbitrator in foundation design is the actual behaviour of the foundation. Our methods of foundation design are based on correlation between pressuremeter tests and records of foundation behaviour, mostly settlements.

4 - When Louis Menard was working out his theses on the pressuremeter in the mid 50's, he could choose between a purely theoretical approach involving the theories of elasticity and plasticity or a direct and synthetic method. He prefered to use the last one, because the analytical methods were, and are, still imperfect.

5 - The Menard Pressuremeter probe can be inserted in a bore-hole drilled with the help of a bentonite slurry or can be driven into the soil or still advanced by the retro-jet method which brings disturbance to a minimum level. The choice between these various methods is made according to the type of problem to solve:

- Design of shallow footing or drilled piles relies on pressuremeter tests carried out in drilled holes,
- Design of driven piles relies on pressuremeter test carried out with driven probes,
- Assessment of slope stability or active earth pressure relies on pressuremeter test, carried out with retrojet probes or self boring probes.

6 - The designer of foundations must not forget the following factors:

- non homogenity of soils which can only be assessed by many in-situ tests and not by refined lab tests on one or 2 carefully chosen samples.
- actual bearing capacity of weak rocks which can only be verified by in-situ loading tests, such as pressuremeter test,
- fatigue of soil, which includes liquefaction due to cyclic loading or rapid rate of loading.

7 - To try to rebuild traditional soil mechanics from pressuremeter testing is not yet feasible. It seems to us much better to keep its specificity to the pressuremeter.

This was the address of Louis Menard to those attending the 9th International Conference in Tokyo - Nothing seems to have evolved since then.

A. van Wambeke, Panelist

CONCLUSIONS

(After intervention by Baguelin, Jamiolkowski,Gambin and the General Reporter Dr. Mori)

I am in complete agreement with the conclusions formulated by Dr. Mori.

I would, however, like to add one or two observations on the problem of soil remoulding.

Trying to fully appreciate the phenomenon of remoulding has always been a difficult exercise since the reference state of any soil is very difficult to describe qualitatively and quantitatively. There are several factors which influence soil disturbance during pressuremeter testing apart from the boring procedure: for example we have probe diameter, the nature of the disturbance, whether consolidation or loosening and the initial structural form of the soil.

Furthermore, we may afford a certain lee-way on the calculation of the soil characteristics due to these perturbations.

But in the case of the undrained shear stress of a clayey soil it is obvious that if we use the undrained cohesive strength Cu, remoulding tends to become an extremely important boundary condition. The initial soil pressure Po, the length of the probe and the accuracy of interpretation are amongst the factors which may also add to the inaccuracy of the result.

Nevertheless, in order to solve problems in practice, the remoulded soil limit is known during testing and we know that the limit pressure is unaffected by remoulding which allows us to dimension the structure in terms of the load bearing capacity of the soil.

Considering the moduli required for the calculation of the settlements, the initial deformation moduli may be estimated with sufficient accuracy by using cyclic loading tests and calculating the corresponding cyclic moduli.

E.E. Alonso and J.A. Gilt (Oral discussion)

SMALL STRAIN DEFORMATION MODULI FROM TRIAXIAL TESTS
Modules de Rigidité à Petites Déformations Obtenues dans des Essais Triaxiaux

Considerable attention has been given in recent years to the small strain behaviour of foundation soils since, in many cases,it plays a fundamental role in their static deformation.Of particular relevance for the prediction problem is the determination of the undrained initial elastic moduli in the laboratory.

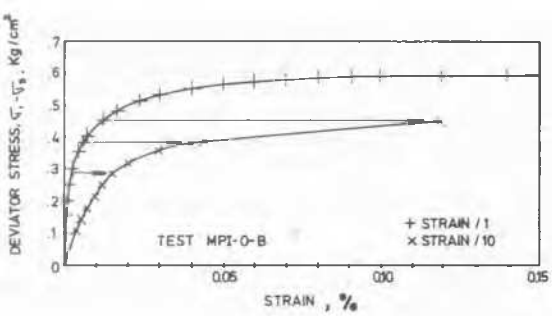

Fig. 1. Stress-strain curve for CIU test on "in situ" specimen. Low-plasticity clay (w_L=29%, I_P=10%).

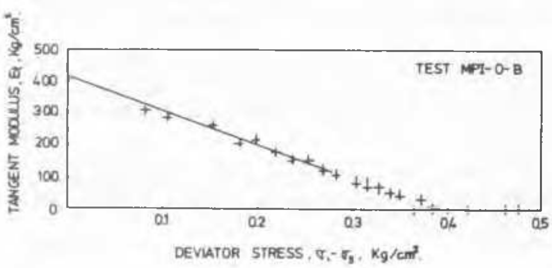

Fig. 2. Variation of tangent modulus with stress level.

In order to investigate the small strain behaviour with conventional (but careful) triaxial tests,let us consider the behaviour of an"in situ"specimen so that both mechanical and stress relief effects are avoided ."In situ" specimens (They never undergo unloading)were consolidated within a triaxial cell in the manner described by Alonso et al(1981) and then loaded (CIU and CAU tests were performed). Setting errors were minimized by this prodedure. However,apparatus compliance(strains were measured by LVDT'S externally located)are not avoided.A typical stress-strain curve is plotted in Figure 1 at two different strain scales.Even with precise and automatic recording systems it is difficult to reach reliable values for the moduli at samall strains.To improve understanding we can use a model for small strians, for instance, assuming a simple exponential or hyperbolic increase of stress with strain. In the exponential model it can be shown that the tangent modulus is linearly related to the current stress level (See Fig.2).This fact allows an easy computation of initial modulus.(Hyperbolic models were also used and resulted in similar values).A better feeling for the behaviour of these samples at small strain can be obtained from Fig.3 in which the normalized secant modulus is plotted against strain.Also,for comparison,is plotted the ratio G/Gmax mostly based in dynamic tesd,proposed by Seed and Idriss(1970) for saturated clays.The difficulties in extending the triaxial results to small (static)strain($\sim 10^{-4}$) are shown.On the other hand in any of the curves drawn the ratios of secant moduli at strains in the order of 0.01% to the moduli at 0.1-0.2% (maximum resolution for conventional triaxial tests)are close to 2.Note,too, that in the hyperbolic and exponential models the "zero strain" modulus is very close to the 10^{-4} strain modulus. Dynamic G values show,apparently, a different trend.

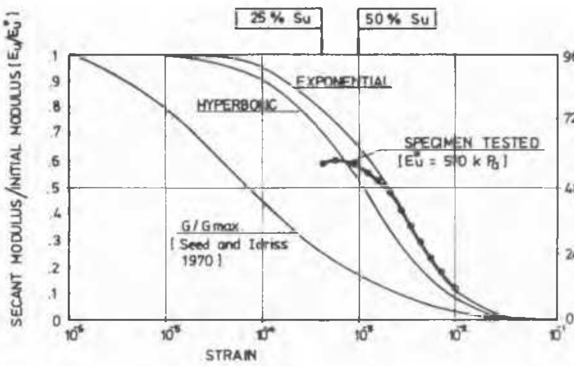

Fig. 3. Normalized secant undrained modulus as a function of vertical strain. $\overline{CIU}$ test on "in situ" specimen.

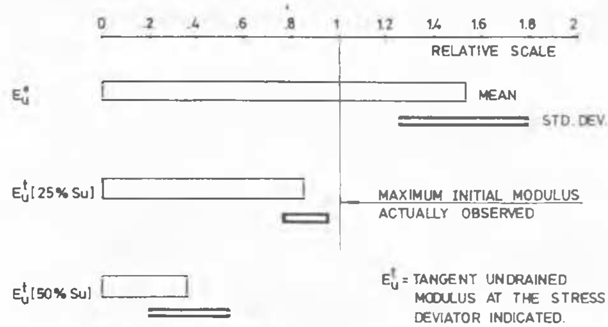

Fig. 4. Results of 9 $\overline{CIU}$ tests on N.C. "in situ" specimen. Low-sensitive clay.

The results of nine tests on "in situ" specimens in terms of undrained moduli, E_u , at different strain levels are presented in Fig.4. If E_u is measured at 50% undrained strength (which is a common practice) the "zero strain" moduli (E_u^o) can be 2 to 4 times larger than the measured value. This ratio is in the order of 1.5-2 if the best direct measurement ($\varepsilon \sim 0.2\%$) of the moduli is made in the triaxial test. For many practical situations involving the deformation of the soil these effects can be considered as a "disturbance" effect to be added to the well known mechanical and stress relief-effects.

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W.M. Kirkpatrik (Oral discussion)

The previous speaker mentioned stress relief and it is to this subject that I will direct my remarks. The changes in total and effective stress and pore water pressure caused by removing a soil element from its insitu stress system in a sampling operation is known to affect the stress strain properties of the material.

Stress relief has received attention from research workers in recent years and I would like to describe some of my experimental work. This work was conducted on laboratory manufactured clay since the nature of the problem prohibits the examination of naturally occurring materials. We looked at two clays - one of high permeability (Kaolin $k = 1 \times 10^{-9}$m/s) and one of medium permeability (illite $k = 7 \times 10^{-11}$m/s). We examined normally loaded and lightly overconsolidated materials. The clays could be considered young, intact, uncemented and from relatively deep insitu positions (for example submerged depths of 55 to 60 m in the case of normally loaded clays). The triaxial compression equivalent of the undrained stress strain characteristics of "samples" were compared with those of the "insitu" soil.

"Insitu" soil was soil which was tested from the "insitu" effective stress state without first unloading to zero total stress whereas a "sample" was one which was unloaded from this "insitu" stress state under undrained conditions and then tested after storage for various times. Physical disturbance of the soil was largely avoided so that stress relief effects could be exclusively examined.

The methods of preparing the "insitu" and the "sample" soil are described in the Paper No. 21, Session 7, by myself and Khan in the present conference (and elsewhere).

Dealing first with the normally consolidated clay, in Fig. 1 stress strain curves (a) and (b) are for "insitu" and for "samples" of illite respectively. The things to note when comparing the "sample" with the "insitu" soil are:

(i) the appreciably lower peak strength of the "sample", and
(ii) the appreciably larger strain to failure and much smaller modulus value in the "sample"
(iii) the "sample strength reduces with age, a fresh sample having about 70% of the "insitu" strength.

All these effects take place nominally without change of water content.

How can we obtain better results, from samples, which can be considered to be more relevant to the insitu condition? The obvious first trial would be to treat samples to some form of consolidation prior to undrained compression. Fig. 1 shows stress strain curves of several consolidated undrained states performed on "samples" for comparison with the straight undrained data for "insitu" and "sampled" clay. Curve A is for a consolidated undrained test in which the sample was consolidated to the same consolidation stresses as the insitu soil. This is ko consolidation and since ko $\neq$ 1 the consolidation stresses were anisotropic. It is seen that this CAU test closely simulates the "insitu" curve although it gives a peak strength 5-10% greater.

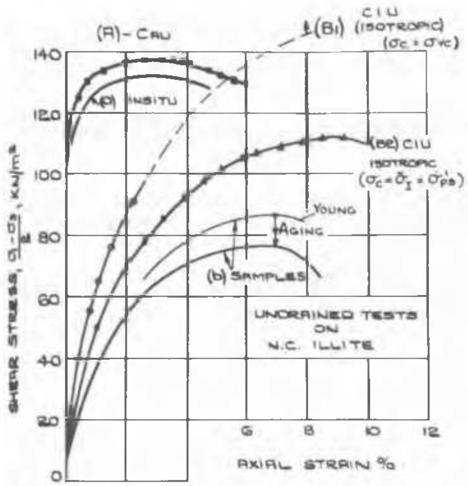

FIG. 1.

The curves for the other two consolidated undrained tests performed diverge widely from the "insitu" curve. Both of these used purely hydrostatic consolidation. In B1 the hydrostatic consolidation stress was made equal to the major (vertical) consolidation stress of the "insitu" soil. The peak strains for this test were appreciably higher than those for the "insitu" soil.

For test B2 the consolidation pressure used was the theoretical value of the negative pore pressure caused to the sample by undrained unloading from the insitu condition by the sampling process. This value is sometimes referred to as σ'ps, - "ps" meaning perfect sampling. σ'ps is very close to the mean principal effective stress acting on the "insitu" element at the end of primary consolidation. This B2 test typically yields peak strengths 10-20% lower than the "insitu" value, but again with failure strains appreciably higher.

We have also experience of lightly overconsolidated clays with OCR's in the region 2 to 3. Lack of space prevents these from being described in detail but the story is similar to that of the normally consolidated clays.

The reason for the fairly serious change in behaviour caused by stress relief is not fully understood. It is suspected however that this is related to the loss of effective stress due to the dissipation of negative pore water pressure during and after sampling. This loss could possibly be assisted by cavitation of the water in the soil pores. Assuming the cavitation pressure in the small pores of the samples is somewhat less than -1 atmosphere the effect would not be present on shallow samples taken from submerged depths of less than say 12 or 13 metres. Samples nearer the surface therefore would not have the same dramatic loss in strength and would possibly conform to models proposed by previous workers and be nearer the behaviour reported by the last speaker.

Generally however a good approximation of insitu behaviour is found if undrained compression is conducted after reconsolidation to the insitu stresses.

M.C. Goel (Written discussion)

SOIL EXPLORATION AND SAMPLING HAZARDS IN EXPLORATION OF STIFF CLAYS

Soil exploration and sampling in fissured stiff clays having interbeddings of very thin plastic clay seams poses a serious hazard for an investigator. In India at Garhwal-Rishikesh Chilla Hydel Scheme, whose gravity intake blocks are founded on upper Shivalik Clay-shales followed by alternate bands of stiff clay and pervious strata, all conventional methods of exploration could not explain the continuous occurrence of land slides during construction period. In this discussion, the methodology adopted for this case would be discussed.

During normal course of exploration before construction, the test results indicated that the foundation material of intake can be classified as clay with medium plasticity preconsolidated to the pressures of the order of 18-20 kg/cm^2. The presence of thin plastic clay seams was indicated neither by percussion nor by rotary drilling. However, during construction slides took place specially during rainy season. Consequent upon occurrence of slides, laboratory tests by collecting undisturbed chunks of clay band material were repeated. Even the repeated tests indicated that clay bands are overconsolidated corresponding to a pressure of 7 kg/cm^2. The calculations indicated that even if a cohesion of 0.5 kg/cm^2 is available, a vertical cut of near about 10 m height would have been stable. But the presence of slope failure indicated that the shear strength as given by laboratory triaxial shear test results on cylindrical specimen of size 37.5 x 75 mm made from undisturbed chunks, was actually not available in the field. Moreover, it was extremely difficult to prepare cylindrical specimens from the chunk. Conventional method of obtaining undisturbed samples by tube insertion was not possible because of the stiffness of clay.

Since the continuity of overconsolidated clays is commonly disrupted by a network of hair cracks, the actual shearing strength may be even much less. Hence, in situ slide tests were conducted on 60 x 60 cm block. But, contrary to our expectation, it was observed that the field values were higher than laboratory values. At this stage, the presence of two very thin highly plastic clay seams was detected in the field. Consequently, efforts were made to test the thin plastic clay seam for shear parameter values by preparing 75 x 38 mm cylindrical samples having seam sand-witched at 45^o + Ø/2 angle, but the samples could not be prepared. Direct shear tests in a 6 cm bore indicate the residual value of cohesion as 0.28 kg/cm^2 and the angle of internal friction as 13^o. Nor could these values explain the slide. In-situ slide tests were neither practicable because of topographical features and rate of construction, nor could they have given representative values, firstly because of uncertainty of forcing sliding actually along weak plane, and secondly because of impracticability of reproducing saturation condition.

As in such type of overconsolidated clays having pressures and slickensides, the laboratory and field test results could not give true representative values of shear parameters of the soil, hence the same were worked back on the basis of observed conditions of slope stability. The presence of an almost vertical slide in stiff clay and an almost parallel slide along bedding planes indicated that failure was taking place along a plastic seam. In the analysis, it was assumed that along a vertical cut in stiff clay, because of the fissures, the shear resistance offered by this type of soil reduces to zero and the cracks, when filled with water during rainy season, also exerts water pressure. The analysis indicated that if value of cohesion is taken equal to zero, then the factor of safety against sliding fell below unity, thus explaining the slide.

Though the occurrence of slides could be explained by working back the slope stability in this particular case and design shear parameter values could be decided, yet the following questions need attention relating to soil exploration:

1. What should be the method of exploration in stiff clays having inter-bedding of thin seams so as to depict the strata precisely well before construction stage?
2. How should the sampling be done so as to include the fissure effect in stiff clays?
3. What should be the procedure of preparing specimen so as to include (i) fissure effect and (ii) weak planes?
4. How to simulate the difficulties of actual field conditions subsequent to construction?

M. Kany and R. Herrmann (Written discussion)

THE METHOD OF SAMPLING IN STIFF COHESIVE SOILS IN THE FEDERAL REPUBLIC OF GERMANY

SUMMARY

In this publication the method of sampling in stiff cohesive soils is described. For sampling in soils of this kind also double tube core barrels or triple tube core barrels resp. are used. These devices work according to the so-called Denison-principle, which proved good. First of all the characteristics, devices and frequency of the most important models used for sampling in the Federal Republic of Germany is described. These results have been obtained in the scope of the review study "boring/sampling" by help of an inquiry among the drilling firms.

The special types of devices developed and used for sampling in stiff cohesive soils described in detail. Both the characteristics and sampling is described. The sample quality achieved by this technology is described in connection with the existing test results and with respect to the special criteria.

1. INTRODUCTION

Since this principle of construction was developed double tube core barrels have been used for sampling in the Federal Republic of Germany.

In order to get an idea of the used devices, procedures and special methods, an inquiry was made for the research project "review study boring/sampling". The inquiry included all firms (drilling firms) which are executing soil investigations by boring and sampling. Moreover, in this research project the actual international state of sampling is described (3).

As to the double tube core barrels (for sampling of all kinds of soil and rock) following results should be mentioned (3,4):

- double tube core barrels are used by 67 % of the drilling firms
- 5 - 10 % of the samples obtained are used as special samples
- 21 % of the devices work according to the so-called wire line core drilling procedure
- the inside diameter ranges between 179 - 48 mm
- the most frequently used medium diameters are 100 and 108 mm
- the bandwitch of the lips of the drill bits was stated to be 4 - 22 mm
- the general medium diameter was defined to be mostly 7 - 11,5 mm

Triple tube core barrels or even better double tube core barrels with immerseable liners are used as follows (3):

- 29 % of the firms are in possession of these devices
- the part of special samples taken by these devices was stated to be 5 - 10 %
- 67 % of these devices are self-developments of the drilling firms. The development was made partly in cooperation with the device-manufacturers
- the inside diameter amounts to 172 - 71 mm, $\bar{x}$ = 75 mm
- the lips of the drill bits range between 3 - 12,5 mm

2. SAMPLERS

The latest devices developed for sampling in stiff cohesive soils work according to the Denison-principle, which proved good (1,3,5,6,8). The cutting edge of the inner sampling tube runs in front of the bit and keeps the core (sample) from the circulating drilling mud.

If one took the definition according to HVORSLEV (1) for the rating of sampling following device-characteristics would result for the inner tube:

ID. = inside diameter of the cutting edge
C_a = area ratio
C_o = outside clearance ratio
C_i = inside clearance ratio
α = cutting edge angle

Type 1: double tube core barrel according to (8)

ID. = 96 mm
C_a = 21 %; (15 % for short penetration)
C_o = 0 %
C_i = 2 %
α = 10° (degree)

Fig. 1

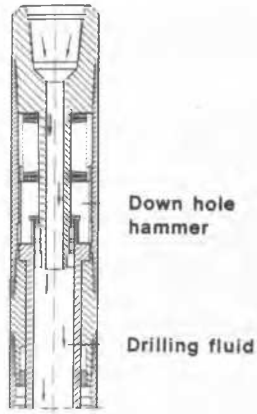

Fig. 1a

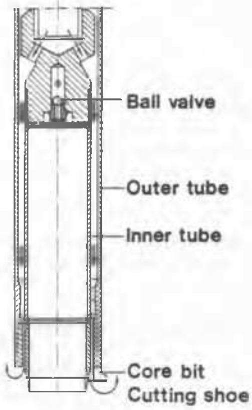

Fig. 1 and 1 a: Type 1 double tube core barrel according to (8)

Fig. 2

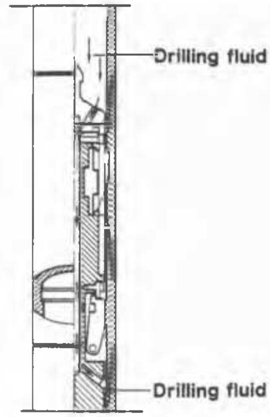

Fig. 2a

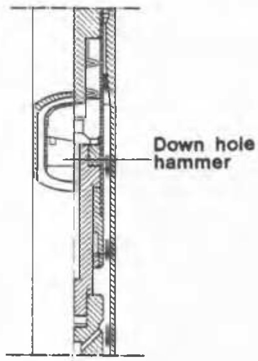

Fig. 2b

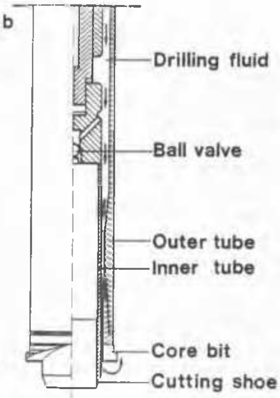

Fig. 2, 2a, 2b: Type 2 double tube core barrel according to (8)

The device is placed on the drill pipe. For the sampling simultaneously to the boring procedure the inner tube is pushed in by help of a down-hole hammer. In this device the cutting edge runs 10 - 50 mm in front of the bit (depending on the kind of soil). The sample is recovered from the inner tube by an injection-device.

Type 2: double tube core barrel according to(8)

ID. = 90 mm
C_a = 30 %
C_o = 0 %
C_i = 2 %
α = 10^o (degree)

As to type 1 the inner tube is pushed in by means of a down-hole hammer, running in front of the bit. In this device, however, the cutting edge runs 10 - 50 mm in front of the bit (depending of the kind of soil). A special feature of this model is, that it can be applied as a wire line core drilling machine (wire line core drilling procedure). Thus sampling is possible without the necessity of the drill pipe being completely removed. As to the sample quality it is of special significance, that using this procedure it is impossible that the soil falls down from the wall of the borehole. Thus, it can be prevented that soil material, which cannot be used, is taken up.

Type 3: double tube core barrel with liners (triple tube) according to (6)

ID. = 72; 87; 101; 113 mm
C_a = 13; 14; 16; 19 %
C_o = 0 %
C_i = 8; 7; 3; 3 %
α = 15^o; 30^o (depending on the kind of soil)

Fig. 3

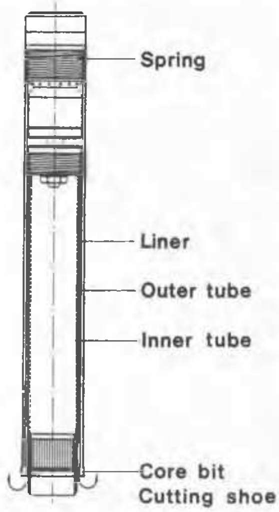

Fig. 3: double tube core barrels with liners (triple tube) according to (6)

The inner tube of this device is continuously pushed in. This device has a special cutting edge construction and a great inside clearance ratio. By this the removed sample will be slightly impaired in the area of the sample tube as a consequence of wall friction. In general, the liners are placed into the device consisting of non-transparent plastic material. For special investigations and in order to make a direct visual examination possible, also transparent plastic liners can be used. The sample can directly be taken out of the liner.

For qualitatively good sampling (the cutting-out of the sample) a special geometrical shape of the cutting edge was developed. During penetration there is the possibility that in account of the increasing inside diameter of the cutting edge, the core behind the cutting edge will relieve stress. This method of sampling corresponds to a careful work-out of the sample.

The relation between cutting edge and area ratio can be represented as follows for the described devices related to the inner tube:

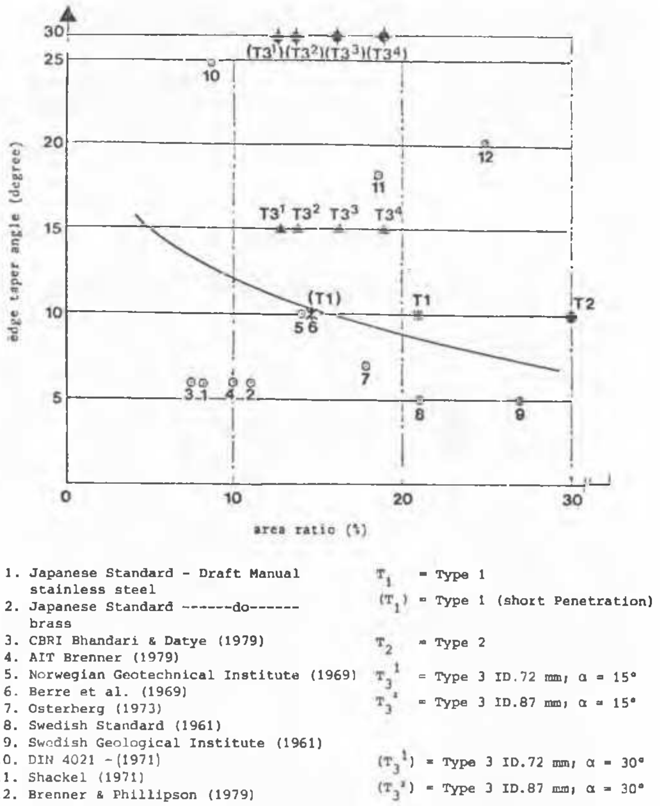

Fig. 4: The relation between cutting edge and area ratio , see (7)

The characteristics of the devices described range near those of the open samplers (for the recovery of qualitatively valuable samples) see (7).

3. SAMPLE QUALITY

An examination of the quality of the samples taken by these devices is not possible in the bulk. The examination criteria according to HVORSLEV (1) are only conditionally valid in account of following facts:

- an easier penetration of the sampling tube is reached if the soil outside the sampling tube is removed (bored)
- the special geometrical shape of the cutting edge
- the possible reduction of the soil quality as a consequence of the circulating drilling mud in dependence on the cutting edge and pressure running in front of the bit
- the special kinematics and kinetics during sampling

Prevailing investigations of samples taken by the Pitscher-sampler according to MORGENSTERN/THOMSON (5) showed values of consistency limits corresponding to those taken by open samplers (Shelby-tube). This is also valid for the dependence on the sampling depth z and the results got by the triaxial test. Greater maximum deviator -stresses ($\sigma_1 - \sigma_3$) resulted for "Pitscher" and "Shelby-tube" with the water content being equal.

Further results got by practise state about 10 - 15 cm/minute as optimum boring velocity in sandy loam and clayey soils (6). This sampling velocity causes a little impairment of the sample in account of the necessary scavenge pressure by the circulating drilling mud. A recovery was made by means of the described samplers from all kinds of soils, partly down to strongly weathered rock. In a certain part of these devices a sample-retention equipment can be built in.

For strong cohesive stiff soils a sample quality-class of 1 according to IDEL/MUHS/v.SOOS (2) can be achieved is strongly dependent on the soil type.

The influence of the special marginal conditions and criteria, determinating soil sampling in strong cohesive stiff soils shall be found out by further scientific investigations in the Institute of Foundation Engineering and Soil Mechanics of the LGA Bayern (Nuremberg).

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A.G. Anagnostopoulos, Panelist

It is well known that in granular soils it is difficult and in many cases almost impossible to obtain undisturbed samples. Therefore results from SPT tests, even characterized as having only a quantitive usefulness, are still widely used by practicing engineers, for engineering estimates in such soils.

For the SPT, despite its widespread use little quantitive information is available to the engineer, concerning all the variables which could possibly affect the outcome of SPT blow count.

VARIABLES AFFECTING SPT VALUES

1. Number of turns of the rope
2. Cathead speed
3. Free-fall distance
4. Rope age
5. Rope release
6. Striking plate

As a result of the above mentioned variables a different kinetic energy could be delivered to the rods during testing, and as Mori (1981) stated in his General Report, the N value varies inversely with input energy. In this respect, Kovacs et al, (1977) reported that the N value could vary by a factor of 2 or more at a given depth and relative density.

The most of the Standard Penetration Tests, as used according to Terzaghi-Peck, were made at about 2/3 of the theoretical kinetic energy for a free hammer fall. In order to do not upset a 40 years engineering correlation with SPT results, Kovacs (1979) suggested that modern hammer release devices shall deliver a kinetic energy equal to the one previously mentioned and by further standarizing the geometric shape of a trip monkey and anvil system, one "standard energy" would be delivered to the sampler regardless of the drill rig model, SPT equipment and operators. This recommendation has to be considered of importance not only for the standarization of the test but also to assist a more uniform interpretation of the test results.

One important factor for the interpretation of SPT test results, is the influence of the generated pore water pressure during driving the sampler. The effect of water table on N values is generally taken into account by a well-known correction $N' = 15+1/2(N-15)$, for $N > 15$, when the SPT is carried out in fine sands or silty sands.

Carrying out the SPT in loose sand deposits positive pore pressures are developed and consequently the shear strength and the N values decrease. The contrary takes place in dense deposits where the negative pore pressures reflect in an increase of the N-values (Schmertmann, 1975). In this case a correction

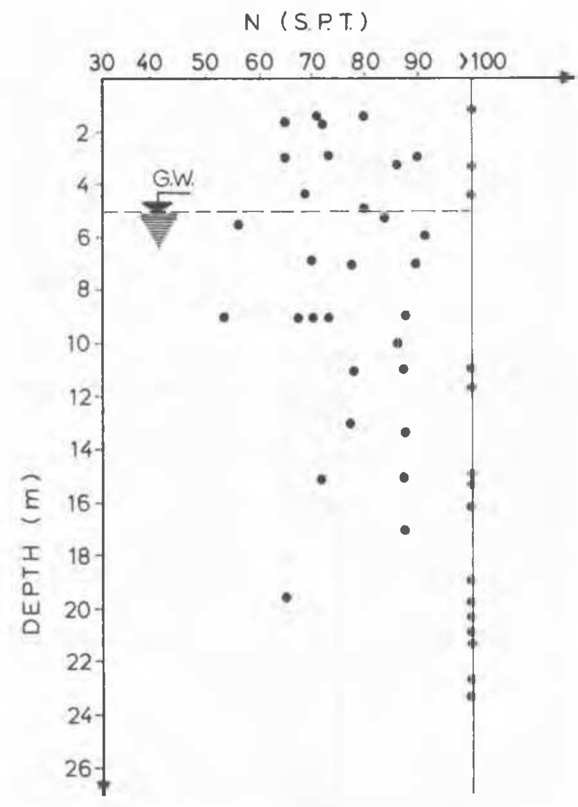

Fig. 1. N values "uncorrected" for the overburden as recorded in a sandy gravel

is desirable. However, it remains questionable whether correction of N values for medium dense deposits are necessary.

The above remarks are summarized in Table 1.

For coarse sands or sand and gravel deposits it is observed that the N values above water level turn out to be higher than in the case of groundwater been present. This could mean that

TABLE I

$N < 15$	$u > o$	Underestimation of N
$N = 15 \div 20$	u ?	"Correction" has no practical meaning
$20 \leqslant N < 30$	u ?	"Correction" is questionable
$N > 30$	$u < o$	Overestimation of N "Correction" is needed

the number of blows for tests carried out below groundwater in such soils, should be corrected, i.e. increased (e.g. see Fig.1)

In such soils the pore water pressures are of no predominance, as this is the case for silty sands or fine sands. The reason should be attributed to a decrease of the side friction on the walls of the split spoon sampler below groundwater, as also on decrease of the point resistance during penetration. Consequently it appears consistent to register lower N values.

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U. Bergdahl (Oral discussion)

REDUCTION OF N-VALUES FROM SPT-TESTS?

The penetration resistance at SPT-test or dynamic probing in fine sands and soils is often very high, especially in medium dense to dense soils. This is why e.g. Peck et al (1967) propose that the N-values from SPT-tests in such soils should be reduced before using the test results for design purposes. Similar reductions are normally not made of the dynamic probing test results in Sweden. However, many experiences show that such a reduction ought to be done. In Fig.1 is shown the results of a dynamic probing (HfA) and a static cone penetration test (CPT). From this last mentioned test it is evident that the point resistance is rather constant from about 4.5-12.0 m depth (between 10 and 12 MPa). However, the dynamic probing resistance in blows/0.2 m of penetration is increasing especially in the silty sands at about 10 m depth.

It has been suspected that these high dynamic resistances are due to changes in the pore water pressure at driving of the penetrometer. Similar experiences are obtained from pile driving in these types of soil in order to find out if it is negative or positive pore pressures that occur during the driving.

In Sweden we have made some investigations with measuring the dynamic pore pressure during driving a rod into a test cylinder filled with a compacted silty sand c.f. a paper by Möller and Bergdahl in session 8 of this conference: the pore pressure was measured at the point and at the shaft. The test results indicated that during each blow a negative pore pressure was generated along the mantle of the rod. At the tip of the rod there was at first during com-

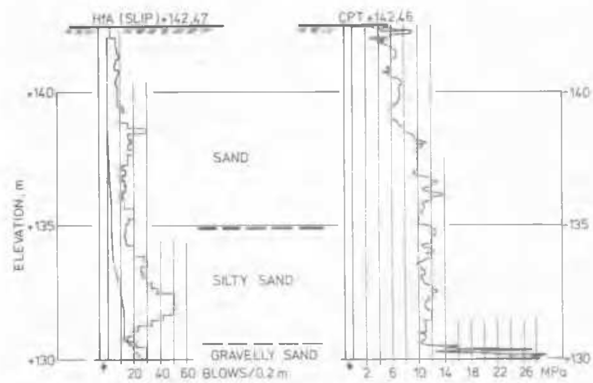

Fig.1 Results of dynamic probing and cone penetration test at Kolari in northern Sweden.

pression generated a positive pore pressure and at shearing of the soil the pore pressure became negative. This negative pore pressure along the mantle and at the point makes the soil sticking to the rod and thus increases the friction and the penetration resistance. This might be part of the explanation for the high blowcount in silty soils.

Also at CPT-test in gravelly soils very high q_c-values can occur due to a scale-effect also when the soil is loose, Fig.2.

The conclusion of this presentation is that the type of soil has to be considered when interpreting the test results from SPT as well as

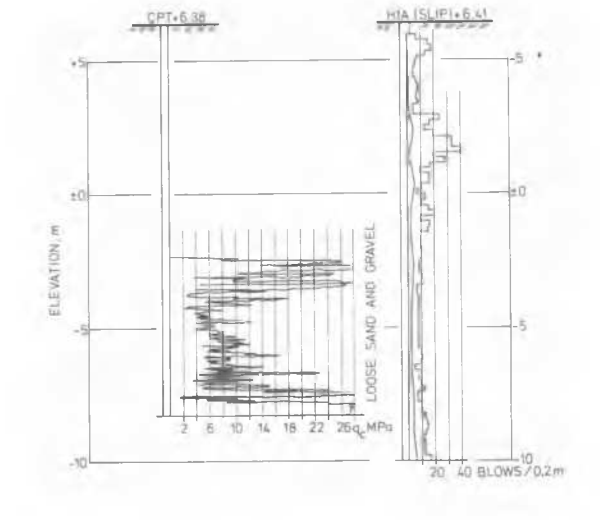

Fig.2 Results of dynamic probing and cone penetration test at Skanstull in Stockholm.

CPT tests. It is recommendable to reduce the N-values from SPT-tests and dymamic probing in fine sands and silty soils below the ground water table. For CPT-tests the minimum values have to be used for design purposes in gravelly soils. However, it is difficult to find out when such reductions have to be done. Therefore it can be recommended to perform both a static and a dynamic test or a combined static-dynamic test as proposed in a paper by Bergdahl and Möller to this session. In this way there will be no doubt about how properties of the soils are effecting the test results.

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M. Tammirinne, Panelist

SOME ASPECTS CONCERNING THE USE OF VANE TEST

Introduction

On basis of articles in this conference it seems the pressuremeter and especially self boring pressuremeter is coming a more and more popular method for determination of undrained shear strength of clays, too. In articles the vane shear test results are used mainly as basic results with which the results of other in situ test methods are compared. But we may not forget that the vane test is the basic method only because it is used so long time all over the world.

This article is based a study made by the Geotechnical Laboratory of VTT in Finland /1/. This investigation was aimed at interpretation of the factors which influence the reliability of the vane test results as well as their applicability to different conditions mainly for practical site investigation purposes and I think the same aspects concerns also for example the pressuremeter tests and testresults, of course applied to the method on question. The in situ measured strengths were compared with results from laboratory tests made on undisturbed samples. The vane test apparatus was equipped with casing tubes for rods and with vane socket during pushing down the equipment (model Geonor). The soils tested were clays and silts (late glacial and postglacial sediments).

Vane strength vs. rotation angle and speed of rotation

In vane torsion or rotation vs. shear strength curve we can usually observe three points of discontinuity (Fig. 1.)

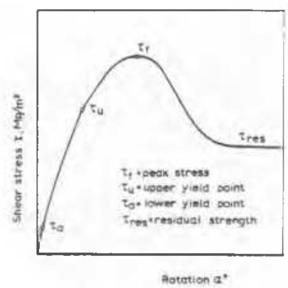

Fig.1.

After peak stress we can still measure the residual strength which is usually much lower than the peak strength but greater than the remolded strength of disturbed soil.

Usually the peak stress is taken as undrained shear strength but this strength needs not always represents the true shear strength of the soil because the peak stress measured also from vane test is affected by many different factors. One of the most influential factor is the speed of vane rotation.

The measured strength (peak stress) increases with an increase in the speed of rotation of the vane. The speed of rotation exerts a clear effect upon the strength values measured especially the youngest fine sediments, on Littorina clay. In other types of sediment the effect is not so clear. As a rule, the more coarsely grained the soil concerned the less marked the time effect. With very low speed it is not anymore question of undrained shear strength. The investigations however indicated that in site investigations for practical purposes there is no need to change the worldwide accepted rotating speed, 0.1^{o}/second (6 degrees/minit).

Some investigators have observed that the upper yield strength is independent of the speed of deformation in compression test. Perhaps the upper yield point also in vane test should be a truer shear strength of soil. The determination of this point is possible by continuous recording of stress and rotation angle.

Vane shape and size

In Fig. 2 it is shown the results from vane tests made with vanes of different shape and size (Fig. 3). The standard sizes of vanes used in the Nordic countries are 65 mm x 130 mm and 55 mm x 110 mm. The conclusions from the tests was that no reason exists for changing the shape of the standard vanes (H = 2D) currently in use. The use of high vanes (H/D > 3) is not

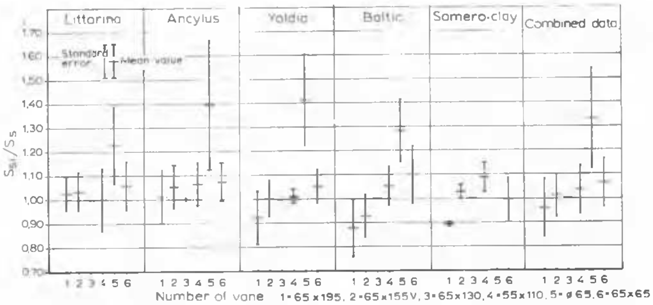

Fig.2.

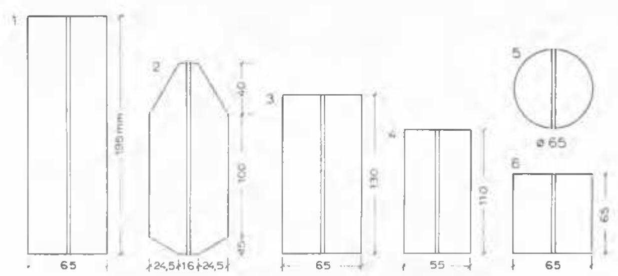

Fig.3.

to be recommended, since the soil layer may be subjected to progressive failure around the vane. The area ratio of the vane should be less than 15 % /4/. With vanes of different shape and size, at least three different sizes, it is possible to clarify the influence of the anisotropy of a soil layer to its shear strength.

Testing technique

The disturbance to the soil structure resulting from pushing the equipment into the ground is not wholly avoidable. Errors arising from the disturbance can be reduced by pushing the vane at a sufficient distance from its protecting socket /3/.

Measurement of the strength should not be made immediately after pressing the vane into the soil because of arised pore water pressure in the soil layer. Under too long delay the consolidation of the soil has certainly some effect to shear strength. In site investigations it would seem necessary in clays always to keep the delay about 3 minutes before starting the measurement. In silty soils the delay does not exert any notable influence upon the strength results.

The effect of the foremen upon the results can be eliminated by ensuring that the testing is always made in conformity with the same given instructions.

In this study a comparison was made between the strength values as measured with the moment key and with the moment gauge. The results obtained imply that the reliability and the serviceability of the moment key strength values are expressly dependent upon the measuring technique. The measured strength with the moment key were 10-20 percent higher as measured with moment gauge. The most important reason for this difference was high and uncontrolled speed of rotation of vane when using the moment key.

The use of vane test results

The vane test seems well suited for the determination of the undrained shear strength of saturated clays and fine silty soils, normally consolidated or slightly overconsolidated of low permeability; under Finnish conditions the strength of the homogeneous Littorina, Ancylus and Yoldia sediments.

However, the anisotropy of the layers need to be taken into account.

Not only the type of soil, but also its state of consolidation should be taken into consideration in the determination of shear strength by vane test. The state of consolidation in the case of fine grained sediments can be estimated approximately from the results of vane test and the plastic properties /2/.

The peak strength measured by field vane test is usually reduced according to Bjerrums method or some other method based on plasticity or liquid limit of clays /2/. Instead of use of reduction factor it is possible and preferable to use a higher factor of safety in the calculations based on peak strength.

When correlating the vane test shear strength (s_s) and the strength values measured with the triaxial compression test (s_u) and with the cone test it has found an appreciably close correspondense in youngest sediments (Fig. 4 and 5). In peaty soil use of vane test is limited primarily to mouldered layers.

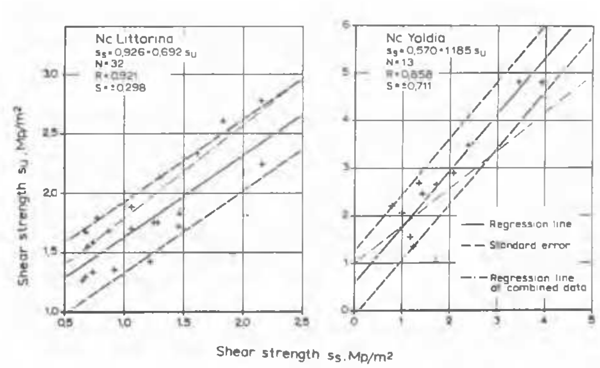

Fig.4.

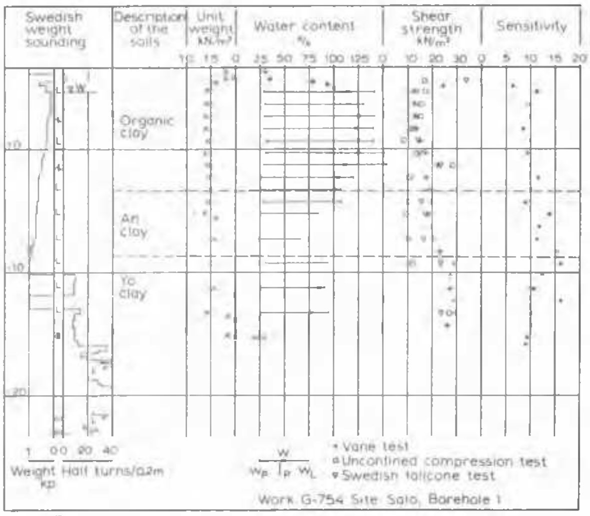

Fig.5.

The vane test cannot be employed for the determination of shear strength parameters of coarse cohesive soils (coarse silt) or of dry crust. In these soils the use of pressuremeters as a "direct" measuring method and the cone penetration test (CPT) as a "indirect" method is preferred.

Conclusions

The vane test is very widely used and very well suited for determination of undrained shear strength of cohesive soils because of its simplicity and low costs. With vane tests we can get easily the whole profile of in-situ strength of soil-layers. Of course, there are difficulties associated with the interpretation of test results; the strength and consolidation state of soils, as it is with other methods,too. It is recommended to take undisturbed samples and make strength-tests in laboratory to ensure the "absolute strength values" of vane test results.

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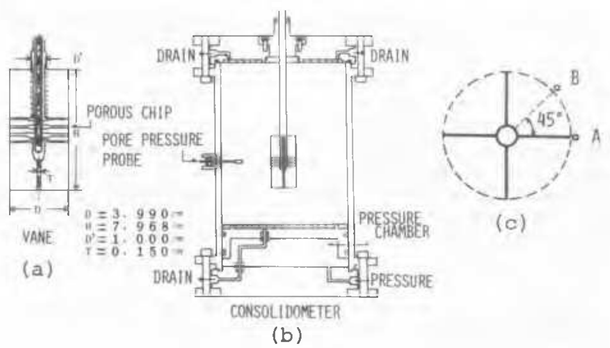
T. Kimura and K. Saitoh.(Oral discussion)

ON THE INTERPRETATION OF STRENGTH MEASURED BY LABORATORY VANE TESTS

The vane shear test was considered to be a useful and innovated technique to measure the insitu undrained strength of saturated clays when it was first proposed. Many workers, however, have cast doubt if the vane test could really yield the true strength. Bjerrum(1977) investigated the vane shear test results from various sites and proposed a correction factor for the vane strength. Kirkpatrick et al.(1981) studied the correction factor by comparing the vane strength with the strength obtained with triaxial tests, and reached a conclusion which contradicted the one by Bjerrum. In spite of many works in this line, ambiguity still remains on the interpretation of the vane test. In this short report the Authors present the behaviour of porewater pressure observed during laboratory vane tests in an attempt to reveal the true features of the vane shear test.

The size of a vane used in the Authors' tests is shown in Fig.1(a). The tests were carried out on two types of cohesive soils; Kawasaki clay with plasticity index(I_p) equal to 48 and a mixture of Kawasaki clay with crushed Toyoura sand with Ip equal to 20. Fully saturated and deaired slurry was poured into a consolidometer with 20cm in diameter(Fig.1(b)). The preliminary consolidation was coducted under the vertical pressure of 78KN /m2 and after the completion two Druck transducers with 6mm in diameter and 13mm in length were inserted from the side of the consolidometer



(a) Vane blade and the size
(b) Consolidometer,vane blade and transducer
(c) Location of transducer

Fig.1

keeping the consolidation pressure on. The attempt was made to locate the transducers as close as possible to the future vertical shear plane to be caused by the vane blade(Fig.1(c)). The consolidation pressure was then increased to the final value of $98KN/m^2$. Subsequently a seal on the top of the consolidometer was broken and the vane was slowly pushed into soil. Porewater pressure was monitored all way through this process with the transducer A(Fig.1(c)).

The observed behaviour of the porewater pressure is given in Fig.2. It can be seen that strikingly high pressure is built up during the insertion of the vane and that the dissipation needs considerable time. The rotation of the vane was carried out varying the time after the completion of the insertion, elapsed time(t_c), as 4.5, 10.0,

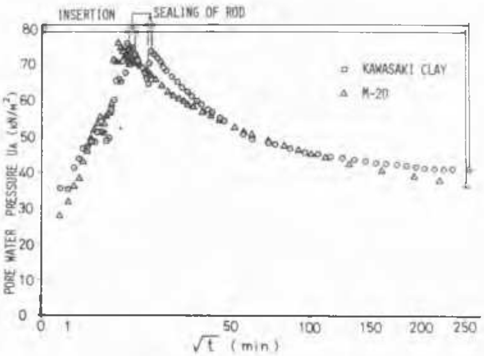
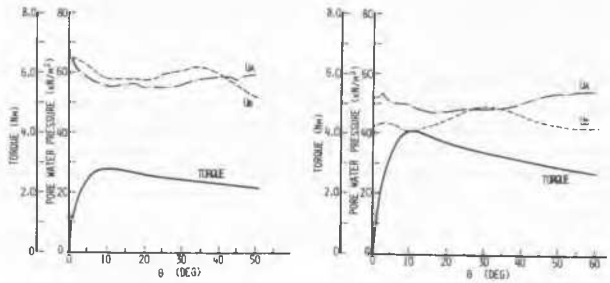


Fig.2 The variation of porewater pressure during and after the insertion of vane



(a) t_c=4.5mins. (b) t_c=60mins.

Fig.3 Variation of torque and porewater pressure during rotation of vane

30.0, 60.0 and 240 minutes. The variation of torque and porewater pressure with the rotation of the vane is shown in Fig.3. It is interesting that the porewater pressure does not vary very much during this process. The ratio of the measured vane strength(s_u) with different elapsed time to the vertical consolidation pressure(p) is plotted against I_p in Fig.4. The triaxial strength for the same materials is also given.

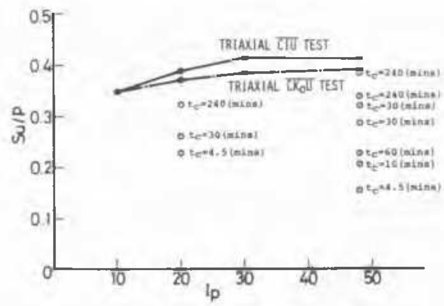


Fig.4 The ratio of undrained strength to consolidation pressure versus plasticity index

This leads to important conclusions that the relationship between s_u/p and I_p is not unique but highly dependent upon the elapsed time and that the ratio s_u/p for the vane test with sufficient elapsed time approaches that for triaxial tests.

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H.L. Jessberger and R. Dörr (Written discussion)

RAM SOUNDINGS AND VANE SHEAR TESTS IN ANTARCTIC ICESHELVES

Vane shear tests combined with ram sounding used in soil mechanics have been adapted to shelfice to determine the short-time strength properties of snow and ice. Such investigations were performed in connection with the construction of the G.v. Neumayer-Station on the Ekstrøm Iceshelf in the Antarctica during the German Antarctic Expeditions1979/80 and 1980/81. The vane shear test device has vane dimensions of h/d = 7,12/3,56 cm. The sounding rod was driven into the ice by a 3 kg-mass hammer with a falling height of 30 cm (Fig. 1)

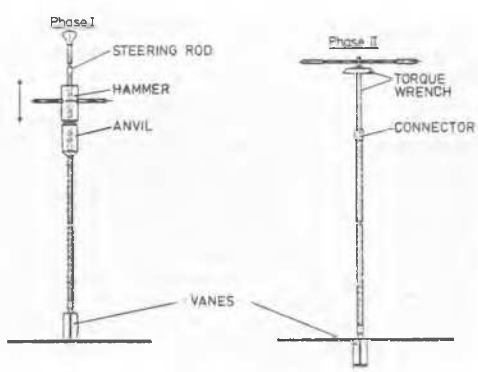

Fig. 1: Ram sounding and vane shear test device

The development of the device for ice and snow was based on the idea that the vanes penetrate into the snow by knife-edges. This avoids mainly compactions of the surrounding snow leading to higher densities and changing the mechanical properties. The number of hammer drops (n_{10}) for the penetration of each 10 cm was determined and the shear stress was calculated from the torque necessary to switch the vane. The results of two tests at different sites (Gold Bay on the Filchner-Ronne-Iceshelf and G.v. Neumayer-Station on the Ekstrøm Iceshelf) shows Fig. 2. The evident different behaviour results from different density distribution.

The comparison between shear strength τ and number of hammer drops n_{10} shows nearly linear relation expressed by the equation

$$\tau = a \cdot n_{10} + b$$

A regression analysis of all soundings independent from the site scatters as shown in Fig. 3

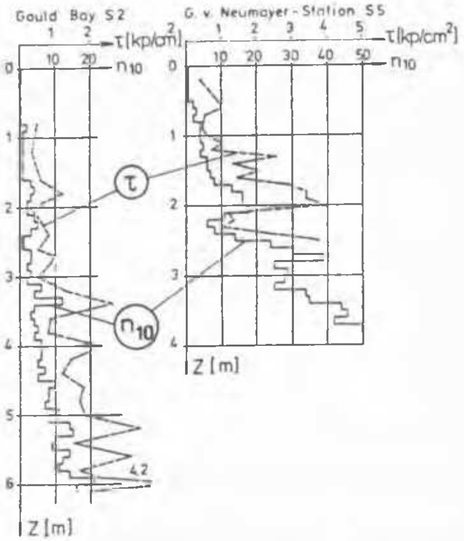

Fig. 2: Results of soundings: shear stress (τ) and number of hammer-drops (n_{10}) versus depth

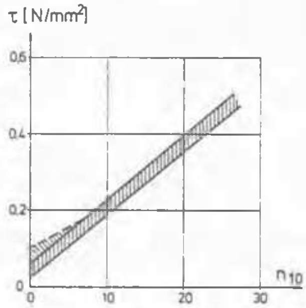

Fig. 3: Scatter of relation between τ and n_{10} from all tests

The derivation in the range of low numbers of hammer drops results from inaccuracies reading low torques in connection with low numbers of hammer-drops near the snow surface.

The shear stresses determined in these tests cover well data published by Mellor /1/ (Fig. 4). Using empirical relations also resonable values of Youngs modulus of elasticity can be found (/2/, /3/).

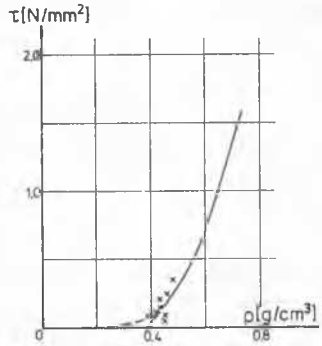

Fig. 4: Shear strength versus density /1/. x values from vane shear tests

From these investigations with the vane shear test combined with the penetration resistance of ram soundings the properties of snow and ice may be deduced well for practical applications. On the other site this combined test method is convenient for application in soft soils.

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F. Tavenas, Panelist

CONCLUSION OF THE PANEL DISCUSSION ON THE RELEVANCE OF SHEAR STRENGTH FROM IN SITU TESTS IN DESIGNS

The discuesion on the relevance of the vane shear strength or of any other test result must be put in a more general perspective, which I would like to point out here.

The first, and most fundamental principle, is that a natural clay has no unique shear strength (Ladd 1967). Rather as pointed out by Tavenas & Leroueil (1979), the shear strength, as well as any other mechanical properties are effective stress path dependent. Consequently, the result of a given in situ or laboratory test will be directly relevant to the design of a prototype only in the case where the effective stress paths in the test and the prototype are identical. The application of this principle has a direct influence on the planning of soils investigations.

Ideally, in deciding on what type of test to use in a given soils investigation we should first establish the type of effective stress path involved in the prototype to design, and then we should select a test in which this stress path is reproduced. Unfortunately we are generally very far from this ideal situation. The number of cases where we know the effective stress paths involved in the prototype is very limited, and our ability to reproduce these stress paths in our routine in situ or laboratory tests is even more limited. Indeed the only case where we can reasonably apply the ideal approach is when designing piles submitted to horizontal loading on the basis of pressuremeter test results.

In the most general case we must develop soils investigations for prototypes involving unknown, effective stress paths, or stress paths of such complexity that they cannot be reproduced in our test. Then we know that the parameters measured in the tests will be different from those governing the behaviour of the prototype. Consequently, we will have to make use of an empirical approach to adjust the test results to fit the observed behaviour of the prototype. The requirements as far as the test methods are concerned are rather simple in this circumstance: the test has got to be simple, reproducible, economical .. and above all, well calibrated against full scale field observations. A number of in situ tests satisfy these requirements for a variety of design problems: the static cone penetration test for the design of piles in sands or the pressuremeter for the analysis of settlements of footings.

For the analysis of the stability of clay foundations, the field vane is, at present, the test which best satisfies the above stated requirements: it is certainly simple and economical. It could be improved in terms of reproducibility by a rigorous standardization of the equipment and procedure. Its results are well calibrated against field performance (Bjerrum 1973; Helenelund 1977) but there is still a need for further improvement of the empirical vane correction factors (Larsson 1980). Therefore, we could be perfectly satisfied with the field vane test, in developing a good design practice for the stability of clay foundations.

On the other hand we have seen the introduction of quite a variety of in situ tests to investigate that problem:screw plate, pressuremeter and selfboring pressuremeter, static cone and piezocone, borehole shear device. In assessing and using these tests, the above stated principle and requirements fully apply. First we should expect each test to give its specific value of the shear strength of the clay, corresponding to the specific effective stress path followed in the test. Second, given the time, expense, and problems associated with the development of good empirical correlations between a new test's results and the field performance of prototypes, and considering the amount of test-prototype correlations already available with the field vane, it will probably be best to first correlate the new test's results with field vane strength before using them in design practice. Finally, great emphasis should be put on simplicity and reproducibility of the new tests rather than on sophistication since the test results will have to be adjusted empirically anyhow.

The limitations put on the development and use of new in situ testing techniques do not really affect the usefulness of some if not all newer tests. In using in situ tests other than the field vane for the investigation of clay deposits we can first observe features not detected by the vane such as detailed stratification by means of the piezocone or in situ horizontal stresses and deformability by means of the selfboring pressuremeter. In addition, given the unavoidable limitations of any one test, we absolutely need multiple profiling to detect erroneous test results or abnormal soil behaviour, as pointed out by our chairman Mr de Mello and by Mr Mori earlier in this panel discussion.

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F. Tavenas, Panelist

ADVANTAGES AND LIMITATIONS OF THE PIEZOCONE (CUPT)

The static cone penetration test has been used successfully for a few decades in the investigation of soil deposits. In recent years this test has been further developed by combining the measurement of the point resistance q_c with that of the pore pressure u generated during the cone penetration. The pore pressure sounding was originally made with a separate probe (Wissa et al. 1975, Torstensson 1975), but combined piezocone probes are now available (Roy et al. 1980, De Ruiter 1981) which make the CUPT an attractive and powerful in situ investigation tool.

1- Principle of the piezocone test

The piezocone consists of a standard cone with a 10 cm^2, 60° conical point fitted with a pore pressure sensor on or just above the conical tip. Both electrical tips (De Ruiter 1981) and hydraulic tips (Parez 1974) may be transformed into piezocones while the mechanical cones are not suitable for such use. With the most common electrical probe, the cone is fixed on a force transducer to provide a continuous record of the point resistance q_c with depth (De Ruiter 1971, 1981). The pore pressure sensor consists of an annular porous element connecting the pore water in the soil around the probe with a very small chamber fitted with an electrical pore pressure transducer inside the cone tip. Investigations by Torstensson (1975) et Roy et al. (1980) have shown that the magnitude of the measured pore pressure depends both on the shape of the tip and on the position of the porous element on the tip or the shaft of the probe. Non standard CPT shapes such as that developed by Torstensson (1975) may have some advantages but

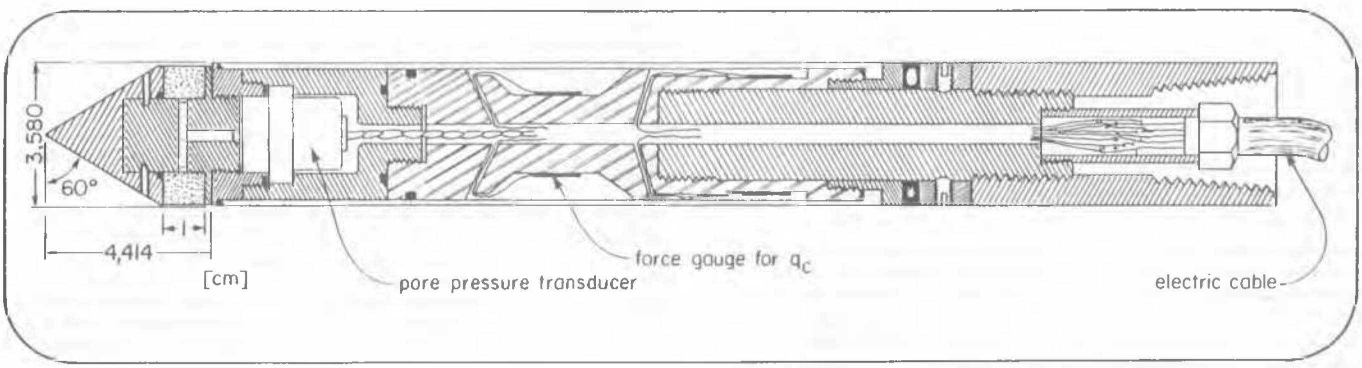

Fig.1—Modified Fugro point used as piezocone at Laval University.

these are more than offset by the need to perform two separate soundings. As for the location of the sensor on the probe, no definite conclusion has been reached, but preference seems to be given to standard CPT probes with a porous element placed in the upper half of the conical tip (De Ruiter 1981) or immediately above the tip (Roy et al. 1980). Figure 1 presents a section of the modified Fugro cone used for a number of years at Laval University.

The piezocone is pushed vertically into the ground at a constant rate of 2 cm/s and q_c and u are continuously recorded as a function of the depth of penetration. To ensure reliable measurements both the force transducer and the pore pressure transducer must be calibrated frequently, preferably under temperature and working conditions similar to those prevailing in the ground (Campanella and Robertson 1981); extreme care should be taken in de-airing the pore pressure sensor (Battaglio et al. 1981). In addition to the determination of continuous q_c and u profiles, the penetration of the probe can be stopped at any depth to observe the rate of dissipation of the excess pore pressure generated during penetration as well as the final equilibrium pore pressure. This latter measurement allows the determination of the in situ groundwater regime or the observation of the present pore pressure isochrone in a consolidating foundation, for example under an embankment (Battaglio et al. 1981).

2- Advantages of the CUPT

a- Determination of stratigraphy

The CUPT provides continuous profiles of the point resistance q_c and the driving pore pressure u. Theories as well as experience have shown that q_c and u are functions of the soil's type, strength and deformability (Vesic 1972). In addition, u is also a function of the soil's permeability. Therefore, all significant engineering properties of the soils being penetrated are indirectly assessed during a CUPT, so that the test provides a detailed picture of the stratigraphy of the site.

The CUPT is also a fast and repetitive test, much more so than any other in situ test; for example a CUPT profile can be obtained in about 1/3 of the time necessary for a field vane profile to the same depth. Therefore, CUPT may be economically performed at reduced spacing to check the homogeneity and the continuity of soil strata.

These advantages may be illustrated by the examples presented in figures 2 and 3.

At the site of an important road embankment in Louiseville Québec, four CUPT to about 32 m depths gave the $q_c = f(z)$ and $u = f(z)$ profiles shown in figure 2. The low q_c and high u values indicate the clayey nature of the deposit. The linear increase with depth of both q_c and u suggest that the clay deposit is free of any significant change in clay properties, but for those due to the increasing effective overburden pressure. The very small scatter indicates the uniformity of properties along the embankment axis. The remarkable homogeneity of this lightly overconsolidated clay deposit was confirmed by high quality undisturbed sampling. From a practical point of view, such results indicated that a reliable settlement analysis could be performed, by making only one borehole for undisturbed sampling and consolidation testing, at the most convenient location and by assuming drainage boundary conditions only at ground surface and at the base of the clay layer.

At another embankment site in Batiscan, Québec, three CUPT to about 30 m depth produced the $q_c = f(z)$ and $u = f(z)$ profiles shown in figure 3. They clearly indicate the stratified nature of the deposit. In the upper few meters a sand layer is evidenced by the high q_c values and driving pore pressures equal to the hydrostatic pressure under the water table at El. 26 m. Below an elevation varying from 24.3 to 23.0 a clay deposit is encountered. Its strength increases linearly with depth down to elevation 6 m, as indicated by the linear and proportional increments in q_c and u. However, the strength varies horizontally, being higher at the southern end of the site. At elevation 16 m at station 18+00, and at elevation 11.5 m at station 19+40, thin sand lenses are encountered, as indicated by the high q_c and the reduced u; their limited extent is evidenced by the fact that they occur in only one sounding in each case. On the other hand, between elevation 6.2 m and 5.0 m a series of continuous sand layers is encountered in all soundings. Finally, below elevation 2 m, another clay deposit is penetrated, the strength of which must be different from that of the upper clay layer as evidenced by offsets in the $q_c = f(z)$ and $u = f(z)$ profiles. Based on these results, it was concluded that laboratory consolidation tests had to be carried out at various locations under the embankment axis and on both the upper and lower clay layers to properly assess the variations in σ'_p and other consolidation properties. It was also decided that only the sand layers between elevation 6.2 and 5 m were significant enough to act as drainage boundary conditions.

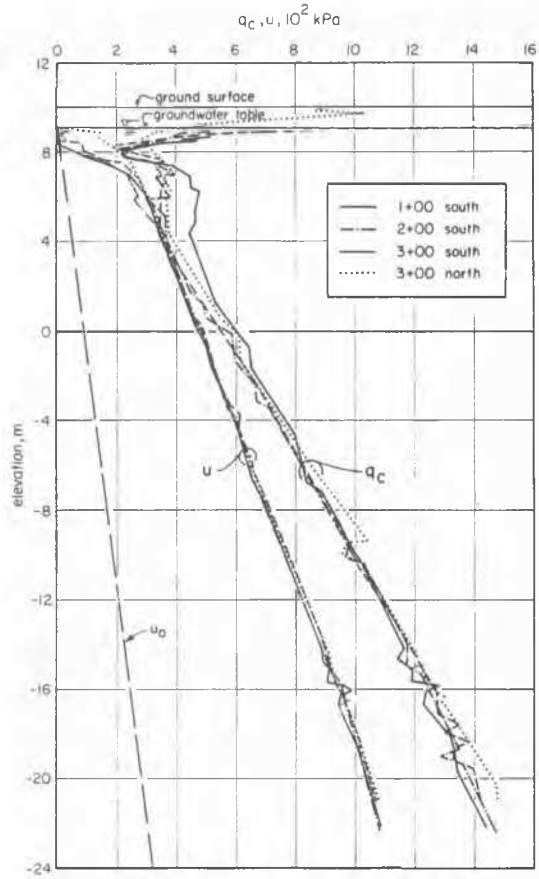

Fig 2— Piezocône profiles at Louiseville

These two examples illustrate the usefulness of the CUPT in helping to design an efficient undisturbed sampling program, as well as to define the drainage boundary conditions for a consolidation analysis. As for the thickness of draining layers which may be detected, it depends on the height of the porous element: experience with the probe shown in figure 1 shows that layers as thin as 5 mm may be easily identified.

Besides this qualitative use of the CUPT, the test results may be used to measure some of the soils parameters, in both clays and sands.

b- Strength evaluation in clays

In clays, various theories or empirical approaches have been used to develop a relationship between q_c and the

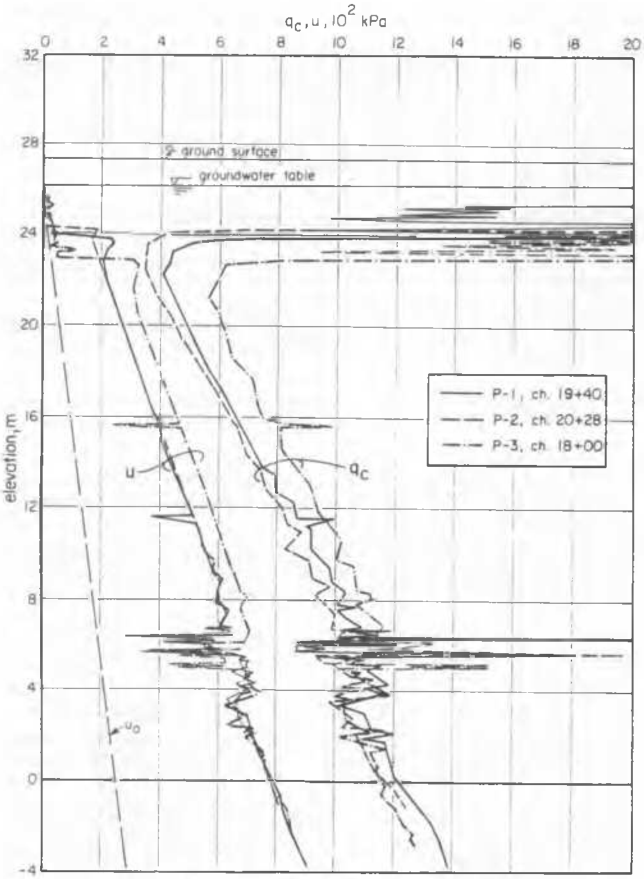

Fig 3 — Piezocône profiles at Batiscan

undrained shear strength measured by field vane c_{uv}:

$$q_c = N_k c_{uv} + p_o$$

The factor N_k is most probably a function of the entire stress strain behaviour of the clay (Roy et al. 1974). It is therefore variable from site to site, and, as noted by Eide (1974) or Lunne et al. (1976), typically in the order of:

$$8 < N_k < 20$$

Similarly the excess pore pressure in the CUPT may be related to c_{uv} by:

$$\Delta u = N_{\Delta u} c_{uv}$$

where $N_{\Delta u}$ is a factor similar in nature to N_k, i.e. variable from site to site as well as with the geometry of the probe. Typically, for the cone shown in Figure 1, it is in the order of:

$$6 < N_{\Delta u} < 10$$

However, both N_k and $N_{\Delta u}$ should be essentially constant for a given clay deposit. Consequently, on large soils investigations a great economic and technical advantage can be obtained by reducing the number of vane profiles to the minimum necessary to define the local N_k and $N_{\Delta u}$ values, and by making a large use of the CUPT which is faster and more reproducible than the field vane, and which gives a continuous c_u profile as well as other indications on the stratigraphy.

Also, recent work (Tavenas & Leroueil 1979 ; Baligh et al. 1980) has suggested that q_c and u/q_c could possibly be related to the stress history of the clay deposit, i.e. to σ'_p and the OCR. Such relationships appear promissing but they need to be checked on a variety of clay types.

c- Deformability and strength evaluation in sands

In granular soils, Schmertmann (1970) has evidenced the relationship between q_c and the modulus of deformability E:

$$E = 2\, q_c$$

Other authors suggest that:

$$1.5\, q_c < E < 4\, q_c$$

while Janbu (1974) points out the possibility of a non unique E - q_c relationship. In addition, it has been shown that both q_c and u may be related to the state of density and stress in sand. The CUPT could thus be a useful tool in assessing the liquefaction potential of sand deposits (Schmertmann 1978). Finally relationships between q_c and ϕ have been tentatively suggested, as summarized by Mitchell & Lunne (1978); however such use of CPT results does not seem to have found a very wide acceptance in practice.

3- Limitations of the CUPT

a- Equipment and test performance

The most serious limitation in the development of the use of the CUPT in practice is probably the high initial cost of the equipment: a fully equipped piezocone costs about 10 times as much as a fully equipped field vane. It is not surprising then, that the commercial availability of the CUPT is rather limited outside Europe.

In performing the test, CUPT are less sensitive to the operator and therefore more reliable than any other in situ test. However, good test results can be obtained only if the probe is calibrated frequently under actual field working conditions and if extreme care is taken in de-airing the pore pressure sensor and maintaining it saturated during handling. Further practical problems in the performance of CUPT are covered by De Ruiter (1981).

b- Interpretation in terms of soil strength

More serious limitations, not of the CUPT itself, but of the methods suggested for its interpretation should be acknowledged. As most in situ tests, the CUPT is difficult to interprete by theory. The major problems in such interpretation may be stated as follows.

As shown by Roy et al. (1974) the actual cavity expanded in the soil by the penetrating cone is neither spherical nor cylindrical. Therefore, the usual cavity expansion theories may give boundaries to the problem,but certainly not a correct solution.

Another major problem is that the effective stress paths in the soil around the probe are unknown and certainly variable from point to point. Since the soil behaviour is effective stress path dependent (Tavenas & Leroueil 1979),it implies that a variety of moduli and strengths should be operative around a penetrating probe.

Finally, the available models for describing the soil's behaviour in the theoretical interpretation of the CUPT are greatly oversimplified. They ignore the effects of anisotropy, strain softening, viscosity, and most generally the effective stress path dependence of deformability and strength of the clays. The use of unique values of E_u and c_u in the present theoretical methods for interpreting q_c and Δu should make any user suspicious of the quality of such interpretations.

As a result of the above stated limitations q_c and Δu are best correlated to c_{uv} directly on each site to define the local values of N_k and $N_{\Delta u}$ instead of using questionable theoretical solutions for N_k and $N_{\Delta u}$. This implies that the CUPT must be used in parallel with,rather than instead of, the field vane. The other advantages of the CUPT certainly greatly offset this limitation.

c- Interpretation in terms of consolidation properties

In recent years, a great emphasis has been put in the literature on the use of the pore pressure dissipation observed in interrupted CUPT to evaluate the coefficient of horizontal consolidation c_{vh}. The theories developed to this en by Torstensson (1977), Randolph & Wroth (1979), are subject to the limitations already stated since they refer to cavity expansion solutions for the initial stress and pore pressure conditions. In addition, they make use of a series of questionable assumptions in the solution of the consolidation problem.

It should first be realized that c_{vh} is not a soil's property but a behaviour, the basic properties being the permeability and the compressibility. c_{vh} cannot possibly be a constant as implied in all theories presently available. It so happens that the permeability, the compressibility and the resulting c_{vh} all vary with location and with time as a result of the locally variable remoulding of the clay around the penetrating probe, and as a result of the consolidation process. None of the theories can account for such variations which may involve orders of magnitude for c_{vh}.

As for the clay behaviour during pore pressure dissipation it is extremely complex and far from the usual assumptions. Insight into this problem may be gained from field observations around piles. First, consolidation and swelling occur simultaneously at different locations around the probe, and in relative amounts which vary with time. Contrary to usual consolidation problems, both the total and effective stresses are variable with time; the boundary conditions are also variable. Finally, vertical consolidation developes during the radial pore pressure dissipation.These phenomena have not been properly considered in the existing theories.

When all these problems are considered one has to be very skeptical about the c_{vh} values determined from such theories as those presented by Torstensson (1977), Randolph & Wroth (1979) or Baligh & Levadoux (1980). Such c_{vh} values are, at best, an educated guess. Before using them in practice there is a great need for serious checks against other measurements and mainly against the field performance of full scale structures. In the absence of such checks, and in keeping with the empirical approach to the q_c - c_{uv} relation, we would be well advised to develop empirical correlations between pore pressure dissipation data and the horizontal permeability for a variety of clay deposits.

4- Conclusion

The introduction of the CUPT represents a great progress in our ability to investigate soil deposits. It provides the geotechnical engineer with a unique tool for defining the detailed stratigraphy of a site and thus performing more efficient soils investigations and more reliable designs. It is also a good complement to the field vane by giving a continuous strength profile in much less time and with greater reproducibility than the field vane.

The main limitation of the CUPT, i.e. the high cost of the equipment, is partially offset by the higher productivity, at least on large projects. The other limitations are related to the theories presently used to interpret the CUPT results. These limitations need not to be, since the test can be properly and fully exploited without such theories, by resorting to the combined use of CUPT, field vane and other in situ tests, and to empirical correlations between the results of these various tests. Indeed greater emphasis should presently be put in developing such correlations which will permit the efficient use of multiple profiling techniques, rather than in elaborating simplistic theories in the vain attempt to have all answers to a soils investigation problem from a single test.

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M. Jamiolkowski and R. Lancelotta, Panelist

ON PIEZOMETER PROBE AND PIEZOMETER CONE TESTS

Panelist Prof. Tavenas made a very clear presentation of some newly developped in situ tests which consist in the steady penetration of some kind of conical tip with the simultaneous measurement of the generated excess pore pressure, Δu (Piezometer Probe = PP) or both Δu and cone resistance, q_c (Piezo-cone = PZC).

On the basis of their experience gained using the PP in soft clays in Italy [Battaglio et al. 1981, Ghionna et al. 1978], the writers wish to add a few remarks to Prof. Tavenas' introduction.

A) The following is a list of some of the recognized aims of the PP and PZC:

1. The pore pressure excess [Δu] during penetration is a useful indication of soil type (detection of soil macrofabric).

2. The simultaneous measurement of Δu and cone resistance [q_c] provides a good index [$\Delta u/q_c$] of soil type, relative consistency and a rough indication of stress-history.

3. The equilibrium pore pressure [u_0] provides important quantitative data to assess ground water conditions.

4. Approximate values of the coefficient of consolidation [c] can be obtained from the analysis of dissipation tests.

Prof. Tavenas appears to be rather sceptical on the possibility to obtain reliable information on the coefficient of consolidation, c, from the observed dissipation of Δu after the process of steady penetration of the PP or PZC has been stopped. This view is at least partly justified because of the extreme complexity of the phenomena which occur around the penetrating tip [Levadoux and Baligh, 1980; Baligh and Levadoux 1980)].
As a consequence the presently used interpretation approaches [Torstensson (1975), Ghionna et al. (1978), Lacasse et al. (1978)] oversimplify to a large extent the consolidation phenomena around the tip.
However, in the writers' opinion, this situation may be largely improved if further research efforts are directed towards the following aspects of the problem:

1) The improvement of our ability to predict by theory the initial value and distribution of Δu. Existing theories simplify rather too much the problem, but nonetheless lead sometimes to a reasonable prediction (see Table 1), at least as far as the situation at the tip is concerned.

2) The definition of the kind of "average" effective stress path the soil follows near the tip during the consolidation process.

3) The importance of uncoupled versus coupled consolidation theory in the interpretation of the dissipation tests.

4) The difficulties encountered in some tests when assessing the initial Δu because of the observed increase in Δu for a short time after the penetration has been stopped.
Is this phenomenon due to the Mandel-Cryer effect or insufficient de-airing of the tip?

On the other hand, if one keeps in mind the well-known difficulties in assessing reliable values of c for natural clay deposits with the presently available simplified interpretation methods, one must acknowledoe that dissipation tests furnish valuable information on the relative value of c and its variation with depth.

TABLE 1

Porto Tolle Silty Clay

measured versus predicted values of the initial Δu

SOIL MODEL	DSS $\Delta u / \bar{\sigma}_{vo}$	SBP $\Delta u / \bar{\sigma}_{vo}$
Elastic perfectly plastic	1.22	1.67
Modified Cam-Clay	1.48	1.89
Henkel's Relat.	1.52	2.01

Average measured value of $\frac{\Delta u}{\bar{\sigma}_{vo}} \cong 1.5$

DSS = soil parameters from CK_0U direct simple shear tests

SBP = soil parameters from quick tests with the self-boring pressuremeter

$\bar{\sigma}_{vo}$ = effective overburden stress

B) A different problem which must be pointed out and which merits further investigation concerns the optimum position of the porous stone on the tip and the geometry of the tip. The points below summarize the writers' experience in this matter and also describe the results of a short review of the available literature.

1) Soil profiling: A porous stone located on the tip will lead to the highest value of Δu when combined with a sharp pointed tip having an acute apex angle (15° to 20°). Such a combination assures a high sensitivity for the detection of soil layering.

2) Consolidation properties: A porous stone located far above the tip makes the theoretical interpretation of results easier, since conditions of one-dimensional flow around an expanded cylindrical cavity are approached. The optimum tip geometry in this respect has not been identified, minimum soil straining and disturbance have to be looked for.

3) Soil index and stress history: Since it is the ratio $\Delta u/q_c$ which is of interest, it is of advantage to use the standard cylindrical CPT tip, with the porous stone located just behind the tip.

In conclusion, it may be observed that tip geometry and the position of the porous stone strongly depend on the scope of the test carried out [Torstensson (1975, 1978), Vivatrat (1978), Baligh and al. (1980), Baligh and Levadoux (1980)].

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J.L. Davidson (Written discussion)

PORE PRESSURE PREDICTION AROUND A CONE PENETROMETER TIP
Prediction des Pressions Interstielles Autour d'une Point à Cone

In Session Number 7 it was suggested by more than one speaker that the initial excess pore water pressure distribution generated during penetration of a Cone Penetration Test tip or piezometer probe could be determined using cavity expansion theory.

In research performed at the University of Florida it has been found from model testing, large scale laboratory testing and field testing, that the initial pore pressure distribution around a cone tip is complex and can exhibit features which would not be predicted by simple cylindrical or conical cavity expansion.

Model testing was performed in which a solid steel probe was penetrated into dry sands of different densities and the resulting density variations around the tip determined using a technique of stereo photography (Davidson 1980, Davidson et. al. 1981). Figures 7 and 8 from the latter paper, presented to this conference, illustrate the resulting complex density regime. Of particular interest is the unexpected zone of loosening along the probe just above the tip in loose sand where densification in all areas might have been expected. For penetration in a loose saturated sand this would translate into a zone of negative pore water pressure above the tip in an otherwise positive pore pressure field.

Schmertmann (1974), while investigating a sensitive organic clayey sand at a site in St Petersburg Florida, performed a series of penetration tests with a Geonor vibrating wire piezometer. This probe had a porous bronze sensing element located 1.5 to 3.5 cm above the base of the cone point. In all tests the cone registered a negative or only a small positive excess pore pressure. On stopping penetration this value rapidly increased, reaching a peak positive value in approximately five minutes and then decreased. Schmertmann concluded that, upon ceasing penetration, the relatively small negative pore pressure zone around the shaft was rapidly overwhelmed by the larger positive zone below the base of the cone.

A series of large scale laboratory penetration tests has been performed in the University of Florida calibration chamber on saturated sands of different density (Gupta 1980). Generation of excess pore water pressure during penetration and dissipation upon stopping were recorded at the probe tip and by several small transducers located throughout the sand deposit. It was not possible to locate a transducer close enough to the line of penetration of the probe to actually measure the negative pore pressure zone noted above. However other readings have indicated its presence.

CONCLUSION

The excess pore pressure regime generated around a penetrating probe appears to be quite complicated, much more so than is predicted by current cavity expansion theory. The distribution depends upon tip geometry, method, rate and depth of penetration as well as on soil type and state. Since effective stresses and q_c also depend on these variables it would seem that more detailed research in this area is required.

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R.K. Bhandari (Written discussion)

RADAR IN GEOTECHNICAL ENGINEERING

Successful Radar (Electromagnetic) probings of the polar ice-sheets in Greenland, Ellesmere Island, Novaya Zemlya, and in Antarctica at frequencies ranging from 30-440 MHz (Gudmandsen, 1971) or of soils in the frequency range of 80-900 MHz (Morey 1974; Bjelm 1981) are two outstanding examples of the enormous potential Georadar holds as a technique of exploration. Aboard an aeroplane, space vehicle, baloon or rocket etc, Radar has found applications in locating sewer lines and buried cables (Morey & Harrington, 1972); examining sub-surface of moon (Porcello, 1974); measuring thickness of sea-ice (Campbell and Organge, 1974); studying structure of sea-waves over large areas of the ocean (Crombie, 1971); evaluating condition of air field pavements (Bertram, Morey and Sandler, 1974); locating ice in perma-frost (Bertram, Campbell & Sandler, 1972); profiling bottoms of Lakes and river (Morey, 1974); studying avalanches; measuring thicknesses of snow bridges over crevasses and searching the avalanche victims (Fritzsche, 1979). The technique cannot only be used for measurement of electrical parameters of earth (Moore & Williams,1957) for surveying of subsoil or searching the raw material (in particular, water) in vast, unknown tracts of terrain with speed and accuracy but may also arm the engineer with an innovative approach to locating fault zones or sliding surfaces in landslides, broken zone around tunnels and the like.

THE PRINCIPLE

The place of Radar frequencies in the electromagnetic spectrum is shown in Fig 1.

A beam of electromagnetic wave energy is transmitted by Radar in the form of periodic pulses of very high power but very short duration. Objects reflect some of this energy back to transmitter. The time delay of a returned echo is a measure of the distance to the reflecting object (Fig 2). The direction of reflecting object can be obtained by the use of directional transmitting and receiving antennas.

The four basic characteristics of an electromagnetic wave are amplitude (power), frequency (wavelength),

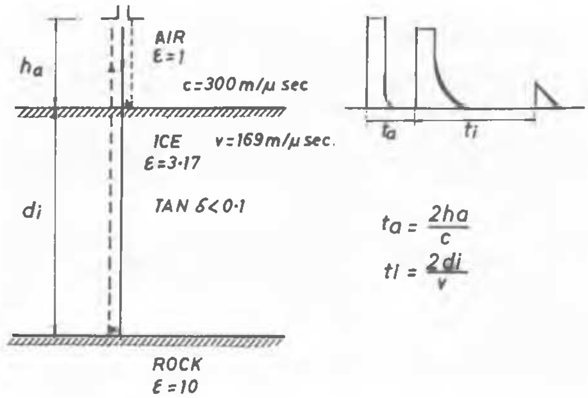

Fig. 2.

phase and polarisation. By examining the behaviour of few or all of these parameters, we can infer some of the properties of surface that scattered the wave. The electrical properties are usually characterised by parameters designated as conductivity, permittivity and permeability. Of these, the conductivity (Mhosper metre)

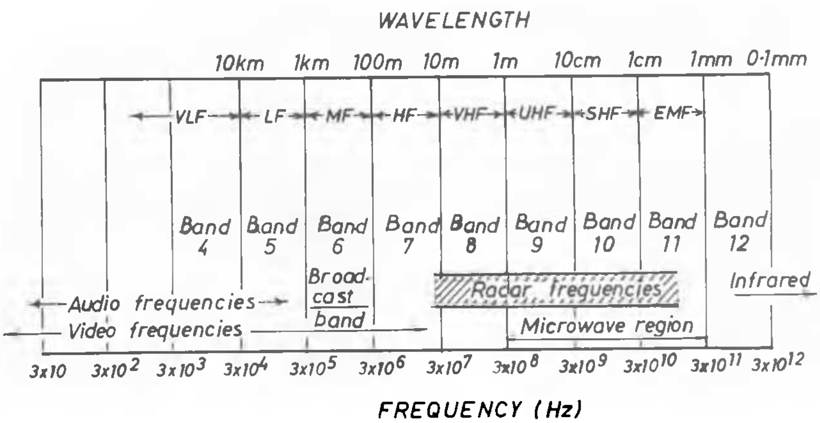

Fig. 1.

varies most widely among various earth materials and on it depends the 'information depth'. The permittivity ($\epsilon = \epsilon' \epsilon_o$) is significantly influenced by water content, can be measured with reasonable accuracy by the radar method which, in fact, effectively averages the dielectric constant (ϵ') over a large area. but only for a small depth below the surface. (ϵ_o) the permittivity of free space is approximately $(1/36\pi)\times 10^{-9}$ The permeability (μ_1), in many locations, approach to that of free space (μ_0) equal to $4\pi \times 10^{-7}$ Henri/metre.

An electromagnetic signal propegating along the surface of the earth suffers attenuation and phase decay as a result of finite conductivity of the soil. Velocity of penetration of electromagnetic waves into ground depends on dielectric constant and information depth on the conductivity. For example, clay which has high conductivity may be penetrated upto only 2-3 metres whereas sand, gravel, granite and moraine with low electrical conductivity may be penetrated upto 20-50m, Bjelm (1981). A relatively distinct conductivity change can be noted in the ground water level. This fact is used to determine the level of ground water table and its configuration.

POINTS TO CONSIDER

* Area covered per unit of time and man power.

* Information depth (D) attainable; loss in ability to distinquish between small strata units as D.

* Quality of information, & expertise required in analysis.

* Weight & size of Equipment.

* Operational safety & site repairability of equipment.

LIMITATION

Earth materials vary a great deal in their 'transparency' to radar. Useful probing distance could be kilometers in glacier ice, igneous and metamorphic rocks; tens of metres in dune sands; several metres in coarse-grained soils and only a few metres in clays even at frequencies as low as 1 MHz. Present equipment is far from approaching limiting radar exploration depth. Several different radar designs are possible.

* Correct operation of equipment and interpretation of signal recordings require more studies, richer field experience and higher level of expertise.

* Long wave lengths require large antenna apertures which would be cumbersome in portable equipment.

* Capability of radar logging of drill holes is still in the stage of development.

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H. Mori (Written discussion)

THE MOSS THIN WALL SAMPLER

1. The MOSS Thin Wall Sampler utilizes wireline and hollow stem auger to take high quality samples from a wide variety of formations at a very high rate of completion.

2. The Sampler assembly consists basically of a Thin Wall Tube and Head with radial bearings, a lead adjustment and a latch assembly. It is swivel attached to a wireline that passes up through the Hollow Stem Auger, the Gimbal Coupling, the large diameter drill spindle and onto a wireline hoisting drum. Hoisting and lowering of the Sampler Assembly is as simple as reeling in or reeling out wire. Changing of tubes or samplers - laden or unladen - is very rapid and is done at waist level with a simple screwdriver.

3. Lead of the tube or sampler can be varied from 0 to 9 inches ahead of the auger cutter head. Leads of from 4 to 9 inches are used for sensitive and cohesive soils. At 0 to 1 inch leads it is quite easy to acquire thin wall tube samples of shale and sandstones because outside friction is cut away.

4. Speed is attained by eliminating the conventional - but unnecessary - operations in sampling. No hole is prebored, no roundtripping of rods or augers is required. No manipulation of the rotary on or off hole is involved. With MOSS the otherwise all important borehole becomes nothing more than a byproduct of the sampling operation. It is quite common to retrieve continuous samples and still maintain rates faster than conventional 1.5 in 5 incremental sampling.

5. Quality of samples is equal to the best quality which could be expected from the sampler used in the assembly. For instance, a thin wall sampler pressed by the MOSS assembly should produce quality equal to a thin wall tube used conventionally. The possibility of better than conventional quality exists due to the outside friction control available by means of lead adjustment.

6. Sample loss is held to a minimum because wireline handling lacks the shock and vibration inherent in rod handling.

7. Sampling below water table is relatively easy by comparison with conventional methods. It is largely a matter of technique and appropriate control. A discussion of technique for use below the water table is beyond the scope of this one-page paper.

The author will be pleased to answer any inquiry regarding usage and technique.

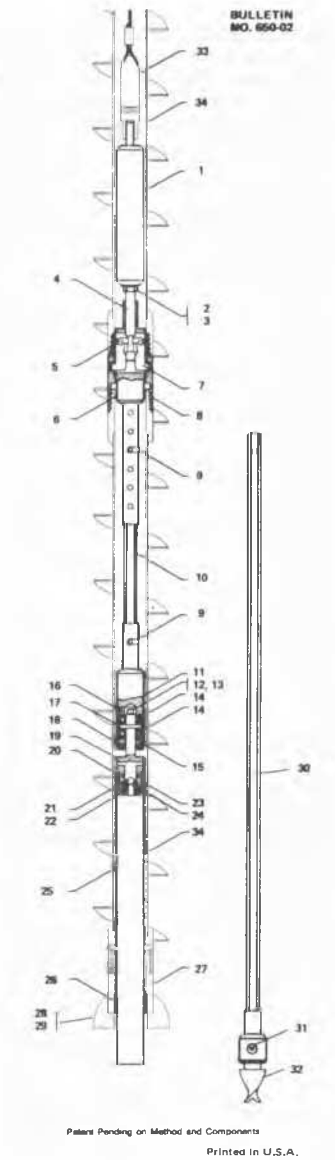

Fig. 1. The Moss Thin Wall Sampler

K. Tanimoto and J. Nakamura (Written discussion)

The paper we presented in this conference (Vol. 2, pp. 573-576) describes the use of acoustic emission (AE) technique in in-situ stress measurement. The followings are some additional comments on advantages and limitations for applying AE technique to this problem.

Figure 1 shows results of a pressuremeter test with AE measurement. The symbols given in this figure are the same as in our paper. From this figure, it is noted that n_l - p relation has two remarkable changing points Y_1 and I_1, which are at precompression stress and the second yielding stress where the sign of dilatancy is considered to change. One of the advantages of AE measurement may be that the sharp changes at those stresses give easier determination than using the customary method.

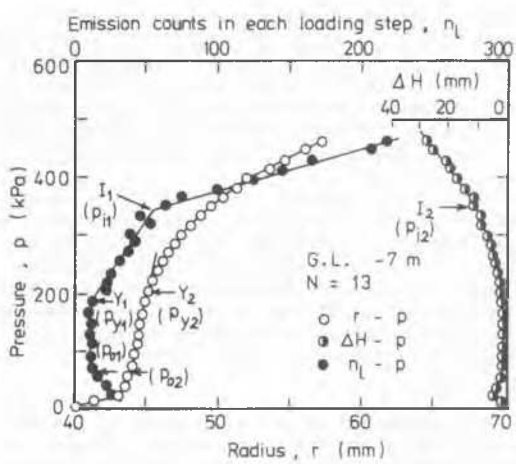

Fig. 1. Results of pressuremeter test with AE measurement

Another advantage of AE measurement is explained with a rough sketch in Fig. 2. If bore-hole has some disturbance or minor cave-in as shown in this figure, the customary method of estimating change in the radius of pressure cell may lead an erroneous result, because the radius is generally estimated from the quantity of water flowing into the pressure cell. On the other hand, the changing points on AE characteristics seem to be less affected by such the disturbance of bore-hole, although absolute count naturally decreases. Therefore, AE measurement may be used for reasonable estimation of in-situ stresses.

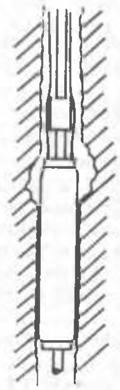

Fig. 2 Pressuremeter test in bore-hole having minor cave-in

The limitation of using AE technique is that the AE measurement should be made in combination with other ordinary measurements. In other words, it is considered that the AE measurement provides one of data for synthetic judgement of the test results. Another problem is that AE measurement in soft clay is generally very difficult.

In spite of such limitations, AE technique can be applied to various types of loading tests and soil investigations as well as instability prediction.

R.K. Bhandari (Written discussion)

LATERAL GROUND STRESS WITH FOCUS ON HYDRAULIC FRACTURING

Hydraulic fracturing is the consequence of the 'local' state of zero effective stress generated by fluid pressures. Under such a state, clean fine sands liquefy because of their single-grained structure. Muds or clays of fluid-like consistency, as in mudflows, develop mud-volcanoes, as the pore water pressure tend to exceed the geostatic stress. Similarly, peats are known to generate 'bog bursts' when acted upon by high pore pressures. Hydraulic fracturing is therefore, seen as a reality only when one speaks of fine-grained relatively impermeable soils. It refers to cracking that takes place on a plane perpendicular to direction of the minimum total principal stress, with effective stress on the plane of crack tending to zero or even negative if soil can withstand some tensile stress. Initially the cracks are small, narrow and scarcely visible but eventually they open up as the hydraulic pressure exceeds total pressure on the plane. If the hydraulic pressure is reduced at this stage, cracks are believed to close and therefore corresponding pressure is termed as 'close-up pressure'. It is inferred to be equal to total lateral pressure. The insitu value of K_0 can thus be computed from the known values of total vertical stress, close-up pressure and porewater pressure, provided $K_0 < 1$, the direction of minimum principal stress is horizontal and soil is relatively impervious and free from fissures.

Of the various approaches to estimating K_0 (Table 1) the one of hydraulic fracturing, described above, has come to be known as convenient and reliable, particularly for normally consolidated clays. Besides, the concept of hydraulic fracturing provides convincing explanations to over-estimation of ground permeability, if measured with hydraulic pressures in excess of 'close-up' or 'fracture-pressure' Bjerrum et.al (1972); Kennard (1970).

There are a number of issues which must be satisfactorily settled if the method of hydraulic fracturing is to get wider recognition towards routine application.

TECHNIQUES FOR PERFORMING FRACTURE TESTS IN SITU

Fracture tests are usually performed either in boreholes or in piezometers. Casagrande open stand-pipe piezometers and Bishop twin tube piezometers have both been successfully used.

For conducting the tests two options are usually open. One could cause hydraulic fracturing either by applying incremental water pressures in short term tests or by performing long term tests. In the former, the close up pressure is a function of total minor principal stress and in the latter case, of average total stress, Vaughan (1972). Close-up pressures have been measured in constant head as well as falling head tests and results are reported to be comparable. It may however be cautioned that use of waterjets or wash boring must be avoided during piezometer installation as that might lead to hydraulic fracturing.

Importance of the effect of shape of borehole and unevennesses of it's surface disturbance due to installation, pressure increment ratio; magnitudes of ultimate hydraulic pressure vis-a-vis close up pressure and length to diameter ratio of the piezometer probe must be recognised. For a piezometer tip corresponding to a 'point', occurrence of horizontal cracking and for that with higher length to diameter ratio, occurrence of vertical cracking are reported, Lefebvre et.al (1981). The questions that emerge are:

(a) What explanation do we advance to the differing pattern of cracks as observed in relation to the length to diameter ratio (L/D) of the piezometer tip?

(b) How much is the minimum value of L/D ratio which could be permitted in a hydraulic fracture test?

Till such time a satisfactory answer is found, I would like to propose a L/D ratio of 15. Besides, the magnitude of applied ultimate hydraulic pressure in relation to the magnitude of close-up pressure is important

LABORATORY MEASUREMENTS

Estimation of K_o with attempts to achieve (a) zero lateral yield and (b) zero vertical shear, in testing of samples under vertical pressure.

* Kjellman (1936) reports tests on cubic specimens of sand enclosed by metallic surfaces (divided into small sections separated by gaps) connected independently to the mechanical loading system. This enabled normal stresses to be applied and normal strains to be controlled on faces which remained free from shear stresses.

*Gersevanoff (1936) reports tests on cylindrical specimens placed in a rigid cell and surrounded with air free, incompressible fluid. The specimens were subjected to vertical stress by loading ram of equal diameter. The pressure-rise in the sealed cell was taken as the measure of lateral pressure at rest.

*Tschebotarioff, Ward, Dibiagio and Watkins (1956) demonstrated that even with elaborate precautions lateral yield becomes significant.

*Kjellman and Jakobson (1955) tested cylindrical specimens of pebbles, enclosed by a series of steel rings with small gaps to achieve lateral yield not exceeding the low elastic extension of rings and insignificant vertical shear.

*Bishop and Henkel (1957) tested cylindrical specimen in triaxial compression by adjusting increments of axial and cell pressures so as to yield zero lateral strain at the mid-height of the specimen.

*Bishop (1958) tested cylindrical specimen in triaxial compression by adjusting increments of axial and cell pressures such that change in specimen length multiplied by its initial cross-section area equalled volume change. Estimation of K_o without attempting zero lateral yield of the specimen.

*Skempton (1961) deduced K_o at different depths in stiff, fissured London Clay by comparing the insitu effective overburden stress σ_v' with isotropic effective stress σ' in the soil after undisturbed sampling:

$$K_o = \frac{\frac{\sigma'}{\sigma_v'} - A_s}{1 - A_s}$$

where A_s represents "relative pore pressure coefficient" defined as the ratio of the change in porepressure in the soil caused by the undisturbed sampling operation to the shear stress released in sampling. Skempton's studies led to a remarkable observation that horizontal effective stress approaches passive resistance of the clay, the finding also confirmed by Terzaghi (1961), Blight (1967). Blight (1967), using the above approach, investigated conditions of horizontal stress in two lacustrine clay deposits of stiff, fissured clays.

*Poulos and Davis (1972) An undisturbed cylindrical specimen is placed in a triaxial cell and reconsolidated in number of stages, to the level of insitu vertical effective stress and a horizontal stress of about 0.35-0.4 times the vertical i.e. a value lower than likely to exist in-situ but sufficiently high to prevent failure of the specimen. The specimen is then subjected to a series of relatively small increments of horizontal stress, the vertical stress remaining unchanged, and the change in volume under each increment is measured. Volume change versus effective horizontal stress relationship is then examined. The point corresponding to a pronounced change in slope of this plot in the case of normally consolidated soils, was inferred to lie in the vicinity of insitu horizontal stress.

IN SITU MEASUREMENTS

* Direct Measurement of pore pressure (u_o) and total vertical and lateral earth pressures.

$$K_o = \frac{\sigma_h - u_o}{\sigma_v - u_o} = \frac{\sigma_h'}{\sigma_v'}$$

Estimation of σ_h' is far more, difficult than estimation of σ_v'. Besides, pressure cells may record intermediate principal stress.

*Wroth (1972,1975) reiterates that the only reliable way to estimate σ_h' was to take recourse to in situ measurement of σ_h and u_o.

*Massarsch, Holtz, Holm and Fredriksson (1975), and Massarsch (1979) report use of spade-like total pressure cells for earth-pressure measurement.

*Rygg and Ostlid (1979) report the use of a steel cube with six earth pressure cells to measure the development of horizontal and vertical stresses.

*PRESSUREMETER; CAMKOMETER

Baguelin, Jezequel, Mee and Mehaute (1972); Wroth and Hughes (1973) Wroth (1975) report use of self-boring pressuremeter (SBP) for measuring σ_h. Procedures for correcting σ_h values to account for disturbance due to installation have been proposed by Marsland and Randolph (1977) and Denby (1978) have been used by Ladd et. al (1979) and Ghionna et.al (1980). The former is considered more appropriate for stiff clays (Ladd et.al 1979) and Latter for normally consolidated/lightly overconsolidated clays (Ghionna et.al 1980: Lacesseet.al (1981).

*HYDRAULIC FRACTURE TESTS

Sufficient information, data and experience lies at our command to yield the firm conclusion that fracturing can be introduced in rock or soil media by hydraulic pressures and that the fracture pressure provides a simple, convenient and powerful tool for estimating total lateral pressures.

Bjerrum and Anderson (1972), Bjerrum et al (1972)

EMPIRICAL & SEMI-EMPIRICAL APPROACHES

Jaky (1944)

$$K_o^{NC} = 1 - \sin \phi$$

Brooker and Ireland (1965)
Lambe & Whitman (1969)

$$K_o^{NC} = 0.95 - \sin \phi'$$

Wroth (1972)

$$0.95 - \sin\phi' \leq K_o^{NC} \leq 1 - \sin\phi'$$

Massarsch (1979) and Flavigny (1980)

$$K_o^{NC} = f(I_p)$$

Alpan (1967)

$$K_o^{NC} = 0.19 + 0.233 \log I_p$$

Alpan; Schmidt (1966)

$$K_o^{OC} = K_o^{NC} (OCR)^{\alpha}$$

where α = empirical constant; According to Flavigny (1980) $\alpha = 1 - K_o^{NC}$; For Italian clays $\alpha = 0.46 \pm 0.06$

Wroth (1972,1975)

$$K_o^{OC} = K_o^{NC}(OCR) - \frac{\nu'}{1-\nu'}(OCR-1)$$

(valid for low values of OCR)

Wroth (1972,1975)

$$m \left[\frac{3(1-K_o^{NC})}{1+2K_o^{NC}} - \frac{3.(1-K_o^{OC})}{1+2K_o^{OC}} \right] = \ln \left[\frac{OCR(1+2K_o^{NC})}{1+2K_o^{OC}} \right]$$

(Valid for high values of OCR)

where m = empirical constant.

TABLE I

as that would govern the extent and nature of cracks. If applied pressures are too high, the crack pattern may turn complex and the consequent estimate of close-up pressure would be in error.

It is therefore desirable to study the crack patterns for different (L/D) ratios of piezometers at different ratios of fracture and close-up pressures. The technique of methylene blue dye (Lefebvre et al, 1981) may prove particularly useful for mapping of cracks.

Attempts are necessary to develop a portable hydraulic fracture test equipment preferably dispensing with drilling rig, and minimising ground disturbances. The equipment and procedure both need to be standardised. Use of fluids more viscous than water might provide some advantage inasmuch as high viscosity fluid will not flow through soil pores unless cracking takes place. Bjerrum and Andersen (1972) describe tests with paraffin instead of water. The estimates of total minor stress using paraffin were found to compare well with those using water where water is used, it should be deaired and preferably be at a temperature higher than that of the ground water to avoid air bubbles which might significantly influence the test results.

INTERPRETATION OF THE TEST DATA

The state at which cracking is imminent marks the transition from the state of 'no crack' to that of the opening of cracks. It is, therefore, essential to have a convincing proof that the hydraulic pressure at which cracking is imminent and the close-up pressure are equal.

What and how much is the direct evidence that the cracks do close or heal up as the hydraulic pressure is brought below the fracture pressure mark. Could the sharp decrease in flow be on account of cessation of crack to propogate or change of fabric in the zone of influence or blocking of the crack by soil particles eroded during the hydraulic flow. Such a possibility has been hinted by Vaughan (1972), while discussing observations on falling head tests; although his observation that fracture tests do not lead to permanent increase of permeability of soil tend to support the view that cracks do close.

Lefebvre et.al's (1981) work reported for this conference is of a special significance and may suggest an answer if additional data on the levels of hydraulic pressure vis-a-vis the close-up pressure, as also the thicknesses of cracks could be reported. The nexus between crack-pattern at closer and the ultimate value of hydraulic pressure in any given case is also of considerable importance.

Besides, it would also be of interest to examine the time dependent redistribution of stresses and the consequent effect on cracks.

It is stated that pressure at which hydraulic fracturing occurs depends on the initial circumferential stress and Poisson's ratio of the soil skelton but is, independent of the modulus of elasticity, Bjerrum et.al (1972). Experiments and measurements are required to prove this.

PATTERN OF CRACKS

A horizontal crack is reported to occur in a highly over-consolidated clay such that fracture pressure equals total vertical pressure on the plane of the crack. For this reason, Bjerrum and Andersen (1972) suggest that hydraulic fracture method cannot be used in over-consolidated clays where $K_o > 1$. Work of Lefebvre et.al (1981) provides evidence that horizontal crack could also occur in normally consolidated clays if piezometer tip has very low height to diameter ratio.

The conclusion that the deposit has $K_o > 1$ just because fracture is horizontal and, fracture pressure and total vertical stress are nearly equal could therefore, be questionable.

EFFECT OF ANISOTROPY

Other things being equal, the probability of hydraulic fracturing is greater in soils which are non-homogeneous with respect to deformability and permeability. Non-uniform deformability helps in development of low total minor stress locally.

Soils containing cementing agent especially susceptible to cracking. The influence of cementing materials in clays on fracture pressure needs to be fully understood because tensile strength will be significant.

RELIABILITY OF K_o VALUES

Comparison of the horizontal effective stress values as measured in a hydraulic fracture test with those obtained by (a) self boring pressuremeter, PAF (b) Jaky's formula for K_o based on triaxial tests suggests that fracture pressure yields in situ horizontal stress values systematically greater than those of PAF Baguelin et al (1978). Not much of information is available to suggest the superiority of one over the other or greater reliability of either. Studies of K_o for the same site by different methods therefore, deserves encouragement. Bjerrum and Andersen (1972) have reported measurements of K_o at six sites. At two of the sites, the clay is quick and at the other four non-quick lean or plastic. The K_o values were found in the range of 0.4 to 0.5 for the quick and 0.5-0.6 for non quick clays. On the basis of this study, the hydraulic fracture test was considered appropriate for routine-type measurements of the horizontal stress in the ground.

ACKNOWLEDGMENT

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G. Flepp (Written discussion)

UTILISATION DE CORRELATIONS DE PARAMETRES POUR UNE RECONNAISSANCE DE SITE PLUS EFFICACE

Use of Parameter Correlations for a More Efficient Site Investigation

RESUME

L'étalonnage des moyens mis en oevre pour reconnaître un remplissage alluvial hétérogène et profond a permis de corréler la vitesse d'avancement de forages destructifs exécutés à poussée constante avec les mesures d'essais de pénétration et d'essais pressiométriques. Il résulte de l'utilisation de forages destructifs avec enregistrement de paramètres une reconnaissance plus efficace, sur l'ensemble du site, des différents matériaux et de leur caractéristiques mécaniques.

Pour un site de barrage la reconnaissance sur une superficie d'environ 20 hectares d'un remplissage alluvial hétérogène dont l'épaisseur maximale dépasse 60 mètres a exigé la mise en oeuvre de moyens importants et divers.

Pour utiliser les procédés les plus adaptés aux terrains traversés et susceptibles de fournir les informations les plus utiles dans les meilleurs délais, on a procédé, en deux points du site, à un étalonnage rigoureux de tous les moyens disponibles.

En chacun de ces deux points d'étalonnage, on a réalisé 7 forages dans un carré de 4 mètres de côté, soit :

- 1 sondage carotté avec échantillonnage intact continu,

- 1 forage destructif à la boue avec essais SPT tous les mètres,

- 1 forage destructif avec enregistrement des paramètres de forage (ENPASOL) et diagraphies de la radioactivité naturelle et du potentiel spontané,

- 1 essai de pénétration dynamique (SERMES),

- 1 forage destructif avec essais scissométriques tous les mètres à partir du premier horizon argileux,

- 1 forage destructif avec essais pressiométriques tous les mètres,

- 1 essai de pénétration statique, (GOUDA).

Ces moyens sont bien connus. On rappelle simplement que dans l'essai ENPASOL quatre paramètres sont enregistrés en continu au cours de la descente du tricone dans le terrain :

- la poussée sur l'outil,

- le couple moteur,

- la pression du fluide de perforation (P),

- la vitesse d'avancement (V).

Dans le cas étudié, on a maintenu aussi constants que possible le couple moteur et la poussée sur l'outil et on a analysé les variations des deux autres paramètres (pression et vitesse).

Sur la Figure 1, on remarque que la pression du fluide de perforation P, est faible dans les sables (de même dans les grès), et élevée dans les argiles (de même dans les marnes). La vitesse d'avancement V diminue avec la compacité.

CORRELATION ENTRE LES DIVERS ESSAIS

Les terrains traversés sont principalement des sables fins, des silts et des argiles.

En séparant les mesures effectuées dans chacun de ces matériaux aux plots d'étalonnage et en procédant à une analyse multifactorielle, on a établi de nombreuses corrélations.

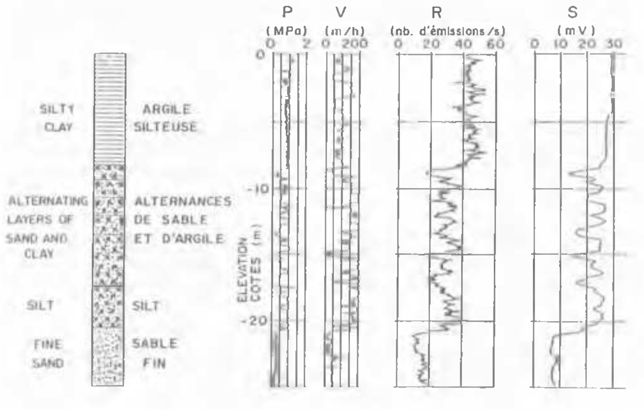


Fig. 1.

Paramètres de Sondage - Drilling Parameters
Diagraphies - Geophysical Borehole Logging

P : Pression du fluide de perforation
Drilling fluid pressure
V : Vitesse d'avancement
Rate of drilling
R : Radioactivité naturelle
Natural radioactivity
S : Potentiel spontané
Spontaneous pontential

Certaines sont classiques, et nous ne présenterons que les corrélations liant la vitesse d'avancement des forages ENPASOL à d'autres paramètres.

Dans les argiles, la vitesse d'avancement V (m/h) des forages ENPASOL est liée :

- au résultat de l'essai, SPT (N) par la relation :

$$N = 1000 / 0,37\ V + 16,3)$$

avec un coefficient de corrélation r = 0,86 (figure 2)

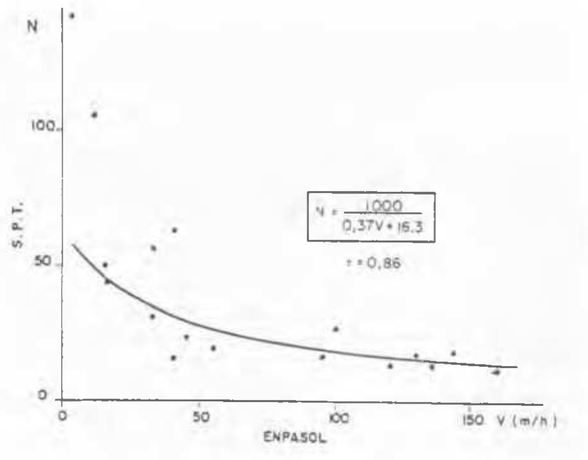


Fig. 2

Corrélation entre le S.P.T. et la vitesse d'avancement (ENPASOL) dans les argiles

Correlation between S.P.T. and rate of drilling in clays

- à la pression limite Pl (MPa) mesurée par l'essai pressiométrique par la relation :

$$\log V = 2{,}2 - 0{,}35\ Pl$$

avec un coefficient de corrélation $r = 0{,}86$

(figure 3)

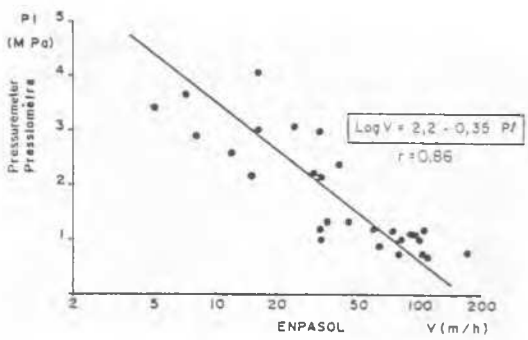


Fig. 3

Corrélation entre la pression limite (Pressiomètre) et la vitesse d'avancement (ENPASOL) dans les argiles

Correlation between limit pressure (Pressuremeter) and rate of drilling (ENPASOL) in clays

Dans les sables, la vitesse d'avancement V (m/h) des forages ENPASOL est liée à la résistance à la pénétration statique R_p (MPa), ou dynamique Q_d (MPa) par la relation :

$$V = 159 - 94{,}6 \log (R_p \text{ ou } Q_d)$$

avec un coefficient de corrélation $r = 0{,}87$

(figure 4)

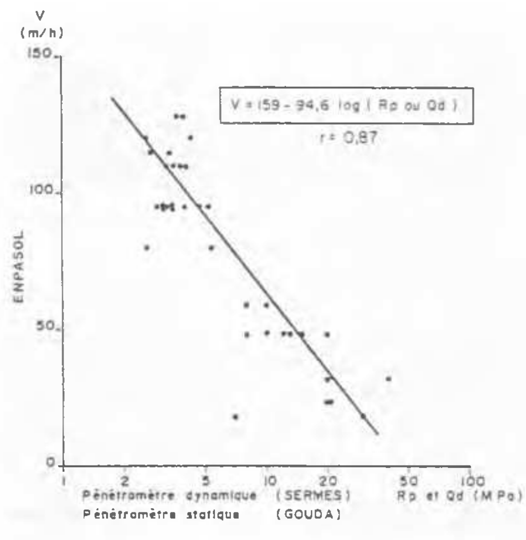


Fig. 4

Corrélation entre la vitesse d'avancement (ENPASOL) et les résistances à la pénétration dynamique et statique dans les sables

Corrélation between the rate of drilling and dynamic and static penetration resistance in sands

Les renseignements fournis par les forages ENPASOL se sont révélés très utiles et ont ainsi permis sur l'ensemble du site :

- la mise en évidence des niveaux sableux, silteux et argileux et du contact avec le substratum.

- la caractérisation des propriétés mécaniques des matériaux (compacité des sables et consistance des argiles).

La rapidité d'exécution et la possibilité de forer à grande profondeur sont des avantages supplémentaires. Il a été ainsi possible d'évaluer la compacité des sables à grande profondeur au delà des limites atteintes par les pénétromètres statiques et dynamiques.

E. Ledeuil (Written discussion)

UTILISATION D'UN PROGRAMME AVEC TRI-STATISTIQUE POUR UNE RECONNAISSANCE DE BARRAGE EN TERRE

RESUME : Ce tri par ordinateur permet de rentrer des données au fur et à mesure de leur disponibilité (1 échantillon en 1 carte peut contenir 14 paramètres de mesures et permet le calcul indirect de 4 autres), il permet l'étude statistique des paramètres et le calcul des volumes disponibles selon différents choix, donc de définir l'exploitation de la ballastière.

1. PRESENTATION DU PROGRAMME :

1.1. Les paramètres sont rentrés sur carte IBM suivant exemple ci-après sans séparateur entre chiffres. On utilise un tableau écrit dont la disposition est identique à celle des cartes pour permettre une vérification de copie visuelle globale et plus sûre.

"LA VERNE"

MESURE	Z	1	2	3	4	5	6	7	8	9	10	11	12	13	14
n° Identif.		γ_s T/m³	W % in situ	Epais.	γ_d Proct. T/m³	W % Proct.	L.L. %	I.P. %	Granulométries < à 50 mm	20 mm	5 mm	80 μ	5 μ	Perm. E−±10 m/s	Surf m²

```
//GO.SYSIN  DD  *
TITRE IRE LIGNE:                                         DONNEES CONCERNANT LES RE
1SUITE:CONNAISSANCES DU BARRAGE '  LA VERNE  '
TITRE 2ME LIGNE:                                         ************************
2SUITE:====================================

VN TR9    B                            99.  95.  87.  15.  6.5
VN TR11   B         0.8                100. 96.5 89.  28.  14.
VN TR18   A         2.8                99.  87.  66.  12.  6.        900.
VN TR28   A         1.8                100. 99.  94.  28.  11.5      1500.
VN TR34   A         1.                 80.  63.  49.5 13.  6.5       1000.
VN TR38   B         2.9                99.5 95.  82.5 16.  7.5
VN TR40   B         2.                 100. 100. 93.  24.5 8.5
VN R1-1   B 2.55 1.8 3.  1.78 14.6 15.  2.  100. 92.5 70.  8.   2.   9E-75000.
VN R3-1   A         0.8                100. 100. 89.  22.  12.5      3200.
```

Le programme s'appelle "SAMPLE" et est complété par un programme de sortie graphique "SKETCH".

1.2. Le numéro d'identification comprend 8 chiffres ou lettres, il n'est pas opérationnel. La dixième position sur la carte indique la zone géographique où se trouve l'échantillon.

1.3. En l'absence de données les cases correspondantes restent vides, ce qui évite les tableaux de zéros.

1.4. Le tri se fait par cartes données par lesquelles on peut choisir la ou les zones à étudier et compléter par 2 critères de qualité agissant conjointement.

```
        Z
1.TITR DATA: RECONNAISSANCES DU BARRAGE & LA VERNE &
2.SUITE....:
3.S/TT DATA:           TABLEAU DES DONNEES
4.SUITE....:
5.PONDER:   VOLMIN=  500.VOLMAX= 5000./XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
6.CRITERES DSORDONN:01>01/02>02/03>03/04>04/05>05/06>06/07>07/08>08/09>09/10>10/
7.SUITE            :11>11/12>12/13>13/14>14/15>15/16>16/17>16/18>18/19>00/20>00/
8.VALIDATION DE CALCUL:01>Y/02>Y/03>N/04>Y/05>Y/06>Y/07>Y/08>Y/09>Y/10>Y/XXXXXXX
9.SUITE               :11>Y/12>Y/13>Y/14>N/15>Y/16>N/17>N/18>N/19>N/20>N/XXXXXXX
CHOIX NO:02/ZONES:A-C-D-M- - - - - - /CRITERE 1:11/-15.  /CRITERE 2:  /      /XX
CHOIX NO:03/ZONES:A-C-D-M- - - - - - /CRITERE 1:11/-10.  /CRITERE 2:  /      /XX
CHOIX NO:04/ZONES:A-C-D-M- - - - - - /CRITERE 1:11/-7.   /CRITERE 2:  /      /XX
CHOIX NO:05/ZONES:A- - - - - - - - - /CRITERE 1:  /      /CRITERE 2:  /.     /XX
CHOIX NO:06/ZONES:A- - - - - - - - - /CRITERE 1:07/10.   /CRITERE 2:  /      /XX
CHOIX NO:07/ZONES:A- - - - - - - - - /CRITERE 1:06/30.   /CRITERE 2:  /      /XX
```

2. RESULTATS OBTENUS :

2.1. Nous obtenons la liste classée des échantillons, retenus par les choix faits en § 1.4., avec leurs caractéristiques.

2.2. Pour chaque choix nous obtenons un tableau donnant les volumes des matériaux disponibles et l'étude statistique de 13 paramètres, le nombre d'échantillons concernés, le même nombre pondéré pour tenir compte de la représentativité de l'échantillon allant de 1 à 10 suivant la grandeur de la zone représentée.

```
-TRI SELON LES ZONES GEOGRAPHIQUES:               ******************
                                                  *                *
    -ZONES RETENUES: E-F-G-H.                     *  CHOIX NO:  9. *
                                                  *                *
-TRI SELON LES QUALITES DES MATERIAUX:            ******************

    -ECHANTILLONS RETENUS:                          VOLUME TOTAL RECONNU  :   140000.M3
                                                    ***********************************
         -MOINS DE 10 % D'ELEMENTS INF. A 80 MU

                                VALEURS STATISTIQUES.
                                *********************
```

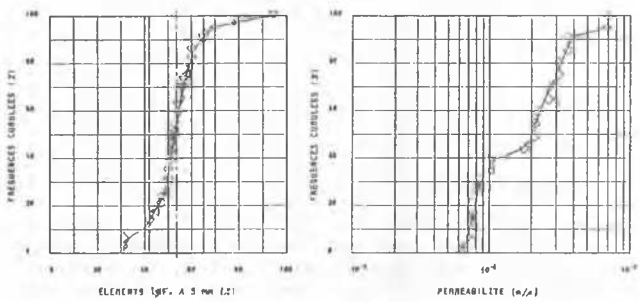
NE SU RE	NOMBRE D'ECHAN-TILLONS	NOMBRE PONDERE	MOYENNE	VARIANCE	ECART-TYPE	APLATIS-SEMENT	COEFF. DE VARIATION	VALEUR MEDIANE
1	0	****	****	****	****	****	****	****
2	0	****	****	****	****	****	****	****
4	5	32	2.145	5.531E-03	7.437E-02	-1.539	3.467	2.098

3. SORTIES GRAPHIQUES :

3.1. Les résultats sont stockés sur disques pour sorties graphiques sur traceur BENSON.

3.2. Le codage permet un graphisme de points différents suivant chaque lettre donc chaque zone, et la grosseur de chaque point varie en fonction de la pondération retenue en § 2.2.

3.3. Fréquence, moyenne, écart type y sont tracés.



3.4. Un graphique spécifique permet si on le veut de placer les échantillons sur un diagramme de classification "CASAGRANDE".

P. Le Tirant (Written discussion)

ESSAIS DU PRESSIOMETRE AUTOFOREUR POUR GRANDES PROFONDEURS D'EAU
Offshore Self-Boring Pressuremeter Tests

INTRODUCTION

Le développement de l'exploration et de la production pétrolière par des profondeurs d'eau de plusieurs centaines de mètres accroît l'intérêt des mesures géotechniques in situ pour la reconnaissance des sols avant l'implantation des ouvrages nécessaires.

Pour répondre aux besoins d'améliorer la qualité des reconnaissances des sols, l'Institut Français du Pétrole a développé, avec le concours des Laboratoires des Ponts et Chaussées, un pressiomètre autoforeur (Fig. 1) :
- applicable par 300 ou 1 000 mètres d'eau suivant le câble ombilical électrique utilisé,
- mis en oeuvre à partir d'un navire non spécialisé (type navire ravitailleur par exemple),
- et permettant d'atteindre des pénétrations dans le sol de 55-60 mètres.

Fig.1 - Pressiomètre autoforeur pour grandes profondeurs d'eau.

1.DESCRIPTION DE L'APPAREIL ET DU FONCTIONNEMENT.

On ne reviendra pas sur le principe de l'autoforage maintenant bien connu. On se limitera au bref rappel de la description et du fonctionnement du pressiomètre autoforeur marin :

1.1. L'appareil comprend essentiellement :

- un bâti autonome posé sur le fond marin d'une hauteur de 8,50 mètres et d'un poids de 16 tonnes (dans l'air),
- une sonde pressiométrique autoforeuse LPC/IFP mise en oeuvre à partir du bâti de fond au moyen d'une tige flexible,
- un câble ombilical de liaison fond-surface transmettant la puissance électrique, les commandes et les données de mesure,
- une cabine de contrôle avec le pupître de commande, le calculateur et ses périphériques (Fig. 2).

Fig. 2-Cabine de contrôle du pressiomètre autoforeur pour grandes profondeur d'eau.

1.2. Le fonçage dans le sol de la sonde pressiométrique est assuré :

- soit en continu sous l'action d'une masse-tige d'un poids de 10 kN, dans le cas des sols mous,
- soit par séquences successives en reprenant la réaction (jusqu'à 35 kN) sur les parois du trou de sondage au moyen d'un packer gonflable, dans le cas des sols plus raides.

1.3. La pénétration dans le sol s'accompagne de l'enregistrement tous les 10 centimètres des paramètres d'autoforage : effort de fonçage, vitesse d'avancement, pression dans la cellule pressiométrique non dilatée.
Les courbes pressiométriques enregistrées en temps réels sont ensuite corrigées pour tenir compte de l'inertie de la membrane, des dilatations des tubulures, de la compressibilité du fluide.

La mise au point des programmes informatiques pour l'exécution des mesures pressiométriques cycliques est en cours.

2. ESSAIS DU PRESSIOMETRE A TERRE ET EN MER.

Après de nombreux tests préliminaires des principaux composants (sonde autoforeuse, centrales hydrauliques immergées en équipression jusqu'à 10 MPa (100 bar), ...) l'appareil a été essayé d'abord à terre, puis dans l'eau.

2.1. Les sondages effectués à terre, dans l'argile à Cran (cohésion de 20 à 50 kPa), jusqu'à une profondeur maximale de 17 mètres (cote du substratum), ont permis de vérifier :
- le très bon fonctionnement de l'appareil (fonctions mécaniques, hydrauliques et électroniques),
- de parfaire les programmes informatiques de commande et d'acquisition des données,
- de s'assurer de l'excellente répétabilité des mesures sur cinq sondages (avec les deux variantes de fonçage).

2.2. Les sondages dans l'eau ont été réalisés à proximité d'un quai du Port Autonome du Havre, la mise à l'eau de l'appareil s'effectuant au moyen d'une grue (Fig. 1).
Les deux systèmes de fonçage par masse-tige et par packer gonflable ont été essayés.
La durée d'exécution d'un sondage de 19 mètres de pénétration dans le sable et le silt (résis-

tance au cisaillement atteignant 100 à 200 kPa), avec des mesures pressiométriques tous les mètres ou les deux mètres, a été de 12 heures environ. Les différents sondages réalisés ont confirmé l'excellente fiabilité du fonctionnement de l'appareil après une centaine d'heures d'immersion sans aucun incident.

2.3. Les essais de l'appareil en Méditerranée, à partir d'un navire à positionnement dynamique, par des profondeurs d'eau de 50 à 300 mètre, auront lieu en octobre 1981.

CONCLUSION

La fiabilité de fonctionnement, les performances réalisées et la qualité des mesures obtenues permettent actuellement de prévoir l'application industrielle du pressiomètre autoforeur en mer dès la fin de 1981.
Un important travail d'ingénierie des fondations marines (soumise à l'action des sollicitations cycliques) au moyen des méthodes pressiométriques est conduit parallèlement au développement de l'appareil.
La convergence de ces efforts devrait permettre de répondre à la fois aux besoins d'une meilleure reconnaissance des sols par des profondeurs d'eau de plus en plus grandes et de mieux évaluer le dimensionnement des fondations d'ouvrages en mer.

M. Maugeri (Written discussion)

ON EVALUATING SITE INVESTIGATION COSTS
Sur la Coute de l Investigation Géotechnique

Introduction

The ground has a complicated "personality" which must first be generally investigated and then in its own particular aspect so as to mitigate the uncertainties in the prediction and performance of geotechnical works: this is one of major conclusions reached in the independent discussions during Sessions 1 and 7 of this Conference.

Among the points discussed in Session 1; the chairman, Dr D'Apollonia, stressed the importance of mitigating the uncertainties, and hoped for fewer financial limitations on geotechnical investigations in the future. Prof. De Beer took up this motion and proposed a formula in which the cost of site investigations should be proportional to the total construction cost (De Beer 1981, a).

In Session 7 the discussion illustrated modern site investigation techniques and the quality of the soil data in relation to specific foundation problems. Dr Wilson, the ISSMFE Sub-Committee chairman announced the imminent publication of an international manual on planning and carryin out site investigations.

This manual will clarify what is to be considered a normal geotechnical investigation program; the cost of such a program must be evaluated however, and it remains to be seen if the cost is accepted by the client.

Choosing criteria for site investigation costs

Theoretically, the optimum site investigation cost is the lowest combination of risk and investigation costs. There is very little literature on this subject however and not even the site investigation costs alone are well documented in case histories. Moreover there are no specific indications on reasonable site investigation costs in the national manuals.

Thus the best way to overcome this situation would be to get information from private geotechnical agencies and public departments to work out a statistic on past site investigation costs.

The Belgian Ministry of Public Works has drawn up three categories for geotechnical investigation costs by using this type of statistio (De Beer 1981, b):
- for small projects, with a total cost of less than $ 200,000: 3% of the total cost;
- for medium projects, with a total cost of between $ 1,000,000 and 4,000,000: 1% of the total cost;
- for large projects costing more than $ 10,000,000: 0.5% of the total cost.

A similar statistical study is being done by the Japanese Ministry of Public Works. From the results so far available and other information from private Japanese geotechnical agencies (Ohya, 1981) it seems that site investigation costs are considerably higher in Japan than in Belgium. For example, 5% of the total cost in constructing a bridge in Japan goes to geotechnical investigation for the bridge foundations, plus a further 1% for monitoring and observation during and after construction.

It is the writer's opinion that these and other differences - which are partly inevitable - can be reduced to reasonable limits once criteria for assembling statistical cata are established.

The first criterion would be to classify the type of work: highway site investigations are generally superficial and cost less than bridge site investigations; on the whole their cost is never higher than 2% of the total highway cost. Table 1 shows a general classification of the kinds of works to which geotechnical investigation costs are to be referred. Obviously these categories can be varied, increased or reduced according to the significance of the data obtained.

The second criterion concerns a reasonable ratio between geotechnical investigation costs and total construction costs. In Table 1 five cost classes for the fifteen work categories proposed are shown. The first category, referring to building foundations, shows, according to the data supplied by the Belgian Ministry of Public Works, the geotechnical investigation costs as a percentage of the total construction cost for the five cost classes proposed. Naturally for total costs lying across class limits the geotechnical investigation costs can be obtained by a linear interpolation between the percentages shown.

Table 1

Cost (millions of dollars)	0.2	1	5	10	100
Bulding foundations	3.0	2.0	1.0	0.7	0.5
Bridge foundations					
Underpinning					
Machine foundations					
Nuclear power plants					
Highway studies					
Airfield pavements					
Tunnels					
Open excavation					
Dams					
Landslides					
Soil improvement					
Waterfront structures					
Pipelines					
Off-shore structures					

The third criterion would be to introduce some adjustment coefficients to take into account the type of ground, the type of the goetechnical investigation and other factors, such as those relative to the country in which one works. A general indication of the influence of the ground conditions on the site investigation cost is shown in Table 2 by means of a multiplicative coefficient of the base cost of Table 1. Attention is drawn to how, in some countries, the greater geotechnical investigation cost may be due to the presence of seismic areas.

In Table 3 a complete geotechnical investigation program is divided into five phases, for each of which a general cost indication is given with reference to the total geotechnical investigation cost. It should not be possible to effect a successive investigation phase unless the preceding one has been finished; moreover the cost of the preceding phases must be added to the cost of any given phase.

Table 2

Type of ground	Coef.
Soil	1.0
Rock	0.8
Soft clay	1.2
Difficult ground	1.5
Seismic areas	2.0

As an example we give the calculation of the cost "C" (in dollars) of a detailed geotechnical investigation, including laboratory tests (excluding monitoring and observation during and after construction), relative to building foundations in a seismic area, with a total construction cost of $ 200,000:

$$C = 200{,}000 \times 0.3 \times 2.0 \times (0.1 + 0.2 + 0.4) = 8{,}400$$

Of course the very simple method here proposed does not intend to give a definite solution to a very complicated problem, but only a general indication for a solution which could be important in practical geotechnical engineering.

Table 3

Stage of investigat.	Coef.
General investig.	0.1
Feasibility study	0.2
Detailed investig.	0.4
Monitoring	0.2
Post Construc.observations	0.1

Conclusions

The establishment, on the basis of well experimented data in the widest international context possible, of what is considered to be a normal cost for geotechnical investigation, can be extremely useful for a geotechnical engineer in order to obtain the money necessary to know the "personality of the ground", in the words of Prof. De Mello. It is far better to spend money beforehand on geotechnical investigation to mitigate uncertainties, than to spend much more afterwards on controversy and lawsuits.

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DE BEER, E.E. (1981, b): Written comunication.

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G. Ranjan (Written discussion)

"MECHANICAL PROPERTIES OF THE GRAVEL OF SANTIAGO FOR STATIC AND DYNAMIC LOADING CONDITIONS" by P. Ortigosa et al. Vol. 2, p. 545

The authors have presented an interesting study on Santiago Gravel. The discusser has also been involved in several studies on boulder-gravel-deposits in India. The experience from these studies is that insitu shear tests are necessary (Prakash and Ranjan, 1975; Ranjan et al, 1980) to have a realistic estimation of shear and other characteristics of the deposit. The authors have used 0.80 m dia x 1.60 m height insitu triaxial tests for the estimation of shear strength. With a maximum particle size of 0.25 m diameter present in the deposit (as reported by the authors), the ratio of sample diameter to maximum particle size is 3.2. This ratio is small and is likely to influence the results. In the opinion of the discusser this ratio should be 10 to 12 (Prakash and Ranjan, 1975). Though triaxial shear tests are certainly better as compared to direct shear tests, bigger size insitu direct shear boxes can, however, be easily assembled at site and conveniently performed. A simple set up (Fig. 1) could be used where tests at two different normal loads can be performed. The discusser has performed insitu direct shear tests with sample sizes of 1.5 m x 1.5 m, and 0.707 m x 0.707 m on boulder deposits. Comparisons have also been made with tests on 0.30 m x 0.30 m samples. The tests have yielded satisfactory results.

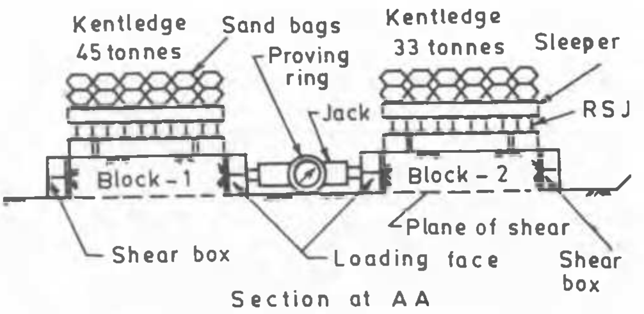

Fig. 1. Test set up for shear box test (After Prakash and Ranjan 1975)

Back analysis of slope failures is a very useful tool to estimate average shear parameters of deep deposits. The authors have also adopted this approach. The technique has also been successfully applied to estimate shear parameters in case of stable slopes (Prakash, Ranjan et al, 1980). However, there is a need to examine in detail the method of assigning suitable values of factor of safety. Also, to adopt a suitable shear parameter, it is necessary to have a clear picture of the type of deposit and of some data from shear tests. This information would be of use when choosing values of C and ø. It will be of interest to know the views of the authors in this regard.

Cyclic plate load tests can be used to estimate the coefficient of elastic uniform compression. The authors have also found this to be useful tool. In discusser's opinion, block resonance test can also be utilized to estimate the values of coefficient of elastic uniform compression C_u and coefficient of elastic uniform shear C_r (Ranjan et al, 1980). Tests have been carried out on a concrete block 1.5 m x 0.75 m x 0.70 m high. However, depending on the size of particles present in the deposit and the capacity of the oscillator, a bigger size block (3.0 m x 1.5 m x 1.0 m high) has also been used (Prakash, Ranjan en al, 1973). Block resonance tests have yielded satisfactory results.

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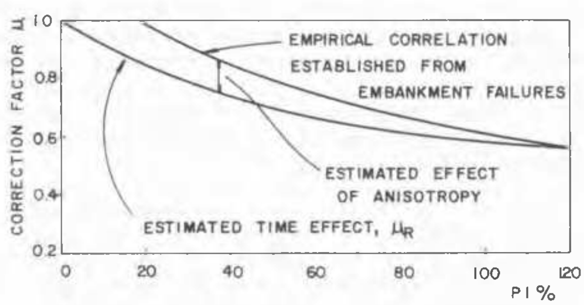
V.F.B. de Mello, Chairman

CONCLUDING REMARKS AFTER ORAL DISCUSSIONS

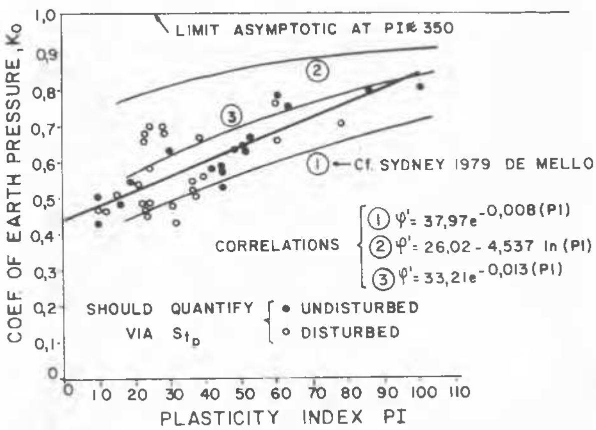
I am happy to have ceded to the General Reporters, Co-Chairman, Panelists, and special invited Discussers most of the time (20 minutes) allotted to myself for the concluding remarks. We all thank them and all participants for their contribution. One inevitable conclusion is that a day-long session is insufficient even for debating even one sub-question, much less for approaching any conclusion. The important exercise is to question and debate. It is quite an experience, to have the self-imposed obligation to stay quiet throughout such a dynamic Session.

The important exercise is to question and debate. Thus, as a final heresy I submit but two examples of concerns, on very local pronouncements, of world-wide influence. One we owe to no less an admirable and dear friend than Bjerrum himself (e.g. VIII ISSMFE, Moscow, vol.3 p 111).



1- APUD BJERRUM, VIII ICSMFE, VOL.3 P.124

Since a correction of s_{vane} is required for stability computations, it was declared and recognized to be imposed by effects of "structure", progressive failure, rate effects, anisotropy, differences between undisturbed and remolded stress-strain, etc. If such a correction is presumed adjusted to PI, are we saying that an index derived from two rudimentary tests on fully remolded material (with different ion contents of pore water) is satisfactory for reflecting undisturbed in situ behavior? Are we saying that S_t, differences of permeabilities of undisturbed vs. remolded, different stress-strain mobilizations of cohesion vs. friction, and so on, are directly related to PI? Obviously very doubtful except under a strictly local or regional empirical coincidence.

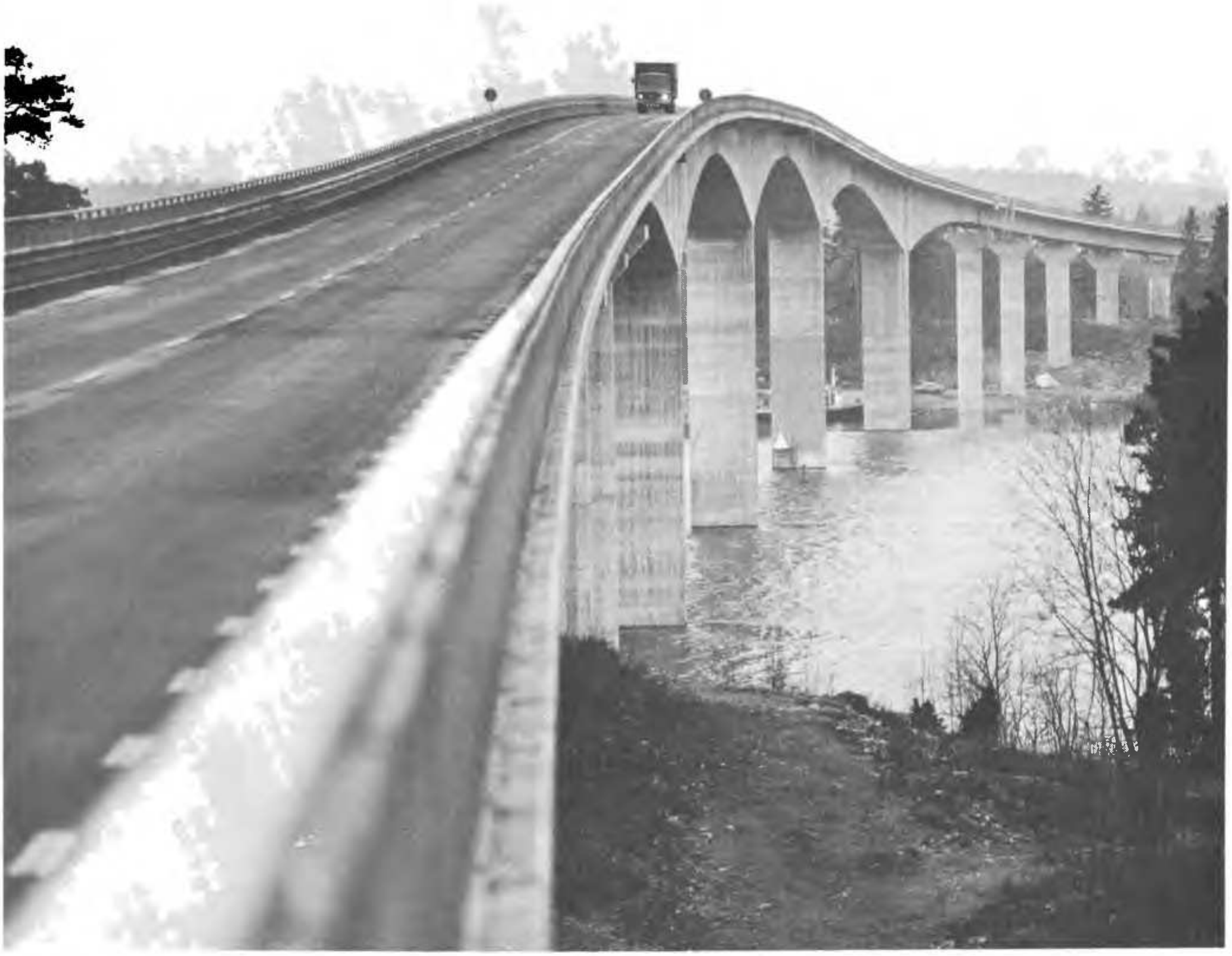


2- APUD MASSARSCH 1979 ADAPTED

The other example concerns the suggestion that in "typical" normally consolidated clays the conventional $K_o' \approx 1 - \sin\phi'$ be substituted by a linear regression $K_o' \approx 0.44 + (0.42)\ (PI/100)$ for $20 < PI < 80$, and this essentially irrespective of being "disturbed" or "undisturbed".

The above Figure is adapted from Massarsch, 7 ECSMFE, Brighton 1979, vol.2 p. 245. Firstly we should desire to substitute the dichotomy disturbed-undisturbed by values of S_t. Next, are we saying that $\phi'_{und} \approx \phi_{rem}$ (at least within $20<PI<80$) and therefore $s_{und} \neq s_{rem}$ only because of differences of c' and U_f? At any rate, adopting exponential exhaustion relationships for ϕ' vs. PI as is intuitively accepted (cf. my Fig. 14 Sydney 1979 ICASP 3, Vol.3 p. 135) we see that the data continue to plot very satisfactorily with reference to non-linear regressions. We must respect the evidences of extreme values of K_o' approximately corresponding to $\phi' \approx 30^o$ for PI $\approx$ 5, and $\phi' \approx 5^o$ for PI $\approx$ 350 (sodium-bentonite), as well as the asymtotic trend $K_o' \rightarrow 1.0$ as $\phi' \rightarrow 0^o$. We should not sacrifice the intrinsic recognition of K_o' as generated as a function of shear stress, and thus limited by shear strenght. Is it not better to use regressions that respect theoretical trends, even for data considered eminently empirical?

I must close, and beg your forgiveness for disturbing comments. We repeat the invitations for written discussions from all, whether or not time precluded your oral participation. I hereby thank everybody once again and declare the session closed.



The new bridge at the town of Strängnäs, about 80 km west of Stockholm. Abutments and Piers founded on either precast concrete piles or footings. Constructed by the Swedish Contractor Skanska.(Technical Visit G)