Discussion leader's report: Direct versus indirect use of in-situ test results

Rapport de l'animateur: Utilisation directe/indirecte des résultats d'essais en place

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INTRODUCTION

To predict the behaviour of a geotechnical structure, the "classical" approach is to determine some basic soil parameters and introduce them into some theory or model which describes the behaviour - or one aspect of the behaviour - of the geotechnical structure. Usually, the soil parameters are determined on soil samples in a laboratory, using deformability or strength tests in which, ideally, simple conditions of loading are developped, e.g. uniform stress fields. This type of approach is shown as path 1 in Fig. 1.

In-situ tests are also often used. But their results can be processed in two different ways to arrive at the behaviour of the geotechnical structure :

- the characteristics of the geotechnical structure are determined directly from the in-situ test parameters, using some correlation formulae. This type of approach can be termed "direct use" of in-situ tests ; it is shown as path 2 in Fig. 1.
- some basic soil parameters are determined from the in-situ test parameters, using some theory or some correlation formula. This is path 3 in Fig. 1. Then path 1 has to be followed, with the soil parameters thus determined. This approach will be termed "indirect use" of in-situ tests.

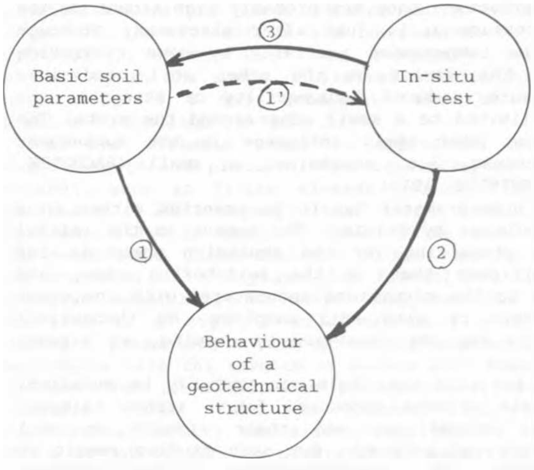

Fig. 1 The classical approach (1), the direct (2) and indirect (3)+(1) uses of in-situ tests.

The view is often expressed that path 1 is the only rational route, of universal value, while path 2 is empirical and of limited value. Thus, when in-situ tests are available, the indirect approach "path 3 + path 1" is often recommended, rather the direct approach "path 2".

Clearly, this view is only as good as the theories behind paths 1 and 3 are valid. Otherwise some adjustment, of empirical nature, as in path 2, will intervene in path 1 or in paths 3 + 1. On the other hand, correlations used in path 2 are not necessarily "blind", or magic box type, correlations ; knowledge gained from theories can be introduced in this type of approach.

It is therefore interesting to try to assess the validity and the role of the theory in the case of geotechnical structures and also of in-situ tests. This will be done by answering the question : <u>what is the present capability of the theory in deriving the behaviour of an in-situ test or a geotechnical structure from an adequate description of the soil behaviour</u>?

This means that the soil characteristics will not consist merely of the traditional "elastic" or "strength" parameters, but may incorporate such features as anisotropy, dilatancy, strain-softening or hardening, creep, etc...

The behaviour concerns the first stage of the loading, at low strains, as well as the ultimate stages of the loading, at large strains, with all intermediate stages.

The question corresponds to our present theoretical ability to follow routes 1 or 1' in Fig. 1.

THEORETICAL MODELLING OF IN-SITU TESTS AND OF GEOTECHNICAL STRUCTURES

Table I provides my personal answer to the question for the main in-situ tests and for a few basic geotechnical structures or problems. The rating is given both in a quantitative and a qualitative form. The explanations for each item are given below.

Table I - Rating the present capability of modelling the main in-situ tests and a few geotechnical structures.

IN-SITU TESTS

Field Vane Test	(FVT)	10 %	Poor
Standard Penetration Test	(SPT)	0 %	Null
Cone Penetration Test	(CPT)	2 %	Very poor
Pressuremeter Test	(SMT)		
- self-boring	(SB-PMT)	75 %	Excellent
- Ménard	(M -PMT)	50 %	Good

Piles	bearing capacity	5 %	Very poor
	settlement	50 %	Good
Footings	bearing capacity	10%to30%	Poor or fair
	settlement	75 %	Excellent
Slope Stability		25 %	Fair

Note that, for all in-situ tests, there is no theory which allows the modelling of the insertion of the probe or sampler within the ground. The effect of this deficiency will vary according to the in-situ test considered ; a comment on the importance of the insertion will be made, in addition to the discussion on the modelling of the loading itself.

Field vane test (FVT)

The traditional interpretation is made using a "limit-analysis" type model, which includes the circumscribed cylinder and the concept of a cohesion c_u (or two cohesions, c_{uv} and c_{uH}, especially when the height/length ratio of the blades can be varied).

The insertion of the probe has probably a significant effect on the initial soil conditions, especially in sensitive soils.

There is no theory which describes the development of the stress and strain fields from the start of the loading, just after insertion, to the ultimate phase where the test is interpreted, so that the failure pattern, and especially the amount and distribution of strains and stresses all along the surface of the cylinder used in the interpretation, are not known. Additionnally, the possible pore-pressuree redistributions, especially near the edges of the blades, are not assessed.

The "peak-value" of cohesion determined in the traditional interpretation of the test is more likely some average value of real post-peak strengths of the soil, corresponding to various rotations of the principal stresses, and different strain rates.

In conclusion, our present capability to model the FVT must be considered as poor.

Standard Penetration Test (SPT)

The test in itself corresponds to the insertion, by driving, of a sampler within the soil. There is no other subsequent loading.

The test is routinely used to provide the blowcount N. In addition to serious uncertainties in the released energy, due to the lack of standardization and to a poor control of the driving conditions, problems arise to convert N into soil physical properties : originally the relative density was exclusively favoured, then corrections of various types appeared for the overburden effect ; now, the angle of friction φ' tends to predominate (see the State-of-the-Art Report of Session 2 at this conference, by L. DECOURT). Perhaps, in the future, a single value of φ' to characterize a sand strength will be recognized as too simplistic and other properties such as dilatancy or dynamic characteristics will be added to the picture,... unless any attempt to derive soil physical properties is abandoned.

Such important fluctuations in the interpretation of the test merely reflects the absence of a theoretical model allowing to describe the penetration process from the basic properties of the soil and to indicate how the soil resistance is generated in the various zones, in other words allowing to understand what is going on.

Hence, the indications 0 % and "null" in Table I.

Cone penetration test (CPT)

The test corresponds to the insertion, by continuous penetration, of a cone within the soil. There is no other subsequent loading.

The result of the test is expressed in terms of a limit pressure, the cone resistance - q_c, corresponding to the average value of soil pressure mobilized at the front face of the cone.

At present, there is no full theoretical model allowing to describe the penetration process from the basic properties of the soil.

In the past, rigid-plastic models have been proposed to derive the tip resistance of a penetrometer or a pile from the shear strength parameters c and of the soil. However, such analyses are not justified from a theoretical point of view, since they require the existence of a plastic zone flowing towards some free boundary where the stresses are known and constant ; second, they were two-dimensional analyses, rather than three-dimensional. Attempts to assign a variable extent to the plastic zone, depending on the angle, do not alleviate the fundamental criticisms which can be addressed to such analyses. In addition, they fail to account for important factures of penetration experimental data e.g. the possibility of quite different penetration resistances of two soils with the same angle (and c=0).

Later, the tip resistance has been assimilated to the expansion of a sphere, for which a satisfactory theoretical modelling can be developped (MENARD, 1963, VESIC, 1967).

More recently, the model of a liquid flowing along a penetrometer has been proposed (BALIGH, 1986).

These two types of model can indeed bring valuable contributions to the understanding of the mechanism of tip resistance, but they cannot compensate for the absence of a theory of a cone (not a sphere) penetrating (not expanding) into a soil (not a liquid).

Considering these models, and also the fact that the penetration resistance concerns a well-defined zone of the probe and is measured directly, the situation has been deemed a little better than in the case of SPT, and not so desperate there is hope in a near future that such difficulties as finite strains, complex soil-probe contact conditions, can be handled successfully with relevant soil properties in a theoretical modelling of the cone penetration.

Hence, the rating 2 % and "very poor" in Table I.

Pressuremeter test (PMT)

The insertion is different in the case of a self-boring pressuremeter and in the case of a MENARD pressuremeter.

The self-boring process tends to minimize the insertion effects ; they are probably significant on the contact pressure (p_o), just after placement, although this can be compensated partially by some relaxation procedure. The impact on the other soil conditions (pore-pressure, reduced deformability or strength) are certainly limited to a small zone around the probe. The theory shows that their influence in the subsequent loading process, i.e. expansion, is small (BAGUELIN, JEZEQUEL, SHIELDS, 1978).

A MENARD pressuremeter has to be inserted either in a prebored hole or by driving. The impact on the initial conditions prevailing for the expansion phase is far more significant than in the self-boring case, and comparable to the situations encountered with the other in-situ tests or with soil samplers. No theoretical model exists for the insertion by driving, as already noted.

The creation of a bore hole can probably be modelled, and the main effects accounted for : stress release, pore water migrations, and their effects on soil deformability and strength. But such studies remain to be carried out systematically, and checked experimentally.

The loading phase, i.e. expansion of the probe, can be modelled easily (BAGUELIN, JEZEQUEL, SHIELDS, 1978), because of the axial symmetry of the problem. Any type of soil behaviour, including time effects, can be taken into account, and the simulation can be carried out from the initial stage of the loading to the ultimate, large displacement stage. Solutions to the ideal pressuremeter loading, i.e. corresponding to an infinite probe, can be corrected, when necessary, to account for the limited length of the probe.

Due to the insertion, different ratings have been given in Table I for the self-boring pressuremeter and the MENARD pressuremeter : excellent capability of modelling the first case (75 %), good capability for the second case (50 %).

Pile under vertical load

The first difficulty for a correct modelling is encountered with the installation phase : either driving, which at present cannot be reproduced in a model, as for penetration tests ; or boring and in-situ casting, for which there are some possibilities of simulation, but they have not been sufficiently explored. Thus, the initial conditions for the second phase, the loading, cannot be derived by some modelling at present, and only scarce indications are available from field observations.

The initial stage of the loading phase can be simulated easily (FRANK, 1974, BAGUELIN, FRANK, 1980), with rather complex soil behaviours if desired, and the analysis can be extended even when slippages occur along the shaft. Hence, the rating 50 % and "good" for the settlement modelling, in spite of the uncertainties in the initial conditions.

When large displacements are produced by the loading, our capability in modelling progressively decreases to almost zero, just as for the penetration tests. However, it should be noticed that, for the loading phase of a pile, it is sufficient to assess the soil resistance developped at large, but limited displacements, say 10 % or 20 % of the diameter, rather than to full penetration. In addition, our capacity to model the shaft friction components in the ultimate phase is not null hence, a rating a little better than for cone penetration tests : 5 %, which remains "very poor".

Footing under vertical load

In general, the installation of a shallow footing does not produce significant alterations of the soil conditions. Hence, it is considered here that the modelling is only concerned with the loading phase.

The initial stages of the loading can be simulated easily, even with complex soil behaviours. The early plastifications and large strains which occur at the edges of the foundation are considered as a minor difficulty, which does not seriously hamper our capability in the range of limited settlements : hence the rating 75 % and "excellent" for the settlement problem of footings in Table I.

As the ultimate stage is approached and larges displacements are produced, the capability of modelling methods, such as finite element methods, is somewhat decreased, and anyway has not been explored systematically. So that, at present, we must rely upon the traditional limit analysis of the bearing capacity, an extension of the plasticity Prandtl's solution for cohesive materials. In these analyses, the soil is modelled as a rigid plastic material, with constant strength parameters c and φ'. This scheme is not in accordance with the results of modern soil rheology and, in the case of sands, has produced bearing capacity formulae which fail to account for important experimental data : the size effect of the foundation width is not linear, as predicted by such formulae, and two sands with no cohesion and the same angle φ', as determined by the routine procedures, can have very different ultimate bearing capacities. Hence, the first ratings : 10 % and "poor" in Table I.

An important progress has appeared recently in the use of the plasticity theory for footings : J. GRAHAM & J.M. HOVAN (1986) have introduced in their calculations the fact that the angle φ' is not a constant, but may vary at one point as the loading of the footing increases, and the stress level changes at the considered point. The model produces a significant non-linear effect of the foundation width, and also a pronounced effect of the at rest stress conditions. If also displays large variations in the mobilized angle of friction throughout the mass of soil (about a 10° range). The model remains to be checked against experimental data. The second rating for the bearing capacity of footing in Table I corresponds to this promising theory : 30 % and "fair".

Slope stability

The models used in practice correspond to a limit analysis, performed on a two-dimensional surface, usually a circular cylinder (the well-known slip-circle analysis). The soils are characterized by their strength parameters c and φ, with the possibility, if desired, to account for soil anisotropy. Note, that in constrast with the footing or pile bearing capacity problems, the stress level in such problems does not change drastically in general, so that the approximation of constant strength parameters c and φ does not appear as a major drawback.

However, at present, little has been done to model the development of the stress and strain fields under the disturbing action, so that, in general, the strains and the type of stresses along the potential failure surfaces are not known.

A few studies of this kind can be quoted ; for instance, the development of plastic zones under the loading of an embankment has been modelled with a soil behaviour not too simplistic, of Cam-clay model type. Such studies seem to be more difficult when the disturbing action is an excavation or, for a natural slope, a change in pore water pressure.

In conclusion, our present capacity in modelling the development of a slope unstability can be considered as "fair" (Table I).

The immediate conclusions which can be drawn from this analysis, summarized in Table I, are that :

- our present capability in theoretical modelling such as defined earlier, is poor, i.e., in deriving the behaviour of a geotechnical system from realistic, thus fairly complex, soil behaviour laws. The rating may seem too severe to many people especially for tests which they use routinely ; but, it is important to remember the question to be answered.
- our understanding of the behaviour of geotechnical systems is based on simplified concepts, which often are not proven ; as a result, this understanding is limited and rather poor.
- an interesting exception is the pressuremeter test, which is the only non-trivial loading (i.e. with a non-uniform stress-field) which can now be modelled throughout a large strain range using complex, realistic soil behavior.
- as a consequence of our poor ability in theoretical modelling, the derivation of soil properties from in-situ test results - path 3 in Fig. 1 - has a pronounced empirical content ; and the indirect use of in-situ-tests- path 3 + path 1 - and the direct use - path 2 - are not so different in nature.

THE DIRECT USE OF IN-SITU TESTS

Of course, it would be unwise to correlate the results of any in-situ test with the behaviour of any geotechnical structure.

A number of conditions are necessary to reach at reasonable and successful correlations. There must be :

- some similarity between the two terms of the correlation ;

- some understanding of the two terms, so that the general knowledge of the two phenomenons can be incorporated into the correlations ;
- a well-documented data base and a reasonable check of the proposed correlation against this data base.

<u>The field vane test (FVT) an the stability of embankments on a soft soil.</u>

The FVT results are converted into undrained cohesion values, c_u, which are subsequently introduced into a slope stability analysis, usually with a circular failure line. Checking the method against 13 actual embankment failures, BJERRUM(1972) proposed a calibration factor μ, which he set as a function of the plasticity index IP of the soft soil.

At first glance, this method seems to follow path 3 and then path 1 of Fig. 1 and thus it would be classified as an indirect use of an in-situ test. And this would probably have been done by the original promoters of the FVT, when they proposed to derive the undrained cohesion c_u. But since then, we have learnt that the concept of a unique cohesion, irrespective of the type of stress change and of anisotropy, cannot be retained any longer ; we also suspect that progressive failure phenomenons, rate effects can be important in the two phenomenons, although we are not able to model them as discussed in the first part of this paper.

It is in fact more relevant, and more realistic, to consider that the method is a direct correlation between the in-situ test FVT and the embankment behaviour at failure, i.e. follows path 2 in Fig. 1.

Indeed, there is some similarity between the phenomenons and the two analysis carried out, as illustrated in Fig. 2. At the ultimate stage, a circular failure line is developped in both cases. When correlating the two phenomenons, it is necessary to account first for the differences in size and geometry :

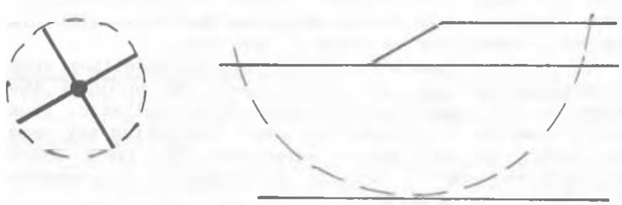

Fig. 2 The field vane test (FVT) and the stability of an embankment on a soft soil

radii R and lengths of the circular arcs. This is basically the job performed by the conventional analysis methods of the FVT and of the embankment stability. The correlation, which comes out of the experimental confrontation of the two problems, is expressed by the coefficient μ, which indeed, is close to unity, an evidence of the similarity of the two phenomenons. The main difference was attributed by BJERRUM to the rate effects ; hence his proposal to introduce the plasticity index.

The method works reasonably well. There is still some scatter. Future improvements require progresses in the understanding and modelling of both the embankment behaviour and the FVT, i.e. paths 1 and 1'.

<u>The cone penetration test (CPT) and the tip resistance of piles</u>

The similarity between the two phenomenons seems obvious (Fig. 3).

If an identity is set, taking account of the difference in sizes (cross -sections), the cone resistance q_c can be equated to the ultimate tip resistance of piles q_p.

Numerous experimental data show that this is not so simple and that a correlation factor k_c is necessary :

$$q_p = k_c . q_c$$

The differences can be attributed to various factors, such as the differences in installation modes, in the rate of loading, the fact that many q_p data correspond to limited relative settlements, as compared to q_c, etc... So that the correlation factor is varied according to the type of soil and the type of pile.

To get a full design of pile capacity, the method is complemented by some correlations between q_c and the shaft friction q_s ; the principle of similarity between the two phenomenons does not seem to be fulfilled here, but numerous experimental data can be gathered to develop correlations. The scatter however is significant.

On the whole, the CPT method of predicting pile bearing capacity can be considered as a good method and is among the best methods available far ahead of the methods based on the basic soil parameters c and φ.

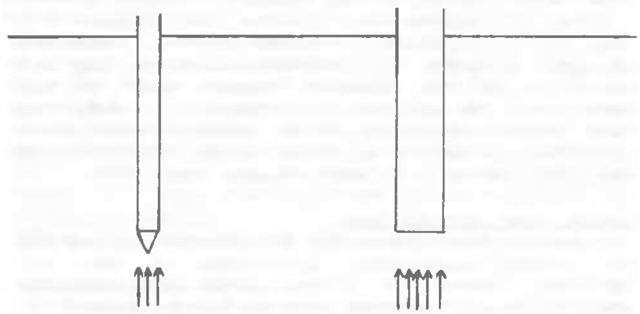

Fig. 3 The cone penetration test (CPT) and the tip resistance of a pile

<u>The pressuremeter test (PMT) and the bearing capacity of footings</u>

The similarity between the two phenomenons does not seem obvious. However (Fig. 4) in both cases there is a general displacement towards the mass of the soil. This analogy between a cylindrical expansion and a vertical displacement of a footing is justified by the results of experiments : many field tests have been carried out in various soils (MENARD, 1963 ; BAGUELIN et al, 1978 ; AMAR et al, 1984, 1985) ; they display a ratio close to unity between the limit pressure p_l of the PMT and the ultimate bearing pressure of footing, q_l, when the embedment is small. The correlation factor, k :

$$q_l = k . p_l$$

varies with the embedment and the soil type.

The method works well. It is probably the most efficient one among the various methods available to redict the bearing capacity of shallow foundations.

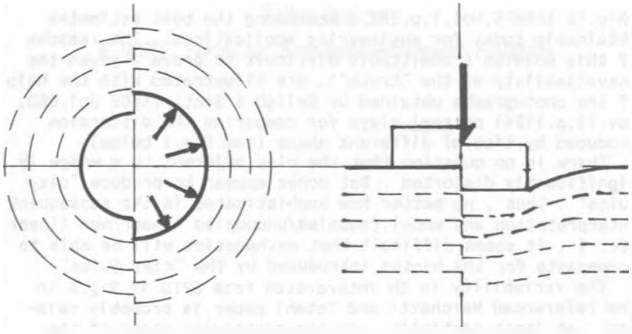

Fig. 4 The pressuremeter test (PMT) and the punching of a footing

CONCLUSIONS

Design methods which are based on a theory are not necessarily valid for this reason.

A key point to the value of any method is its check against a sufficiently wide data base. This is rarely done.

Empirical methods, when based on a proper analogy and when duely calibrated, may be more efficient than theoretical methods.

The direct use of in-situ tests for certain problems is, in such cases, superior to the conversion of their results into basic soil parameters, which are then introduced in a theoretical method. In this last approach, errors in the interpretation of the in-situ tests and in the theory of the geotechnical problem are accumulated.

Present research on in-situ tests seems to put more and more emphasis in the derivation of basic soil parameters from in-situ test results by empirical correlations. Personnally, I am not sure this direction of research will lead to important progresses in the use of in-situ tests, and, more generally, in our understanding of soil Mechanics problems.

I would rather advocate more research in two domains, in which the present situation is poor :

1) the theoretical modelling of the behaviour of various geotechnical systems - in-situ tests, but also ogeotechnical structures - using the modern soil rheological models,

2) the making up of extensive experimental data bases on the behaviour of geotecnnical structures, against which checks or calibrations or any design method could be made.

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Discussion

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This discussion addresses some issues , concerning DMT interpretation , raised by the paper 2/19 by Houlsby and Wroth to this Conference (Vol.1,p.227).

1. INTERDEPENDENCY OF Po AND P1

Based on calibration chamber (CC) results , the Authors conclude that , in a given sand , Po - P1 - P (P = Pushing force , not considered in this discussion) are interrelated, and therefore do not provide independent information . This discusser would like to emphasize the implications of the restriction " in a given sand " , often not fully appreciated by CC experimenters , who tend to generalize the interdependency of Po and P1.

Two quantities (such as Po and P1) are said to be dependent when , given one of them (e.g. Po) one can predict the second (e.g. P1) . This discusser has hundreds of results in natural sand deposits in which , for a given Po , P1 varies widely . E.g. if Po is 2 bar , P1 can easily be 6 or 14 or 22 bar (i.e. Id = 2 , 6 , 10) . Thus , in a new sand site , it is impossible to predict , even approximately , P1 from Po. Hence measuring P1 does provide an additional information , and permits calculating Id=(P1-Po)/Po .

In a sense it is encouraging to see that CC researchers have confirmed the interdependency Po-P1 . It reinforces the soundness of the definition of Id as Material Index . In fact , if the "universe" of sands tested in the CC is only one "material" , if Id is an index representing well the material , the interdependency of Po and P1 is just what one would expect .

(Moreover, in CC research , often different stress histories are imposed , and the despite-that interdependency of Po,P1 , i.e. the constancy of Id , points towards the ability of Id to reflect material type independently from stress history).

If two natural sands have Po=2 , but one has P1=6 ,the other P1=22 , the second one is much stiffer,having requested much higher δP for the same 1.1 mm displacement . Thus Id is some kind of rigidity index . To help illustrating Id , one may recall e.g. that often correlations based on Qc (from CPT) refer to broad categories of sand (e.g. "predominantly quartz angular sand" etc.). The information provided by Id is a quantification (non subjective and reproducible) of these categories,in a scale of Id from 1.8 to 10 (generally).

One might even argue (though a little provocatively) that the material classification based on Id may be more useful , engineeringwise , than the strict grain size classification , being more reflective of the "rigidity index" . It is well known that a little mica considerably decreases sand rigidity. The mica addition would be briskly reflected by Id (and would affect engineering behaviour) , but modestly by grain size analysis.

2. CPTU vs DMTA DISSIPATIONS IN CLAY

The Authors indicate broadly CPTU as their first choice for testing clay in situ . This discusser would like to express an alternative opinion , restricting however herein the discussion to the evaluation of rate of consolidation properties ("Ch") .

This discusser is in fact of the opinion that the evaluations of Ch provided by the Marchetti and Totani DMTA method (Rio,12 ICSMFE,Vol.1,p.281) are among the best estimates obtainable today for engineering applications . The reasons of this opinion (admittedly difficult to prove , given the unavailability of the "truth"), are illustrated with the help of the photographs obtained by Baligh & Scott (ASCE Jnl.GED, Nov.75,p.1124) on real clays for comparing the distorsion produced by tips of different shape (see Fig.1 below).

There is no question that the clay adjacent to a wedge is significantly distorted . But cones appear to produce "clay juice" . Thus , no matter how sophisticated is the subsequent interpretation and model (coupled/uncoupled linear/non linear etc.) , it seems difficult that mathematics will be able to compensate for the hiatus introduced by the "clay juice".

The variability in Ch interpreted from CPTU in Fig.6 in the referenced Marchetti and Totani paper is probably related , at least partially , to the particular shape of the "curls" around the cone , local quirks of nature ,lenses etc. Total pressure decay in DMTA tests appears more stable, being an integral quantity (similar to the different smoothness of diagrams of pore pressures or settlements below embankments).

Finally , CPTU difficulties associated to filter clogging, smearing and unsaturated stone do not have an equivalent in the DMTA dissipations . The decay of A (total stress) depends substantially on pore pressure dissipation outward of the clay bulb facing the membrane . Such dissipation is virtually unaffected by the permeability of the smear zone (the membrane surface is anyway a non draining boundary).

3. COMPARISON OF Su OBTAINED BY DIFFERENT TESTS

The Authors discuss the ability of DMT to predict Su . Since their Fig.6 , comparing Su obtained by various methods at Madingley , does not include Su by DMT (Su by DMT at Madingley was published in 1979, 7 ECSMFE,Brighton,Vol.2, p.243) , it was considered of interest to superimpose Su by DMT in their plot (Fig.2 below) . In this plot the agreement with triaxial appears very good . (It is noted , in passing, that , for higher Su , the laboratory values may become a doubtful reference).

4. IDENTIFICATION OF SAND FROM Id

The Authors discuss the ability of Id to indicate sand . They express some comments based on 31 tests on the same sand on CC . In this discusser's opinion this methodology is not ideal . In fact the sand they tested has Id near borderline 1.8 , with obvious instability of the outcome Id < or > 1.8 . (Moreover calibration chamber effects can easily shift somewhat Id artificiously) .

Had the Authors tested a say Id≈4 sand , they would have probably found a 100% correct "prediction" as sand . But this would not have been a correct methodology either.

A methodology believed more correct is to compare Id and grain size analysis in many different natural sands sites . Such comparison has generally shown that Id provides material indications accurate enough for many practical applications.

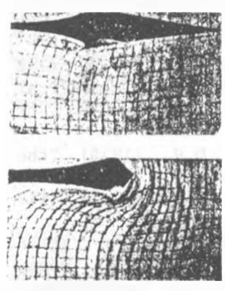

Fig. 1

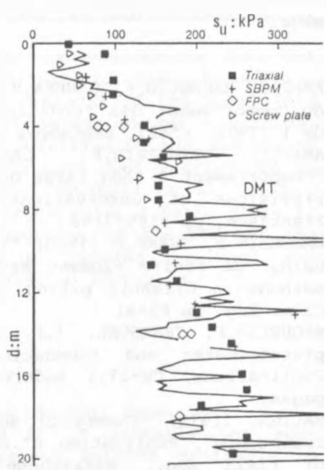

Fig. 2

Discussion

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In discussion session 2 the question was raised if the joined model described by Imre et al /1989/ for one dimensional case could be used as explanation of the total stress variation experienced by piezo lateral stress cell /see Figure 1.a/. The answer is given as follows.

As it has been discussed in the paper /Imre et al 1989/ the joined model consists of a consolidation part-model and a relaxation part-model. The consolidation part can be enlarged to a coupled model with boundary conditions of constant displacement beside of fluid pressure and fluid flow boundary conditions. The system of differential equations for coupled consolidation in axisymmetrical case:

$$W\left(\frac{\partial^2 v}{\partial r^2} + \frac{1}{r}\frac{\partial v}{\partial r} - \frac{1}{r^2}v\right) - \frac{\partial u}{\partial r} = 0 \qquad /1/$$

where v = radial displacement, r = radial distance, u = pore water pressure, $W = 2G\ /1+\frac{\nu}{1-2\nu}/$, G = shear modulus of soil, ν = Poisson' ratio in terms of the effective stress.

$$\frac{\partial^2 v}{\partial r \partial t} + \frac{\partial v}{r \partial t} - L\left(\frac{\partial^3 v}{\partial r^3} + \frac{2\partial^2 v}{r \partial r^2} - \frac{\partial v}{r^2 \partial r} + \frac{1}{r^3}v\right) = 0 \qquad /2/$$

where $L = W\frac{k}{\gamma_v}$.

Boundary conditions are as follows:

/a/ $v(r_o) = \text{const.}$ /b/ $v(r_1) = 0.0$

/c/ $\partial u/\partial r|_{r=r_o} = 0.0$ /d/ $u(r_1) = 0.0$

where r_o, r_1 are the inner and outer boundaries.

Relaxation part of the model has been described by the following equation:

$$\Delta\sigma_r(t) = \sigma_{ro}\, s \log(t/t_o) \qquad /3/$$

where σ_r = radial total stress, s = coefficient of relaxation, t = elapsed time in minutes, the o subscript refers to the initial state.

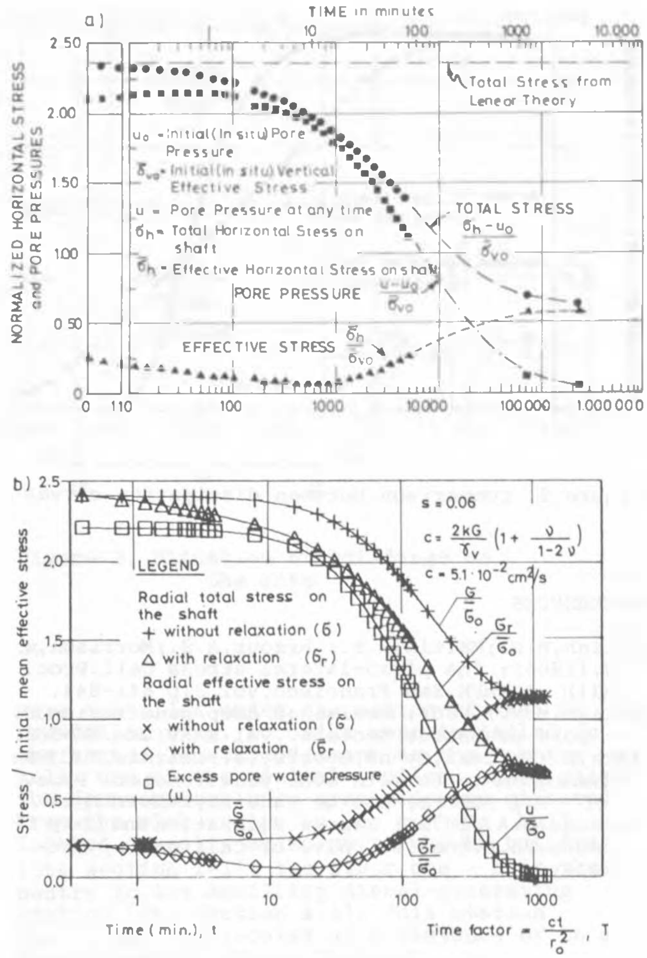

Figure 1. Stress changes in soil after penetration /a/ measured by piezo-lateral stress cell /Baligh et al, 1985/ /b/ determined by the coupled consolidation and by the joined model

System of equation /1/ and /2/ has been solved by finite difference method /implicite scheme/ taking the initial pore water pressure distribution from strain path predictions /Baligh 1986, p 494,Fig.4./ and using a coefficient of consolidation such that the elapsed time in minutes is equal to the time factor. Boundary condition /a/ has been arbitrarily taken. Then the solution at the pile-soil interface has been decreased by the relaxation term $\Delta\sigma_r(t)$ /see Eq 3 / using the principle of superposition and a reasonable-value for s /Lacerda 1972/.

Figure 1 shows that results are in reasonable agreement with tendencies experienced by the piezo-lateral stress cell /Baligh et al 1985/. Figure 2. shows that the dissipation curve is markedly different from solution of Soderberg /1962/ in terms of shape and dissipation time.

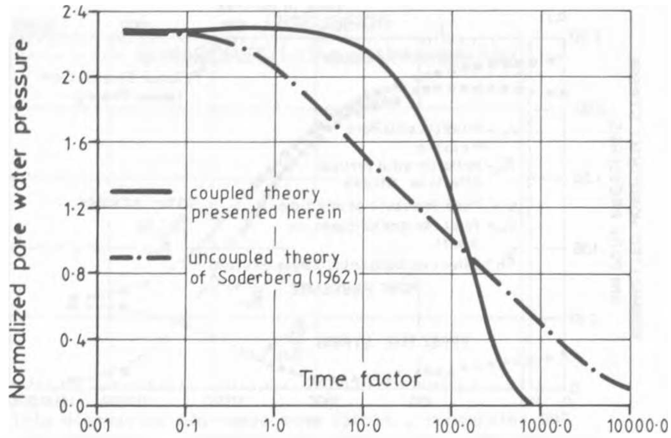

Figure 2. Comparison between dissipation curves

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Discussion

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In the process of our research work,see vol.I p.259 soil dynamometers (earth pressure cells) of the SDKS type were installing on the surface of the ground under the monolithic foundation slab. Before the installation the dynamometers had been screwed up into small reinforced blocks Figure I.

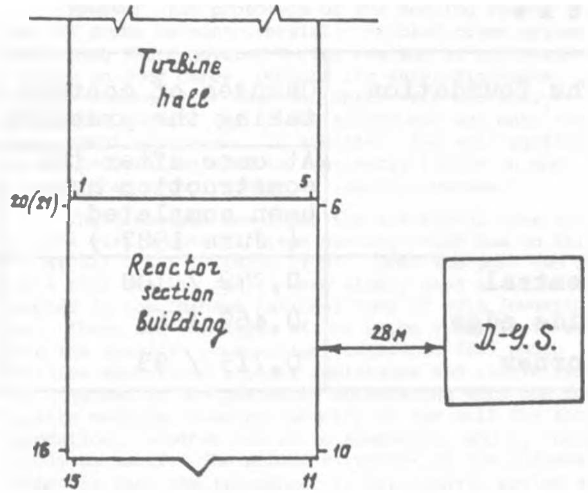

Figure 2. Situation of buildings on the site

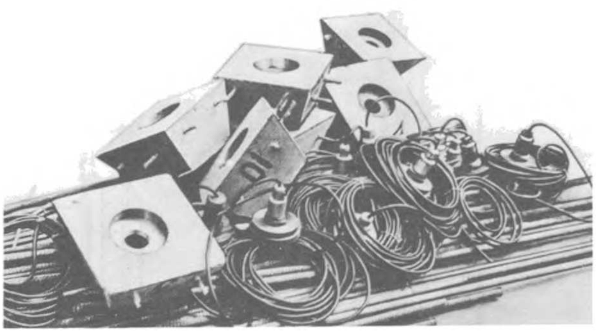

Figure I. Small reinforced concrete blocks

Contact surface of each sell was just flush (on the same plane) with the low side of the block. The blocks were grinding into the ground by means of the light movements. Then concreting of the foundation slab was been accomplished. As soon as the concrete hardening had been over, the blocks and the dynamometers formed the monolith together with the slab construction.

Dynamometers of the SDKS type are quite stiff. While graduating by means of soil pressure their data don't depend on the soil stiffness (on the deformation modulus) due to its density.

Soil dynamometers were installing in the foundation slab corners of the reactor section (RS) building and on I/4 part of its area. At the same time, the principle of duplicating of the measuring points was strictly keeping that is most of the dynamometers had "the double", installed symmetrically to the centre of the foundation slab. 3 earth pressure cells were installed under the centre of the RS foundation slab. IO cells were installed in the corner located adjacent to the turbine hall. One of the central sections,along which the dynamometers were arranged, was directed from the slab centre to the turbine hall, located directly adjacent to the RS building (the section I-I); the other one - from the centre to the auxiliary diesel-generating station (the section A-A). This station (D.- G. S.) is located at a distance of 28 m from the RS building, Figure 2.

The inclination of the RS building reached about 2,3 cm to the end of our investigation is directed to the turbine hall side. This inclination and availability of the foundations of the turbine hall columns led as well to the considerable increasing of the pressures under the slab edge directed to the turbine hall side. Ordinates of the soil pressure pattern under the slab edge,directed to the D. - G. S. are considerably smaller than those, obtained under the edge near the turbine hall.

While calculating the foundation slab it should be remembered that for the foundations consist of the fine-grained soils the inclination takes place in most cases,what should be taken into consideration by averaging of the ordinates obtained experimentally in the sections I-I and A-A.

In the process of exploitation of the RS building ordinates of the pattern of soil pressure had been some changed. The contact pressures under the central part of the foundation slab had been increased but under the side edges and under its corners - on the contrary decreased. The changes of ordinates of the pattern of contact pressures under the foundation slab of the RS building in time are shown in the Table.

T a b l e

The foundation	Changes of contact pressures in time, MPa and per cent, taking the pressure in the centre of the slab for I00 %		
	At once after the construction had been completed (June I987)	In 3 months (September I987)	In I9 months (January I989)
Central	0,772 / I00	0,874 / I00	0,986 / I00
Side edge	0,469 / 6I	0,478 / 55	0,473 / 48
Corner	0,7I5 / 93	0,667 / 76	0,676 / 69

Discussion

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CPT

- better sealing of the probe to be adopted.

Since CPT has been popularized more and more world-widely from time to time, and the uses of it are basically semi-emperical, standardization of CPT is absolutely necessary. To this end, the Technical Committee TC-16 of ISSMFE has been contributing a lot.

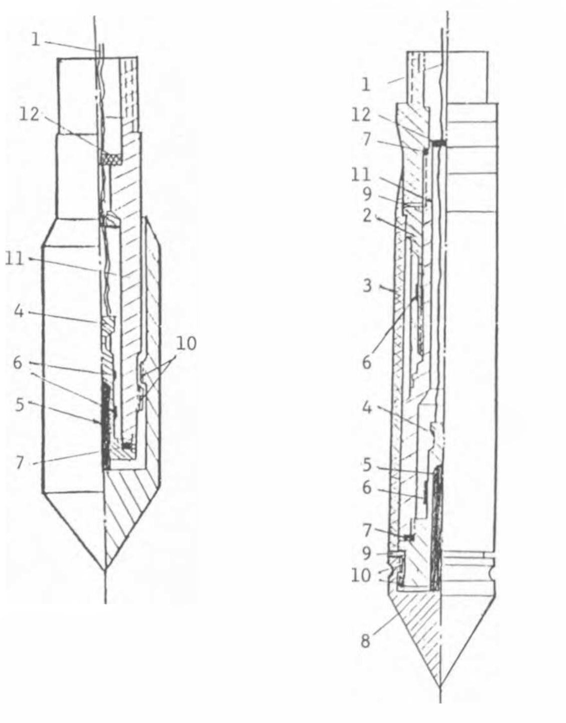

Fig. 1 Schematic presentation of CPT probes (without O-ring) used inChina

1. cable
2. sleeve friction transducer
3. friction sleeve
4. point resistance transducer cylinder
5. point resistance transmitting piston
6. strain guage
7. sealing packing
8. conical point
9. deformation gap for transducer cylinder
10. check thread
11. transmitter rod
12. cable packing

However, the prototype of the sealing system of the CPT probe is controversial. Problem often arises that using the suggested O-ring sealing is not satisfactory in some cases, because its water-tightness stands together with friction which will certainly deduce the sensitivity of the transducer and make the measurement erraneous. In addition, the soil particles during penetration used to inevitably insert in and cause either more friction or ineffectiveness.

The discussor developed the electrical cone early in 1964 with a special fixed sealing which has no friction at all (Fig. 1) (WANG 1979). Over the past two decades this sealing system were widely used in China and adopted in the Chinese National Code of Site Investigation. There are two types of the probe - One is to measure the specific penetration resistance (Ps) which includes specifically point resistance and sleeve friction together as one parameter correlating with the compression modulus, bearing capacity of the soil for footing foundation. Another one is to measure q_c and f_s respectively as usual. The unique structure of the Chinese probes is that the transducer is air-tightly sealed and acting as a perfectly elastic member. The conical point and the friction sleeve can move freely, and by the aid of "check thread" they can be indirectly linked with transducer without any friction to prevent falling down when taking it up from the test hole.

This sort of sealing is seriously suggested to be introduced to the proposed CPT standard.

PRESSUREMETER

- anisotropy of soil should be put into consideration.

As a consequence of the promotion of pressuremeter, more people used to apply it as an insitu testing for settlement analysis, bearing capacity determination and the like of vertical design parameters regardless the anisotropy of soils.

However, it is most important to notice that the pressuremeter test is actually operated horizontally. Serious mistake may be introduced by using the horizontal loading-deformation behavior of the tested soil to calculate its vertical parameters without knowing the anisotropy of soil.

The discussor developed a new pressuremeter in 1979 which consists of wave velocity measuring devices and can be operated during the same measuring. By using special formulation with the measured (Vs) and (Vp) in vertical, horizontal and diagonal direction, two conversion factors of soil anisotropy can be calculated. Then the vertical bearing capacity and the vertical modulus of compression can be obtained by converting from the horizontal ones. (WANG 1985)

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