Session 2: New developments in field and laboratory testing of soils
Séance 2: Nouveaux développements en matière des tests de sols sur terrain et en laboratoire

Closures to discussions to Sessions 2A (In-situ testing techniques), 2B (Laboratory testing – New procedures and data acquisition techniques) and 2C (Centrifuge testing and its application)

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I. LABORATORY TESTING

Discussions by Leonards, Kavazanjian, Mesri, and Leroueil et al. are concerned with topics treated in Section 2.5.2 of the theme lecture entitled Time Effects During One-Dimensional Consolidation. The writers will first respond to comments related to creep behaviour after the end of primary consolidation and then to the more controversial topic of creep (strain rate) effects during primary consolidation.

In reply to Leonards' first question, there is complete agreement that $C_{\alpha e} = de/d \log t$ cannot be measured during consolidation [as also stated by Mesri in his discussion of Eq. (1)]. Hence the answer to whether $C_{\alpha e}/C_c$ is constant for a given clay must come from data obtained after dissipation of excess pore pressures.

Mesri and Choi (1984) present extensive data on seven clays showing that $C_{\alpha e}/C_c$ is essentially constant for both overconsolidated and normally consolidated samples of the same clay in spite of very significant changes in C_c. The writers therefore would not expect anomalous behaviour in the vicinity of the preconsolidation pressure, although precise evaluation of $C_{\alpha e}$ and C_c near σ'_p is difficult, as noted by Mesri and Choi (1984). Further, the writers disagree with Leonards' view that use of R_s/R_{100} provides a more reliable approach for estimating secondary compression settlements in the field, because of its extreme dependence on the load increment ratio (LIR).

Kavazanjian, Leonards, Leroueil et al. and Mesri offer divergent views on the relative importance of strain rate effects during primary consolidation and specifically on the validity of Hypothesis A (negligible creep effects during primary as per Fig.22 in the Theme Lecture) and the uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship independent of the time required for primary consolidation (t_p). These views lead to very different conclusions about the reliability of conventional laboratory tests to predict field settlements during primary consolidation.

Mesri's discussion emphasizes the fact that the EOP uniqueness concept is empirically based on observed behaviour of soils and may involve compensating factors regarding the relative importance of the compressibility and creep components in the first term of his Eq. (1). Although the laboratory data used to support the concept come from measurements of e-σ'_v at essentially zero excess pore pressure (e.g. Mesri and Choi, 1985a), he also uses it to predict field settlements during primary consolidation (e.g. Mesri and Choi, 1985b). The writers agree that the unique EOP concept provides the most reasonable model for most practical field problems even though the actual e-σ'_v paths during consolidation may differ from the EOP relationship. Such divergence was recognized by Mesri and Feng (1986) and is emphasized in the contributions by Kavazanjian and Leroueil et al. discussed below. The writers also agree with Mesri's view that the preconsolidation pressure should be defined as the value of σ'_p obtained from incremental oedometer tests at t_p or CRS tests run with $\dot{\varepsilon}_p$ as defined in his Eq. (2). This definition provides the most useful and consistent approach for settlement analyses and for empirical c_u/σ'_p correlations. Leonards appears to concur with this view.

Kavazanjian considers Hypothesis A and the EOP uniqueness concept as misleading generalizations, based on both practical and theoretical considerations. He cites examples of anomalous pore pressure behaviour attributed to creep effects, but many of the cases involve states of stress deviating significantly from a one-dimensional (K_o) condition. He also correctly points out that Hypothesis A poses theoretical inconsistencies for generalized soil models which incorporate creep effects. Nevertheless, Kavazanjian apparently agrees with the writers' opinion that field consolidation settlements can be reliably estimated from laboratory EOP data for most soils, even though this approach may involve compensating errors.

Leroueil, Kabbaj and Tavenas disagree that Mesri and Choi's (1985a) laboratory test data support the EOP uniqueness concept and with the writers' use of this concept to assert that Hypothesis A is valid. The writers' reply to their three arguments follows.

1) The writers agree that $C_{\alpha e}$ is lower for isotropic than for 1-D consolidation if C_c is also reduced, as was obviously true for the Louiseville clay. Nevertheless, the virgin

compression ratio, $CR = C_c/(1+e_o)$, for the three sets of data presented in Mesri and Choi (1985a) equalled about 0.25 to 0.3 for Saint Alban and San Francisco Bay Mud and ranged from 0.4 to 1.0 for Louiseville. These CR values are representative of medium to very high compressibility clays ($CR = 0.25 \pm 0.1$ and 0.35 ± 0.1 for "ordinary" CL and CH clays, respectively). This fact, combined with the $C_{\alpha e}/C_c$ values quoted in the paper, means that the results should be representative for clays of practical interest.

2) Leroueil et al. argue that the differences in the end-of-primary void ratios predicted from Hypotheses A and B may lie within the experimental scatter. This may be true for Saint Alban but certainly not for Louiseville at void ratios greater that about 1.4 or for the S.F. Bay Mud.

3) The discussion correctly points out that the data in Fig. 6(b) of Mesri and Choi (1985a) contradict theory A since the four sublayers within the 50.8 cm thick Louiseville clay specimen do not follow the EOP $e-\sigma'_v$ relationship. Mesri and Feng (1986) also noted this discrepancy. The writers agree that Hypothesis A represents a simplification of true behaviour, but question whether a ± 4% variation in σ'_p is of great practical concern.

Leroueil et al. compare laboratory EOP stress-strain curves with in situ data from clay sublayers beneath four test embankments. The results show in situ vertical strains averaging 5 times larger than predicted from the laboratory curves (range of 1.4 to 8.9) and hence the discussers conclude "that theory A is not valid and that the EOP curve obtained in the laboratory cannot be considered to present the field behaviour". The writers concur that the results indeed show very dramatic deviations. But are they both reliable and representative of typical in situ behaviour considering the following two features? (Note: Leroueil kindly furnished $\varepsilon_v - \sigma'_v$ plots similar to their Fig.1 for the other case histories).

1) The field values of strain quoted in Table 1 involve very small stress increments beyond the in situ preconsolidation pressure, namely:

$$\text{Virgin } \Delta\sigma'_v = \sigma'_v - \sigma'_p = 1\ 4 \pm 6 \text{ kPa}$$

It is difficult to accurately measure such small changes in σ'_v given the uncertainties usually associated with the applied total vertical stress and the average pore pressure within the sublayer. This potential error, plus possible deviations from truly 1-D in situ strains, may provide a partial explanation for the large differences.

2) The case histories all involve extremly high field compressibilities since small stress ratios equal to $\sigma'_v/\sigma'_p = 1.25 \pm 0.1$ produced virgin strains of $\varepsilon_v = 10 \pm 5\%$. The corresponding values of $CR = 1.2 \pm 0.4$ are several times larger than typical for clays of low to moderate sensitivity.

Creep effects during primary consolidation may indeed be of great practical significance in clays having extremely high compressibility, as concluded by Leroueil et al. But the results presented in their discussion do appear to contradict prior conclusions regarding in situ values of σ'_p for the Eastern Canadian marine clays. Specifically, Leroueil et al. (1983) state: "From a practical point of view, the in situ preconsolidation pressure can readily be estimated from conventional ... oedometer tests ..." (meaning the use of one day curves) and such values obtained from 70 mm piston samplers equal the field σ'_p for Champlain clays having OCR = 1.2 to 2.5. The case histories cited in Table 1 had OCR values within this range, yet the principal reason for the discrepancy between the measured versus predicted strains lies in lower in situ values of preconsolidation pressure. Part of the difference (say 10%) can be attributed to plotting EOP rather than one day curves. Is the rest due to differences in sample quality, reinterpretation of the field data or what?

In conclusion to this portion of the Closure, the writers thank the discussers for their highly simulating contributions and make the following observations.

1) The field cases presented by Leroueil et al. indicate that primary consolidation at stresses slightly greater than σ'_p does not follow Hypothesis A and the EOP $e - \sigma'_v$ uniqueness concept for clays having extremely high values of virgin compressibility. Whether this major difference occurs because of significant creep effects during primary and/or lateral deformations and/or possible errors in effective stress are not clear. In any case, the writers consider the results to be nonrepresentative of more typical clay behaviour.

2) The empirically developed EOP uniqueness concept represents a very reasonable and practical model of soil behaviour for most soft clays, although it may involve compensating errors. Hypothesis A, which also assumes the EOP $e - \sigma'_v$ relationship to apply during primary consolidation, oversimplifies true clay behaviour. Nevertheless, Hypothesis A is still recommended for 1-D settlement problems until another approach can be shown to be both feasible in terms of testing requirements and more reliable for a range of clay types.

II. CLOSURE TO TAVENAS AND LEROUEIL ON "STRUCTURAL EFFECTS ON THE BEHAVIOUR OF NATURAL CLAYS"

Tavenas and Leroueil discuss several issues related to the question: is the behaviour of resedimented Boston Blue Clay (BBC) or the marine clays of Eastern Canada more typical of that for "normal" natural clays of low to medium plasticity? The writers agree that resedimented soils cannot reproduce the complex geological processes experienced by natural clays. But that does not preclude them from having structural effects. The resedimented BBC exhibited an unexpectedly high degree of thixotropy (σ'_p almost doubled after two years storage at constant water content) and it possessed a very pronounced inherent anisotropy as shown by the undrained stress-strain data plotted in Fig. 14(b). On the other hand, its liquidity index versus preconsolidation pressure relationship ($I_L = 1.0$ for 1-D consolidation to 100 kPa) is significantly lower than that for the Eastern Canadian clays tested at Laval University, where I_L typically equals 2 ± 0.7 at $\sigma'_p = 100 \pm 50$ kPa (from data in Leroueil et al., 1979 and 1985).

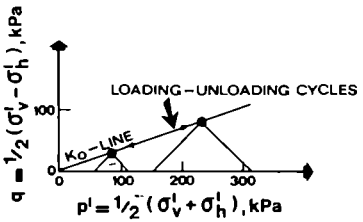
Tavenas and Leroueil compare normalized yield surfaces and undrained stress-strain data for "intact" (Recompression type testing) versus "destructured" (implied SHANSEP type testing) samples to demonstrate that the destructuration caused by SHANSEP leads to a decrease in the size of the yield surface and a "significant reduction in the strength and modulus". The writers have already stated (p.76 of Theme Lecture) that natural overconsolidated clays develop a structure that cannot be precisely duplicated via mechanical overconsolidation in the laboratory. The differences shown for the Saint Alban and Bäckebol clays are indeed significant for purposes of predicting in situ pore pressures and deformations during undrained loading. But these two clays have I_L values of 2.7. and 1.4, respectively, so that one would expect significant changes in their structure and behavioural characteristics when strained well beyond their in situ preconsolidation pressure. The Atchafalaya clay, with I_L =0.8, shows less difference. Moreover, the testing procedures used for the destructured samples did not conform to the recommended SHANSEP technique in the following respects: 1) maximum consolidation stress less than 1.5 - 2 times the in situ σ'_p; 2) used constant $K_c = \sigma'_{hc}/\sigma'_{vc} = 0.65$ during loading and unloading rather than maintaining a K_o condition; and 3) rebounded samples to stresses significantly less than the preshear σ'_{vc}, i.e. samples reloaded rather than unloaded to the final OCR. The last deviation can cause a noticeable decrease in the measured undrained strength. Finally, the results presented in Fig.12 of the Theme Lecture show little differences in undrained strength and modulus between intact (Recompression) and destructured (SHANSEP) specimens from a block sample of an overconsolidated varved clay.

Tavenas and Leroueil state that the much higher quality sampling used for their studies is one reason why structural effects are reported for Canadian clays "while remaining ignored in other soils". However, these special large diameter samples are either too expensive for most geotechnical investigations or are not feasible for very deep sampling and for offshore exploration. Thus most sampling programs must employ procedures that may yield samples of less than ideal quality. Therefore, the real question from a practical viewpoint is the degree to which the SHANSEP reconsolidation procedure will provide reasonable estimates of relevant design parameters, especially when compared with other testing methods commonly used in practice.

To help answer this question, the writers recommended (p.77 of the Theme Lecture) systematic research studies comparing results from Recompression and SHANSEP testing on block samples subjected to varying degrees of disturbance. The program should include clays having a wide range of liquidity index versus preconsolidation pressure relationships and hence varying sensitivities.

III. IN-SITU TESTING

The writers substantially agree with Leonards about the inability of almost all in situ techniques to detect with a sufficient degree of sensitivity the stress and strain history of a deposit. This history influences to a very large extent the magnitude of any type of deformation modulus. Thus, all correlations between penetration test results and deformation



TEST N°	DR %	σ'_{vc} kPa	K_D -	E_D MPa	M_D MPa	E' MPa	Prestraining
118	37	100	1.48	19.9	17.9	19.5	before
			3.55	28.8	44.2	118.3	after
119	40	100	1.50	17.5	14.9	20.7	before
			2.46	24.6	23.0	114.5	after
123	65	513	1.76	33.0	28.1	62.4	before
			2.09	35.5	35.3	243.1	after
142	35	313	1.10	27.0	23.0	31.7	before
			1.35	27.9	23.7	173	after
143	30	111	1.34	14.6	12.5	10.9	before
			1.59	14.8	12.6	114.9	after
144	30	111	1.00	14.2	12.1	16.4	before
			1.30	13.7	11.7	108.9	after

σ'_{vc} = VERTICAL CONSOLIDATION STRESS DURING DMT

Fig.1: EFFECT OF PRESTRAINING ALONG THE K_o-LINE ON DMT RESULTS IN CALIBRATION CHAMBER TESTS ON TICINO SAND

moduli are of questionable reliability. This statement also applies to some extent to the dilatometer (DMT), whose penetration obliterates, at least partially, the strain history of sand. The following experimental data from calibration chamber (CC) tests illustrate this point.

On the basis of a large number of DMT run in Ticino (TS) Hokksund (HS) sands, Baldi, et al., (1986), have found:

NC: $\frac{M_o}{M_D} = 1.3$ to 1.5; OC: $\frac{M_o}{M_D} = 1.9$ to 2.9

where:

M_o = measured constrained modulus, and
M_D = constrained modulus as inferred from DMT according Marchetti (1980).

For NC sand M_o corresponds to the tangent modulus of the last load increment before the DMT penetration; in OC sands, it corresponds to the secant modulus of the entire unload stage performed before DMT penetration.

The difference in the ratios, as reflected by the higher values of M_o/M_D for the OC sands, suggests that dilatometer penetration at least partially cancels the strain history of the sand.

In order to show the importance of this phenomenon, a number of special CC tests have been performed on specimens of TS. The objective was to separate the strain history effect, as re-

flected by the magnitude of the accumulated plastic strain ε^P, from the increase in the horizontal effective stress σ'_h resulting from mechanical overconsolidation. These special CC tests had the following steps:

- one dimensional consolidation of the sand specimen;
- DMT penetration of one half of the specimen while taking dilatometer readings every 10 cm;
- performing several loading-unloading cycles along the K_o-line (see Fig.1);
- completion of the DMT penetration into the CC specimen while taking dilatometer readings every 10 cm.

This testing sequence meant that:

- the first sequence of dilatometer tests was performed in NC sand specimens;
- after cyclic stressing along the K_o-line, the stresses acting on the CC were identical to those which existed before on the NC specimens;
- hence the difference in soil moduli observed before and after prestressing are due to ε^P accumulated during the loading-unloading cycles.

This results of these special tests, summarized in Fig.1, indicate that:

- E_D is almost completely insensitive to the imposed strain history.
- K_D retains some sensitivity to the strain history; this sensitivity seems to decrease with increasing relative density D_R.
- Hence the $M_D = f(E_D, K_D)$. The limited sensitivity of K_D to prestraining is not sufficient to compensate for the corresponding increase of the measured E'.

In the above:

E_D = dilatometer modulus (see Marchetti, 1980),
K_D = dilatometer lateral stress index (Marchetti, 1980),
E' = Young's modulus computed from the formula:

$$E' = \frac{\Delta\sigma'_v - 2\nu' \, \Delta\sigma'_h}{\Delta\varepsilon_v}$$

$\Delta\sigma'_v$ = change in the vertical effective stress
$\Delta\sigma'_h$ = change in the horizontal effective stress
$\Delta\varepsilon_v$ = corresponding axial strain
ν' = Poisson's ratio (assumed equal to 0.3)

Because during reloading and unloading along the NC K_o-line, the CC specimen is no longer strained one-dimensionally, it is necessary to compute E' using the theory of elasticity.

The data shown in Fig.1 explain why even for the same sand, non unique relations between E_D and M_o or E' can exist.
Further verification of the above premise can be found by considering the existing empirical correlations between E' and static cone resistance q_c (Baldi, et al., 1985). From a large number of CPT's performed in the CC on TS and HS, the following results have been obtained:

$$\text{NC: } \frac{E'_{25}}{q_c} = 2.0 \text{ to } 3.7; \quad \text{OC: } \frac{E'_{25}}{q_c} = 8.2 \text{ to } 16.0$$

where:

E'_{25} = drained Young modulus from triaxial CK_oD compression tests evaluated at one-fourth of the deviator stress at failure

The difference in the two ratios is again due to the obliteration of the effect of the strain history of the sand deposit caused by cone penetration. The special CPT's run in the CC following the same procedure already described in

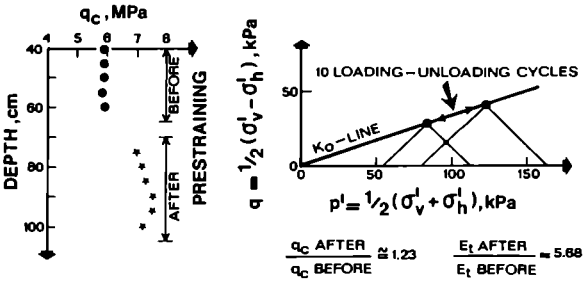


FIG.2: EFFECT ON CONE RESISTANCE OF PRESTRAINING ALONG THE K_o-LINE.
TS, TEST No.71, D_R = 36%, BC-1

connection with the DMT's, support the above statement; see the example shown in Fig.2.
The above experimental evidence shows the extreme uncertainty involved in the evaluation of deformation parameters on the basis of all penetration test results.
The writers share Gudehus' point of view about the difficulties in attempting measurements of σ_{ho} in sands and stiff clays using the available in-situ techniques. It must be emphasized that all push-in devices cause a large increase in σ_h, which with these soil types is compensated only to minor extent by subsequent relaxation.
Therefore the interpretation of σ_{ho} from these tests always requires empiricism.
This point is clearly shown by the contribution to the panel discussion by Marchetti, who presented a tentative correlation for K_o for sands based on DMT results. This correlation allows an estimate of K_o as function of K_D and q_c to be made, with the later normalized with respect to σ'_{vo}. The proposed approach represents a possible improvement of the K_o from DMT results presented by Schmertmann (1983) and discussed in our Theme Lecture. In both cases, the basic idea behind these correlations is to separate the influence which both σ'_{ho} and D_R existing in the ground prior the dilatometer penetration have on the measured K_D. The results of the recent CC tests [Baldi, et al. (1986); Houlsby (1986)] have indicated that an approach of this kind allows at least a qualitative assessment of the K_o from DMT's in natural uncemented predominantly quartz sand deposits. The evaluation of its reliability however requires further validation from field measurements and additional CC tests.

The use of shear moduli as obtained from pressuremeter tests, especially from the self-boring types, is potentially of great practical significance in design. This is particurly true if one considers that:

- In this test all strained elements follow very similar effective stress paths allowing G to be computed from theory.
- The SBPT's during which the pore pressure at the expanding cavity wall is monitored represent the only in situ technique allowing a rational assessment of a fully drained G in sands at medium and large shear strain levels.
- The properly programmed (Wroth, 1982) unload-reload loop performed during a drained SBP expansion test allows the determination of G_{ur} which may be considered to be a measure of the

sand elastic shear stiffness below the current yield locus.

Despite these advantages, the use of G values obtained from SBPT results is still difficult, as pointed out by Lacasse in her contribution to the panel discussion. This situation may be attributed to the following facts:

- The G depends on the current levels of the average mean effective stress σ'_o and engineering shear strain γ existing around the expanding cavity. Therefore in order to relate the pressuremeter moduli to the specific design problem, it necessary to know the current average σ'_o and γ during in situ tests. As shown by Robertson (1982); Robertson and Hughes (1986); Bruzzi et al. (1986); and Bellotti et al. (1986), these values are difficult to assess.
- The G measured during SBPT represents the shear stiffness of soil when stressed and strained in the horizontal direction $G = G_{HH}$, while for the majority of practical problems, the designer is interested in the G_{VH}, the shear stiffness of the soil when stressed and strained in the vertical direction. The G_{HH} and G_{VH} may differ appreciably due to inherent and/or induced soil anisotropy. Therefore the use of the pressuremeter modulus in design requires in many situations a knowledge of the degree of sand anisotropy (G_{HH}/G_{VH}), which is also very difficult to assess.

In these circumstances, the use of pressuremeter moduli in practice must still be made on a quite empirical basis, as mentioned in the discussion by Leach and Thompson. The writers share the opinion of Lacasse that much additional work and experience are needed to improve the situation.

REFERENCES

Baldi G. et al. (1985). Laboratory Validation of In Situ Tests. Geotechnical Engineering in Italy, ISSMFE Golden Jubilee, Associazione Geotecnica Italiana.

Baldi G., Bellotti R., Ghionna V., Jamiolkowski M., Marchetti S. and Pasqualini E. (1986). Flat Dilatometer Tests in Calibration Chambers. ASCE Spec. Conf. In-Situ 86, Blacksburg, VA.

Bellotti R., Ghionna V., Jamiolkowski M., Lancellotta R. and Manfredini G. (1986). Deformation Characteristics of Cohesionless Soils from In-Situ Tests. ASCE Spec. Conf., In-Situ 86, Blacksburg, Va.

Bruzzi D., Ghionna V., Jamiolkowski M., Lancellotta R. and Manfredini G. (1986). Self-Boring Pressuremeter Tests in Po River Sand. ASTM, STP 950, The Pressuremeter and Its Marine Applications, Texas A & M Univ.

Houlsby G. (1986). Research on DMT performed at Univ. of Oxford, UK. Presented at Second Seminar on Calibration Chamber Research, Milan.

Marchetti S. (1980). In Situ Tests by Flat Dilatometer. Journal of the Geotechnical Div., ASCE, GT3.

Mesri G. and Feng T.W. (1986). Discussion to Stress-Strain-Strain Rate Relation for the Compressibility of Sensitive Natural Clays" by Leroueil et al. (1985), Geotechnique, V35, no.2. In press.

Robertson P.K. (1982). In-Situ Testing of Soil with Emphasis on Its Application to Liquefaction Assessment, Ph. D. Thesis, Univ. of British Columbia, Vancouver.

Robertson P.K. and Hughes J.M.O. (1986). Determination of Properties of Sand from Self-Boring Pressuremeter Tests. ASTM STP 950, The Pressuremeter and Its Marine Applications, Texas A & M Univ.

Schmertmann J.H. (1983). Revised Procedure for Calculating K_o and OCR from DMT's with ID > 1.2 and which Incorporate the Penetration Force Measurement to Permit Calculating the Plane Strain Friction Angle. DMT Workshop 16-18 March Gainesville, Fla.

Wroth C.P. (1982). British Experience with the Self-Boring Pressuremeter. Proc. Symp. on the Pressuremeter and its Marine Applications, Paris.

In-situ testing to evaluate the deformation characteristics of residual soils

Une épreuve en place pour l'évaluation des caractéristiques de déformation des sols résiduels

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The residual soils and partially weathered rock in the southeastern U.S. generally maintain the many relic features and structure of the parent rock. Crumbled pieces of rock in a matrix of micaceous silty sand compose the "undisturbed" Shelby tube sample. Local practice depends on the standard penetration test, local experience and empirical correlations to aid in foundation design. Recent research results have provided a better understanding of residual soil behavior and demonstrated the practical utility of in-situ testing.

Testing by Schmertmann (1985) confirms the difficulty of conventional sampling. Using Fluoroscopic techniques to pass x-rays through Shelby tube samples of residual soils, it was possible to record a 360° image of an entire Shelby tube and its contents. Inherent sample structure, sampling induced fractures, and discontinuous zones of gravel-sized pieces were detected prior to extrusion and testing. Using these procedures it was possible to locate zones within each tube that were suitable for trimming and testing. As double core and block sampling procedures are seldom used in local practice, laboratory tests are conducted on severely disturbed samples. Therefore, the observation that conventional test results are not reliable indicators of in-situ performance is not unexpected.

In-situ testing in residual soils is not a new concept but it has been slow in gaining acceptance as a routine alternative to current practice. Martin (1977) suggested a correlation between the pressuremeter modulus and SPT results. Using these correlations in the Atlanta area has generally led to an overprediction of foundation settlements. A more acceptable approach is to use in-situ testing techniques directly and forego the intermediate correlations. The pressuremeter and the flat plate dilatometer are two such tests which offer a significant potential in evaluating the deformation characteristics of residual soils. Five significant observations, based on ongoing research are submitted:

1. The pressuremeter loading response of residual soils at similar standard penetration resistance can be quite different. Using a high pressure pressuremeter which has a caliper measuring system at the center of the expanding membrane, the inflation volume is measured and compared to the measured centerline displacement. It is clear that in some instances the membrane bulges outward significantly and in other cases the centerline is restrained relative to the ends.

2. Comparison testing of the pressuremeter and flat plate dilatometer is very encouraging. As the dilatometer tests a much smaller area than the pressuremeter, variation of modulus is anticipated. The dilatometer modulus correlated reasonably well with the unload-reload modulus of the pressuremeter tests.

3. The results of the pressuremeter tests conducted in the residual soils are generally consistent with previously reported results of tests conducted in soft, fractured rock. Specifically, the ratio of intact rock to rock mass modulus, i.e., unload-reload to initial load modulus, varies typically between 2.5 and 4.0. The mass modulus is generally 2 to 3 times the value measured in the laboratory. The strength calculated from pressuremeter data yield undrained strengths consistently greater than 2 to 3 times the lab determined strength.

4. The high pressure pressuremeter shows tremendous application in the partially weathered rock. Unless extreme care is taken in coring, the recovered core run is usually characterized by 10-40% recovery and 5-10% RQD. The pressuremeter test results indicate that the material is quite competent and often exceed 10-100 times the values used in conventional practice.

5. The preparation of the pressuremeter borehole has a marked influence upon the pressuremeter curve. Unconfined samples of partially saturated residual soils will soften and degrade upon contact with water. The effect of true cohesion can be irrecoverably lost, thus reducing the actual mass strength and stiffness.

REFERENCES

Martin, R.E., "Estimating Foundation Settlements in Residual Soils", ASCE, Vol. 103, No. GT3, March, 1977.

Schmertmann, G.R., "Use of a High-Pressure Pressuremeter for Determination of Properties of Piedmont Residual Soils and Partially Weathered Rock", Masters Research Problem, Georgia Tech, June, 1985.

Discussion of Theme Lecture 2: 'New developments in field and laboratory testing of soils', by Jamiolkowski et al. (Insitu tests)

G.A.LEONARDS

For over 15 years I have been teaching my students, and attempting to persuade practitioners (1), that conventional procedures for estimating the settlement of footings on granular soils--such as the Terzaghi and Peck, or Schmertmann procedures--grossly overestimate the settlements if the soil had been prestressed. In the beginning, this assertion was based purely on physical reasoning, but in 1978 Dr. Lambrechts and I published solid evidence of its validity using the results of miniature cone tests in triaxial samples subjected to known stress histories (2). I am very glad that the extensive and precise tests conducted by Prof. Jamiolkowski and his co-workers, so elegantly reported in the Theme Lecture, have fully confirmed our conclusions, namely: that penetration resistance tests of whatever nature (SPT, CPT, etc.) are inherently incapable of sensing the effects of prestress on the deformation modulus of granular soils, a conclusion that is further reinforced by the results of controlled tests in the laboratory reported in a recent paper by Clayton, et al. (3). As many more natural sand deposits are prestressed than is commonly realized, and as all compacted granular fills are strongly prestressed, it is time to account for the effects of prestressing on the deformation of sand in routine practice.

In view of the foregoing, it is necessary in practice to adopt insitu tests that sense soil compressibility directly. At shallow depths a limited number of plate load tests (correlated to penetration resistance tests) will suffice, but at greater depths the use of such devices as screwplates, dilatometers, self-boring pressuremeters, or measurement of the excess pore pressures generated during undrained penetration testing (piezocone) can be considered. Unfortunately, interpretation of the results from all these latter devices involves either empirical correlations of limited local applicability, and/or analyses based on unproven assumptions. Large calibration chambers help to improve the reliability of these interpretations, but it must be kept in mind that the influence of such factors as different types of granular soils, aging, prior strain history, cementation, etc. remain to be evaluated. For this reason, it is important to continue development of improved techniques for undisturbed sampling of granular soils in order to provide a sound basis for evaluating the results of insitu measurements. Work along these lines is progressing (4), but in the meantime, it is recommended that practitioners select a suitable insitu device and proceed to develop local correlations consistent with field experience.

Defining practicability as an optimum combination of sensitivity, reliability, and cost, my experience suggests that--where plate bearing tests are not feasible--the dilatometer is the most practical tool currently available to correlate with field experience. An improvement that would greatly enhance its usefulness for this purpose would be to use a liquid in place of gas to actuate the pressure on the membrane and to provide for continuous measurements of membrane pressure vs. deformation, both in loading and unloading modes. Even without these improvements, the dilatometer may be equal to the task. It is now up to practitioners to lead the way!

REFERENCES

(1) Leonards, G. A. (1975). Keynote Address - Fundamental Misconceptions in Modern Practice of Soil and Foundation Engineering. 5th Asian Regional Conference, Bangalore, India, Vol. II, p. XVII.

(2) Lambrechts, J. R. and Leonards, G. A. (1978). "Effects of Stress History on Deformation of Sand", Journal, GT Division, ASCE, Vol. 104, No. GT11 (November), pp. 1371-87.

(3) Clayton, C.R.I., Hababa, M. B., and Simons, N. E. (1985). "Dynamic Penetration Resistance and the Prediction of the Compressibility of a Fine-grained Sand - a Laboratory Study", Geotechnique, Vol. XXXV, No. 1, pp. 19-31.

(4) Schneider, H., Chameau, J. L. and Leonards, G. A. (1986). "Undisturbed Sampling of Sand", (to be published).

Practical determination of insitu stress and deformation parameters
Détermination pratique in-situ des contraintes et des paramètres de déformation

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The title for the discussion session was chosen, in consultation with the Theme Lecturer, Professor M Jamiolkowski, because it was considered the essential design parameters of initial stress state and deformation characteristics remain difficult to determine with confidence. This is particularly so in practical, or everyday, soil mechanics.

In introducing the discussion session it was considered appropriate to be a little prevocative in order to stimulate discussion, as well as to provide some direction for the subsequent contributions from the floor. In addition to brief comments presented by the Discussion Leader, a panel of four was invited to present brief comments pertinent to the session theme.

It was submitted to the session that insitu testing provides the greatest hope with respect to the satisfactory determination of the above parameters. However, it was noted that of approximately 30 papers presented to the conference relating to the more general theme of "insitu determination of soil behaviour", only six discuss direct insitu testing methods applicable to the general theory of the discussion session. The papers by Baligh et al, Konrad et al, Campanella et al, and Stefanoff et al, are considered to be of interest in this regard.

Whilst considered to be peripheral to the general theme of the session, some time was also devoted to the advances represented by the piezo-cone as a practical insitu testing device for site investigations and geotechnical design. In order to stimulate discussion a table prepared by Campanella and Robertson (1983), and reproduced here as Table I was presented. During subsequent informal discussion the presentation of this table was criticized, on the premise that it had not been subjected to review. This was countered on the basis of its having been reproduced by Wroth (1984) in his Rankine Lecture.

Notwithstanding possible variations in the possible ranking of the various devices shown, the table illustrates the value of the piezo cone, and in particular the piezo friction cone as an investigation tool.

From the table, it can be seen that the piezo friction cone is surpassed in its general relevance in determining engineering parameters only by the self boring pressuremeter. However, given that the table lists the various insitu tests in perceived ascending order of complexity and unit cost, it can also be noted that the two testing devices are almost at opposite ends of the cost-complexity scale.

Whilst acknowledging this potential of the piezo cone, it was recognised that the location and nature of the porous tip remains a contentious issue. A variety of cones reported in the literature were described to the session, illustrating this point.

It was further noted that it is necessary to correct measured cone resistance for the effects of generated pore pressure, in particular to that generated immediately behind the shoulder of the cone. Location of a porous element immediately behind the cone tip was therefore suggested as being logical. It was nevertheless acknowledged that the gerenated pore pressure shows marked variation along the face of the cone, with different responses in different soil types. Despite this, it was suggested that standardisation of the position, size and nature of the porous stone is essential if the full value of the piezo cone is to be realised, and useful, albeit semi empirical, correlations developed for use in practical soil mechanics.

The panelists invited to contribute to the Discussion Session, and their general discussion topics were:

- Prof. R G Campanella, University of British Columbia, Vancouver. "Factors Which Affect Pore Water Pressure and its Measurement around a Penetrating Cone".
- Prof. S Marchetti, Instituto di Idraulica Facolta di Ingegnèria, L'Aquila, Italy. "Field Determination of K_o in Sand".
- Dr S Lacasse, Norwegian Geotechnical Institute, Oslo. "Insitu Determination of Modulus by Direct Methods".
- Prof. K Stokoe, University of Texas, Austin, USA. "Insitu Determination of Soil Properties by Seismic Measurements".

Each of the above has indicated an intention to publish an abridged version of their discussions in this volume, so their comments will not be repeated here.

Discussion from the floor was initially taken on the piezo cone, with much of the discussion related to the location of the porous stone.

Lunne from N.G.I. presented data indicating that whilst it has commonly been accepted that the pore pressure immediately behind the cone is about 70% to 80% of that at the face of the cone, his work has shown that significantly greater differences can occur. For example, in London clay negative pore pressures have been measured behind the cone. This was consistent with data produced by Campanella. In subsequent discussion others also confirmed this observation in highly over consolidated clays and dense sand.

The apparent variation in the ratio between the observed pore pressure on the face of the cone and that immediately behind it, and indeed its

TABLE I

Perceived applicability of in situ test methods* - update 1982 (after Campanella & Robertson, 1983, taken from Wroth, 1984)

← Increasing complexity and cost	Soil type	Profile	Density D	Angle of Friction ϕ	Undrained Shear Strength C_u	Pore Pressure u	Stress History OCR	Modulus: Young's and shear (E and G)	Compressibility m_v and C_c	Consolidation c_h	Permeability k	Stress-strain curve	Liquefaction resistance
Dynamic cone	C	A	B	C	C	-	C	-	-	-	-	-	C
Static cone													
Mechanical	B	A	B	C	B	-	C	B	C	-	-	-	B
Electrical friction	B	A	B	C	B	-	C	B	C	-	-	-	B
Electrical piezo	A	A	B	B	B	A	A	B	B	A	B	B	A
Electrical piezo/friction	A	A	A	B	B	A	A	B	B	A	B	B	A
Acoustic probe	C	B	B	C	C	-	C	C	-	-	-	-	C
Dilatometer	B	A	B	C	B	-	B	B	C	-	-	C	B
Vane Shear	B	C	-	-	A	-	B	-	-	-	-	-	-
Standard penetration test	B	B	B	C	C	-	-	-	C	-	-	-	A
Seismic cone penetration test downhole	C	C	C	-	-	-	-	A	-	-	-	B	B
K_o blads	-	-	-	-	-	-	B	-	-	-	-	-	-
Resistivity probe	B	B	A	B	C	-	C	C	C	-	-	C	A
Borehole permeability	C	-	-	-	-	A	-	-	-	B	A	-	-
Hydraulic fracture	-	-	-	-	-	B	B	-	-	C	C	-	-
Screw plate	C	C	B	C	B	-	B	A	B	C	C	B	B
Seismic downhole	C	C	C	-	-	-	-	A	-	-	-	B	B
Impact cone	C	B	C	C	C	-	C	C	C	-	-	-	C
Borehole shear	C	C	-	B	B	-	C	C	-	-	-	C	-
Menard pressuremeter	B	B	C	B	B	-	C	B	B	-	-	C	C
Self-boring pressuremeter	B	B	A	A	A	A	A	A	A	A	B	A	A
Self-boring devices													
K_o meter	-	-	-	-	-	-	A	-	-	-	-	-	-
Lateral penetrometer	C	C	B	B	B	-	B	B	C	-	-	-	-
Shear vane	B	C	-	-	A	-	B	-	-	-	-	-	-
Seismic cross-hole	C	C	B	-	-	-	-	A	-	-	-	B	B
Nuclear tests	-	-	A	B	-	-	-	C	-	-	-	-	C
Plate load tests	C	C	B	B	C	-	B	A	B	C	C	B	B

* A, high applicability; B, moderate applicability; C, limited applicability

sign, then led to several contributions suggesting that to fully realise the potential of the piezo cone at least two filter elements should be incorporated in the design. Dr J De Ruiter (Holland) suggested that the marginal cost of this is small, but in view of the size constraints advised his organisation is now using a 15cm^2 cone.

Several contributions were received regarding ideal filter element pore size, pore fluid, material and de-airing requirements. It is hoped the detail of these contributions will be separately submitted for inclusion in the proceedings.

In summary, the writer concluded that the adoption of two filter elements is desirable and should be incorporated as a standard as soon as possible. The concept of a 15cm^2 cone is accepted for its practical basis, but if possible it is considered desirable that the 10cm^2 cone should be maintained if possible so that the wide range of empirical data remains applicable. It is also considered desirable that the size of the cone and push rods be kept as small as possible to maximize the achievable depths of penetration. In many instances penetration of deep sand deposits or residual clays is required and whilst the piezo cone may not be appropriate in these circumstances, use

of as much common equipment as possible is desirable.

After a brief recess, the discussion was devoted to insitu stress and modulus determination. The inter-relationship of these parameters was acknowledged and much of the discussion centred around this.
Dr J Schmertmann (USA) presented a plea that K_o should be measured, not guessed. He suggested the frequent assumption that $\sigma_2 = \sigma_3$ is invalid and showed that measured value of K_o are well outside the values often adopted. It was indicated that a range in K_o of between 0.2 and 6 exists. From this and subsequent contributions the importance of the state of stress to other correlations was emphasized.

Some very interesting discussion was received on the use of pressuremeters in determining design parameters. Recent developments in self boring instruments were discussed by Wang (China) and Bassett (UK). The latter discussion introduced data from X-ray observation of an expanding soil mass (sand) around a pressuremeter. This suggested the behaviour may be governed more by dilatancy and rupture than that indicated by expanding cavity theory. Hopefully this very interesting work will be presented for publication.

Dr J Hughes (Canada) introduced some observations he had made regarding a full displacement pressuremeter, and in particular some surprising results indicating that despite the very great disturbance associated with the introduction of such a device the subsequent pressure expansion curve is similar to that observed from a conventional self-boring pressuremeter test. Whilst this result is perhaps surprising, Hughes emphasized that with most direct insitu testing techniques (specifically the pressuremeter) it should be recognised that real data is being obtained. As such if it doesn't fit with expectations, it should not be dismissed but an attempt made to understand what is occurring. To this end a test with known boundary conditions (eg. full displacement pressuremeter, Self Boring Pressuremeter) has advantages over that with less well defined boundary conditions (eg. Push in Pressuremeter).

Following Hughes' discussion a brief presentation of the Full Displacement (cone) Pressuremeter developed by Fugro, was given by Withers (Holland). It should be noted, however, that the basis of this instrument is understood to have been developed by Robertson and others at UBC, Vancouver, Canada in about 1983.

It had been hoped sufficient time would have been available to allow some discussion on the relevance of direct insitu testing methods to determination of design parameters in residual and non-saturated soils. Happily the discussion was spirited and free flowing, but consequently little time was available for this topic. One invited discussion was received from Ass. Professor R Bachus (USA) on this topic. Bachus is submitting this discussion for publication in this volume, so it does not warrant elaboration here. His work is worthy of note, however, because he is extending the conventional analysis of the pressuremeter to include C-ϕ relationships and discontinuous behaviour.

Such research is considered by the writer to be invaluable for extension of routine insitu testing into the somewhat unknown area of weathered or soft rock.

Numerous discussions were presented to the session which have not been mentioned here, yet assisted greatly in the dialogue which developed. The writer thanks all who contributed for this valuable participation. It is hoped all who contributed will have formally submitted their discussion for publication in this volume.

REFERENCES

Baligh M M, Martin R T, Azzouz A S, Morrison M J. "The Piezo-Lateral Stress Cell", Proc XI ICSMFE, San Francisco, P841.

Campanella R G and Robertson P K (1983). "Flat Plate Dilatometer Testing: Research and Development", Soil Mechanics Report 68, University of British Columbia.

Campanella R G, Robertson P K, Gillsepie D G, and Grieg J (1985). "Recent Developments in In-situ Testing of Soils", Proc XI ICSMFE, San Francisco, P849.

Konrad J M, Bozozuk M, Law K T. "Study of In-situ Test Methods in Deltaic Silt", Proc XI ICSMFE, San Francisco, P879.

Stefanoff G, Ahpatelov D M, Jellev J, Fedorova S. "Determination of the Initial Soil Stress State", Proc XI ICSMFE, San Francisco, P937.

Wroth C P (1984). "The Interpretation of In-situ Soil Tests", Geotechnique, 34 No 4, pp 449-489.

Discussion Session 2A: In situ determination of deformation parameters
Essais in situ: Modules de déformation

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A reliable determination of stiffness in situ is very significant in view of the difficulties to obtain deformation modulus from tests on laboratory specimens. Modulus is even more sensitive to sample disturbance than undrained shear strength. Difficulties are however also encountered in situ, because (1) modulus depends on effective stress and stress history, (2) the in situ test conditions (i.e. stress level, drainage and direction of loading) cannot be controlled, and (3) reference modulus values to assess in situ results are very scarce, or seldom documented. The literature has witnessed confusion as to which modulus one refers to (for example, applicable to drained or undrained conditions), the stress and strain levels the modulus is valid for, and its use in practice. In the following, the in situ test methods to obtain deformation parameters, their usefulness and the reliability of the derived modulus are reviewed. No mention is made of the initial modulus, G_{max}, since the subject is covered by other panelists.

Figure 1 summarizes five test methods in use, the loading applied, possible drainage conditions, and the range of applicability. For each test, a field curve is assumed and the modulus derived from the measurements is given.

For the cone penetration and dilatometer tests, the derived modulus is obtained empirically. In most cases, although the field test is undrained, correlations to the constrained modulus M from a (drained) oedometer test have been preferred because the laboratory M-values are easier to obtain, generally more consistent than other moduli, and easier to judge whether reliable or not if one uses reference data from the literature. Loading direction, however, has very little in common with the manner in which the reference modulus is obtained.

The dilatometer provides reasonable empirical correlations for the constrained modulus in soft clay, overconsolidated clay (within a factor of 2 or 3) and loose sands, as illustrated in Fig. 2. The correlations are, however, only as good as the reference values used. For cone penetration tests, the correlations have also worked "well" in loose sands (Fig. 2c). Little experience exists for other material types.

The screw plate, the pressuremeter and plate load tests have a theoretical basis for interpretation, and each should, at least conceptually, measure deformation modulus in a less disturbed soil than the cone penetration and dilatometer tests (Fig. 1). The screw plate test has worked well in Norwegian sands, based on experience acquired by Janbu and Senneset (1973), Aas (1977) and Lunne et al. (1985). The pressuremeter test (self-boring) has given promising results for loose sand (unpublished NGI results) and more recently in dense sands (Bellotti et al., 1986; Bruzzi et al., 1986). But in both cases, although the devices have been in use 15 years, much further work and evaluated experience are needed.

The most useful modulus from pressuremeter testing is the unload-reload modulus, which corresponds to the "elastic" shear stiffness of the sand. In practice, the modulus has to be related to a shear strain level which is very difficult to assess.

The self-boring pressuremeter has not given such promising results in clays, although the English experience in stiff clays is good. Figure 3 attempts to illustrate the reason for this (Wroth, 1985). For an overconsolidated clay, loading will take place within the yield surface and one measure an "elastic" modulus. For a slightly overconsolidated clay, expansion takes place outside the yield surface and a secant modulus (for example CE) would not correspond to any conventional reference modulus.

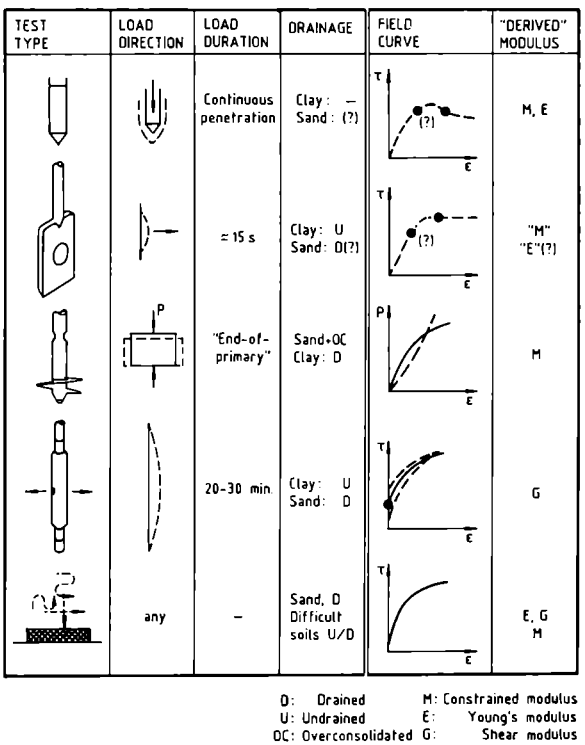

Fig. 1 Description of in situ methods to determine deformation parameters.

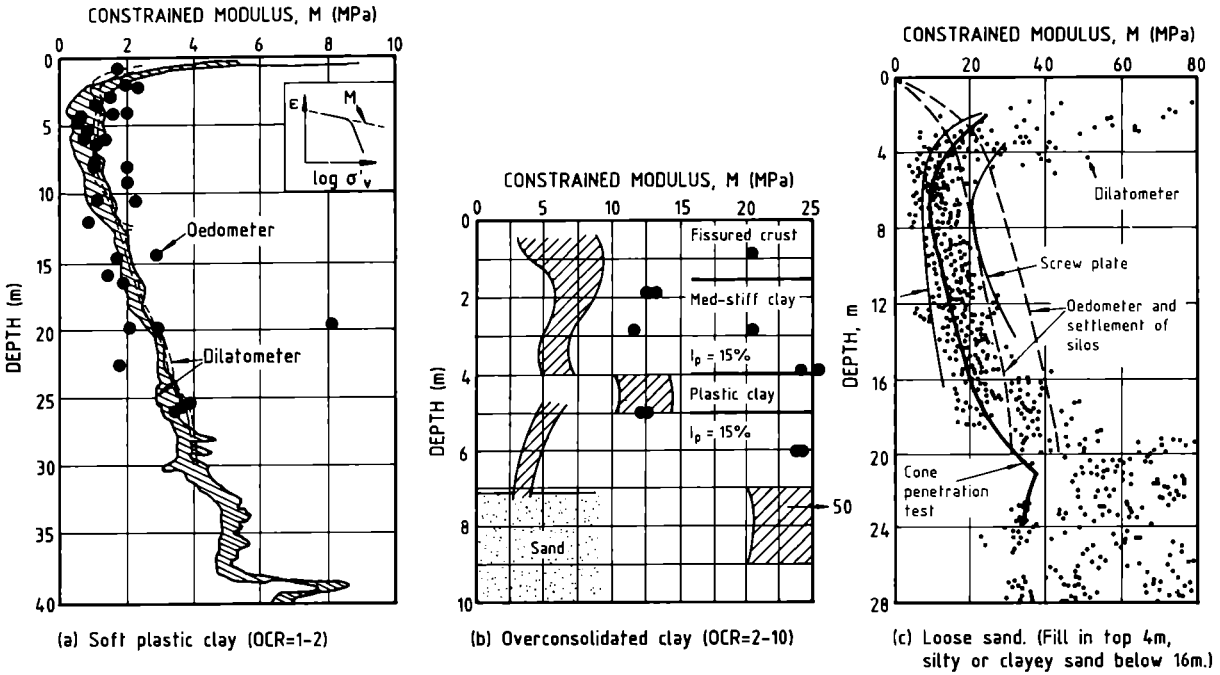

Fig. 2 Constrained modulus from dilatometer and cone penetration tests (Lunne et al., 1983; Lacasse and Lunne, 1986).

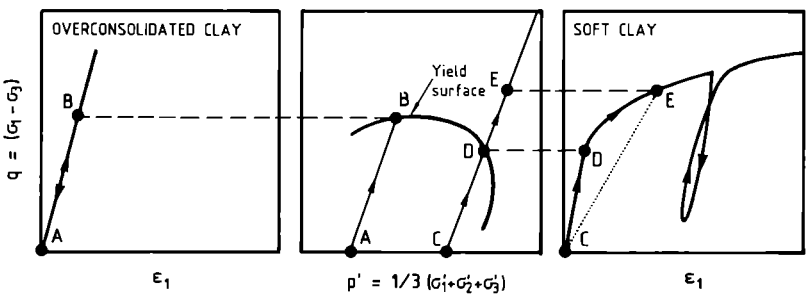

Fig. 3 Pressuremeter expansion in soft and overconsolidated clays.

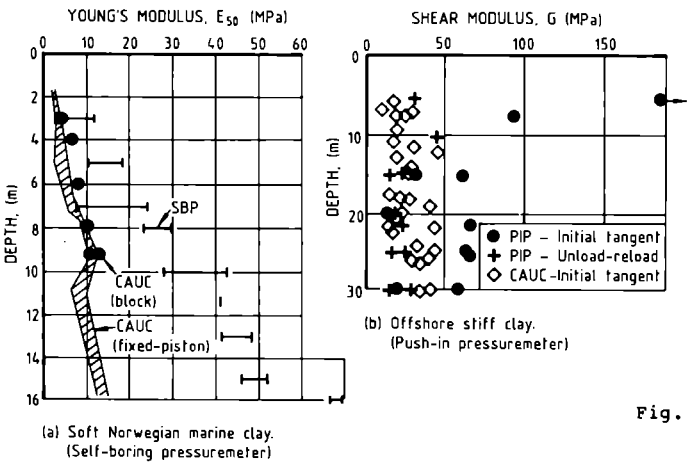

Fig. 4 Young's and shear modulus from pressuremeter tests in clays (Ghionna et al., 1983; Lunne et al., 1983).

TEST METHOD	TEST PROCEDURE/ COST	SOIL TYPE	INTERPRETATION	"EVALUATED" EXPERIENCE
	Simple Cost-effective	Sand	Empirical	Site-specific
	Simple Cost-effective	Sand Clay Silt (?)	Empirical	"Checked" w/ lab data
	Simple "Medium" costs	Sand	Theoretical Empirical	Yes
(SBP)	Expensive Complex	All	Theoretical	Checked w/ lab data
	Expensive, but in some cases necessary	Sand OC clay	Theoretical	Yes

Fig. 5 Evaluation of in situ methods to determine stiffness.

Figure 4 gives examples of pressuremeter measurements of Young's and shear modulus in soft and stiff clays. The trends shown may be reasonable, but reference moduli are also questionable, especially when one considers that the laboratory test specimens are biased by sample disturbance effects.

Plate load tests have the advantage of greater versatility than the other four in situ methods, but they are expensive and are generally limited to very special soil conditions and large projects. Since the plate loads a larger soil volume than the other in situ devices, the method is more reliable in fissured soils for example.

Figure 5 summarizes the cost-effectiveness of the five in situ test methods reviewed. Results from tests interpreted empirically should be used with caution. Because of disturbance during penetration, the cone penetrometer and dilatometer may obliterate any sensitivity of stiffness to stress history.

In all cases, although some experience exists, fundamental evaluations are desirable, to reduce the amount of uncertainty inherent today in the determination of an in situ stiffness.

ACKNOWLEDGMENT

The assistance of several NGI colleagues, especially Tom Lunne's, is gratefully acknowledged.

REFERENCES

Aas, G (1977). Sammenstilling av styrkeparametre og spennningsmoduler for sand bestemt ved in situ målinger og laboratorieforsøk. NGI, Geoteknikkdagen, Oslo, pp. 32.1-32.20.

Belloti, R., V. Ghionna, M. Jamiolkowski, R. Lancellotta and G. Manfredini (1986). Deformation characteristics of cohesionless soils from in situ tests. ASCE Spec. Conf. IN SITU 86, Blacksburg, VA.

Bruzzi, D., V. Ghionna, M. Jamiolkowski, R. Lancellotta and G. Manfredini (1986). Self-boring pressuremeter in Po River sand. ASTM STP, Austin 1986.

Ghionna, V., M. Jamiolkowski, S. Lacasse, C.C. Ladd, R. Lancellotta and T. Lunne (1983). Evaluation of self-boring pressuremeter. Int. Symp. Soil and Rock Investigations by In Situ Testing. Paris 1983. Proc., Vol. 2, pp. 293-301.

Janbu, N. and K. Senneset (1973). Field compressometer - principles and applications, Proc. 8th ICSMFE, Moscow, Vol. 1.1, pp. 191-198.

Lacasse, S. and T. Lunne (1986). Dilatometer tests in sand. ASCE Spec. Conf. IN SITU 86, Blacksburg, Va.

Lunne, T., S. Lacasse, G. Aas and C. Madshus (1985). Design paramaters for offshore sands; use of in situ tests. Int. Conf. Offshore Site Investigation. London.

Lunne T., T.-I. Tjelta and S. Lacasse (1983). Soil investigation for a North Sea gravity platform. Pan Am. Conf. Soil Mechanics and Foundation Eng., 7. Vancouver 1983, Vol. 1, pp. 227-241.

Wroth, C.P. (1985). Personal communication, San Francisco, August.

Use of pressuremeter in soft rock

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We would like to make a contribution concerning the use of the pressuremeter for assessing in situ vertical stiffness of soft rocks. At Allott & Lomax we have built up a wide experience using Ménard 'type' instruments in such materials as Bunter Sandstone, Keuper Marl and chalk which occur extensively below the drift cover of the British Isles. These weak rocks are notoriously difficult to sample and to test in the laboratory. The pressuremeter is probably the only in situ method available that can be carried out at depth in a borehole. from which reasonable assessments of material properties for engineering design can be made.

Despite the difficulties inherent in carrying out pressuremeter tests in these materials some success has been achieved and it has been reported by Leach, et al (1976) and in our Paper No. 1/A/60 to this Conference.

Those familiar with pressuremeter work will know that, while there are many examples in the literature of comparisons between various properties measured by the pressuremeter and the same properties assessed by other test methods, there is a dearth of examples where actual large scale performance has been compared with that predicted solely from pressuremeter test results. The two papers referred to above constitute two such examples and confirm the generally accepted usefulness of this test.

In the case of the assessment of the vertical stiffness of weak rocks from pressuremeter tests carried out in pre-formed pockets, our experience has shown that the modulus deduced from the initial load cycle, G_i, has consistently underestimated the vertical in situ stiffness and that the modulus derived from unload/reload cycles, G_{ur}, has consistently given an overestimate. This experience is at variance with that reported by Dr. Lacasse in her contribution. We have therefore been faced with the problem over the years of how to relate these two 'pressuremeter' moduli to a design situation. In our paper we quote the relationship:

$$\text{Vertical } G = G_i \left[\frac{G_{ur}}{G_i}\right]^{\frac{1}{2}}$$

which we have found to give satisfactory agreement with full-scale performance in these soft rocks. The full-scale performance referred to has been in the main vertical settlement at working stresses measured in tests on large diameter bored piles.

We appreciate that there is no theoretical justification for such a relationship but for practising engineers we believe it provides a useful method for assessing a vertical modulus which will apply to the full-scale performance of structures. No doubt the fact that the pressuremeter test is carried out in a horizontal direction is a significant contributory factor to our experience that the pressuremeter does not give directly a modulus which can be applied in the vertical direction. We suggest that the method given is more realistic than the direct application of stiffness parameters from pressuremeter testing to a design situation.

Reference

Leach, B. A., Medland, J. W., and Sutherland, H. B. (1976). The Ultimate Bearing Capacity of Bored Piles in Weathered Keuper Marl. Proc. 6th ECSMFE, (1.2) 507-514, Vienna.

On the field determination of Ko in sand
Sur la détermination in situ de Ko dans les sables

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SYNOPSIS Existing methods of field determination of Ko in sand are subdivided into direct,semidirect,indirect methods. One particular indirect method (Schmertmann,1983) is discussed in detail, based on the parallel measurement and interpretation of matching pairs of Ko from DMT and q_c from CPT. A compact Ko-chart is worked out using this metod,permitting to read directly Ko from K_D and q_c. Sensitivity diagrams are shown illustrating the different sensitivity of K_D and q_c to Ko and Ø. Additional calibration information, in the form of an additional scale,is added in the Ko chart,summarizing the results of recent extensive CPT-DMT investigations in the Po river sand.

1 INTRODUCTION

The determination of the coefficient of earth pressure at rest Ko in sand is probably one of the most difficult tasks of in situ testing.The action of the measurement itself alters what is being measured.
In a recent paper in the Prof.Osterberg Volume, Schmertmann (1985) lists 17 methods of Ko determination.However most of them are for clays,while only few are applicable to sands.
The laboratory methods suffer from the well known difficulty of recovering samples of adequate quality and are generally considered inadequate for predicting Ko in sand.
At present,only field methods are believed to have the potential for such determination,despite their inherent disadvantage of requiring the insertion of some type of instrument.

2 CLASSES OF FIELD METHODS

From the methodological point of view,the field methods for the determination of Ko may be subdivided into 3 classes:

- -Direct methods
- -Semidirect or back extrapolation methods
- -Indirect methods

2.1 Direct Methods

Direct methods try to measure Ko directly,by attempting insertion with zero disturbance. The only existing instrument able in principle to measure directly Ko is the Self Boring Pressuremeter.Some researchers,however,question this possibility even in principle. E.g. Fahey and Randolph (1985) argue that,in sand,even the penetration of an infinitely thin hollow cylinder would produce significant stress alteration. Because of interlocking of sand grains,some movement of the grains still has to occur to allow passage of the cylinder. According to Fahey and Randolph such movement is sufficient to alter substantially Ko. The question has conceptual value,because by technology we cannot hope to outperform the infinitely thin hollow cylinder.

2.2 Semidirect or Back Extrapolation Methods

This designation is reserved herein to methods which,by back extrapolation,try to figure what the response would have been in absence of instrument. These methods still have a "direct" philosophy because,if the extrapolation is successful, then Ko can be determined separately from other parameters. An instrument well exemplifying this class is the Handy stepped blade (Handy et al. 1982). The principle of this instrument is to measure the lateral stress against sections of the blade of different thickness and to back extrapolate the lateral stress to zero thickness.
Such extrapolation,however,is not free from problems.In particular

- -A blade of zero thickness does not mean no blade,because it still causes movement of the grains,as discussed for the SBPM.
- -The lateral pressure does not always increase with blade thickness,as presupposed by the method.
- -Even thin blades may bring soil conditions far from the origin,and back extrapolation may not work (Fig.1).

2.3 Indirect Methods

Other in situ penetration tests (SPT,CPT,DMT) bring the soil even further up in the stress - strain curve (Fig.1),thus calling into play the entire stress-strain-strength behaviour of the soil.The direct determination of an isolated parameter independently from others is no longer possible. The rigorous interpretation becomes a formidable task,as it requires the complete theoretical solution of the penetration problem and a soil modeling involving soil properties yet to be determined. One simplified way of attacking the problem may involve the following steps:

- -To pursue the determination of a few simple (or simplified) parameters,such as "modulus", "friction angle", Ko.
- -To measure in situ a number of independent soil responses,possibly each one dominated by one of the simplified parameters.
- -To infer from such responses the unknown parameters.

On the other hand factors such as the shape of the stress strain curve,volumetric strain properties,unloading behaviour etc. may have considerable influence on the penetration response. Thus, only appoximate correlations can,at best,be obtained in this way.
The rest of this contribution will deal with one particular method of Ko determination,based on DMT plus CPT results,developed by Schmertmann in 1983,on which the writer had some first hand experience. Hereunder,reference will always be to the following conditions:

- -DMT advanced quasi-statically (jacked)
- -CPT performed with electrical cone
- -Clean sand (drained conditions)

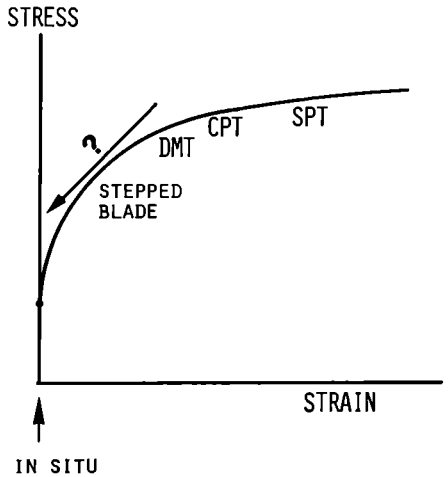

Fig.1 Schematic Diagram Illustrating Qualitatively Insertion Effects.

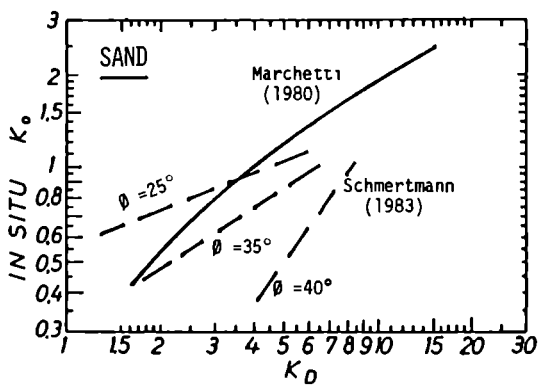

Fig.2 Correlations K_o vs K_D

-Solid curve : Initial 1980 Correlation

-Dashed curves : Schmertmann's 1983 correlations (re-plotted).

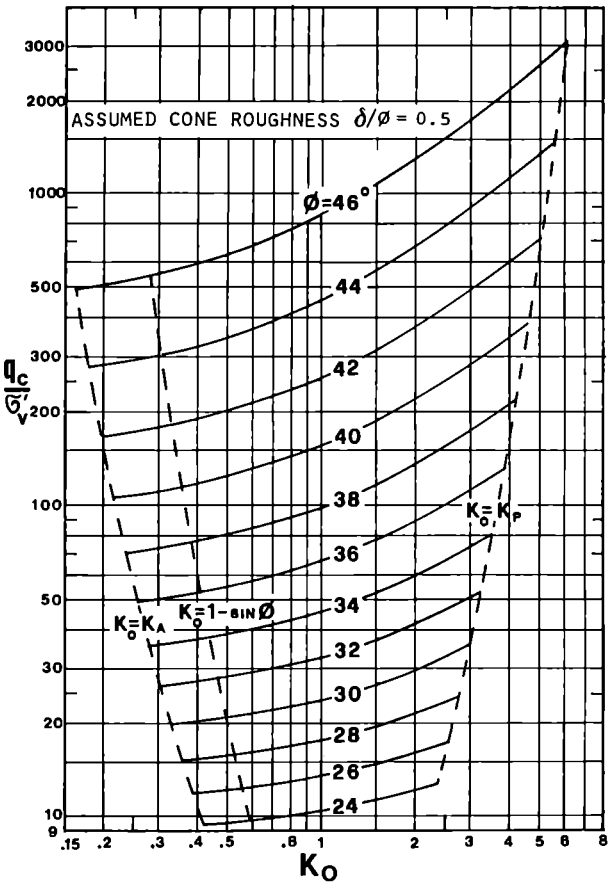

Fig.3 Chart for Interpreting Ø from CPT requiring an evaluation of K_o (worked out by the writer from the Durgunoglu and Mitchell 1975 Equations).

3 THE DMT & CPT METHOD

3.1 The Initial (1980) Ko vs K_D Correlation

When the dilatometer blade penetrates into sand, it causes lateral displacement and, in general, an increase of the pre-existing horizontal stress σ_h to a higher value p_o, measured by DMT. In non dimensional terms, the pre-insertion Ko is increased to K_D. E.g. in a NC sand, where K_o=0.4-0.5, typically $K_D \cong$ 2 to 4 (some 5 times higher).
Early in the development of DMT it was noted that, in OC soils, where Ko is higher, even K_D was higher. Hence the correlation Ko vs K_D was investigated. The solid line in Fig.2 shows the correlation based on the data points available in 1980 (Marchetti, 1980). However most of these data points were for clays and only two for sands. Thus, in sands, the then available data were insufficient to draw any conclusion. Later data referring to sands, obtained from calibration chamber (CC) tests, clearly indicated the necessity, for sands, of introducing in the correlations Ko vs K_D the relative density Dr (or Ø) as a parameter, as Dr played a major role in the correlation.

3.2 The Schmertmann Ko-K_D-Ø Correlation (1983)

Based on the CC data available up to 1983, Schmertmann draw tentative correlations Ko vs K_D with Ø as a parameter, which are superimposed in Fig.2 to the initial 1980 correlation (it may be noted that the 1980 correlation corresponds, according to the 1983 correlations, to loose sands). Such Ko-K_D-Ø correlation was expressed analytically by Schmertmann as follows:

$$K_o = \frac{40+23\cdot K_D-86\cdot K_D(1-\sin Ø_{ax})+152(1-\sin Ø_{ax})-717(1-\sin Ø_{ax})^2}{192-717(1-\sin Ø_{ax})} \quad (1)$$

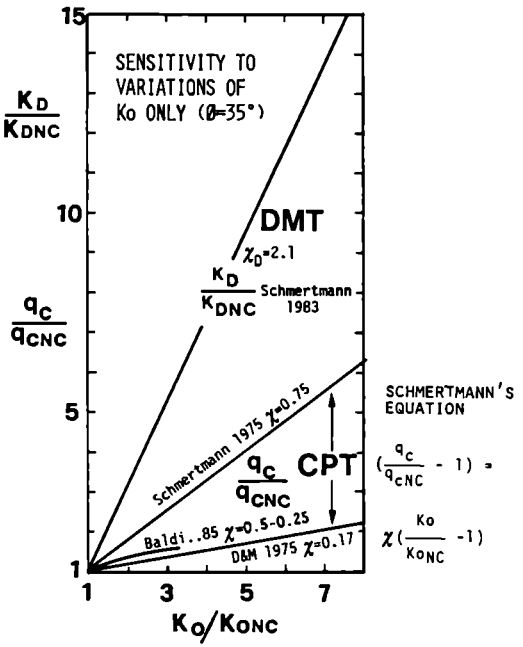

Fig.4 Normalized Sensitivity to K_o of K_D and q_c.

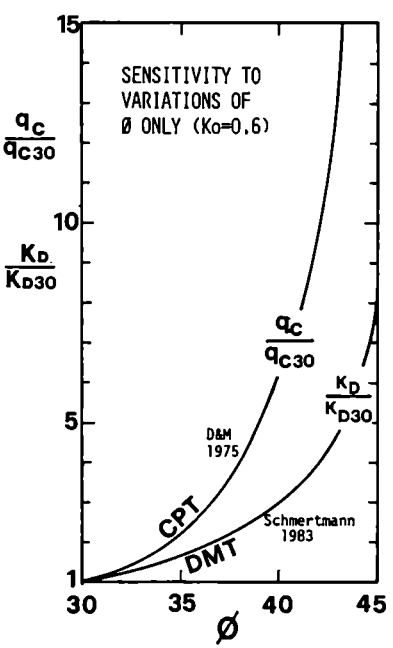

Fig.5 Normalized Sensitivity to Ø of q_c and K_D.

where $Ø_{ax}$ is the angle of shearing resistance as determined by standard triaxial compression tests (same as Ø herein).

3.3 The Schmertmann's Durgunoglu and Mitchell Method (1983)

Eq.1, to be used, requires the knowledge of Ø, usually unknown too. Therefore Schmertmann suggested to measure simultaneously K_D from DMT and q_c from CPT (or q_D, the dilatometer tip resistance), from which both the unknowns Ko and Ø could be simultaneously determined. For such determination Schmertmann suggested to combine Eq.1 with the Durgunoglu and Mitchell (D&M) theory (1975), also expressing q_c as a function of the two unknowns K_o and Ø. Thus he obtained the following system of two equations in the two unknowns Ko and Ø :

$$\begin{cases} K_D = f_1(K_o, Ø) & \text{(Eq.1 above, solved for } K_D) \\ q_c = f_2(K_o, Ø) & \text{(D\&M theory)} \end{cases} \qquad (2)$$

The system 2 can be solved by an iterative procedure described in detail by Schmertmann(1983). Here it is only noted that the D&M equations are matematically complex so that the iterative procedure is generally performed by computer.

3.4 Compact Graphical Form of the D&M Equations

The D&M equations, used in the Schmertmann's method, have been summarized by the writer in the chart in Fig.3. This chart permits to estimate Ø from q_c if an evaluation of K_o is also available.

(The D&M theory predicts, except at very shallow depths, a linear increase of q_c with depth, for given values of K_o and Ø. Thus, except at very shallow depths, q_c can be normalized to σ'_v, thereby eliminating one variable. An analysis of the chart error at shallow depths has shown:

- for z=2m, the maximum difference between $Ø_{D\&M}$ and Ø predicted by the chart is 0.2 degrees
- for Z=1m the maximum error is 0.8 degrees

These errors are found in the most unfavourable zone in the chart, i.e. for Ø=46° and $K=K_p$. For less extreme values of Ø and K_o the error is much smaller and, even for z=1m, less than the chart reading error).

3.5 Sensitivity of K_D and q_c to K_o and Ø

For a better understanding of the Schmertmann D&M method, it is instructive to examine the different sensitivity of K_D and q_c to the two unknown variable K_o and Ø.
The sensitivity graph in Fig.4 shows how q_c and K_D react to changes of K_o alone. Both axes display variables normalized to their NC value. In all cases it has been assumed Ø=const=35°.
Despite some differences in χ according to various authors, Fig.4 clearly shows that K_D is several times more responsive than q_c to changes of K_o.
The sensitivity graph in Fig.5 shows how q_c and K_D react to changes of Ø alone. In the vertical axis q_c and K_D have been normalized to their values for Ø= 30°. In all cases it has been assumed K_o=const=0.6. Fig.5 shows that q_c is considerably more responsive than K_D to changes in Ø.
In conclusion both q_c and K_D depend on both K_o and Ø, but q_c reflects more Ø, K_D reflects more K_o.

3.6 The Compact K_o Chart

As noted earlier, The Schmertmann's D&M method requires complex computations, generally performed by computer. For quick and direct applications the writer has found useful to draw the chart in Fig.6, obtained using the Schmertman's D&M method. The only unknown in the chart is K_o, having eliminated the other unknown Ø. The K_o chart in Fig.6 permits to read directly K_o from K_D and q_c. Once K_o has been estimated, then Ø can be read from the chart in Fig.3.
The chart in Fig.6 may be used readily by engineers unfamiliar with the complex computer programs otherwise needed. The chart may also be helpful for parametric studies and for identifying trends. E.g. it permits to note that some uncertainty in q_c is tolerable without a significant loss of definition in determining K_o.
Even more importantly, Fig.6 provides an interesting alternative format in which the data points may be drawn. In fact:

- The chart expresses K_o as a function of 2 highly reproducible measurements (K_D and q_c), bypassing the intermediate determination of Ø (or worse Dr), representing an unneeded potential source of ambiguity.
- The combined use of the Schmertmann K_o-K_D-Ø correlation plus the D&M theory brings in the inevitable approximations inherent in both, which are probably corrected most efficiently by plotting the experimental results in the format of Fig.6, having, at least partially, a theoretical origin.

It should be noted that, in Fig.6, the q_c/σ'_v = constant curves are not curves of constant Ø (or Dr), because the D&M equations by which q_c is calculated, already account for the dependence of q_c/σ'_v from K_o, besides Ø.
When sufficient data will justify refinements, it will be worth verifying if the use of q_c/σ'^m_v (with the exponent m between 0.6 and 0.8), rather than q_c/σ'_v, may lead to better correlations. It is noted that there are many alternative forms in which Fig.6 may be drawn, e.g. K_D vs q_c/σ'_v with K_o as a parameter on the curves, K_o vs K_D with the ratio q_c/p_o as a parameter (p_o=first corrected DMT reading) etc.

3.7 The Ko Chart vs the Po River Sand Data

An opportunity of evaluating the chart in Fig.6 was offered to the writer by the availability of some 90 pairs of parallel close DMT and CPT (electrical) soundings, in the Po river valley sand. This sand is a recently sedimented, geologically normally consolidated, slightly overconsolidated sand, with the preconsolidation mechanism due to aging and GWL oscillations, with an evaluated OCR ranging from 1.3 to 1.7 and an evaluated K_o ranging from 0.5 to 0.6 (Jamiolkowski et al., Section 3.2.4, 1985).
From the large mass of available data, the writer selected 25 pairs of values of matching q_c and K_D. During the selection the overriding concern was to pick up values from well characterized and definitely corresponding layers (this concern would have been avoided if a multiple sensor probe was available). These pairs of q_c and K_D are listed, with additional information, in Table I and are plotted in Fig.7.
From q_c and K_D values of K_o have been interpreted, using the Schmertmann's D&M procedure (or Fig.6). These values are also listed in Table I and plotted in Fig.7 (C). It is noted:

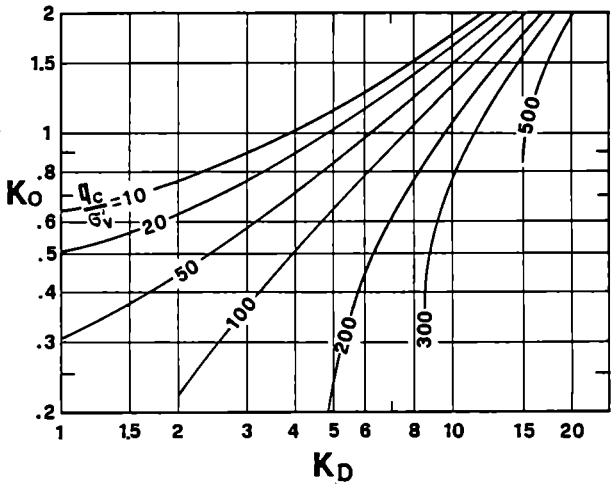

Fig.6 Chart for interpreting K_o from K_D (DMT) and q_c (CPT) worked out by the writer using the Schmertmann's Durgunoglu and Mitchell procedure.

1 The average of the predicted K_o is 0.92, considerably higher than the estimated 0.55.

2 The coefficient of variation of K_o (~30%) is attenuated compared with the coefficient of variation of K_D (~41%). This because, at this site, high K_D are generally accompanied by high q_c/σ'_v, so that, in the Schmertmann's D&M interpretation, part of the responsibility of the high K_D is attributed to a high Ø, and not entirely to K_o.

3 Despite this attenuation, the variation in the interpreted K_o is still considerable. An important question requiring clarification is if such variation reflects:

a Actual variations of K_o in the ground

b Local prestressing (at least in the loosest layers, where prestressing increases K_D and hence the interpreted K_o)

c Local cementation (but no evidence of cementation was noted sofar in this intensely investigated site)

d Other effects

4 The data listed in Table I carry a heavy experimental weight for several reasons:

a They are representative of a much larger mass of accurately taken field measurements

b They have been collected in the field, so they are certainly free from boundary conditions uncertainties, as it is the case with calibration chamber data, especially at high Dr

c If it is accepted that the in situ value of K_o in this deposit is nearly 0.55 (and indeed it is difficult to find reasons why K_o should be appreciably outside the range 0.50 to 0.60), then the 25 data points in Table I are equivalent to 25 CC data points

TABLE I

Results of Parallel CPTs and DMTs in the Po River Valley Sand

#	TEST	Zi (m)	Zf (m)	Zave (m)	σ'_v (bar)	Kd	Id	Ed (bar)	Qc (bar)	fs (bar)	Qc/σ'_v	ϕ_{SDM}	Ko_{SDM}	Ko'
1	4031	5.5	6.5	6.0	.52	6.0	2.5	300	60	.30	115	39.24	.73	.33
2	5025	6.0	7.0	6.5	.58	9.0	2.3	420	95	.45	164	40.22	1.02	?
3	4047	8.0	12.0	10.0	.91	6.5	1.8	350	100	.50	110	38.81	.82	.53
4	5041	10.2	10.2	10.2	.80	5.0	2.7	400	75	.25	94	38.38	.67	.44
5	4010	10.0	12.0	11.0	1.00	7.5	2.1	550	105	.50	105	38.35	.97	.70
6	4027	10.0	12.0	11.0	1.00	12.0	1.8	800	150	1.20	150	39.16	1.46	.84
7	5009	11.5	12.5	12.0	1.02	6.0	2.2	450	110	.50	108	38.87	.76	.50
8	4031	11.5	12.5	12.0	1.10	8.0	2.1	650	130	.60	118	38.81	1.00	.64
9	4005	12.0	14.0	13.0	1.22	7.0	1.7	500	130	.80	107	38.52	.90	.64
10	5014	12.0	14.0	13.0	1.06	10.3	1.6	680	145	1.00	137	39.04	1.27	.74
11	5038	12.0	14.0	13.0	1.05	11.0	1.4	650	160	1.10	152	39.42	1.33	.70
12	5033	12.0	17.0	14.5	1.20	8.0	1.6	650	150	.85	125	38.93	1.09	.70
13	4027	15.0	15.0	15.0	1.45	12.0	1.6	1000	250	4.00	172	39.79	1.42	?
14	5009	15.5	16.5	16.0	1.42	6.0	1.9	550	140	.80	99	38.38	.79	.54
15	5041	15.0	18.0	16.5	1.33	6.0	2.6	600	170	.90	128	39.73	.70	?
16	4002	18.0	18.0	18.0	1.82	12.0	1.4	1015	300	2.30	165	39.59	1.43	.70
17	4002	17.0	19.0	18.0	1.82	10.0	1.7	1050	270	2.05	148	39.50	1.20	.48
18	5009	19.5	20.5	20.0	1.86	5.5	1.9	680	140	1.00	75	37.05	.80	.68
19	5038	18.0	22.0	20.0	1.79	5.0	1.5	480	120	.90	67	36.53	.76	.67
20	5040	20.0	24.0	22.0	2.04	4.7	1.7	600	110	1.00	54	35.35	.78	.70
21	4010	21.0	25.0	23.0	2.23	4.5	1.6	500	150	.80	67	36.68	.70	.60
22	4022	20.0	26.0	23.0	2.12	4.5	1.6	620	140	1.20	66	36.56	.70	.60
23	4002	22.0	26.0	24.0	2.53	3.0	1.8	500	140	1.00	55	35.96	.55	.48
24	5003	25.0	29.0	27.0	2.63	2.6	2.0	500	115	1.15	44	34.63	.55	.50
25	5036	32.0	34.0	33.0	3.00	2.6	2.0	500	115	.75	38	33.80	.58	.52
												Average	0.92	0.60

Legend

Zi, Zf, Zave = initial, final, average depth of layer

σ'_v = vertical effective overburden stress

K_D, I_D, E_D = intermediate DMT parameters

q_c, f_s = CPT tip resistance and sleeve friction

$K_{o,SDM}$ and ϕ_{SDM} = Ko and Ø derived from K_D and q_c using the Schmertmann D&M procedure (or Figs.6 and 3)

K'_o = value of K_o derived from K_D and q_c using Fig.9 with modified scale

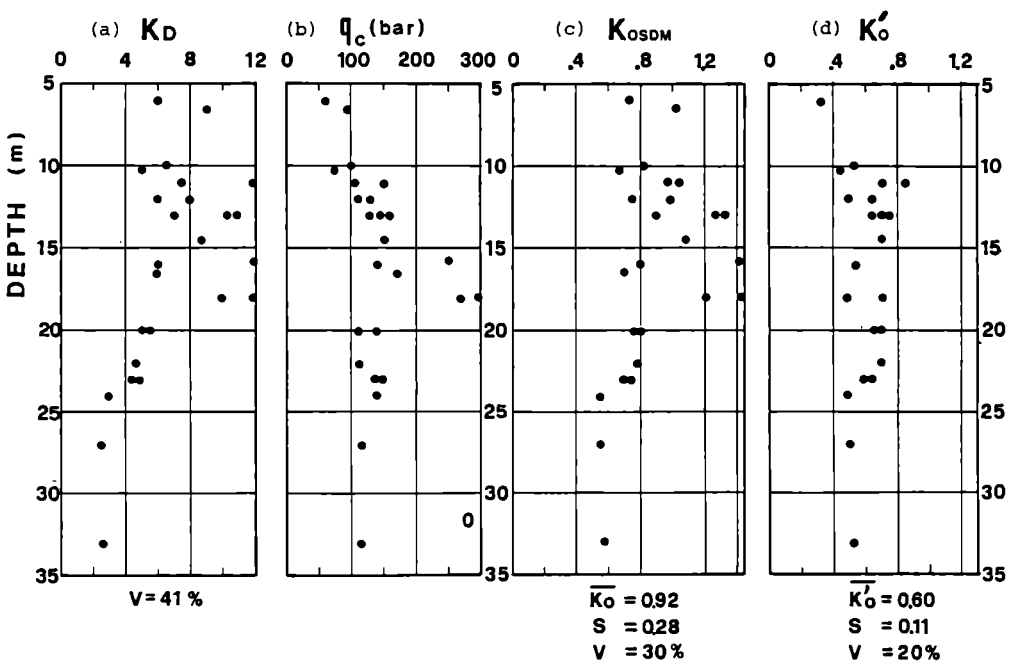

Fig.7 Results of parallel DMTs and CPTs in the Po River Valley Sand

(a) and (b) : Pairs of values of K_D and q_c in corresponding layers
(c) : Ko derived from K_D and q_c using the Schmertmann D&M procedure (or Fig.6)
(d) : Ko derived from K_D and q_c using Fig.9 with modified scale

$\bar{X}$ = average of X S= Standard Deviation V= Coefficient of variation = $S/\bar{X}$

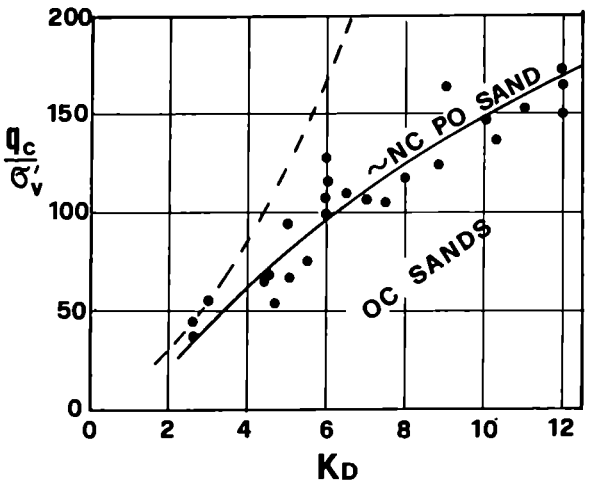

Fig.8 Relationship q_c/σ'_v vs K_D for :
Solid line : nearly NC Po River Sand (OCR 1.5)-least square parabola
Dashed line: as predicted from Fig.6 for K_o=0.55

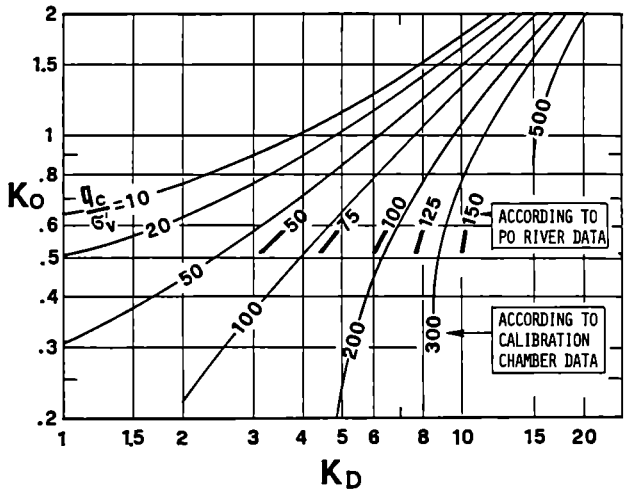

Fig.9 Chart for Interpreting K_o from K_D (DMT) and q_c (CPT),with a dual scale :
(1) According to Calibration Chamber data
(2) According to Po River Field data.

3.8 The Dual Scale Ko Chart

In view of the above,it is possible that the field data may reflect reality more than the CC data on which Fig.6 is based. It was therefore considered of interest to investigate how Fig.6 would modify if it had to accomodate the Po river data. To do this,it was assumed for the deposit the field value K_o=0.55. The horizontal line Ko= 0.55 was drawn in Fig.6. The intersection of this line with the curves in Fig.6 define the correspondance existing,according to Fig.6,between q_c/σ'_v and K_D for Ko=0.55 (dashed line in Fig.8). However the Po river data define such correspondance too (solid line in Fig.8). The difference is considerable,especially for the denser (high q_c/σ'_v)layers. A simple way of modifying Fig.6 for a better agreement with the Po data is to assume that the shape of the curves in Fig.6 is correct,but the q_c/σ'_v values for each curve are those prescribed by the solid line in Fig.8. By so doing,an additional scale for q_c/σ'_v is obtained,as shown in Fig.9.
In all,the Po river data suggest a shift of the curves towards the right,especially for high K_D values (i.e. in the zone where items 3b and 4b in section 3.7 suggest that the field data may be more representative than the CC data).
Fig.7d shows K_o' values that one would obtain from the pairs of K_D and q_c using the modified scale in Fig.9. It is noteworthy that not only has the average K_o decreased from 0.92 to 0.60 (expectable) but also the coefficient of variation has deceased appreciably,from 30% to 20%,lending some support to the modified scale.

In conclusion the K_o chart in Fig.9,with its dual scale,summarizes all the experimental information available sofar -to the writer- and is the one he would use today to evaluate K_o from K_D and q_c. On the other hand it should be emphasized that Fig.9 still requires considerable CC and field verification.

REFERENCES

Baldi,G.,Bellotti,R.,Ghionna,V.,Jamiolkowski,M., Pasqualini,E.(1985). Penetration Resistance and Liquefaction of Sands. Proc. of the XI ICSMFE,San Francisco.

Durgunoglu,H.T. and Mitchell,J.K.(1975). Static Penetration Resistance of Soils,I-Analysis, II-Evaluation of Theory and Implications for Practice,ASCE Spec. Conf. on In Situ Measurement of Soil Properties,Raleigh NC,Vol.I, pp.151-189.

Fahey,M. and Randolph,M.F.(1985). Discussion of "Effect of Disturbance on Parameters Derived from Self-Boring Pressuremeter Tests in Sand", Géotechnique 35,No.2,pp.221-222.

Handy,R.L.,Remmes,B.Moldt,S.,Lutenegger,A.J., Trott G.(1982). In Situ Stress Determination by Iowa Stepped Blade. ASCE Jnl GED,GT11,pp. 1405-1422.

Jamiolkowski,M.,Ladd,C.C.,Germaine,J.T. and Lancellotta,R.(1985). New Developments in Field and Laboratory Testing of Soils. SOA Report, XI ICSMFE,San Francisco,Aug.1985.

Marchetti,S.(1980). In Situ Tests by Flat Dilatometer. ASCE Jnl GED,GT3,pp.299-321.

Schmertmann,J.H. (1983). Revised Procedure for Calculating Ko and OCR from DMT's with I_D>1.2 and which incorporate the Penetartion Force Measurement to permit Calculating the Plain Strain Friction Angle. DMT Workshop 16-18 March,Gainesville,Florida.

Schmertmann,J.H. (1985). Measure and Use Insitu Lateral Stress. Osterberg Volume,Northwestern University,Dept.of Civil Engineering,pp.189-213.

Discussion on Theme Lecture 2: 'New developments in field and laboratory testing of soils', by M.Jamiolkowski et al.

JOHN P.SULLY, INTEVEP, S.A., Venezuela

The theme lecture by Jamiolkowski et al presents a wealth of information on the present progress of laboratory and in-situ testing in geotechnical engineering and will provide a useful reference for those working in this field. This discussion limits itself to the interpretation of the results from piezocone (CPTU) tests.

The Bq parameter suggested by Senneset et al (1982) and given by:

$$Bq = \frac{U_{max} - U_o}{q_t - \sigma_{vo}} = \frac{\Delta U}{q_t - \sigma_v}$$

can be used for soil classification purposes as described in the theme lecture, where the authors concentrate mainly on its variation related to changes in overconsolidation ratio. In addition, variations in Bq can also be linked to stratigraphical changes and variations in undrained shear strength (Sgambatti & Sully, 1985). Futhermore, as suggested by Wroth (1984), the Bq parameter is the only meaningful pore pressure - cone resistance ratio for rational interpretation of piezocone test results.

Senneset et al (1982) suggest values of Bq for varying soil types; these range from less than 0.1 for sand and sandy gravels to 0.8 - 1.0 for soft clays and muds. Piezocone test results for soil types presented in the theme lecture, i.e. soft to stiff clays, have Bq values in the range 0.3 (stiff) to 1.0 (soft) according to Senneset. It is important to note that Senneset calculates the Bq values using the uncorrected point resistance, qc; lower Bq values will be obtained using the corrected point resistance, qt. The Bq parameter can also be expressed as:

$$Bq = \frac{N_u}{N_k}$$

where

$$N_u = \frac{\Delta U}{S_u}$$

$$N_k = \frac{qt - \sigma_v}{S_u}$$

From cavity expansion theory (for a spherical cavity), N_k and N_u have values in the following ranges:

$$N_k = 16 \pm 2$$

$$3 < N_u < 7$$

which would suggest a theoretical range of Bq between 0.16 and 0.5 for the possible range of soil stiffness (G/S_u = 6 - 400).

Considering now the results presented in the theme lecture (Figs. 56-61) for a piezocone with the pore pressure filter located at the tip of the cone (presumably representing conditions for analysis by spherical cavity expansion methods) the range of Bq values quoted is between 0.3 (OCR = 9) and 1.2 (OCR = 1). Based on the previous argument, it would appear that existing simplified cavity expansion models do not adequately represent (or predict) soil behaviour during cone penetration and that, as suggested by Tavenas et al (1982), empirical correlations provide the most appropiate form of interpretation, at least until more comprehensive soil models can be incorporated into the analysis.

It should also be emphazised that sole use of the Bq profile is unlikely to provide a good interpretation of soil conditions except in thick homogeneous clays; rather the interpretation should be performed using all the available information, i.e. induced pore pressure profile with depth, cone factors, dissipation records and Bq profiles, in order to optimize soil identification.

REFERENCES

Senneset, K. et al (1982) Strength and Deformation Parameters from Cone Penetration Tests. ESOPT II, Vol 2, pp 863 - 870.

Sgambatti, J. & Sully J.P. (1985). Piezometer Probe Testing in the Orinoco Clay, Offshore Venezuela. Proc. Int. Sym. Offshore Engrg, Brazil Offshore 1985, Rio de Janeiro, Sept.

Tavenas, F. et al (1982) The Piezocone Test in Clays: Uses and Limitation. ESOPT II, Vol 2, pp 889 - 894.

Wroth, C.P. (1984) Rankine Lecture: The Interpretation of In-Situ Soil Tests. Geotechnique, Vol 34, N° 4, Dec. pp 449 - 489.

Discussion of Theme Lecture 2: 'New developments in field and laboratory testing of soils', by M.Jamiolkowski et al. (Laboratory tests)

G.A.LEONARDS

Time effects in the consolidation of clay have been a subject for discussion at all previous International Conferences. In Tokyo and in Stockholm the General Reporters specifically discussed the issues, and for the Jubilee Conference the Theme Lecturers have again fanned the flames of controversy. In the belief that evidence is available to examine the key issues on a rational basis, I now raise three questions and give my answers to them. In the limited space provided, the answers could be documented only by specific references; the interested reader will need to consult the sources of this evidence, as cited in the references.

Question 1. Is C_α/C_c constant?

Answer: $C_c = \partial e/\partial \log\sigma'$; $C_\alpha = \partial e/\partial \log t$

At the present time, neither of these coefficients can be measured separately during the consolidation process, i.e., during the pore pressure dissipation phase. To answer the question, then, C_α has been measured during secondary compression, i.e., when $d\sigma'/dt = 0$, and C_c was interpreted at the instant C_α was measured from the e vs. σ' data obtained during the consolidation period when $\partial e/\partial \log t$ is unknown. For this reason, it is manifestly impossible at present to answer the question in general on the basis of direct measurements.

In 1961, Leonards and Girault (1) presented data to show that a plot of R_s/R_{100} vs. load increment ratio (LIR) showed an anomalously high value for the load increment that crossed the preconsolidation pressure, p'_c, where $R_s = \Delta e/\Delta \log t$ when σ' = constant (which equals C_α), and $R_{100} = \Delta e$ during consolidation when the load is increased from σ'_o to $\sigma'_o + \Delta\sigma$. Thus, $R_{100} = C_c$ (ave) $\log(\sigma'_o + \Delta\sigma)/\sigma'_o$, and

$$\frac{C_\alpha}{C_c(\text{ave})} = \left\{\frac{R_s}{R_{100}}\right\} \log(1 + \text{LIR}) \qquad (1)$$

Having formulated this relationship, I searched my files on an earlier occasion and evaluated the right-hand side of eqn 1 from consolidation tests on a variety of soil types in the undisturbed, artificially sedimented, and reconsolidated condition. For all practical purposes, I found C_α/C_c (ave) to be approximately constant for a given soil sample except for the load increment that crossed the preconsolidation pressure (2). This is the only definitive statement that I can make about C_α/C_c given the present state of my knowledge.

The foregoing discussion illustrates why it is important to distinguish between $\partial e/\partial t$ during consolidation when $\partial\sigma'/\partial t \neq 0$, which I proposed be called 'secondary consolidation', from its value when $\partial\sigma'/\partial t = 0$, which I have called 'secondary compression' (3). In any case, the relation between R_s/R_{100} and LIR offers a viable means of estimating secondary compressions in the field (4), while the reliability of using C_α directly remains to be established (5).

Question 2. During consolidation (i.e. the p.p. dissipation phase), is the e-σ' relation strain rate ($\dot{\varepsilon}$) dependent?

Answer: This is a crucial question because if the answer is yes, then the compressibility a_v would not only be dependent on the stress level σ' but also on the magnitude of $\dot{\varepsilon}$, which would greatly complicate predictions of both the total and time rate of settlement. In 1964, Leonards and Altschaeffl (6) presented an analysis of the following four idealized cases of consolidation:

1. a single cell filled with water whose piston is supported by a linear spring;
2. 20 such cells in series;
3. a single cell whose piston is supported by a linear spring and dashpot; and
4. 20 such cells in series.

It was shown that while the rates of consolidation of cases 1 and 3 were markedly different, those of cases 2 and 4 were essentially identical, and we pointed out that if the rate of consolidation in lab tests was too rapid, the field rate of consolidation would be grossly underestimated.** But how rapid is "too rapid"? We did not know; however, it should be mentioned that up to that time, most of our consolidation tests were being performed on samples that were 5 cm thick.

Over the years I became increasingly convinced that in most conventional lab tests the strain rate effects during consolidation were too small to be of practical importance. This was based on good agreement between predicted and observed settlement of buildings (4), carefully conducted tests on artificially sedimented clays (7), interpretation of controlled tests on undisturbed samples of different height (3,8), and on re-interpretation of creep consolidation tests on block samples of sensitive clay (9). The conclusion I reached was that e-σ' was independent of $\dot{\varepsilon}$ except for the load increment that straddled the preconsolidation pressure (9). Test results presented by Mesri and Choi to this Conference (10) indicate that the latter caveat is unnecessary if the samples are thick enough so that the strain rate is not "too rapid". Recently, solid evidence has become available that $\dot{\varepsilon} < 10^{-6}$ to 10^{-8} s^{-1} is not "too rapid". This evidence is reviewed in answer to the next question.

Question 3. When will the applied vertical stress exceed the preconsolidation pressure, p'_c?

Answer: p'_c is defined as a condition of "yielding", i.e., the onset of significant irrecoverable strains generally accompanied by a sharp reduction in stiffness. For many years p'_c was considered to be the value obtained from oedometer tests only, but recently measurements made beneath embankments with S.F. = 1.3 or less, have been reported to validate a proposed procedure for obtaining p'_c from oedometer tests. In this connection, it is important to emphasize that yielding will occur at particular combinations of normal and shear stress, which are demonstrably strain rate dependent (11,12,13). However, if the yield locus is plotted in terms of changes in volumetric and shear strain, then yielding can be considered to be independent of strain rate (14). To support this conclusion further, one need only note the

** It was also shown, for the first time, that if a_v was not constant then the degree of consolidation expressed as $1 - u/u_o$ was not equivalent to $\delta/\delta_{ultimate}$.

large number of independent oedometer tests in which p'_c occurs at the same strain--but not at the same stress--as the strain rate during the test is varied (e.g. see 15, 16,17,18). Thus, if the strain rate is too rapid, p'_c measured in the lab will be greater than the field value; on the other hand, if secondary compressions are allowed to occur in the lab that will not be extant in the field, the lab values will be too low. Fortunately, recent tests by Leroueil, et al. (19), Sällfors (20), and by Sylvestri and Morgavi (21), strongly indicate that the strain rate will be too rapid if $\dot{\varepsilon} > 10^{-6}\ s^{-1}$ for silty clays and $10^{-8}\ s^{-1}$ for more plastic clays. Therefore, to obtain reliable values of p'_c from laboratory tests:

a) obtain the best possible samples;
b) model the field stress path; and
c) test samples thick enough so that $\dot{\varepsilon} < 10^{-6}$ to $10^{-8}\ s^{-1}$, without inducing secondary compressions. Tests on samples 2 cm thick do not always satisfy this criterion.

REFERENCES

(1) Leonards, G.A. and Girault, P. (1961). "A Study of the One-Dimensional Consolidation Test", Proceedings, 5th International Conference SMFE (Paris), Vol. 1, pp. 213-218.

(2) Leonards, G.A. (1979). Letter to Prof. R. Mesri dated January 23, 1979.

(3) Leonards, G.A. (1977). Panel Discussion in Main Session 2 - Behavior of Foundations. Proceedings, 9th International Conference SMFE (Tokyo), Vol. 3, pp. 384-386.

(4) Leonards, G.A. (1968). "Predicting Settlement of Buildings on Clay Soils", Proceedings of Lecture Series on Soil Mechanics and Foundation Engineering, Illinois Section ASCE (Jan-May), pp. 41-50.

(5) Leonards, G.A. (1973). Discussion of "The Empress Hotel, Victoria, B.C. Sixty Five Years of Foundation Settlements", by C.B. Crawford and J.G. Sutherland. Canadian Geotechnical Journal, Vol. 10, No. 1, pp. 120-122.

(6) Leonards, G.A. and Altschaeffl, A.G. (1964). "The Compressibility of Clay", Journal, Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM5 (September), pp. 133-155.

(7) Leonards, G.A. (1972). Discussion on Shallow Foundations. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth Supported Structures, Purdue University, W. Lafayette, Indiana, Vol. III, pp. 169-173.

(8) Aboshi, H. (1973). "An Experimental Investigation on the Similitude in the Consolidation of a Soft Clay, Including the Secondary Creep Settlement", Proceedings. 8th International Conference SMFE (Moscow), Vol. 4.3, pp. 88.

(9) Leonards, G.A. (1980). Discussion of "Strain Rate Behavior of Saint-Jean-Vianney Clay", by Y.P. Vaid, P.K. Robertson, and R.G. Campanella, Canadian Geotechnical Journal, Vol. 17, No. 3 (August), pp. 461-462.

(10) Mesri, G. and Choi, Y.K. (1985). "The Uniqueness of the End of Primary (EOP) Void Ratio--Effective Stress Relationship", Proceedings, 11th International Conference SMFE (San Francisco), Vol. 2, pp. 587-590.

(11) Lo, K.Y. and Morin, J.P. (1972). "Strength Anisotropy and Time Effects of Two Sensitive Clays", Canadian Geotechnical Journal, Vol. 9, No. 3 (August), pp. 261-277.

(12) Tavenas, F. and Leroueil, S. (1977). "Effects of Stresses and Time on Yielding of Clays", Proceedings, 9th International Conference SMFE (Tokyo), Vol. 1, pp. 319-326.

(13) Crooks, J.H.A. (1981). "A Qualitative Stress-Strain Time Model for Soft Clays", ASTM, STP No. 740, pp. 685-699.

(14) Leonards, G.A., Chang, M.F., Dewey, R., Mundell, J., Gonzalez, R., and Harris, A. (1981). Discussion of "Yielding of Weathered Bangkok Clay" by A.S. Balasubramanian and Hwang Zue-Ming, Soils and Foundations (Japan), Vol. 21, No. 2 (June), pp. 129-131.

(15) Crawford, C.B. (1964). "Interpretation of the Consolidation Test", Journal, Soil Mechanics and Foundation Division, ASCE, Vol. 90, No. SM5 (September), pp. 87-102, Fig. 3.

(16) Sällfors, G. (1975). "Preconsolidation Pressure of Clay Soils", Thesis, Chalmers University, Gothenburg, Sweden, 231 pp., Fig. 49.

(17) Ref. 10, Fig. 5; Ref. 12, Fig. 5; and Ref. 19, Fig. 7.

(18) Vaid, Y.P., Robertson, P.K. and Campanella, R.G. (1979). "Strain Rate Behavior of Saint-Jean-Vianney Clay", Canadian Geotechnical Journal, Vol. 16, No. 1 (February), pp. 34-42, Fig. 5.

(19) Leroueil, S., Tavenas, F., Samson, L., and Morin, P. (1983). "Preconsolidation Pressure of Champlain Clays, Part II, Laboratory Determination", Canadian Geotechnical Journal, Vol. 20, No. 4 (November), pp. 803-816, Fig. 7.

(20) Sällfors, G. (1986). "Automatic Continuous Consolidation Testing in Sweden", ASTM STP 892, Consolidation Behavior of Soil, Fig. 15.

(21) Silvestri, V. and Morgavi, R. (1986). "Measurement of Lateral Stresses in One-Dimensional Consolidation Tests on Sensitive Clays of Eastern Canada", ASTM STP 892, Consolidation Behavior of Soil, Fig. 9.

K_o behaviour during secondary aging

JOHN L.DAVIDSON

In 1983 Schmertmann posed the following question: Does K_o of a normally consolidated cohesive soil increase, stay the same or decrease during secondary compression (aging)? In a survey of 32 prominent engineers worldwide, all three answers received support (16 increase, 9 same, 4 decrease, and 3 didn't know).

To settle this soil behavior disagreement, the straightforward solution seemed to be to build an apparatus capable both of maintaining the K_o boundary conditions and of measuring the lateral stresses during secondary aging. Results from three such testing programs are reported by Jamiolkowski et al. in their Theme Lecture - New Developments in Field and Laboratory Testing of Soils. In one study K_o increased significantly, in another a small but consistent increase was measured, while in the third no significant chance was observed.

Quite different results have been found in a research program being conducted at the University of Florida. Testing on two soils, Edgar Plastic Kaolinite Clay and Novaculite (a ground silica with more than 98% passing the #200 sieve) have consistently showed decreasing K_o during secondary aging. The test cell used was designed, based on the controlled volume triaxial cell concept. Simplified schematics of the cell and of the system are shown in Figures 1 and 2, respectively.

The 3.00 inch diameter, 0.75 inch high soil sample is sandwiched between stainless steel porous discs and upper and lower platens and is surrounded by a rubber membrane. The vertical loading piston which is supported by linear ball bushings (not shown) has the same diameter as the sample. The vertical stress is provided by a modified Soil Test oedometer. No lateral strain conditions are met by maintaining a constant fluid volume in the small cavity around the sample. This is monitored with a small bore mercury manometer connected to the cell cavity with copper tubing. The pressure required to keep the mercury level constant is the lateral total stress acting on the sample. Both a gas supply and a mercury pot system are used to provide this pressure. When rapid pressure adjustments are needed, as when new loads are first applied, the gas source system is more convenient. During long term testing, the mercury pot system is used. This is not susceptible to the pressure fluctuations and the danger of accidental switch off as is the gas supply. The back pressure set-up also consists of both gas and mercury pot systems.

As shown in Figure 2, the cell chamber pressure is recorded by a regular pressure transducer and on the positive side of a differential pressure transducer. The pore water pressure at the base of the sample is recorded on the negative side of the same transducer. The applied back pressure is also measured by a pore pressure transducer. Tests were performed in a temperature control room, in which the temperature was maintained within a mean deviation about the desired temperature of less than ±0.2°F.

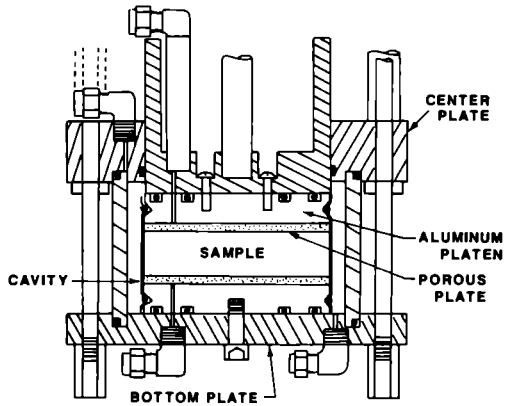

Figure 1 The UF Test Cell

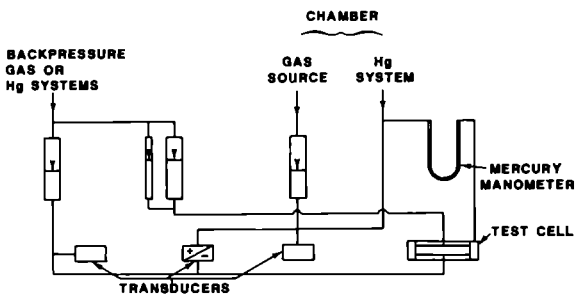

Figure 2 K_o-Testing System

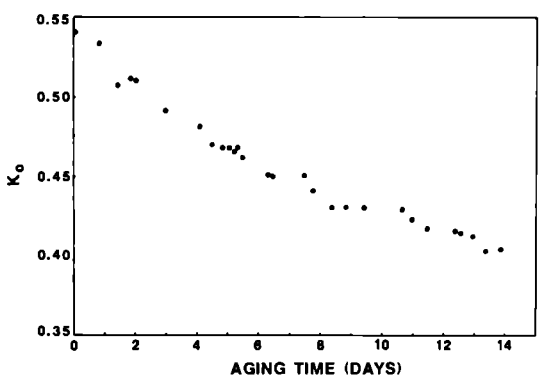

Figure 3 K_o Versus Aging Time

Tests consisted of a back pressure saturation phase, followed by a K_o loading to a vertical effective stress of 2 kg/cm^2 (196 kPa). The sample was then allowed to age, under K_o conditions, for a period of at least 2 weeks and the lateral stress monitored. Small vertical stress increments were then applied to establish the quasi-preconsolidation stress and the stress path back to the virgin curve.

Six tests have been performed on the two most updated test systems. Three were run on Kaolinite specimens and three on Novaculite. Figure 3 shows a typical plot of K_o versus aging time, from one of the Kaolinite tests. In this test K_o decreased from 0.54 to 0.41 over a period of fourteen days, a decrease of 24.3%. In the other two Kaolinite tests, the decreases were 36.1 and 29.9%. The three Novaculite tests gave decreases in K_o of 43.2, 31.7 and 37.2%.

The behavior during secondary aging found in this research is quite different from that reported by Jamiolkowski et al. This may be a consequence of the testing techniques, which were quite different, or of the soils tested - perhaps soil plasticity is an important parameter.

Recent developments in laboratory equipment
Développements récentes des équipements de laboratoire

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I would like to give some background information to the paper 1A26 "Field and Laboratory Measurements of Soil Stiffness." The work described in the paper provides an example of how field measurements, insitu testing and laboratory testing have all played a part in developing our understanding of the stiffness properties of overconsolidated clays.

As a result of my work at the Building Research Establishment, in which I carried out many field measurements of ground movement around structures founded on or in stiff clays and weak rocks, I came to seriously doubt the value of laboratory determined stiffness values. Such values were invariably much lower than stiffnesses deduced from the field measurements - often by an order of magnitude or more.

Hence the approach has been to use values of undrained stiffness (E_u or G_u) derived from the back analysis of field measurements using linear elastic analyses. The reason for employing linear analysis was because high quality research testing on stiff clays had indicated that, at low strains, the stress-strain behaviour was essentially linear (Burland, 1975). This approach seemed to work quite well. For example the class A predictions of the inward displacements of the diaphragm retaining walls of the New Palace Yard car park at the Houses of Parliament (Fig. 1) agreed well with the measurements as shown in Fig. 2 (Burland and Hancock, 1977).

However, certain anomalies began to appear when using linear elasticity. For example, the shape of the predicted surface settlement profile around the New Palace Yard car park differed significantly from the measured profile (Fig. 3). Simpson et al (1979) showed that by using a bi-linear stress-strain law with a high initial stiffness the agreement could be greatly improved as shown in Figs. 2 and 3.

Simultaneously with Simpson's theoretical work, laboratory experimental studies began at Imperial College in which strains were measured locally on the samples. These gave much larger stiffnesses at small strains than obtained from the average strains across the end plattens. The techniques for measuring local strains were greatly improved by making use of electrolytic levels (Burland and Symes, 1982) and Fig. 4 shows a photograph of the latest version.

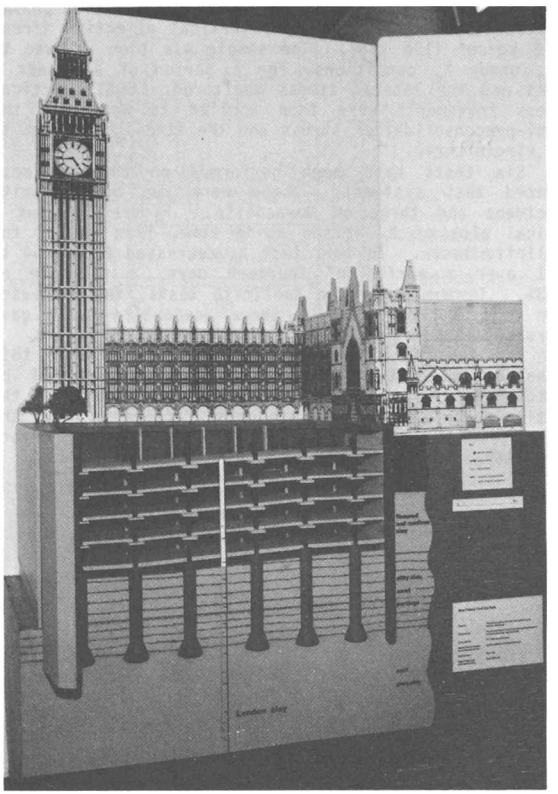


Fig. 1 Model of car park at New Palace Yard

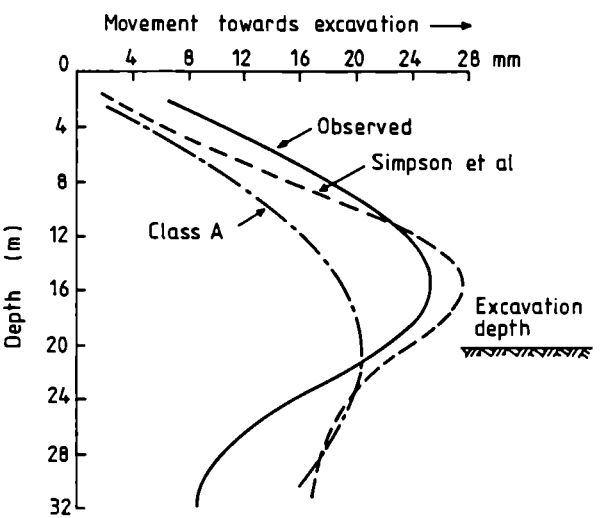


Fig. 2 Predicted and observed displacements of New Palace Yard car park

Fig. 5 shows a comparison between the undrained stress-strain behaviour for external and local measurements of strain and is typical for stiff overconsolidated clays (Jardine et al, 1984). The difference between the two relationships at small strains is very marked. The external measurements give apparently linear behaviour at small strains with E_u/C_u = 188. However, the local measurements give a non-linear response with initial secant values of E_u/C_u greater than 2000 for strains less than 0.01% reducing to more familiar values at larger strains.

It is shown in paper 1A26 that the stiffness-strain relationships derived from local strain measurements on undisturbed samples of stiff

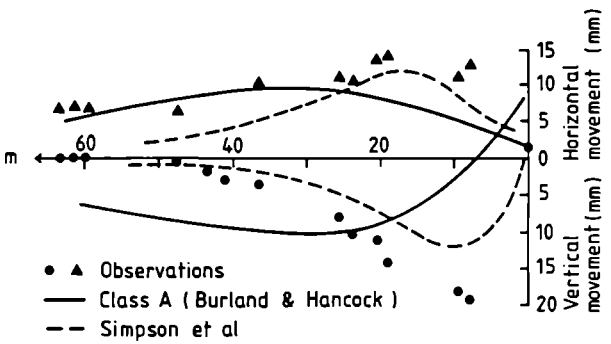


Fig. 3 Predicted and observed ground surface displacements outside New Palace Yard car park

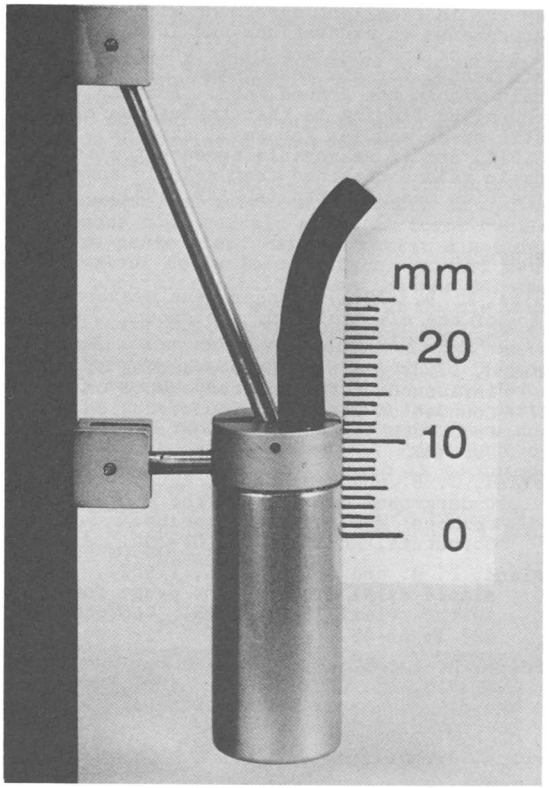


Fig. 4 Electrolevel strain gauge

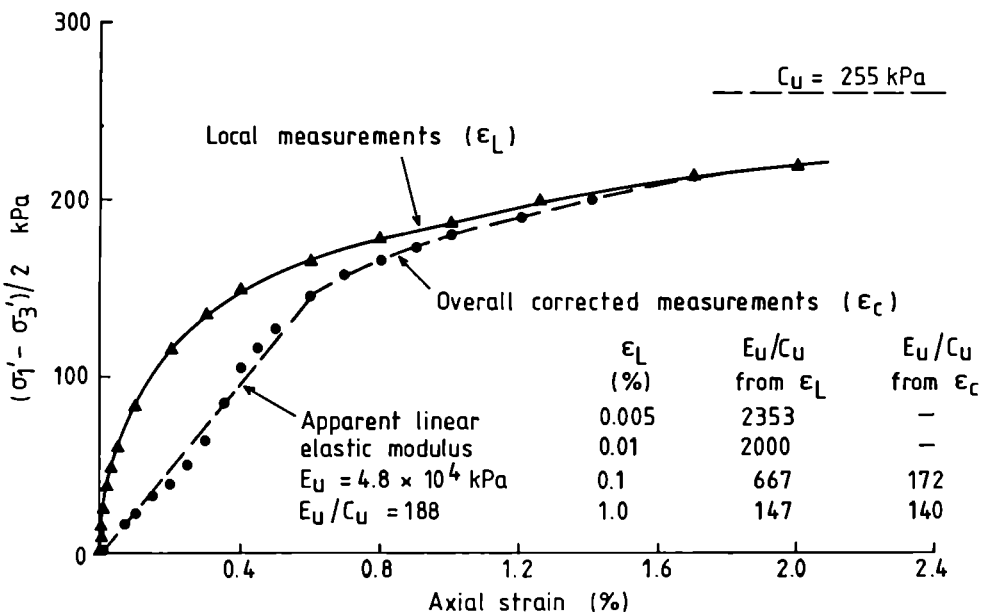


Fig. 5 Comparison of locally and externally measured strains

clay are in excellent agreement with field measurements on excavations and foundations and also on footings and piles which have been instrumented for the measurement of local strain within tne ground mass. Another very encouraging finding is that the values of static shear modulus for small strains (<0.01%) are in reasonable agreement with dynamic values obtained from seismic shear refraction measurements (Abbiss, 1981).

REFERENCES

Abbiss, C. P. (1981). Shear wave measurements of the elasticity of the ground. Geotechnique 31; 1; 91-104.

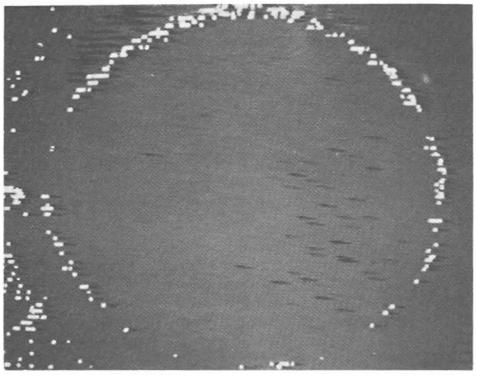
Burland, J. B. (1975). Some examples of the influence of field measurements on foundation design and construction. Proc. 6th African Reg. Conf. SMFE, Vol 2, Durban.

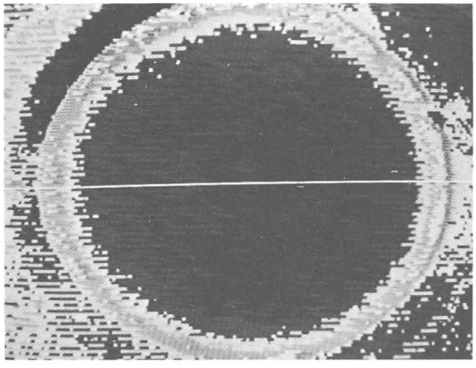
Burland, J. B. and Hancock, R. J. R. (1977). Underground car park at the House of Commons: geotechnical aspects. Structural Engineer 55; 87-100.

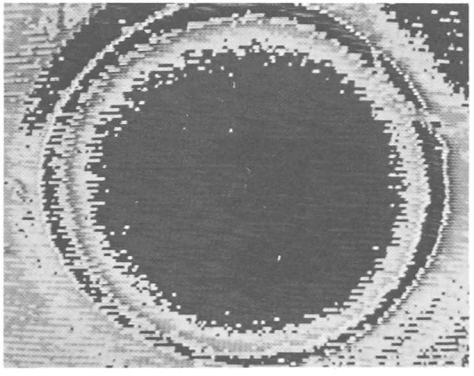
Burland, J. B. and Symes, M. L. (1982). A simple axial displacement gauge for use in the triaxial apparatus. Geotechnique 32; 1; 62-65.

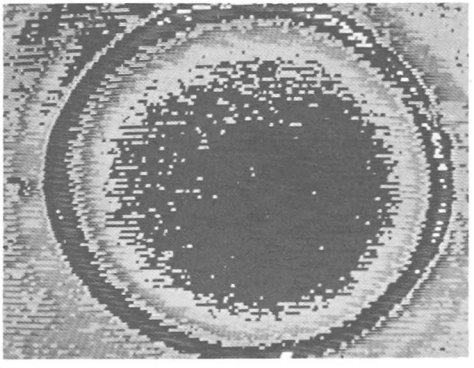
Jardine, R. J., Symes, M. L. and Burland, J. B. (1984). The measurement of soil stiffness in the triaxial apparatus. Geotechnique 34; 3; 323-340.

Simpson, B., O'Riordan, N. J. and Croft, D. D. (1979). A computer model for the behaviour of London Clay. Geotechnique 29; 2; 149-175.









Post-cyclic shear strength of a silty clay (Written discussion)

A.F.L.HYDE

The paper by K. Yasuhara "Undrained and drained cyclic triaxial tests on a marine clay" presents an interesting development of the work by Mayne (1980).

An attempt has been made to apply Yasuhara's equation for post cyclic undrained strength reduction to a series of undrained cyclic loading tests on a silty clay, Keuper Marl. The soil tested had the following properties:

Specific gravity	$G_s = 2.80$
Liquid limit	$w_\ell = 36\%$
Plastic limit	$w_p = 19\%$
Compression index	$C_c = 0.168$
Swelling index	$C_s = 0.048$
Critical state parameter	$M = 1.074$

One way cyclic triaxial tests were carried out on samples normally consolidated to 700 kPa and 38 mm x 75 m in size. Tests were carried out undrained at varying maximum deviator stresses at a frequency of 0.1 Hz. After 10,000 cycles the cyclic loading ceased and samples were taken to failure under strain-controlled monotonic conditions. Figure 1 shows the Hvorslev surface and critical state determined from single stage monotonic tests without cyclic loading. The effective stress paths are shown for normally consolidated samples and both axes are normalised with respect to the equivalent pressure, p_e, which for the normally consolidated samples is the same as the effective consolidation pressure, 700 kPa. Cyclic loading induced a permanent pore water pressure in the samples resulting in a state of overconsolidation. Figure 2 shows the effective stress paths for the post cyclic monotonic tests which clearly indicate a strength reduction with failure apparently occurring on the Hvorslev surface as would be expected for overconsolidated soil, Hyde and Ward (1983). There is considerable scatter in the results but the average permanent pore water pressure developed under cyclic loading is 132 kPa and the mean value of $\frac{q}{p_e}$ at failure was reduced to 0.7.

Rewriting Yasuhara's equation (2) as

$$\frac{q \text{ (post cyclic)}}{q \text{ (single stage)}} = \left[\frac{1}{1 - \frac{(\Delta u)_{cy}}{p_i}}\right]^{\frac{\Lambda_o}{1-\lambda} - 1}$$

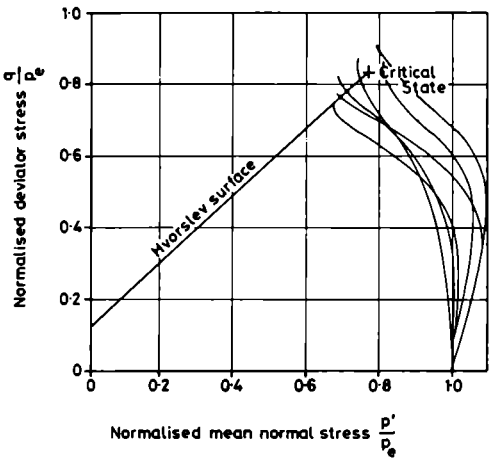


Figure 1 Single stage monotonic effective stress paths

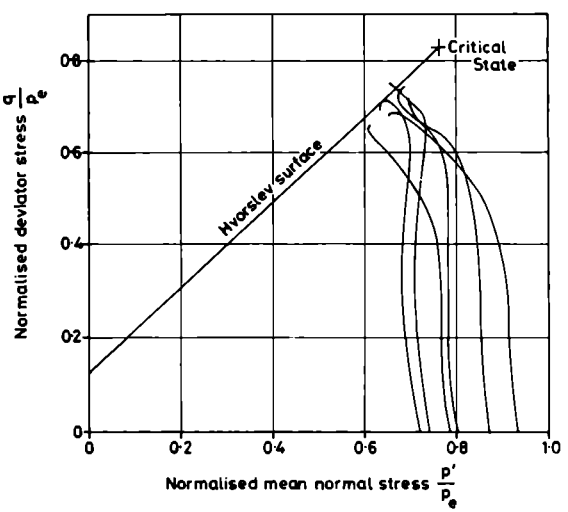


Figure 2 Post-cyclic monotonic effective stress paths

gives a back figured value for Λ_o of 0.17. This is much lower than the value of 0.720 obtained by Yasuhara. This may well be due to the differing properties of the clays under comparison and also the larger apparent loss in strength for Keuper Marl under cyclic loading.

References

Hyde, A.F.L. and Ward, S.J. (1983) "Yield boundary for cyclic loading of a silty clay", Proc. Conf. Geotechnical Practice in Offshore Engineering, Austin, ASCE, pp. 99-110.

Mayne, P.W. (1980) "Cam-clay prediction of undrained strength", Proc. ASCE, Vol. 106, No. GT.11, pp. 1219-1242.

Discussion Session 2B: Testing of 'special' soils: Ocean floor soils
Soils 'spéciaux': Sols marins

S.LACASSE, Norwegian Geotechnical Institute, Norway

Ocean floor soils cannot really be classified as "special" soils, but the testing techniques from onshore practice have been adapted to problems specific to offshore conditions. Four topics are briefly addressed: (1) sampling disturbance and means of qualitative assessment; (2) comparisons of laboratory tests run offshore and onshore; (3) testing of shallow samples and (4) measurement of initial modulus G_{max} by piezoceramic bender elements.

Table 1 summarizes current North Sea practice for sampling and laboratory testing of soil samples from the site of a proposed major offshore structure. Great improvements have been achieved in undisturbed sampling. Percussion or hammer operated wireline sampling, first developed in the Gulf of Mexico, was originally most common practice. Push sampling, using the weight of the drill string, provides samples of better quality, but it is restricted to fairly soft material, due to the limited penetration force. Sea-bed jacking units help increase penetration force. Fixed piston samplers are now available for offshore sampling of soft to stiff material (s_u up to 200 kPa).

For important site investigations, the sample can be radiographed, extruded, classified, photographed and tested already on board the drill ship. This enables

- continuous updating of the soil properties in order to modify the sampling/testing program
- immediate control of disturbance of samples
- stress history and strength profiles available on board

1. Sampling disturbance

Figure 1 shows the compressibility curve of two offshore soft clay specimens from a depth of 4 m below mudline, sampled in 1981 (push samples) and in 1983 (fixed piston samples) at very large water depths. Sampling disturbance and the possibility of high quality sampling (confirmed by radiography) are clearly demonstrated, even at water depths as large as 350 m. The comparison is however imperfect, because of different weather conditions, drill ships, heave compensating equipments and geotechnical contractors.

Andresen and Kolstad (1979) suggested the following criterion to evaluate test quality from

TABLE I Laboratory testing of offshore samples

Sampling
- Percussion (usually poor quality)
- Push } (high quality)
- Fixed-piston } (high quality)
- Gravity core (top 3-6 m, some cases 30 m, good quality)

On-board laboratory tests
- Radiography
- Description
- Geology
- Geochemical analyses
- Index strength tests
- Oedometer
- Direct simple shear tests (static)
- CIU and CAU triaxial tests (static)

On-land laboratory tests
- As above, often in light of X-ray photographs
- Cyclic strength tests
- Resonant column test (G_{max})

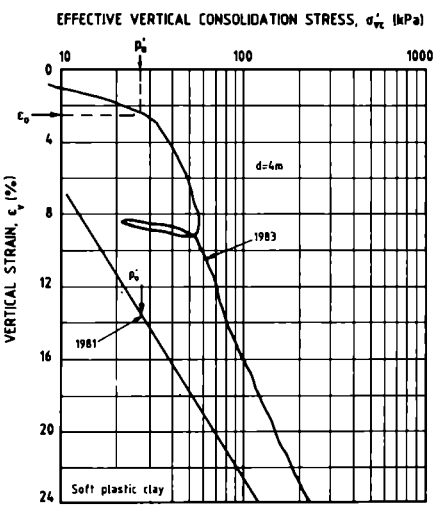


Fig. 1 Oedometer tests on disturbed and undisturbed specimens of soft plastic clay.

the volumetric strain of soft shallow (< 20 m) onshore clay specimens, measured during one-dimensional consolidation to the in situ stresses.

ε_o (%)	Test quality
< 1	Very good - excellent
1-2	Good
2-4	Fair
4-10	Poor
> 10	Very poor

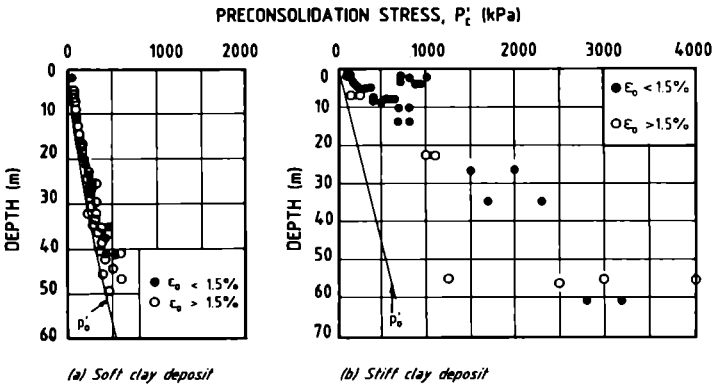


Fig. 2 Preconsolidation stress from high quality offshore clay specimens (NGI files).

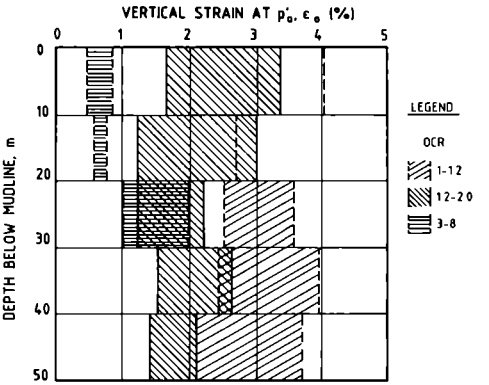


Fig. 3 Strain at p_o' for high quality offshore clay specimens (Kleven, 1985).

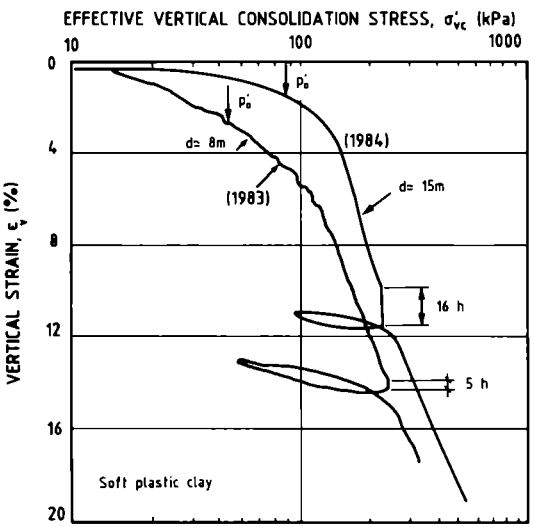


Fig. 4 Comparison of constant rate of strain oedometer tests offshore (Sandbækken et al., 1985).

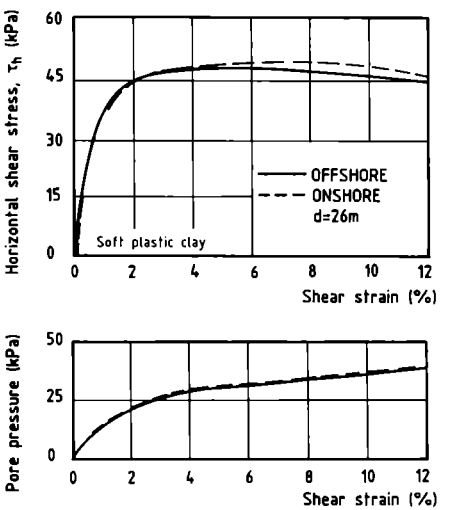


Fig. 5 Comparison of direct simple shear tests offshore.

If one uses a vertical strain, ε_o, of 1.5% as the maximum allowable strain on two example offshore sites, one would reject 78% of what is considered high quality test data in the case of a soft clay deposit and 20% in the case of a stiff clay deposit, as shown by Fig. 2. (The preconsolidation stresses were obtained by averaging Casagrande's and Janbu's method.) A criterion of 1.5% may thus be too strict in practice. Data compiled by Kleven (1985) at NGI (Fig. 3) show that one should consider allowable strain as a function of stress history (ratio p'_c/p'_o) and depth.

2. Offshore and onshore laboratory tests

Advanced laboratory testing of specimens on board the drill ship has developed especially since 1983 when oedometer, simple shear and CAU triaxial testing became more common in North Sea practice. Equipment also improves from year to year as exemplified by Fig. 4. Two constant rate of strain tests were run on soft clay specimens from the same site, depths 8 and 15 m. Modifying lever arm loading to uniaxial loading and improvements in the data logging techniques resulted in much smoother compressibility curves (Sandbækken et al., 1985).

The strengths obtained from tests run offshore and onshore compare very well, as shown by only one of the many examples available (Fig. 5).

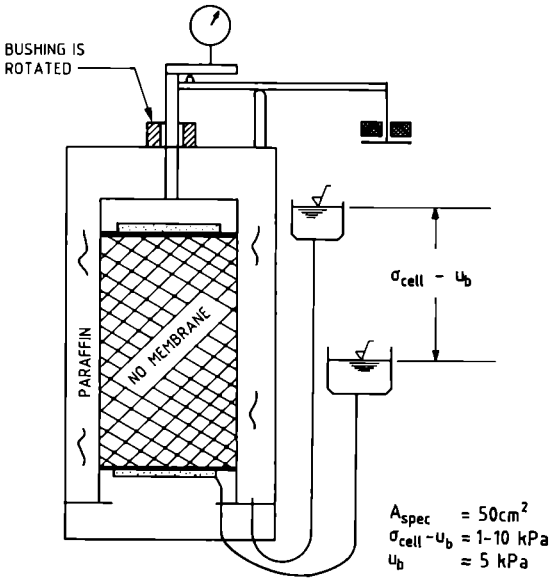


Fig. 6 Example of stiesses for triaxial testing of very shallow specimens (Berre, 1985).

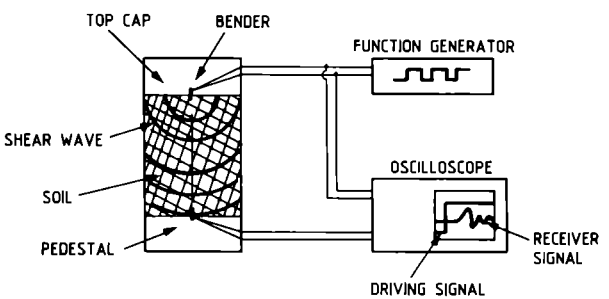


Fig. 7 Measurement of initial modulus by piezoceramic bender elements (Dyvik and Madshus, 1985).

Testing offshore and onshore can help determine any change in the sample quality and behaviour with time, in addition to using effectively the technical crews on board the site investigation vessel. Oedometer and advanced strength testing done offshore also provide a sound basis for establishing soil design parameters at an early stage, and supplies information (like stress history) for planning the main laboratory test program onshore.

3. Testing of shallow samples

In the case of foundation design for pipelines

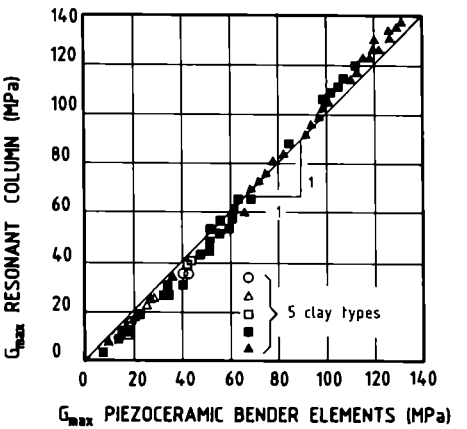


Fig. 8 Comparison of initial modulus from resonant column and bender element techniques (Dyvik and Madshus, 1985).

for example, extremely low stresses need to be applied to soil samples. Figure 6 illustrates one such case, where the difference in cell pressure and back pressure may be of the order of 1 to 10 kPa, the back pressure of the order of 5 kPa, the piston load to be applied during consolidation may be negative, and the load increase at failure may be only one kilogram! For such low load, the specimen was tested without membrane and the cell fluid was liquid paraffin. Measurement difficulties for such small quantities are to be expected.

4. Initial modulus from bender elements

This new technique, developed at NGI (Dyvik and Madshus, 1985) consists of having a pair of piezoceramic bender elements at each end of a soil specimen. The bender element at one end generates a shear wave pulse which propagates along the length of the specimen. The other element is used to determine the arrival time and the shear wave at the other end of the specimen, thus producing a direct measurement of shear wave velocity and, in turn, G_{max}. The measurement is simpler than a resonant column test and can be made in any of the standard geotechnical testing devices.

The bender element, placed in a slot in the pedestal, protrudes edge first into the soil specimen. The surrounding soil particles move in the same back and forth movement as the tip of the bender element (Fig. 7). This results in shear waves propagating through the specimen. An oscilloscope of high accuracy is needed to record the results. Under a square wave pulse, the signal from the receiver marks the arrival of the vertically downward propagating shear waves at the base of the specimen.

Figure 8 compares the shear modulus measured on five clays with the resonant column and the piezobender techniques. These G_{max} values apply to shear strain levels of 10^{-3}% and below. The

agreement is excellent. NGI is now considering the use of the bender element technique in oedometer tests offshore.

ACKNOWLEDGEMENT

The assistance of NGI colleagues, especially Tom Lunne, Arne Kleven, Toralv Berre and Rune Dyvik, is gratefully acknowledged.

REFERENCES

Andresen, A. and P. Kolstad (1979). The NGI 54 mm sampler for undisturbed sampling of clays and representative sampling of coarser materials. Int. Symp. SOA of Current Practice of Soil Sampling. Singapore. Proc., pp. 13-21.

Berre, T. (1985). Personal communication, NGI, July 1985.

Dyvik, R. and C. Madshus (1985). Laboratory measurements of G_{max} using bender elements. ASCE Nat. Conv., Detroit, MI, Oct. 1985.

Kleven, A. (1985). Personal communication, NGI, July 1985.

Lacasse, S., K. Iversen, G. Sandbækken and P. Morstad (1984). Radiography offshore to assess sample quality. NGM (Northern Geotechnical Conference), Linköping (Sweden). Proc. Vol. 2, pp. 575-584.

Sandbækken, G., T. Berre and S. Lacasse (1985). Oedometer testing at the Norwegian Geotechnical Institute. ASTM STP889, Symp. Consolidation of Soils. Laboratory testing. Fort Lauderdale, Florida.

Hydraulic fracturing study (Written discussion on Session 2B)

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Hydraulic fracturing tests have been performed at IWHR by injecting water into a borehole of the cylindrical specimen in triaxial cell for investigating the factors controlling the fracturing pressure u_f. The following conclusions have been obtained.

1. Hydraulic fracturing occurs on the minor principal plane where the effective stress becomes tensile.

2. The increase of pore pressure may not necessarily induce tensile effective stress in soil where the pore pressure is uniform and the deviator stress is larger enough to induce shear failure before the minor principal effective stress approaches negative.

3. Fracturing pressure is smaller for the specimen having higher coefficient of permeability, larger borehole and tested under slower rate of applying water pressure.
4. The preexisting cracks in the soil do facilitate the hydraulic fracturing but the cracks may not preexist in the soft soil, because they will be healed under pressure.

The condition under which hydraulic fracturing occurred in this kind of test is quite different from that occurred in the earth core under the reservoir water. Therefore other tests were carried out to simulate the field condition. The specimens were also cylindrical in shape, and had different shape of intake where water pressure applied to induce hydraulic fracturing. Four kinds of intake were chosen, as shown in Fig. 1. The stress applied to the saturated specimen in triaxial cell was such that the axial stress was equal to one and one half of the cell pressure σ_3.

The u_f versus σ_3 curves for different shapes of intake are shown in Fig.2, which shows that the u_f of the specimens with plane intake (b) is higher than that of the other specimens having curved intake. However the value of u_f is not as high as expected because the water pressure acting on the surface of plane intake (b) causes not only the pore pressure buildup but also the transient radiating flow in the specimen as shown in Fig. 3a, both result in effective stress reduction.

In order to prove whether the radiating flow was one of the important factors controlling hydraulic fracturing, the water pressure was applied on the entire surface of the end of the specimen which was placed in an enhanced permeameter as shown in Fig. 3b. The flow lines in the specimen were parallel, and the tensile stress would not be induced. Though the test results did confirm that the specimen would not be fractured, the specimen in this permeameter was tested under

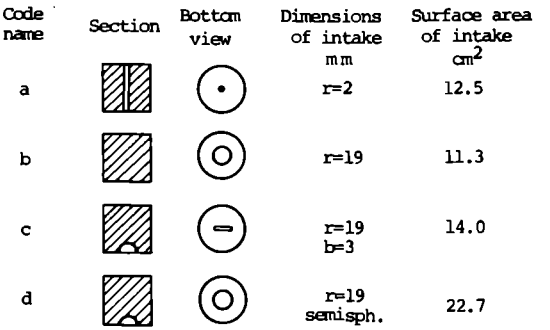

Fig. 1 Specimens Tested

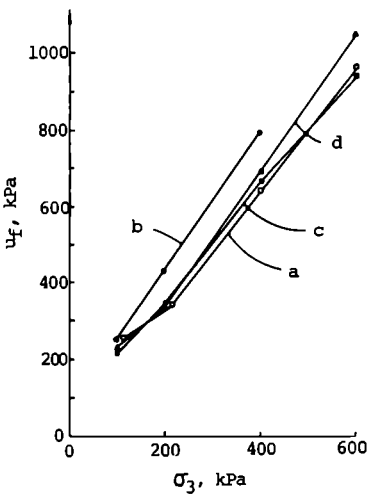

Fig. 2 u_f Versus σ_3 Curves

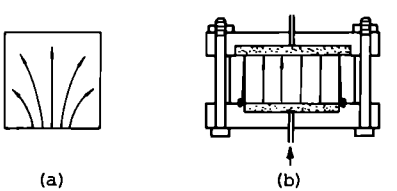

Fig. 3 Flow Lines in the Specimens

deformation control, but not under stress control as in the triaxial cell, so the total stress was being changed when the water pressure was applied. This made direct comparison impossible.

During the early stage of reservoir filling the effective stress in the core depends not only on the transient radiating flow or the pore pressure buildup, but also on the change of total stress induced by the reservoir water pressure, because the core is stressed under the intermediate condition between stress control and deformation control during the filling of reservoir.

It can be anticipated that if a relatively pervious layer is accidentally placed in the core near the upstream part, the rapid filling of reservoir water may enter this layer and create radiating flow which may induce hydraulic fracturing. Therefore keeping uniform permeability of the core seems to be one of the preventive measures of hydraulic fracturing.

Laboratory testing – New procedures and data acquisition techniques

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The hypothesis of a unique EOP void ratio-effective stress relationship which is independent of t_p, is not yet a result of rigorous analysis. It is an empirical concept based on the observed behavior of soils. Empirical concepts arise from the need to solve immediate construction problems. Therefore, those who wish to disagree with the EOP uniqueness concept, should offer an alternative approach that can predict settlement at least as well as it is done by the use of the EOP uniqueness concept.

The reason for the lack of a rigorous analysis of the EOP uniqueness question has been presented to this conference (Mesri and Choi, 1985b) in terms of the following equation.

$$\Delta e = \int_0^{t_p} \left[\left(\frac{\partial e}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma'} \right] dt + \int_{t_p}^{t} \left(\frac{\partial e}{\partial t}\right)_{\sigma'} dt \qquad (1)$$

The first term in Eq. (1) represents the compression that takes place during the increase in effective stress (primary consolidation stage), and the second term is the compression that follows at constant effective stress (secondary consolidation stage). The reason for the absence of a rigorous proof for the EOP uniqueness concept is that at present it is not yet possible to evaluate $(\partial e/\partial \sigma_v')_t$ and $(\partial e/\partial t)_{\sigma_v'}$ when $d\sigma_v'/dt \neq 0$. Equation (1) clearly shows that compressibility associated with time, $(\partial e/\partial t)_{\sigma_v'}$, as well as compressibility associated with effective stress, $(\partial e/\partial \sigma_v')_t$, contribute to the compression during the primary consolidation stage. However, Eq. (1) does not necessarily contradict the EOP uniqueness concept because for any value of t_p it may be possible to have such a combination of values of $(\partial e/\partial \sigma_v')_t$ and $(\partial e/\partial t)_{\sigma_v'}$ that produce a $(\Delta e)_p$ independent of the duration of primary consolidation.

In the absence of a rigorous analysis, the EOP uniqueness concept has been established using observed behavior of soils. The contribution to this conference by Mesri and Choi (1985b) reviews the existing data and presents new results on three natural clays, including consolidation tests on 1/2-m thick specimens. Unique EOP void ratio-effective stress relations were obtained for values of t_p in the range of a few days to one year.

In recent years the influence of secondary compression in load-controlled tests and strain rate in strain-controlled tests on the measurement of σ_p' has been confused with the EOP uniqueness question. It has been known for a long time that e-log σ_v' curves corresponding to decreasing rates of strain or increasing secondary compression time produce a decrease in measured σ_p' (Crawford, 1964, 1965). However, it has been pointed out by Mesri (1977) that such results do not necessarily disprove the EOP uniqueness concept. The following strain rate is recommended for determining EOP e-σ_v' relation from the CRS test:

$$\dot{\varepsilon}_p = \frac{k_{vo}}{2\, C_c/C_k\, H^2} \; \frac{\sigma_p'}{\gamma_w} \; \frac{C_\alpha}{C_c} \qquad (2)$$

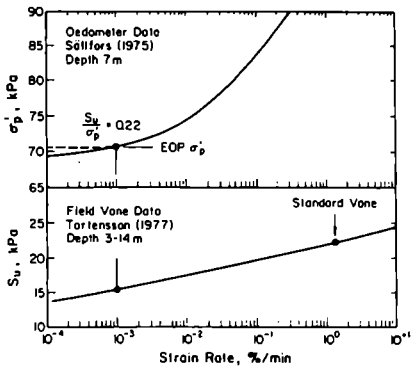

Fig. 1 Strain rate effect on σ_p' and s_u

where k_{vo} is the initial coefficient of permeability, $C_k = \Delta e/\Delta \log k_v$, $C_c = \Delta e/\Delta \log \sigma_v'$, $C_\alpha = \Delta e/\Delta \log t$, H is maximum drainage distance and σ_p' is the preconsolidation pressure corresponding to EOP e-log σ_v'. The CRS tests with strain rates faster than $\dot{\varepsilon}_p$ require the use of average void ratio and average effective stress for interpretation of data and therefore are not expected to provide reliable information on fundamental soil behavior. On the other hand, CRS tests that are slower than $\dot{\varepsilon}_p$ only confirm the effect of secondary compression on the measurement of σ_p' (Mesri and Choi, 1984).

The so-called 'field e-σ_v' relationships' computed by S. Leroueil can be misleading because the computations rely heavily on the position and measurements by field pore water pressure piezometers. It is unreasonable to suggest that field settlements could be as much as 8 times larger than settlement predictions using laboratory EOP e-σ_v'. In contrast to such a pessimistic conclusion, successful applications of the EOP uniqueness concept have been reported by Mesri and Choi (1985a).

In view of the recent extensive discussion on 'strain rate effect' on the e-σ_v' relationship, G. A. Leonards appropriately asks: what is preconsolidation pressure? There are two answers to this pertinent question. A preconsolidation pressure from an e-log σ_v' relationship corresponding to any time equal or greater than t_p, or strain rate equal or slower than $\dot{\varepsilon}_p$ is a consolidation pressure at which major changes in soil structure begin to take place. On the other hand, the preconsolidation pressure is the value of σ_p' from the e-log σ_v' relationship corresponding to t_p in incremental loading tests and $\dot{\varepsilon}_p$ in CRS tests. This σ_p' has traditionally been measured using oedometer tests on 2-cm thick samples. It is this latter σ_p' which has been used to normalize measured undrained shear strength by laboratory and field tests and the undrained shear strength which is mobilized on a failure surface in the field. This important point is illustrated in Fig. 1 for Backebol clay. The EOP preconsolidation pressure of 70.5 kPa also corresponds to the σ_p' from a CRS test with an imposed strain rate of 10^{-3} %/min. The undrained shear strength from the standard field vane test is 22.5 kPa. The resulting s_u/σ_p' of 0.32 has traditionally been correlated to the plasticity index. The EOP σ_p' is also used in the empirical equation s_u (mobilized) = 0.22 σ_p' (Mesri, 1975), which gives s_u (mob) = 15.5 kPa. For the Backebol clay data in Fig. 1, a strain rate correction for the vane strength also produces a s_u/σ_p' = 0.22. The σ_p'-strain rate data in Fig. 1 dramatically show that if any value of preconsolidation pressure other than EOP σ_p' is used to normalize either measured or mobilized undrained shear strength, the results will be inconsistent with the well-established empirical framework and can be expected to lead to confusion.

REFERENCES

Crawford, C. B. (1964). Interpretation of the consolidation test. ASCE J. Geot. Engr. Div., 90, 87-102.

Crawford, C. B. (1965). The resistance of soil structure to consolidation. Canadian Geot. J., (2), 90-97.

Mesri, G. (1975). Discussion: New design procedure for stability of soft clays. ASCE J. Geot. Eng. Div. (101), 409-412.

Mesri, G. (1977). Discussion: Stress-deformation and strength characteristics. Proc. 9th Int. Conf. Soil Mech. Found. Engrg., (3), 354-355, Tokyo.

Mesri, G. and Choi, Y. K. (1984). Discussion: Time effects on the stress-strain behavior of natural soft clay. Geotechnique (34), 439-442.

Mesri, G. and Choi, Y. K. (1985a). Settlement analysis of embankments on soft clays. ASCE J. Geot. Engr. Div., (111), 441-464.

Mesri, G. and Choi, Y. K. (1985b). The uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship. Proc. 11th Int. Conf. Soil Mech. Found. Engrg., (2), 587-590, San Francisco.

Discussion on Theme Lecture 2: 'New developments in field and laboratory testing of soils', by M.Jamiolkowski et al.

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F.TAVENAS, Dean, Faculty of Engineering and Sciences, Laval University, Québec, Canada

In their paper to this conference, Mesri and Choi have compared End-of-Primary (EOP) e - lg σ' curves obtained from isotropic consolidation tests on specimens of different heights and conclude that EOP curves are unique. In their State-of-the-Art report, Jamiolkowski et al. use these results to assert that theory A is valid. However, it seems to the writers that it is not possible to conclude so easily.

1] Due to the structural arrangement of natural clay particles, the compression index C_c is higher in one dimensional (1-D) compression than in isotropic compression (fig.5d of Mesri and Choi's paper). If it is accepted with Mesri and Choi, 1984 that $C_{\alpha e}/C_c$ is identical for 1-D and isotropic compressions, it follows that secondary effects are much less important under the test conditions used than under 1-D conditions.

2] What would be the void ratio differences between the EOP curves corresponding to various thicknesses if theory A would not be valid? An extreme case is given by the theory B (Jamiolkowski et al., 1985) with a constant $C_{\alpha e} = \Delta e/\Delta \lg t$ value and a duration of primary consolidation proportional to the square of the specimen height. In such conditions, the void ratio difference Δe_{A-B} between 2 EOP curves corresponding to heights H_1 and H_2 would be:

$$\Delta e_{A-B} = 2\left(\frac{C_{\alpha e}}{C_c}\right) C_c \lg \left(\frac{H_2}{H_1}\right) .$$

Considering C_c and $C_{\alpha e}/C_c$ values given by Mesri and Choi 1985, Δe_{A-B} for specimens 12,7 and 50,8 cm in height would be 0,015 for the Saint-Alban clay, 0,055 for the San-Francisco bay mud and 0,03 for the Louiseville clay at a void ratio of about 1,3. These differences are indeed very small, in the order of the variation of natural void ratios; they may hardly be observed in the graphical presentation of test results. For the Louiseville clay, at a void ratio of about 1,7, Δe_{A-B} would be 0,085 between the 50,8 cm and the 12,7 cm curves, 0,14 between the 50,8 cm and the 5,1 cm curves and 0,18 between the 50,8 cm and the 2,5 cm curves. Such values are measurable and in fact differences of this order have been measured, indicating that theory A is possibly not valid. However, as noted by Mesri and Choi, the behavior of thin specimens could also be influenced by end constraints.

3] Theory A also means (Jamiolkowski et al., 1985) that the effective stress-strain curve followed by the clay is unique, independently of sample thickness or position of the considered soil element within the specimen. However, fig.6b of Mesri and Choi is indicating that this is not the case. This figure shows the excess pore pressures u' measured during consolidation at various depths within a 50,8 cm thick specimen versus the logarithm of time. The curves present well defined steps at increasing values of u' from 16 kPa, 12,7 cm from the drainage boundary to 27 kPa at the impervious boundary. The corresponding effective stresses are 136 kPa and 125 kPa. As shown by Kabbaj et al., 1985, and Leroueil et al., 1985, these effective stresses associated with the steps are equal to the preconsolidation pressures of the clay at the corresponding depths. It follows that the preconsolidation pressure and thus the stress-strain curve depends on the position of the clay element within the specimen, which is not consistent with theory A.

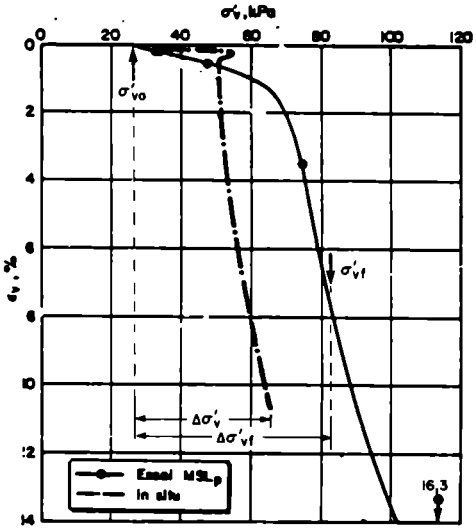

Figure 1. Stress-strain curves obtained in laboratory and observed in situ. (Saint-Alban, 3,1-4,9 m, Kabbaj, 1985).

Table I. Comparison between strains observed in situ and deduced from EOP laboratory curves (after: 1) Kabbaj, 1985; 2) Leroueil et al., 1983; 3) Leroueil et al., 1985)

	$\frac{\Delta\sigma'_v}{\Delta\sigma'_{vf}}$	in situ ε_v under $\Delta\sigma'_v$	lab. ε_v under $\Delta\sigma'_v$	lab. ε_v under $\Delta\sigma'_{vf}$
Berthierville[1] 3,0 - 3,9 m	0,85	11,7%	8,5%	10%
Berthierville[1] 3,9 - 4,8 m	0,89	10,5%	4,3%	5,7%
Gloucester[1 & 2] 2,4 - 4,9 m	0,95	5,6%	1,3%	1,4%
Saint-Alban[1] 3,1 - 4,9 m	0.69	10,6%	1,3%	7,5%
Väsby[3] 4,25- 7,3 m	0,50	16,0%	1,8%	15%

It thus appears that the data presented by Mesri and Choi (1985) can hardly be used to validate the theory A, as stated by Jamiolkowski et al., 1985.

On the other hand, the writers have compared EOP stress strain curves obtained in the laboratory on high quality samples with in situ stress-strain curves of clay sublayers under test embankments. Figure 1 shows a typical result; it clearly evidences a significant difference between the stress-strain curve followed in situ and the laboratory EOP curve. The results obtained on four sites are summarized in table 1 in which $\Delta\sigma'_{vf}$ is the stress increase due to the embankment and $\Delta\sigma'_v$ is the present increase in effective stress. It can be seen that in all cases, the in situ strain is larger than the strain in the

laboratory under the same stress increase $\Delta\sigma_v'$ and even larger than the strain in the laboratory under $\Delta\sigma_v'f$. This clearly indicates that the theory A is not valid and that the EOP curve obtained in the laboratory cannot be considered to represent the field behavior.

REFERENCES

Jamiolkowski, M., Ladd, C.C., Germaine, J.T., Lancellotta, R. (1985). New developments in field and laboratory testing of soils. State-of-the-Art Report, Proc. XI ICSMFE, San Francisco, Vol.1, 57-153.

Kabbaj, M., (1985). Aspects rhéologiques des argiles naturelles en consolidation. Ph.D. thesis, Laval University, Québec, Canada.

Kabbaj, M., Oka, F., Leroueil, S., Tavenas, F. (1985). Consolidation of natural clays and laboratory testing. ASTM Symposium on Consolidation of Soils - Laboratory Testing. Fort-Lauderdale. In press.

Leroueil, S., Samson, L., Bozozuk, M. (1983). Laboratory and field determination of preconsolidation pressures at Gloucester. Canadian Geotechnical Journal, Vol.20 (3), 477-490.

Leroueil, S., Tavenas, F., Kabbaj, M. (1985). Discussion to "Settlement analysis of embankments on soft clays" by Mesri, G., and Choi, Y.K. ASCE Journal of Geotechnical Engineering. Submitted for publication.

Mesri, G., Choi, Y.K. (1984). Discussion to "Time effects on the stress-strain behaviour of natural soft clays" by Graham, J., Crooks, J.H.A. and Bell, A.L. Géotechnique, Vol.34(3), 439-442.

Mesri, G., Choi, Y.K. (1985). The uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship. Proc. XI ICSMFE, San Francisco, Vol.2, 587-590.

Discussion on Theme Lecture 2: 'New developments in field and laboratory testing of soils', by M.Jamiolkowski et al.

Structural effects on the behaviour of natural clays

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In their paper, Jamiolkowski & al. present results of laboratory tests on resedimented Boston blue clay (fig.3) and on intact B-6 marine clay (fig.4) to illustrate the concepts of normalized stress-strain and stress paths and of yielding, and to discuss the effects of clay structure. Implying that the resedimented Boston blue clay is typical of most clays, they note important differences in behaviour between this clay and the B-6 clay. They suggest that all Canadian clays are special materials subject to destructuration. The distinction between "cemented" Canadian clays and other presumably more normal clays is then repeatedly made in the theme lecture. The writers would like to take strong issue with this part of the theme lecture and present evidence in contradiction to the statements by Jamiolkowski & al.

It must first be realized that a residemented clay can not be in any case considered as representative of natural clays, since the complex geological history (rate and duration of deposition, creep, thixotropy, leaching and other physico-chemical changes) cannot be reproduced. Indeed, residemented material is probably the least structured state of clay which may be encountered. As for the B-6 clay, it is the writers experience that this material represents an extreme case of structured soil, not at all typical of the marine clays of Eastern Canada.

The writers have been investigating structural effects in natural clays for ten years. Original studies were concerned with Champlain sea clays from Eastern Canada (Leroueil et al., 1979) and led to the "destructuration" concept. Since then, similar studies have been conducted on clays from Atchafalaya (USA), Bäckebol and Lilla Mellösa (Sweden) and Cubzac-les-Ponts (France). Figure 1 shows the yield envelopes obtained on four natural clays from various geological origin. The similarity between the four envelopes is striking. In low stress region, the envelopes are curved and high above the Mohr-Coulomb line, i.e. similar to the B-6 data in the theme lecture. All the natural clays tested by the writers have shown a similar behaviour, indicating an effect of structure.

A number of the natural clays tested by the writers have been submitted to destructuration by consolidation under a few kPa above their preconsolidation pressure before being unloaded and tested at the same OCR as in situ intact clays. In all cases, a significant reduction in strength and moduli was noted. Figure 2 shows the strong effects of destructuration on the stress-strain curves of three natural clays: reduction by a factor of 2 or more of the modulus, flattening of the curve at the peak strength, increase by a factor of 2 of the strain at failure.

Figure 1 shows the changes in the shape of the yield surface caused by destructuration in the same three natural clays. While the magnitude of these changes varies from clay to clay, they are qualitatively similar in all cases: in the low stress region, the yield curve is

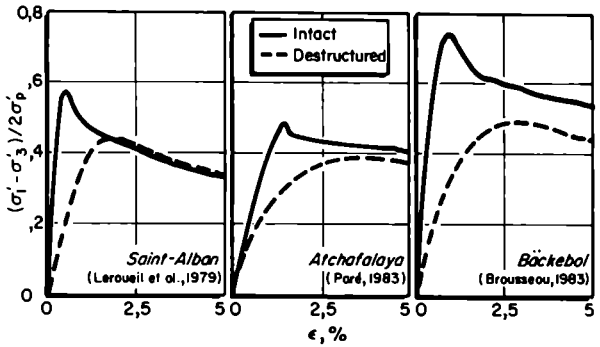

Figure 1. Stress-strain curves in CIU tests on intact and destructured natural clays.

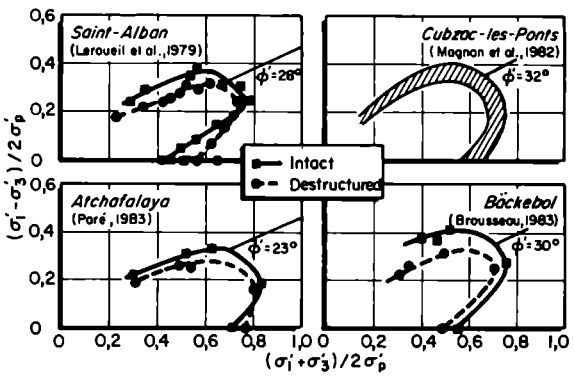

Figure 2. Limit state curves of intact and destructured natural clays.

lowered to the vicinity of the Mohr-Coulomb line. It should be noted that the Atchafalaya clay is the same material which has been tested by Ladd & Foott (1974) in establishing their SHANSEP concept. The data shown here in figures 1 and 2 contradicts the assumption underlying SHANSEP as well as the theme lecture, that some natural clays would not be subject to destructuration. Indeed, the writers have yet to find a natural clay in which the effects of destructuration described by Leroueil et al. (1979) would not apply.

Based on their experience, the writers would like to suggest that the marine clays from Eastern Canada are not so different from other natural clays of low to medium plasticity. The repeated reference to their "cemented" state may not be justified in view of the available experimental evidence. For instance, Bjerrum (1973) presented the Saint-Jean-Vianney clay as a typical example of cemented clay; yet, Bouchard et al. (1983) have shown that the high preconsolidation pressure observed in that region can be easily explained by sedimentation and erosion. One of the reasons why structural effects have been reported in Canadian clays while remaining ignored in other soils is that a much higher quality of sampling has been practiced in Canada, using tools such as those described by Lefebvre & Poulin (1979) or La Rochelle et al. (1981).

REFERENCES

Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others). Proc. 8th ICSMFE, Moscow, Vol.3, pp.111-159.

Bouchard, R., Dion, D.J., Tavenas, F. (1983). Origine de la préconsolidation des argiles du Saguenay, Québec. Can.Geot. J, Vol.20(2), pp.315-328.

Ladd, C.C., Foott, R. (1974). New design procedure for stability of soft clays. ASCE, J of GED, Vol.100(GT7), pp.763-786.

La Rochelle, P., Sarrailh, J., Tavenas, F., Roy, M., Leroueil, S. (1981). Causes of sampling disturbance and design of a new sampler for sensitive soils. Can. Geot. J, Vol.18(1), pp.52-66.

Lefebvre, G., Poulin, C. (1979). A new method of sampling in sensitive clays. Can.Geot.J, Vol. 16(1), pp.226-233.

Leroueil, S., Tavenas, F., Brucy, F., La Rochelle, P., Roy, M. (1979). Behaviour of destructured natural clays. ASCE, J of GED, Vol.105(GT6), pp.759-778.

Magnan, J.P., Shahanguian, S., Josseaume, H. (1982). Etude en laboratoire des états limites d'une argile molle organique. Revue française de Géotechnique, n° 20, pp. 13-19.

Discussion on 'Centrifuge study of spill-through abutments', by Randolph et al.

Discussion sur: 'Etude de centrifugation des appuis ouverts', par Randolph et al.

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The authors have presented an interesting paper which describes a long awaited alternative to the original design approaches proposed by Chettoe and Adams (1938) and Huntington (1957). However, the applicability of an ultimate failure mechanism created by the removal of the support from beneath the sloping side of the embankment would appear to be somewhat unrepresentative of the conditions experienced by the many spill-through abutments in use today.

Since 1982 the University of Surrey has been monitoring the behaviour of the full size spill-through abutments at Wisley, Surrey, UK, under contract to the Transport and Road Research Laboratory [Lindsell (1984)]. The earth pressures have been measured with vibrating wire earth pressure cells and the bending of the concrete columns has been recorded using vibrating wire strain gauges. Abutment movements were recorded using inclinometer tubes and precise surveying techniques. The structures were monitored throughout the period of construction and the effects of traffic loading and seasonal temperature effects are still being recorded.

This full size investigation has shown no evidence to suggest that ultimate conditions have been developed. However, it has indicated that certain stages of construction have had a significant influence on the behaviour of the structure and on the development of the earth pressures. The compaction of the backfill around the columns was found to produce balanced pressures on back and front column faces, and these were considerably greater than those predicted by an "at rest" assumption of 9.4H. The temperature rise due to hydration of the concrete in the crossbeam caused it to expand during the first 48 hours after casting, resulting in a transverse loading to the top of the columns. The expansion was resisted by the soil that surrounded the columns. Similarly, the expansion of the 76m long concrete deck slabs after casting was found to push the abutments backwards as a result of the connectivity provided by the formwork to the end of the deck slab.

The abutment rotated about a point above its base and was again supported by the large pressures generated from the backfill around the columns. The construction of the embankment up to road level was found to cause large compaction pressures (as high as 120kPa) on the rear of the crossbeam. The forwards movement of the abutments at this stage was restricted by the propping force from the end of the deck slab which was generated as a result of the compression of some remaining packing material within the expansion joint. The pressures on the rear of the crossbeam were found to increase further still during the summer months as the deck slab expanded but were found to drop to near active pressures during the winter when the deck contracted and allowed the top of the abutment to move forwards. At no time has a pressure distribution been created that increases linearly with depth. Instead, the abutment was found to rotate about a point above the level of the base slab thus creating an increase in earth pressure on opposite sides towards the top and the bottom. Furthermore, displacements never exceeded 14mm at the top of the abutment, which corresponds to 1/570 of the height. Active conditions were not created at this time because the movement was caused by the large pressures resulting from the compaction of the backfill at the rear of the capping beam. It was deduced that the centre to centre spacing of the colums of 4 times the width of a rectangular column was not close enough for the effects of column interaction to be significant. The displacements, however, were likely to be adequate to mobilize fully the shear stress of the soil against the side faces of the columns.

The design approach for ultimate conditions as has been proposed by the authors does not account for the conditions which have so far been observed for the Wisley abutments. Specifically, it does not model the effects of backfill compaction, deck propping, thermal effects, soil resistance at the front of the columns, abutment rotation about a point above base level, or the effects of shear stresses on the column side faces. In conclusion, the proposed design approach should only be applied to the design of structures for ultimate conditions as suggested and cannot be assumed to be representative of the working conditions that are more often considered for the design of spill-through abutments.

Reference

Lindsell, P. (1984) "Monitoring of Spill-Through Bridge Abutments", Internal Report, Dept. of Civil Engineering, University of Surrey.

Discussion to Session 2D: Field instrumentation and field measurements

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ABSTRACT

Session 2D was divided into two topics: the evaluation of piezometers and stress cells respectively. Each topic was introduced by lectures by panel members, followed by a free discussion from the floor.

GENERAL

The state-of-the-art-report "New Developments in Field and Laboratory Testing of Soils", written by Jamiolkowski and co-authors, formed the base for four discussion sessions, sessions 2A-D. Session 2D concentrated on measurements of pore pressures with piezometers and horizontal stresses with pressure cells. The development of automated data acquisition was included in each of the subjects.

The panel of session 2D consisted of
Professor A. Rico, Mexico. Chairman
Dr. J. Hartlén, Sweden, discussion leader
Dr. J.T. Germaine, USA, panel member
Professor G. Gudehus, West Germany, panel member
Dr. A.D.M. Penman, United Kingdom, panel member
Dr. S. Leroueil, Canada, panel member

TOPIC 1 EVALUATION OF PIEZOMETERS

Different types of piezometers are available on the market, based on either open or closed system. Electrical instruments are increasing in popularity, and then the possiblity of collecting data automatically becomes relevant. In the introduction to session 2D, Hartlén (Sweden) focused on the following questions not yet being solved and therefore suggested for the discussion:

- natural variations of pore pressures with time
- decay of excess pore pressures
- long-term performance and stability of different systems
- choice of measuring system, absolute contra relative piezometers
- control functions during operation and the reliability of data collection systems.

All these questions are more or less directly related to the reliability of pore pressure measurements, especially for long-term periods.

Hartlén (Sweden) started the presentation from the panel by showing that the time needed for pore pressures to stabilize after installation varies from system to system. Open systems with thin plastic tubes are quicker than many electrical systems based on vibrating wire or electrical pressure transducers. It was noted that some of the instruments needed several weeks to stabilize even when the filter stones were well saturated before installation. After the stabilization period the open and the closed systems showed very good agreement.

Hartlén (Sweden) also presented a comparison where both absolute and relative pore pressures were registered. The absolute pore pressure showed large variations with time and this variation followed the change in atmospheric pressure. The variation of the relative pore pressure calculated by subtracting the atmospheric pressure from the total pressure, however, became small. This calculated pore pressure agreed well with the pore pressure measured by a relative piezometer. The experience in Sweden is that the instruments measuring relative pore pressure are not reliable for longer time periods, probably due to bad communication in the connection tube that compensates for atmospheric pressure. According to the results presented, the total pore pressure corrected for current atmospheric pressure may be a more reliable system.

Leroueil (Canada) described a field test were hydraulic, electrical and pneumatic piezometers were installed underneath an embankment with a width greatly exceeding the clay thickness. During construction the hydraulic piezometer showed an increase during the night due to a time lag while the electric piezometer showed a decrease due to consolidation. The pore pressure increased even after the completion of the embankment. Leroueil presented the idea that this was caused by creep in sensitive clays. Bergdahl (Sweden) presented an example where the pore pressures had been just as high as the total overburden pressure. This was obtained in the shear zone of a slope which had a low safety against failure. The high pore pressures occurred during the work to improve the stability by placing a berm at the toe of the slope. The occurrence of high pore pressure at an initial phase after completion of an embankment may also be caused by other factors than creep according to the discussion.

Richards (the Netherlands) had found pore pressures in excess of hydrostatic pressures in cohesive seabed soils at depths beneath some metres below sea bottom. If this paradigm is general it means that seabed soil (by defintion) is not normally consolidated regardless of water depth.

It was pointed out by Massarsch (Sweden) that the excess pore pressure during installation of the tip depends on soil profile, soil permeability and response of the device. Before a piezometer is installed, pore-pressure soundings should be performed to identify proper installation depths. Penman (United

Kingdom) showed, in his panel contribution, examples of measuring pore pressure in non-saturated soils. He compared the results from using coarse-and fine-grained filter stones. High negative pore pressures were measured using the fine filter stones. Andresen (Norway) pointed out that fine filter stones are generally used by NGI.

Karlsrud (Norway) showed an example where pore pressures were measured during the installation of a pile. Some field piezometers were saturated using "standard" procedures, just placing filter and piezometer in water some time before installation. Other piezometers were vacuum saturated in the laboratory and carefully kept saturated during handling and installation. The vacuum saturated piezometers showed immediate response (within seconds or minutes) during pile installation, whereas the "standard" saturated piezometers showed a delayed and reduced pore pressure response. However, after one day or so the vacuum saturated and the "standard" saturated piezometers coincided. The conclusion drawn by Karlsrud (Norway) was that when short-term variations (seconds, minutes, hours) are of interest great care must be taken with saturation of filters. The vacuum saturation procedure seems to be very suitable for this purpose.

During long-term measurements it is important that the transducers can be checked. As a consequence, Bergdahl (Sweden) stated that the BAT system is advantageous. In the BAT system the transducers can also easily be replaced.

Patton (Canada) presented a modular piezometer system. The system allows many completion zones. The pore pressure had in one case been read at 50 depths in a total depth of about 270 m. The different zones are separated by hydraulic packers and it is also possible to take water samples. An interesting point is that the reading units can be calibrated inside the hole.

Geotechnical engineers become more and more involved in environmental questions. Painter (USA) pointed out that new Swedish and Canadian piezometers allow collection of water samples. Painter especially emphasized the possibility of indentifying the presence of methane and other explosive gases during tunnelling.

Methane is commonly contained in solution under significant hydraulic pressures in formations beneath or above organic layers. The gas normally comes out of solution if groundwater samples are not maintained at in situ pressures when raised to the ground surface. According to Painter tunnelling standards for mining and construction should be formulated to include geotechnical studies of methane presence as many explosions have occurred with disastrous consequences.

The conclusion of the first part of session 2D was that, although pore pressures have been measured for a long time, there is still a need to continue the research. This especially applies to long-term measurments. The need of determining the permeability in situ and taking water samples is increasing.

TOPIC 2 EVALUATION OF PRESSURE CELLS

In the SoA report Jamiolkowski et al show different methods to evaluate the horizontal stresses in soil. In session 2A the self-boring pressuremeter and the dilatometer were discussed. In session 2D the interest was directed to pressure cells. Two essential questions were brought forward by the discussion leader Hartlén (Sweden), namely

- disturbance due to installation

and

- the reliability of measurements made in stiff clay and sand.

Introductions before the free discussion were given by three panel members. Gudehus (West Germany) stated that the measurement of the horizontal stress (σ_h) in sand by any driven-in device is impossible because the introduced shear stresses will change the value of σ_h. Any relaxation later on will not correct the measured value. In soft clays, Gudehus agreed with the SoA report that in soft clay the K_o-value measured in field is very close to the K_o-value determined in laboratory. As a consequence K_o measurements are not needed in the field in case of soft clays. In stiff clays the K_o-value varies with the horizontal direction and Gudehus recommended in this case the use of a self-boring pressuremeter, whereas laboratory tests do not give reliable values.

Germaine (USA) showed interesting results where the geometry (tip geometry and cross-section) of a pushed-in device had been varied in a slightly overconsolidated clay. With a tip angle of 20 degrees a K_o-value close to that from the laboratory tests was achieved. The tip with an angle of 40 degrees gave a lower value and an enlarged tip an even lower value. Two shapes were investigated, one spade-like and one circular. The pore water dissipation after installation was alike for these two shapes. Germaine drew the conclusion that the disturbance was then of the same order and that the geometry of the cell should then not influence the K_o-value.

Penman (United Kingdom) defined a critical pressure from hydraulic fracture. The critical pressure became smaller than that determined by the Marchetti probe. Between these two values a value was obtained using a cell with a thickness of 6 mm. In a core of a dam the horizontal stress was measured by hydraulic fracture, a 2 mm thick spade and the Camkometer. The lowest value was obtained by hydraulic fracture (critical pressure) and the highest value by the Camkometer.

The bedding technique was mentioned by several contributors. Penman (United Kingdom) described measurements in a dam core. Lower stresses were registered underneath than above the cell. Tests presented by Montanez (Mexico) showed that the edge effect, also in a large test chamber, influences the measured values due to an uneven vertical stress distribution. A scatter of $\pm 15\%$, correlated to applied stress, was detected. In a triaxial cell the stress distribution will be different than in a stiff ring.

Leroueil (Canada) reminded of the tests by Audibert & Tavenas in 1975. The relationship between the bedding material (E_b) and the surrounding natural soil (E_S) showed the following values between measured and applied vertical stress

E_b/E_S	$\sigma_{measured}/\sigma_{applied}$
4	1.08
1	1.01
0.5	0.87

It is thus very important how the soil is compacted around the stress cells.

Schober (Austria) emphasized the need to calibrate the stress cells after being placed in an embankment. In Austria a special technique is used. The degree of error by this technique falls within the order of $\pm 10\%$. In Mexico calibration tests have been performed in water and between loading plates, resulting in a recommendation to make the test in water.

It is important to realize that the K_o-value must not be looked upon as a soil constant (Massarsch, Sweden). Stress changes may change the K_o-value and stress measurements should be combined with those of the pore water pressure. It was further brought up whether the low horizontal stresses sometimes measured by self-boring pressuremeters were due to arching effects.

The conclusion of the discussion on the second topic concerning stress cells was that the use of pressure cells is to a high degree questionable. Use of pushed-in cells in frictional material cannot be recommended at all. To determine the in situ horizontal stresses the self-boring pressuremeter is more reliable and maybe also the dilatometer. However, these instruments have also several drawbacks, well expressed in the SoA report.

Pressure cells are built into the soil to measure the stresses in e.g. earth dams. In that case it is important to calibrate the cells in a proper way and there are several pitfalls. The bedding technique is also of greatest importance. Finally, the pore pressure should be measured simultaneously with stress measurements.

FINAL REMARKS BY RICO (MEXICO), CHAIRMAN OF SESSION 2D.

Rico (Mexico) closed the session as follows: "In México we have installed many piezometers in saline waters with a very high content of salts. In that case the metal-cells have had a very poor performance in very short time. We needed to develop and install pneumatic piezometers of plastic.

Under high embankments a certain number of effects contribute to give some extra results in the development of pore pressures. For instance, the lateral flow in natural soils under the embankment, which is rapid, produces an initial change of pore pressures generated from shear stresses. This effect is later overcome by the primary consolidation effect and the pore pressure begins then to increase in a normal way. This effect is related to the sensitivity of clays.

Also we observed systematically the elevation of the groundwater table under the embankment because of the lack of evaporation that the embankment itself causes.

With respect to total stress measuring cells I can conclude that much attention should be given to some so-called minor details. I can assure that in the near future much work on calibration of measuring devices will be done, mainly when measuring horizontal stresses.

We hope the discussion was of interest to all of you. Thank you for your attention."

Pore pressures generated during cone penetration testing in heavily overconsolidated clays

JOHN L.DAVIDSON

Pore pressure dissipation plots are presented which support the conceptual pore pressure distribution presented to this conference by Campanella et al., and shown in Figure 1. For a heavily overconsolidated soil, their figure indicates high positive excess pore pressures measured on the face of the tip and low pressures measured behind the tip. The high positive pressures are a consequence of the large normal stresses existing during tip penetration. Behind the tip there is a normal stress relief and large shear stresses which dominate the response.

In research at the University of Florida designed to determine insitu coefficients of consolidation from pore pressure dissipation behavior, (Gupta, 1983) a total of nine soundings were performed in stiff overconsolidated soils at three sites with near surface water tables. In all soundings, negative pore pressures were measured during probe penetration. At a total of 30 depths, penetration was stopped and the dissipation of excess pore pressures monitored with time. In most cases, when the negative excess pore pressure was seen to approach zero, the dissipation testing was discontinued. However, in a few tests, when the time allowed was greater than necessary for dissipation to zero, it was observed that rather than leveling off at zero excess pore pressure, the pore pressures crossed the axis and became positive. It was therefore decided to allow a number of tests to dissipate for much longer periods of time. Nine such tests were run. In all cases dissipation started from an initial negative excess pore pressure, reached a positive peak, then decreased back to or towards zero. Results from four tests at two sites are included in Figure 2.

Tests at both these sites were performed using a Fugro piezocone with the pore pressure sensing element located just behind the tip. The soil at the Deerhaven Power Plant was a stiff overconsolidated medium plasticity clay. At the Lake Wauberg site the soil was a stiff overconsolidated very plastic clay.

The shape of these curves can be explained by the phenomena illustrated in Figure 1. During penetration the excess pore pressures behind the tip are negative, a consequence of the high shear stresses and the heavily overconsolidated nature of the soils. Upon stopping penetration, these negative excess pore pressures are quickly swamped by inflow from the higher positive pore pressure zone in front of the tip. A peak positive pore pressure is reached which then dissipates with time back towards zero.

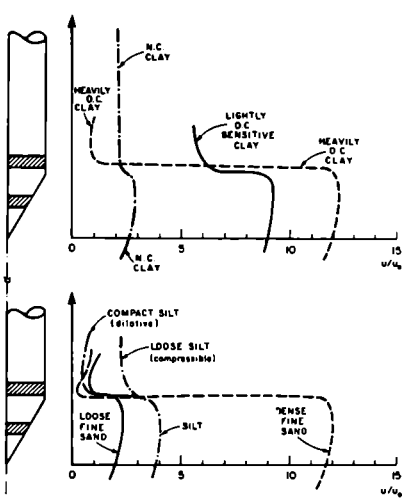

Figure 1 Conceptual Pore Pressure Distribution in Saturated Soil During CPT Based on Field Measurements (from Campenella et al., 1985)

REFERENCES

Campanella, R.G., P.K. Robertson, D.G. Gillespie, and J. Grieg, "Recent Developments in In-Situ Testing of Soils," 11th ICSMFE, Vol 2, p. 849, San Francisco, 1985.

Gupta, R.C. "Determination of the Insitu Coefficient of Consolidation and Permeability of Submerged Soils," Ph.D. Dissertation, University of Florida, Gainesville, Fla. 1983.

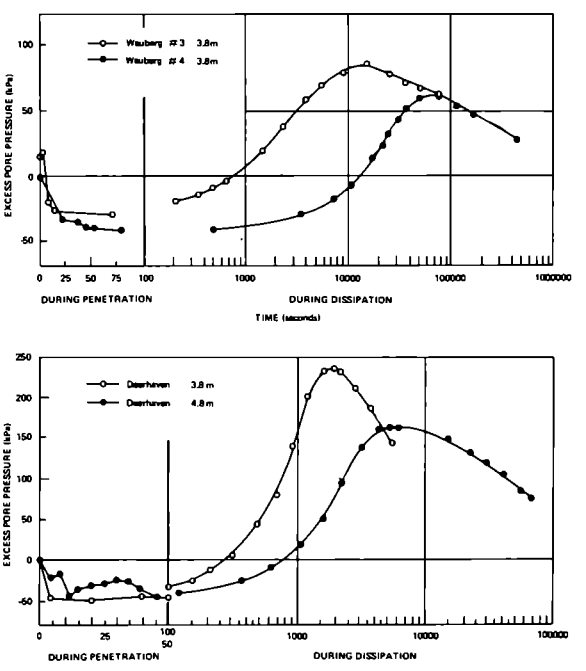

Figure 2 Excess Pore Pressures During Penetration and Dissipation

Discussion to Session 2D
Calibration of total stress measurements in situ

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In order to investigate the bearing behaviour of embankment dams extensive measurements of total stresses have been carried out in Austria. FIG. 1 shows the instrumentation of the Finstertal-dam in Tirol {1} as a typical example.

83 total pressure cells have been installed at horizontal and vertical positions as well as under an inclination of 45° into the different dam zones (even into the rockfill).

For the calibration in situ the method of F. List {2} has proved well. It was adapted successfully for modelling gauges too by B. Lackinger {3} .

FIG. 2 shows the example of a calibration diagram of a cell in a horizontal position. The measuring data were transformed into the 45°-straight-line by using the calibration formula

$$\sigma_v = A_K \cdot \overline{\sigma_v} + B_K$$

Only that part of the fill is to be used for a calibration range in which no deviation of the calculated vertival stresses $\gamma.h$ as a result of the load transfer arises. According to the experiences it is possible up to an approx. h_{max} of 10 m.

The calibration of vertical gauges can only be done via the calculated horizontal stress σ_h' , using the formula :

$$\sigma_h = (1-\sin \phi') .\gamma.h$$

In case of an inclination of 45° the gauges can be calibrated by using the state of stresses in the horizontal halfspace and the noted principal stresses $\sigma_{1,3}$.

In order to check the degree of accuracy in calibrating the horizontal gauges in a mea-

SYMBOL	MONITORING EQUIPMENT	NUMBER
	surface monuments	111
	vertical plate gauges	7
	horizontal plate gauges	8
	extensometers	38
	groups of 4 strain meters	4x4
	fluid level settlement devices (permanently installed)	7
	plumb - lines	2
	measuring devices for the determination of core-wall thickness variations	3
	two-dimensional group of earth pressure cells	83
	three - dimensional group of earth pressure cells	18
	pore pressure cells	14
	resistance thermometers	9
	strong motion accelerographs (three dimensional)	2
	peak recording accelerographs	3
	piezometer tube in borehole	13
	recording of seepage water losses	3

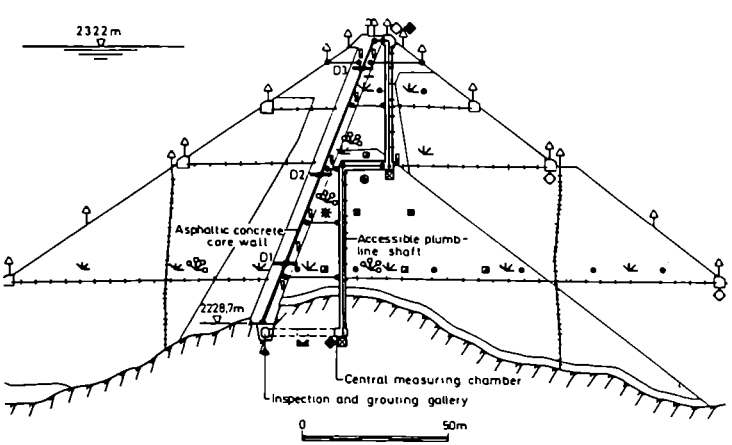

FIG. 1 :
Finstertal-dam :
Monitoring Equipment

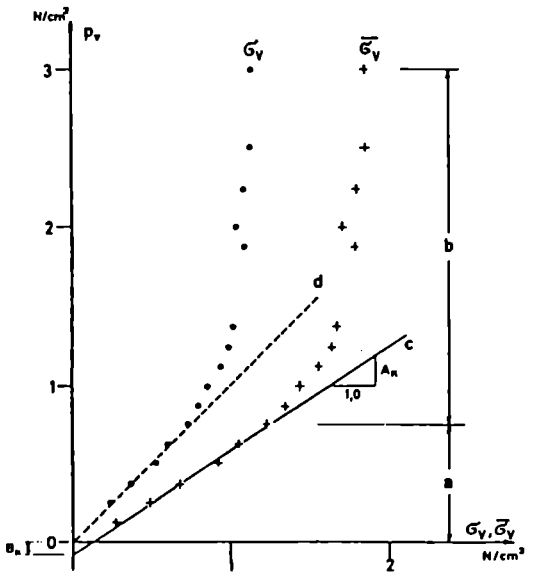

FIG 2 : Calibration curve p_v vers. $\overline{\sigma_v}$ and curve p_v vers. σ_v

$p_v = \gamma . h$: overburden pressure
+ ..... $\overline{\sigma_v}$: vertical pressure on the gauge
o ..... σ_v : real vertical pressure in the fill
a .....: calibration range
b .....: range of load transfer
c .....: regression line $\sigma_v = A_K . \overline{\sigma_v} + B_K$
d .....: theoretical straight line $p_v = \overline{\sigma_v}$

suring level, the total overburden load can be compared with the load calculated on the basis of measurements. A dregree of erros less than 10 % has been ascertained - a satisfying result for such measurements.

{1} PIRCHER,W.: Behaviour of an Asphaltic Concrete Core Wall. Discussion to Q 55, 14th ICOLD, Rio de Janeiro, 1982, p. 617-624

{2} LIST,F.: Das Projekt des Staudammes Mauthaus. Dissertation,University of Innsbruck, 1978.

{3} LACKINGER,B.: Das Tragverhalten von Staudämmen mit membranartigen Dichtungen. Dissertation, University of Innsbruck, 198o.

Discussion to Session 2
Pore pressure – Comparison of different piezometers

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J.HARTLÉN, Phd, Director General, Swedish Geotechnical Institute, Linkoeping, Sweden

Very often the engineer has to measure the pore pressure in a soil deposit in order to be able to solve the problems which can arise during the different phases of a project. These measurements have to be adapted according to the type of problems expected, and the instruments used for this purpose must satisfy different criteria of reliability, accuracy, cost and flexibility.

In order to compare different types of piezometers a study including 12 tips has been made. The tips were installed in a clay deposit and the pore pressure was read once a week over a period of six months. The results are presented in Fig. 1; the piezometers have been classified in four major groups (open standpipe, vibrating wire, closed system with continuous reading, closed system with removable transducer), and only the mean values are shown on the figure. As seen from the results, there is a stablization period following the installation of the instruments, but after this period the values show a rather small scattering. Thus, it seems that the natural variation of pore pressure can be measured with good accuracy and reliability regardless of the type of piezometer employed.

Four of the piezometers used in the preceding study were connected to a recording system which could take one reading every 20 minutes; we used this installation to study the effect of the variations in atmospheric pressure on the measurements. Two of the piezometers were equipped with an absolute pressure transducer, while the two others were installed with a relative pressure transducer. A barometer was also connected to the recording system for measurement of atmospheric pressure. Fig. 2. presents the measurements during a period of more than 20 days. The results show clearly that the use of an absolute pressure transducer without taking into account the variations in atmospheric pressure may lead to noticeable errors on the pore pressure value.

Considering the different problems related to the long-term behaviour of relative pressure transducers, especially the obstruction of the tube transmitting the atmospheric pressure on the back of the membrane, it is preferable to use abosolute pressure transducers. However, the present study has shown the necessity of combining the pore pressure measurement with atmospheric pressure reading, to obtain the correct value of the pore pressure.

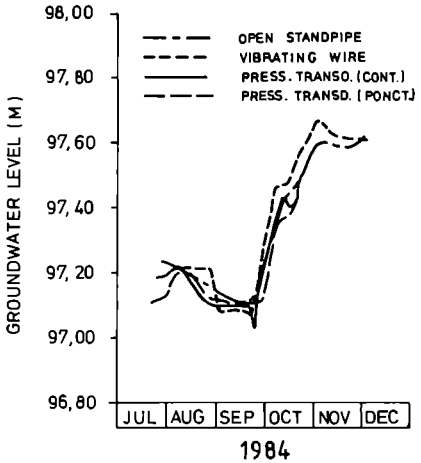

Fig. 1. Results from measurement of piezometric levels.

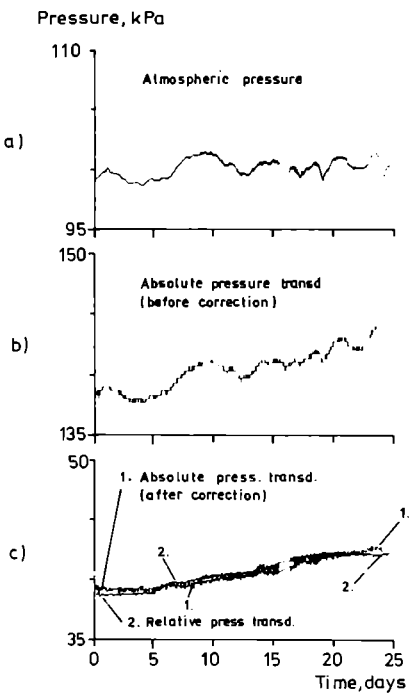

Fig. 2. Influence of atmospheric pressure.

Discussion to Session 2D
Pressometer test and k_o determination

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The pressurameter test cannot be correctly interpreted without considering triaxial deformations. This becomes evident if we consider the radial deformation u_α of a cylindrical cavity on plain strain condition in a elastic medium surrounded by a rigid cylindrical boundary at R_α, Fig. 1 (u_α is normalised to u_∞ corresponding to R_∞. We can deduce that no deformation may take place for finite values of R_α when Poisson ratio is $\nu = .5$ as in undrained expantion in saturated soils.
Deformation becomes possible only if we move R_α to infinity. This is physically absurd, in particular considering that:

1. When we performe a pressurameter test we expect that only a relatively thin shell of soil participates;
2. Should be true that probe expantion depends on strains propagating to infinity then whatever high might be the length to diameter ratio or the probe depth, conditions for plain strain assumption cannot be obtained.

If we refer to the elastoplastic model developed by APPENDINO , DI MONACO (1981) which assumes plain stress expantion and a rigid boundary at a radious R_α moving away as the plasticised shell radious R_p increases so to keep constant the ratio $\beta = R_p^2 / R_\alpha^2$, fig. 2, to simulate the effect of strains on shear modulus ($\beta \cong 1/6$).
We have for $\nu = .5$

$$(1) \quad p = p_o + c_u \left[1 + 3\beta + \ln \frac{1 + \Delta^* - R_o^2/R_F^2}{\frac{1-\beta}{I_r} + \Delta^*} \right]$$

where :

$p_o = k_o \sigma_v$ is the original confining pressure,

Δ^* is a term allowing for volumetric and vertical strain in the plasticized shell

$I_r = E/3c_u$ is Vesic'rigidity index

p is the pressure inside the cavity

If $R_F/R_o = \infty$ we obtain P_{lim} which is intermediate between Vesic' plain strain cylindrical and spherical solutions when $\beta = 1/6$.
If $R_\alpha = \infty$ and $\Delta^* = 0$ (1) coincides with Vesic'cylindrical plain strain solution (2)

$$(2) \quad p = p_o + c_u \left[1 + \ln \frac{1 - R_o^2/R_F^2}{1/I_r} \right]$$

Solutions(1) and (2) are similar but (1) contains terms that cannot be found in (2).When we ignore them (as we do using either Gibson and Anderson model or Baguelin et al. or similar no model procedures) we get erroneous estimates of k_o, c_u, E_p and P_{lim}. In the determination of k_o we are particularly interested in the early stage of the test when deformation is elastic,then the expantion is regulated by a confining pressure exceeding the original p_o of Δp_o where

$$(3) \quad \Delta p_o = (p - p_o) \; 4\beta/1+3\beta$$

Δp_o grows up with p until plastic yeld develops , then it remains constant allover the expantion.
We consequently measure k_o from a confining pressure equal to $(p_o + \Delta p_o)$; then fair agreement with the expected value is likely to be the consequence of errors cancelling.

References:

M.APPENDINO , DI MONACO, 1981, unpublisced report

BAGUELIN,F., JEZEQUEL, J.F., LE MEE, H. and LE MEHAUTE, A., (1972), "Expansion of Cylindrical Probes in Cohesive Soil", JSMFD, ASCE. Vol. 98, No. SM11 pp. 1129 - 1142

GIBSON, R.E. and ANDERSON, W.F., (1961), "In situ Measurement of soil properties with the pressuremeter", Civil Engineering Public Works Review, Vol. 56 , No. 658, pp. 615 - 618

VESIC, A.S., 1972 - Expantion of cavities in indefinite soil mass, J., SMFD., Proc.Asce, Vol. 98 No SM3, March, pp. 265 - 290

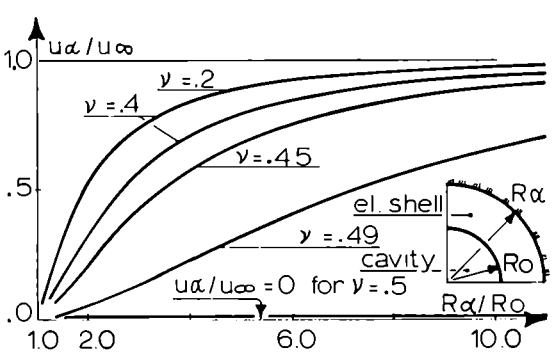

Fig. 1 - Elastic expantion with rigid boudary

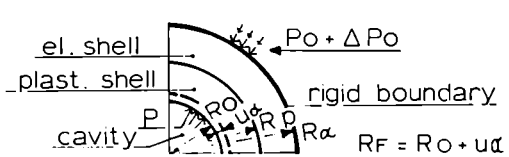

Fig. 2 - Elasto-plastic model with rigid boundary

Some limitations for horizontal stress measurements
Restrictions pour la mesure des contraintes horizontales

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It appears that measuring horizontal stress σ_h in sand by any driven-in device is properly impossible. Even a very thin blade or a self-boring tube produces shear stresses changing σ_h. The deviation from the undisturbed σ_h does not disappear by relaxation. Thus the initial soil response of the instrument is inevitably influenced by placement errors.

Vibrations of sufficient amplitude and/or duration bring back σ_h to the value of unpreloaded sand (we found this with stress cells in the laboratory). Therefore it is not always necessary to determine σ_h in a deposit if it has presumably been subjected to vibrations.

One can conclude from the theme lecture by Jamiolkowski et al. that K_o in soft clays is very close to the laboratory value. This can be explained by relaxation of minor placement disturbances. Consequently, field measurement of σ_h is not urgently needed in unpreloaded clay deposits.

Whereas σ_h in preloaded clay cannot be predicted from lab tests. In general it will also vary with horizontal directions (e.g. due to ice shift or recent tectonics). Reliable σ_h-measurements are of practical importance (e.g. for tunnels). Even if relaxation of placement disturbance is not as complete as in soft clay, self-boring pressuremeters and equivalent devices are a very promising aid for preloaded clays.