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# Technical session 1c: In-situ testing

## Séances techniques 1c: Tests in-situ

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## 1 INTRODUCTION

The primary objective of this report is to review the papers offered to this conference on the topic of in situ testing. By way of a general introduction there will be a general discussion on Ground Investigation (GI) and the role of in situ testing within it.

Successful geotechnical design and construction require a good knowledge of the mechanical behaviour of the ground including its spatial variability. The requisite information is gathered as part of a ground investigation programme.

The objective of any subsurface exploration programme is to determine the following:

1. Nature and sequence of the subsurface strata (geological regime)
2. Groundwater conditions (hydrogeological regime)
3. Physical and mechanical properties of the sub-surface strata (engineering regime)

For geo-environmental investigations of ground contamination there is the additional requirement to determine the

4. Distribution and composition of contaminants (geoenvironmental regime)

These requirements vary depending on the nature and extent of the proposed project and the perceived ground related risks. There are many techniques available to meet the objectives of a ground investigation and these include both field and laboratory testing of the ground. Laboratory tests include those that test elements of the ground, such as triaxial tests and those that test prototype models, such as centrifuge tests. Field tests include drilling, sampling, in situ testing, full-scale testing and geophysical tests.

An ideal ground investigation should include a combination of tests to classify the ground and to determine engineering parameters. In the past, this has led to an emphasis on laboratory testing for both classification and parameter determination. In practice, in situ tests have now evolved to the extent that they can complement other forms of GI. In situ tests can, however, often offer significant advantages over laboratory tests, for example:

- they can be quicker, easier and cheaper than sampling and laboratory testing,
- the soil can be assessed in its natural environment without the potential problems of sample disturbance,
- and the spatial variability of the deposit can be more fully investigated.

Table 1 (from Lunne et al., 1997) is an updated version of a table originally devised in the early 1980s and presents a list of some of the major in-situ tests and their applicability for use in different ground conditions. If this table were compared with the earlier versions it would be noticeable that the ratings of some tests have improved as the database of experience has built up, but others have declined as the initial predictions of their capability have been found to be wanting. There is little doubt that the levels of applicability given in Table 1 can now be attained or exceeded provided the tests are selected and used correctly. This applies to all levels of in situ testing from the simplest basic tests such as dynamic probing and the SPT, through tests such as the CPT and piezocone (CPTU), to the more complex devices such as the self-boring pressuremeter (SBP). Table 1 shows that a wide range of parameters can be reliably obtained from in situ testing. Ideally we would want all assessments in Table 1 to be 'A' and researchers and others continue to strive to raise the levels of applicability, however this may not always be achievable if there is a fundamental weakness in the appropriateness of the test.

It would appear that many practitioners have felt that too often the capabilities of in situ tests have been over-sold or inappropriately applied. This has resulted in dissatisfaction when the tests failed to deliver what was promised. The fact that we continue to see papers with more and more correlations for in situ devices often does little to improve this feeling; however it is often not the test that is as fault but its operation and application. Before anything else the results from an in situ test must be repeatable within the bounds of ground variability. There must be consistency in both equipment and operation wherever it is specified and used not just in one country or soil type, but around the world and this is particularly important if we are to transfer experience from one country to another or develop generic correlations. Consistency in data reporting and level of detail is also of primary importance.

We have seen with the CPT how the realisation of the effects of porewater pressure on measured cone resistance (Lunne et al., 1997) has enabled both consistency of results between devices and the potential for reduced scatter in new correlations.

In the SPT we have over the years seen how the results can show significant scatter resulting not just from the simplicity of the test and operator influences but more importantly from how equipment variations, and particularly the energy input from different equipment, influences the results.

Once we can produce measurements from in situ tests that are repeatable to the extent that it is inherent ground variability that is the controlling factor then experience gained can reliably be re-used elsewhere and confidence can be developed. The applicability of different tests in various soils and the reliability of parameters determined from those tests can be better assessed.

Table 1: The applicability and usefulness of in situ tests. (Lunne et al., 1997)

| Group | Device | Soil Parameters | | | | | | | | | | | | | Ground type | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Soil Type | Profile | u | *ϕ' | Su | $I_D$ | $m_v$ | $c_v$ | k | $G_0$ | $\sigma_h$ | OCR | σ–ε | Hard rock | Soft rock | Gravel | Sand | Silt | Clay | Peat |
| Penetrometers | Dynamic | C | B | - | C | C | C | - | - | - | C | - | C | - | - | C | B | A | B | B | B |
| | Mechanical | A | A/B | - | C | C | B | C- | - | - | C | C | C | - | - | C | C | A | A | A | A |
| | Electric (CPT) | B | A | - | C | B | A/B | C | - | - | B | B/C | B | - | - | C | C | A | A | A | A |
| | Piezocone (CPTU) | A | A | A | B | B | A/B | B | AB | B | B | B/C | B | C | - | C | - | A | A | A | A |
| | Seismic (SCPT/SCPTU) | A | A | A | B | A/B | A/B | B | AB | B | A | B | B | B | - | C | - | A | A | A | A |
| | Flat Dilatometer (DMT) | B | A | C | B | B | C | B | - | - | B | B | B | C | C | C | - | A | A | A | A |
| | Standard (SPT) | A | B | - | C | C | B | - | - | - | C | - | C | - | - | C | B | A | A | A | A |
| | Resistivity probe | B | B | - | B | C | A | C | - | - | - | - | - | - | - | C | - | A | A | A | A |
| Pressuremeters | Pre-Bored (PBP) | B | B | - | C | B | C | B | C | - | B | C | C | C | A | A | B | B | B | A | B |
| | Self boring (SBP) | B | B | A[1] | B | B | B | B | A[1] | B | A[2] | A/B | B | AB[2] | - | B | - | B | B | A | B |
| | Full displacement (FDP) | B | B | - | C | B | C | C | C | - | A2 | C | C | C | - | C | - | B | B | A | A |
| Others | Vane (FVT) | B | C | - | - | A | - | - | - | - | - | - | B/C | B | - | - | - | - | - | A | B |
| | Plate load | C | - | - | C | B | B | B | C | C | A | C | B | B | B | A | B | B | A | A | A |
| | Screw plate | C | C | - | C | B | B | B | C | C | A | C | B | - | - | - | - | A | A | A | A |
| | Borehole permeability | C | - | A | - | - | - | - | B | A | - | - | - | - | A | A | A | A | A | A | B |
| | Hydraulic fracture | - | - | B | - | - | - | - | C | C | - | B | - | - | B | B | - | - | C | A | C |
| | Crosshole /Downhole / Surface seismic | C | C | - | - | - | - | - | - | - | A | - | B | - | A | A | A | A | A | A | A |

Applicability: A = high, B = moderate, C = low, - = none
*ϕ' = will depend on soil type; [1] = only when pore pressure sensor fitted; [2] = only when displacement sensor fitted.
Soil Parameter definitions: u = in situ static pore pressure; ϕ' = effective internal friction angle; $s_u$ = undrained shear strength; $m_v$ = constrained modulus; $c_v$ = coefficient of consolidation; k = coefficient of permeability; $G_0$ = shear modulus at small strains; OCR = overconsolidation ratio; σ-ε = stress-strain relationship; $I_D$ = density index

## 2 REALISING THE FULL POTENTIAL OF IN SITU TESTING

So how can we improve the situation and get the best from existing tests without developing new ones?

- firstly, whoever is specifying a ground investigation programme should always consider in situ testing.
- they should at least have a basic understanding of the various tests and their strengths (and weaknesses or limitations).
- they should be able to select the right test for the situation and once a decision has been made then be able to specify the correct equipment and procedures for achieving the desired results. They need standards available to them that will ensure that the tests are carried out with consistency (NB. For confidence in the results there must always be a way of checking the quality of the data determined).
- they should also consider whether additional information might be required later, for example as a result of design changes. (It may well cost very little extra to gather that data at the same time thus avoiding having to make the best of the original data or incurring re-mobilisation costs later).

We are seeing increasingly in some countries and in CEN and ISO standards currently in preparation, that specifications for test procedures are now trying to help guide the specifier, for example having various specified classes of accuracy for CPT based on soil type and data use (profiling or soil parameters). Furthermore, we should be encouraging accreditation procedures for in situ testing. If current procurement has resulted in cost cutting and bad practice, then this should be firmly discouraged even if some small additional costs are incurred.

With in situ testing we have at our disposal very powerful tools that can yield a great deal of valuable information as part of a well planned GI provided they are specified and used correctly. We should not be specifying them without due thought to the end result and the reliability we can put on the data gathered. The lessons learnt from the past must be used to ensure that as other in situ tests are developed they are validated with reliable databases and that specifications and procedures allow their full potential to be developed.

We do not all have to be experts in in situ testing but a sound understanding of the test methods and equipment, coupled with improved specifications and better guidance will ensure that the engineer is able to realise the full potential of these tests.

By selecting the right configuration of tests, in situ testing will give advantages over the traditional combination of borings, sampling and other testing, and can provide complementary data. namely:

1. continuous or near continuous data
2. repeatable and reliable data
3. speed of operation (potential for shorter GI timescales)
4. cost savings.
5. A wide range of parameter data from a single test

Table 2. The papers to session 1c.

| *Device* | *Authors* | *Title* |
|---|---|---|
| CPT/CPTU | Powell, J.J.M. & Lunne, T. | A comparison of different sized piezocones in UK clays |
| | Karlsrud, K., Lunne, T., Kort, D.D., & Strandvik, S. | Revised correlations for interpreting CPTU is clay soils; effects of sensitivity, OCR and $I_p$ are considered |
| | Hamza, M.M., Shahien, M.M. & Ibrahim, M.H. . | Characterization and undrained shear strength of Mile Delta soft deposits using piezocone |
| | Mlyknarek, Zb., Tschuschke, W.,Wierzbicki,J, & Wolymski, W | Statistical criteria of determination of homogenous geotechnical layers. |
| | Lee, W.J., Kim, T.J. & Kim, S.I. | CPTU dissipation behaviour of overconsolidated clay. |
| | Mayne, P.W. & Campanella, R.G. | Versatile site characterization by seismic piezocone. |
| Seismics | Areias, L. & Van Impe, W.F. | Selecting a seismic source for the SCPT test. |
| | Stokoe, II, K H. & Rathje, E.M. . | Development of an in situ method to measure the nonlinear shear modulus of soil |
| | Yu, X., Drnevich, V.P. & Nowack, R.L. | Near surface soil properties using electromagnetic and seismic waves. |
| | Di Benedetto, H., Geoffroy, H.,Duttine, A., Sauzeat, C. & Chau, B. | Anisotropic behaviour of soils and site investigation based on wave propagation tests. (In French) |
| | Lee, J.S., Carlos Santamarina, J., Li, Z. & Kutter, B.L. | Geophysical process monitoring in scaled models. |
| MPM & SBP | Bahar, R., Aissaoui, T. & Kelanemer, S. | Comparison of some methods to evaluated the undrained cohesion of clays from in situ tests. (In French) |
| | de Sousa Coutinho, A.G.F. & Lukdovico Marques, M.A. | Ménard and Cambridge self boring pressuremeters: correlations between mechanical parameters in Lisbon Miocene clayey soils. |
| | Tan, N.K. & , Miller, G.A. | Pressuremeter testing in a calibration chamber with unsaturated Minco Silt. |
| DMT | Akbar, A., Kibria, S. & Clarke, B.G. | The Newcastle dilatometer testing in Lahore cohesive soils. |
| SPT, LPT, DP etc | Aoki, N., Esquivel, E.R., Neves, L.F.S. & Cintra, I.C.A. | SPT sampler static penetration resistance in the case of a sandy soil. |
| | Kulhawy, F.H. & Chem, J.R. | Evaluation of penetration tests and their correlations in gravely soils. |
| | Ampadu, S.I.K. | A correlation between the dynamic cone penetrometer and bearing capacity of a local soil formation. |
| | Stanichevsky, M. & Bosio, J.J. | The dynamic penetration cone index as an alternative for the control of sub grade surface. |
| PLT | Zhusupbekov, A.Z., Zhakulin, A.S. & Bakenov, H.Z. | Determination of the mechanical characteristics of soils by results of plate load tests. |
| | Teme, S.C. & Eton, G. | Determination of soil bearing pressures using a Modified Plate Load Testing in the Nigerian Niger Delta. |
| | Souza Neto, J.B., Coutinho, R.Q., & Lacerda, W.A. | Evaluation of the collapsibility of a sandy soil by in situ collapse tests. |
| Case History | Viana da Fonseca, A., Carvalho, J., Ferreira, C., Santos, J., Almeida, F. And Hermosilha, H. | Combining geophysical and mechanical testing techniques for the investigation and characterization of ISC'2 residual soil profile |
| | Steenfelt, J.S., Hansson, L. & Dakheel, A.L.A | Site characterization for Sheikh Al Jaber Al Ahmed Causeway Project |
| Rock Mass | Kovacevic, M.S., Skazlic, Z.,Szavits-Nossan, V. . | Case histories of very hard fissured soils stiffness determination |
| Lab & field | Ejjaaouani, H., Magnan, J.P. & Shakhirev, V. | Changes in the physical and mechanical properties of expansive clays during wetting (in French). |
| other | Aitaliev, Sh.M., Baimakhan, R.B., Sydykov, A.A. & Muzdakbaev, M.M. | Moving Factors of Regional Geodynamics of the Caspian Sea |

It should also be remembered that the power of in situ tests is not restricted to soil parameter determination; there are also many examples of their use in indirect design applications where parameters unique to a particular test type can be used directly in design procedures.

## 3 REVIEW OF PAPERS

Gathering information on the use and experiences of in situ testing in an ever wider range of geomaterials allows us to both consolidate and hopefully improve what we have, to have greater confidence in their applicability (this might mean rejecting their use in some materials) and to develop additional techniques where there is a gap to be filled. To this end this session has 27 papers from 19 countries covering all 6 of the populated continents (Antarctica would have completed the set) which will be helping us to gain additional in-sight into the application of in situ tests. Table 2 lists the papers to this session and the breadth of topics covered is a good example of their versatility and potential usefulness for ground investigations. The papers offer experience in:

- Equipment development
- Improving existing correlations
- Developing new correlations
- Increasing the range of soil types
- Combining different devices for greater coverage and detail.

The papers can most conveniently be divided by the primary device type as Table 2 and as follows;

- 6 CPT and derivatives
- 5 assorted geophysical
- 3 pressuremeters
- 2 SPT
- 2 DP
- 3 Plate
- 1 DMT
- 2 case histories/range of tests
- 1 rock classification
- 2 others/misplaced

Although not a primary test in terms of papers, reference is also made for correlation purposes to vane testing and of course laboratory testing.

In the following author references without a date assigned to them and shown italicised and underlined, refer to papers listed in Table 2 and contained within these proceedings.

The reader should of course also read the excellent state-of-the-art report on in situ testing to this conference by Schnaid and the recent proceedings of ISC2 (Viana da Fonseca and Mayne, 2004).

### 3.1 *Cone Penetration testing*

We have 6 papers covering cone, piezocone and seismic cone testing.

In their paper *Powell and Lunne* look at the repeatability of 5 different cones of 10 and 15 cm$^2$ sectional area on 4 well documented soft to stiff clay soils and also consider how the cones conform to the requirements for accuracy of the International Reference Test Procedure (IRTP, 1999) (this document will soon be superseded by the new CEN/ISO standard on Cone and Piezocone testing, EN ISO 22476-1;2006). The results clearly confirm the now well known fact that measured cone resistance ($q_c$) is a function of individual cone inner geometry as there is a need to allow for pore water pressure effects on the measured cone resistance, this is shown when an apparently large scatter in results (in terms of $q_c$) becomes a very good agreement between the cones when data are compared as corrected cone resistance ($q_t$). They also show that good consistency can be achieved for sleeve friction ($f_s$) and pore water pressure behind the cone ($u_2$) irrespective if cone type or size. This consistency in $f_s$ may to some extent be the result of a generally larger scatter in the results probably related to lack of sensitivity in the measurements and zeros (Lunne and Powell 2005, suggest that in a separate investigation then there may be evidence pore water pressure effects can become significant on friction sleeve results also and this would have implications for fiction ratio and soil identification). It should be pointed out that all the work reported by *Powell and Lunne* was undertaken by the same operators and all cones were calibrated to a single source which would eliminate some of the variables in routine practice.

They conclude that the most consistent parameter especially in soft soil is the pore water measurement provided good saturation is achieved and maintained. Provided care is taken in calibration and cone set up then results of corrected data from 10 and 15 cm$^2$ cones are generally comparable in clay soils. The general implications for practice are that correlations based on 10 cm$^2$ can be used with the same confidence for 15 cm$^2$ cones.

However there are general considerations of accuracy applicable to all cones when the new accuracy classes of the IRTP 1999 are considered, especially in soft soils. Once we want to generate parameter information then we need to know the accuracy of the measured results to have an idea of the potential inaccuracies in the interpreted results.

The paper also shows that there is still a need for better resolution in measurements if the highest accuracy classes are to be achieved especially in soft soils. It also shows the excellent detail than can be achieved when in CPTU profiles if reading frequency is increased.

*Karlsrud et al* look at redefining our basic correlation parameters for obtaining undrained shear strength and overconsolidation ratio in soft to medium stiff clays from CPTU tests. The correlations are based on a very high quality database of undrained triaxial compression strength and preconsolidation pressure determined on very high quality block samples taken with the Sherbrooke 250 mm sampler. The data base covers 17 different sites having plasticity indices from 10 to 50 % and sensitivities from 3 to about 200 (note the soils are generally normally to lightly overconsolidated) . For undrained strength they investigate the use of 3 different cone factors, the well known $N_{kt}$, as well as $N_{ke}$ and $N_{\Delta u}$ as defined below:

$$N_{kt} = \frac{q_t - \sigma_{v0}}{s_u}$$

$$N_{ke} = \frac{q_t - u_2}{s_u}$$

$$N_{\Delta u} = \frac{u_2 - u_0}{s_u}$$

where $N_{kt}$ is the cone factor for corrected cone resistance
$N_{ke}$ is the 'effective' cone factor
$N_{\Delta u}$ is the 'excess pore pressure' cone factor

They comment that their data clearly show that the measured excess pore pressure gives the best and most consistent results; the cone resistance shows fairly large scatter, which may partly be an equipment/measurement problem and partly reflect that the cone resistance is a more complex parameter than the pore pressure response and which depends on more subtle clay characteristics. In developing their correlation factors they also suggest that they depend on the clay sensitivity and the plasticity index of the clays tested and also the type of cone used. They also consider the potential accuracy or more correctly inaccuracy in the measured CPTU parameters on their correlations.

As a result they propose a new set of correlations for $s_u$ based on the above 3 cone factors but also related $S_t$ and $I_p$ but also OCR and, in the case of $N_{ke}$ , $B_q$ ($B_q= (u_2 - u_0)/(q_t - \sigma_{v0})$). Figure 1 shows this latter correlation. They give new tentative

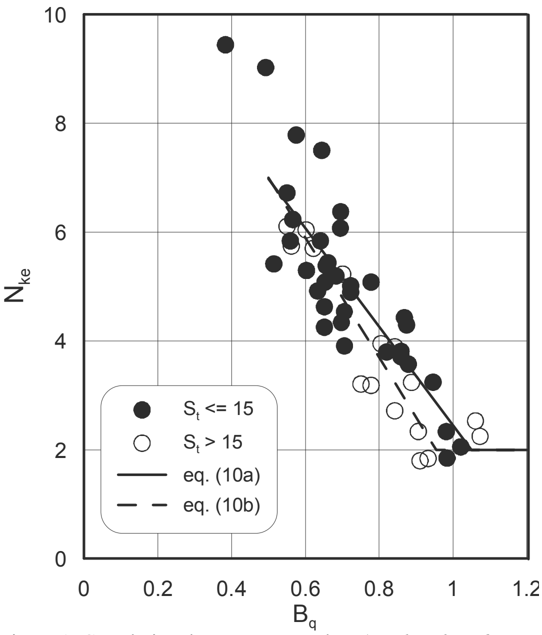
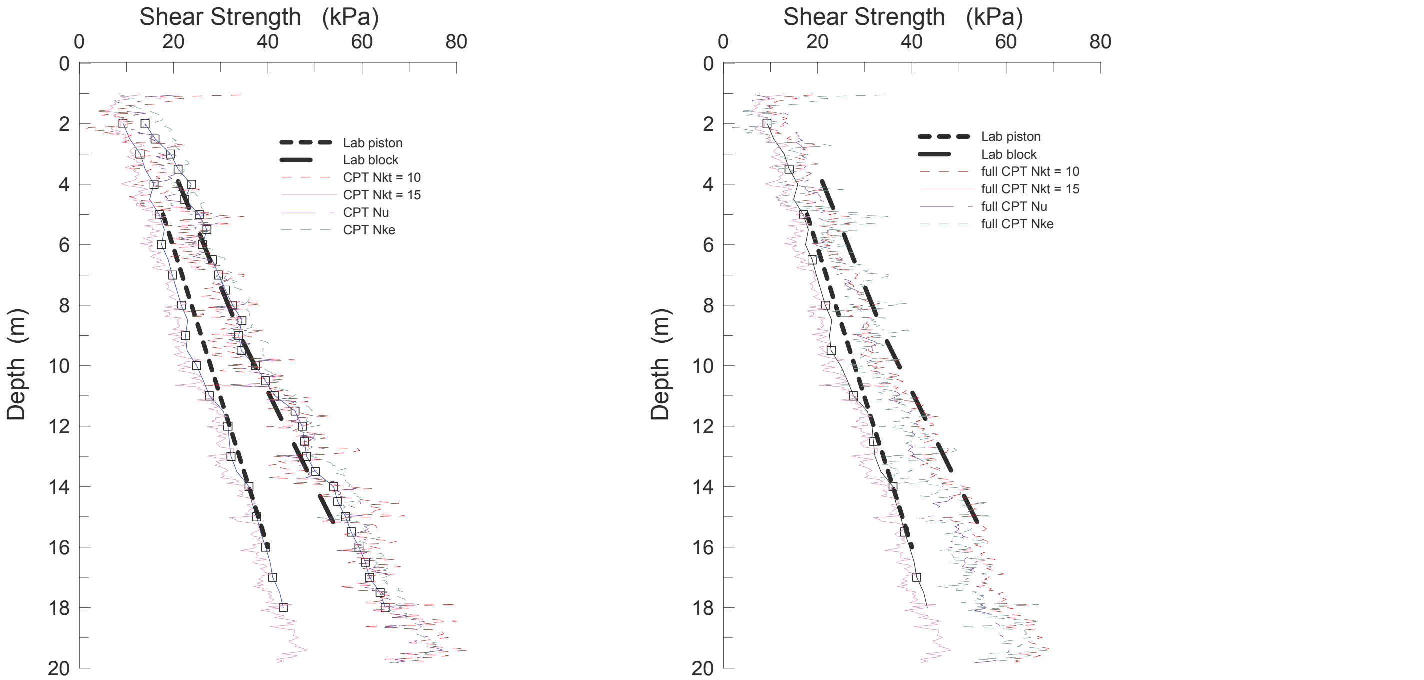



Figure 1. Correlation between $N_{ke}$ and $B_q$ (*Karlsrud et al*)




(a) (b)

Figure 2. Shear strengths determination at Bothkennar. a) using previous correlations (Powell 2001) b) based on the new correlations

equations for determining the cone factors which are split by sensitivity (above and below 15) and which also contain OCR and $I_p$. As a result information on OCR is required and they look at correlations between cone parameters and OCR and conclude that the best involve normalised cone factor $Q_t$ and OCR but the reliability of this is less than for shear strength (maybe a more site specific approach would be more appropriate - see Lunne et al., 1997).

They suggest that the reliability in $N_{\Delta u}$ determined strengths (±10%) may be higher than others as a result of the more consistent pore water pressures mentioned above.

It is interesting in Figure 2 to compare profiles of shear strength at the Bothkennar test site (one that the authors find does not always fit their correlations). Figure 2a shows how their earlier correlations related entirely to $B_q$ seemed to have performed extremely well with good consistency between the results based on all 3 factors; in Figure 2b the same level of consistency appears but the strengths are now slightly lower on average. Also shown in the figures are strengths based on more traditional interpretation of cone profiles ($N_{kt}$ from more general charts) and these are seen to more closely relate to strengths measured on piston samples rather that the high quality block samples (showing a need to link the interpretation with the desired type of strength). The authors' rightly comment that 'when using the strengths derived from the proposed CPTU correlations, a designer must keep in mind that the undrained strength determined represents the peak undrained triaxial compression strength on high quality samples'.

The authors' have presented some high quality work and offer some very promising correlations that will be an enhancement to CPTU interpretation. They rightly say that care should always be taken when using any correlation in understanding what the original source data was derived and that further validation will be required.

*Hamza et al* present a case history combining the use of CPTU, vane and sampling to study Nile delta deposits to determine stratigraphy, soil properties and natural variability. Deposits comprise silty sand and layered clay, organic silt and sand. They show how well the CPTU profiles in these deposits and also how the Robertson 1990 charts in terms of normalised cone parameters identify the changing deposits except for the organic silts which are missed, as such, on the chart. It should be noted though that they would have been identified as a distinct layer in the CPTU profiles, and could then have been identified in borehole logs. This is probably a case of never look at CPTU data in isolation, it always needs a level of calibration. They also look at interpretation of test results for undrained shear strength. They correlate corrected vane strengths to net correct cone resistance to establish $N_{kt}$ values (see above). They show that the Aas et al (1986) correlation of $N_{kt}$ to $I_p$ fits remarkably well at low $I_p$ but needs adjustment at higher $I_p$ and they offer a new correlation as shown in Figure 3.

The significant difference between the $N_{kt}$ values from this work compared to those from *Karlsrud et al* further highlights the importance of knowing the source of the shear strength measurements in empirical correlations as well as knowing which strength you wish to predict from the CPTU.

With any in situ test that gives a near continuous profile there is always a desire to be able to group the data mathematically to establish the stratigraphy both vertically and laterally.

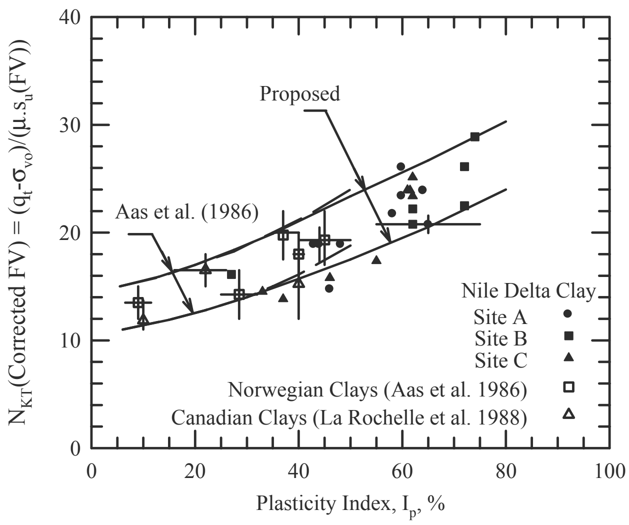



Figure 3. $N_{KT}$ against $I_P$ (*Hamza et al*)

Mlynarek et al apply statistical methods to CPT data to try to establish distinctive homogeneous layers within their CPT profiles. They look at different techniques and use cluster analysis based on both normalised cone resistance, $Q_t$ (= $(q_t - \sigma_{vo})/ \sigma'_{vo}$) and friction ratio, $R_f$. They show that if cluster integration is taken too far it can be detrimental and detail within the profile can be lost. Whilst this type of approach can help our overall view of the data it is important that this type of treatment is used as a tool and that details that are evident in the original CPTU profiles are not lost during interpretation.

Lee et al look at the problem of interpreting CPTU dissipation tests in heavily overconsolidated clays. The problems of dilation that generally accepted theories based on normally or lightly overconsolidated clays may not apply. They say they have developed the method of Burns and Mayne (1998) (based on a hybrid cavity expansion-modified Cam clay model) to include the effect of the initial anisotropic stress state. To verify the model they undertook piezocone dissipation tests in a calibration chamber at OCRs of 1, 5, 10 and 20. They found that spherical cavity predictions of initial excess water pressure were much too high when compared to the calibration chamber results and that a cylindrical cavity approach gave much better fit. The model needs to take account of the relative sizes of the plastic and shear zones and they show that the initial radial variation of excess pore pressure is predicted reasonably well but slightly lower than measured at OCRs of 10 and 20. It will be interesting to see how this is developed into modelling dissipation test results.

Mayne and Campanella propose that the seismic piezocone test with dissipation phases offers the optimal SI technique to obtain up to 5 independent readings relating to soil behaviour ($q_c$, $f_s$, u, $V_s$, decay of pwp). Table 1 supports this idea showing how this configuration gives the highest overall rating of applicability for any one test. (Note: a new guideline on seismic cone testing has just been produced by TC10 of ISSMGE and is published in these proceedings, Butcher et al 2005).

They propose that this level of testing should be the minimum level of effort during geotechnical investigations as we can obtain the standard CPTU data with the additional seismic test data. We see that the addition of geophones allows the measurement of arrival times of shear waves to be measured and hence their travel velocities to be calculated in 'downhole mode' ($V_{vh}$) (Figure 4 shows the different orientations of typical seismic tests which are discussed later); from these small strain shear modulus values can be obtained by:

$$G_{vh} = \rho V_{vh}^2$$

where ρ is soil mass density.

It should be remembered that the ground is anisotropic and this same equation is used with other orientations of shear waves to establish small strain stiffness anisotropy, i.e. $G_{vh}$, $G_{hv}$ and $G_{hh}$ and equally when establishing any correlations between parameters this must be remembered (Powell and Butcher 2004).

Mayne and Campanella show how correlations exist linking estimates of total unit weight, small strain shear modulus, effective friction angle, preconsolidation pressure, undrained shear strength equivalent soil modulus and consolidation and permeability and all can be estimated from a CPTU test with due regards to the source of the correlations.

### 3.2 *Geophysics and related items*

The previous paper above leads logically into the use of geophysical testing to determine soil stiffness. For convenience Figure 4 shows the different types of shear waves that can be generated. In the following the subscripts refer to firstly the direction of propagation and secondly the direction of polarisation of the waves e.g. $V_{vh}$ refers to vertically propagating horizontally polarised shear wave – a downhole test.

Areias et al look at the effect of source type on the results of seismic cone penetration testing (SCPT). They look at a statistical analysis of arrival times from differently constructed shear beams for downhole seismic cone work. By undertaking multiple strikes they are able to produce a normal distribution curve for arrivals from each beam and show that a solid steel beam gives more statistically reliable and consistent data than a steel H beam (both using swing hammers falling through $15^o$). However it is a little surprising that not only is there a difference in the repeatability (standard deviation) of the two beams but also the mean of the arrival times is also different, they say about 7% but this could be even greater if fewer repeat strikes were made. This would have implications for the accuracy in single over dual array seismic cones (see Butcher and Powell, 1995). Rather surprisingly, early in the paper they cite that decoupled beams give higher energy transfer to the ground but the experiments on the selection of beam use coupled systems! Field tests on a different site (a dense sandy soil) used a decoupled beam and non uniform (sledge) hammer strikes! They show that arrival times measured in the SCPT test method constitute a continuous random variable that can be represented by a normal distribution. This finding has obvious implications on the accuracy of measured arrival times and should be considered in the future to improve the test method. Might a dual element seismic cone have mitigated these problems here also?

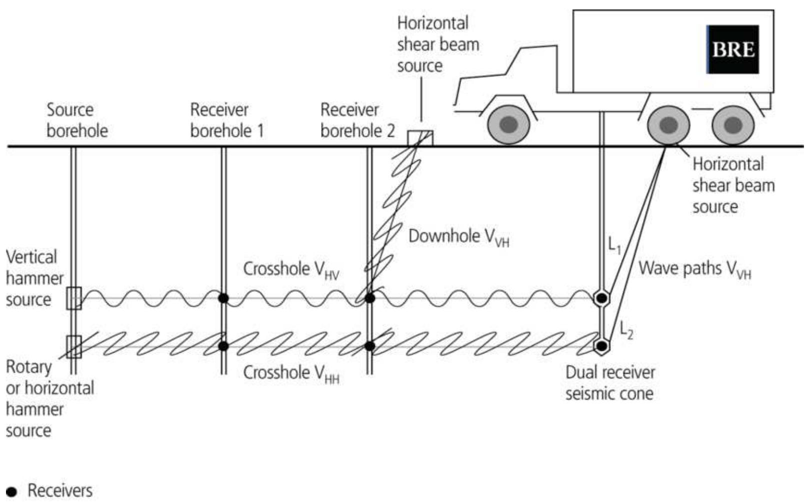



Figure 4. Typical shear wave orientations

The paper by *Stokoe et al* reports some interesting field scale experiments to assess, in situ, modulus and modulus degradation using seismic waves. The method involves applying static and dynamic loads at the surface of a soil deposit and measuring the dynamic response of the soil mass beneath the loaded area using embedded instrumentation. They use a vertically loaded pad which is struck laterally by a swing hammer as a source for downhole testing, and two adjacent boreholes are used for crosshole measurements. The buried instrumentation beneath the pad is used to pick up seismic waves. Testing is conducted at a range of vertical loads (time being allowed at each load to equilibrate) and also a range of dynamic loads (achieved by varying the fall height of the swing hammer). The impact loads on the pad yield $V_{vh}$ and the crosshole tests give both shear wave velocities, $V_{hv}$, as well as compression wave velocities. The soil studied is poorly graded sand which was heavily overconsolidated due to excavation. Changes in shear wave velocities with changing stresses are recorded and a relationship between shear wave velocity $V_{hv}$ and the stresses as $\sigma_v x \sigma_h$ established. Changing the lateral loads allows different strain levels to be applied and through this the shear modulus decay at varying strains can be established (data was recorded up to 0.01% shear strain, which is significantly higher than normal seismic waves). Combining all information allows a shear modulus strain decay curve to be established. The authors' comment that with better understanding then larger strains could have been achieved. The results are very encouraging and the data fits well with Seed et al (1986) curves for sands (see Figure 5); this work could be a significant step forward in in situ testing for modulus and modulus degradation; future developments will be interesting. Currently has only been tried at shallow/near surface depths but would appear to have potential as a practical field method if extended greater depths. Could also have applications if used with Rayleigh waves with receivers further a field.

*Yu, et al*. investigated new techniques for assessing the near surface properties of fills. They undertook laboratory and field experiments to find all parameters for soil modulus and modulus degradation using in situ tests. They use 2 geophysical techniques to assess soil properties, Time Domain Reflectometry (TDR) which can be used to assess soil water content and density, and seismic waves for modulus and modulus degradation. The test setup uses a series of short spikes (around 210mm) pushed into the ground; initially they are used for TDR measurement using electromagnetic waves; then having removed the top cap one of them is struck, the others having had accelerometers attached to them and a pseudo crosshole test undertaken. By varying the tapping force they establish a modulus reduction curve. Tests were undertaken on 2 soils, silty loam in the field and Ottawa sand in the laboratory, the results are very encouraging at laboratory scale with obvious potential for field scale.

The paper by *Benedetto et al* is very interesting using a new soil model in conjunction with field crosshole data to predict behaviour in granular soils. The new hypoelastic soil model (DBGS) was developed based on laboratory testing of Hostun and Toyoura sands in hollow cylinder apparatus. It establishes the very small strain domain a symmetrical tensor stress strain matrix to model the behaviour in the laboratory tests. The model is then validated by using the results of field crosshole tests on a site for a new nuclear power station. The profile comprises sands and gravels overlying grey marls and sands, then limestone and with granite and finally granite. They use both P and S wave data to allow them to identify the parameters of the model assuming isotropic elasticity and cross anisotropic elasticity. They found that by using both data and taking anisotropy into account then the model allowed them to make better assessments of Poisson's ratio variations down their profile. The procedures would appear to have significant potential.

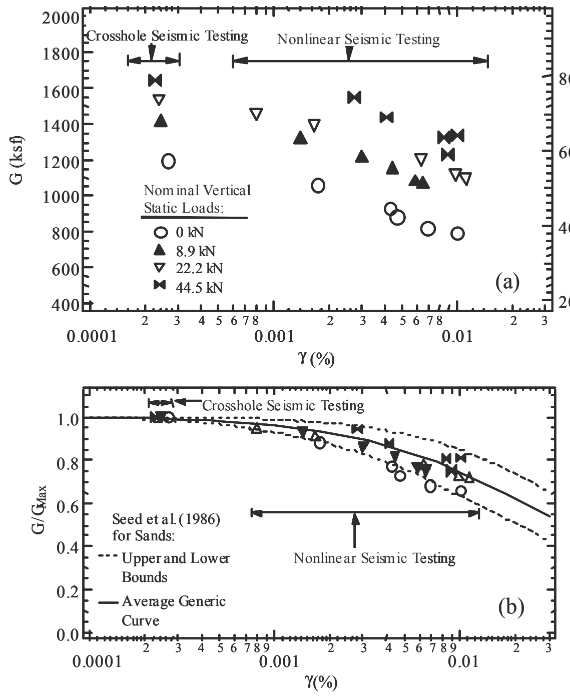



Figure 5. Stiffness decay curves (from *Stokoe et al*)

In their paper *Jong-Sub Lee et al* state that the intent of their work was to develop a set of versatile wave-based technologies that cause none or minimal disturbance to the soil mass or to the processes taking place within it. The soil properties they wished to establish were: P-wave velocity (a function of saturation, porosity, skeletal stiffness), S-wave velocity (a function of skeletal stiffness: cementation, effective stress), and electrical resistivity (a function of porosity, pore fluid conductivity, soil type and specific surface). The techniques rely on complementary elastic and electromagnetic waves. Whilst the work was carried out at 1g they see applications with shaking tables and the centrifuge.

The work was laboratory based at small scale mainly on cohesionless deposits, they use P-waves to detect layering, piezobenders to measure shear waves, including a look at the near-field effects, and they show that piezobenders with trans-illumination can show the stiffness changes during and subsequent to liquefaction in a sand column. In addition they look at the use of S waves for tomography and also use a needle type impedance probe intended to show layering. Finally they discuss the development of a system of electrical resistivity tomography. The systems all seem to be evolving and show some level of potential for model scale testing. There appears to be a great potential in complementary wave based test systems that now need to be adapted for field use.

## 3.3 *Pressuremeters*

As can be seen from Table 1 the various pressuremeters are potentially one of the more reliable / powerful GI techniques with regards to the range of data that can be obtained. We have 3 papers to the session that concentrate on pressuremeter testing.

*Bahar et al* look at the interpretation of Menard pressuremeter tests on 3 Algerian clay sites with shear strengths in the range 50 to 500 kPa. They compare the results from 3 methods for interpreting undrained shear strength from the test results namely; the empirical method of Amar & Jezequel (1972), the numerical method called PRESSIDENT which is a numerical program taking into account the Duncan and Chang (1970) model and the method developed by Bahar and Olivari (1993).

The latter 2 use modelling to curve fit to the test results and thereby the $s_u$ from the input parameter to the model; they give similar results with the Bahar and Olivari appearing simpler to use. However these two give much higher strengths than the Amar & Jezequel results, in some cases by more than a factor of 2. The correlations ideally need to be validated against some other from of shear strength assessment in order to give confidence in their applicability. The authors do compare their results at one site with CPT assessed shear strengths using they say $N_k$ factors of 10 -15 but these also should be validated as often on very stiff clays higher values for $N_{kt}$ have been reported (Lunne at al., 1997).

The second pressuremeter paper by Sousa Coutinho and Marques compare the results of MPM and SBP tests in Lisbon's Miocene oveconsolidated clayey soils with shear strengths up to 2000kPa (one would think more a soft rock). They computed strength, stiffness and $K_0$ values from the different test results. As the SBP tests give a wider range of parameters directly from the test interpretation, they use these results to cross correlate to the MPM test data. For example using strength results from the SBP (based on Gibson and Anderson, 1961) they conclude that strength from the MPM can be obtained using the expression:

$$s_u = \frac{p_{LM} - \sigma_{ho}}{5.75} + 175$$

where $p_{LM}$ is the Ménard Pressuremeter limit pressure.

They compare this correlation with others in the literature and show that most others significantly under-predict the strengths but rightly admit that most were probably developed for lower limit pressure ranges. However is should be noted that, for example, the Marsland and Randolph correlation was developed to be used with the limit pressures at infinite expansion and so should be used only with that limit pressure; which as shown by Sousa Coutinho and Marques can be up to 40% higher than the MPM value (their fig 3). If at selected depths on their Figure 3 the MPM limit pressure are taken and the corresponding SBP $p_L$ values then used in the Marsland and Randolph equation in order to calculate shear strength, then the new line in Figure 6 is obtained. Marsland and Randolph give:

$$s_u = \frac{p_L - \sigma_{ho}}{6.18}$$

where $p_L$ is the infinite expansion limit pressure.

There appears then to be much closer agreement with the authors' correlation; whether this is fortuitous or not is not known. For comparison also shown in the Figure is the correlation of Bahar et al mentioned above, but this was only developed for $p_{LM}$ values up to 1800kPa. This shows danger if correlations are not related to their original source data ranges and soil types. One should remember that it may not always be correct to assume that soft rocks behave in an undrained cohesive way and 'shear strengths' can be over predicted (Mair and Wood, 1987).

This is an interesting approach correlating two pressuremeters to each other to improve the data quality of the more robust one. The authors' conclude that the correlations developed greatly improved the understanding of the unit and will enhance future site investigations using the MPM test.

The third pressuremeter paper by Tan et al looks at the interpretation of pressuremeter tests in unsaturated soils. The work was undertaken in a calibration chamber with a miniature pressuremeter test (MPMT) on unsaturated Minco silt (clayey silt). The effect of varying the matrix suction as well as the net normal stress was studied. Cavity expansion theory was incorporated in the analyses of the experimental results. This interesting study showed that matrix suction, net normal stress and dry unit weight have a strong influence on the limit pressure; the limit pressure increases with suction, net normal stress and dry unit weight. In the preliminary analysis of the PMT data the cylindrical cavity expansion equations for unsaturated soil have been utilized. The results obtained from the analysis seem promising for the future development of a method to interpret PMT results in unsaturated soil.

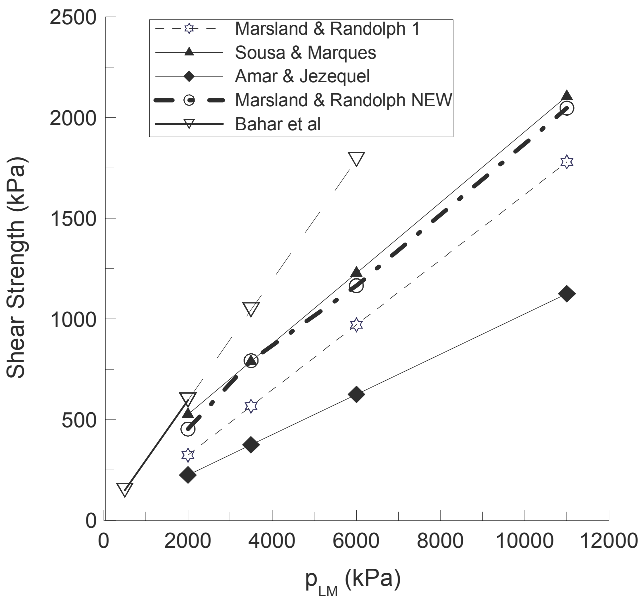



Figure 6. Shear strength from $p_{LM}$ modified from *Sousa Coutinho and Marques*

### 3.4 *Dilatometers*

One paper to the session looks at a development of the Marchetti Dilatometer test. Akbar et al report the development of a new version of the dilatometer which uses a rigid disc as the membrane in place of the flexible one and called the Newcastle Dilatometer test (NDMT). The movement of the disc and the pressure applied are monitored throughout the displacement to 1.1mm. Testing was performed on cohesive soils in Lahore.

The results were correlated with measurements from SPTs and 'undisturbed' soil samples. From the test pressure expansion curve they determine 3 pressures (($p_B$, $p_E$, $p_{1.1}$ ) that do not directly relate to Marchetti's $p_0$ and $p_1$. As a result the Marchetti parameters namely, Horizontal stress Index ($K_D$), Material Index ($I_D$) and Dilatometer Modulus ($E_D$) are redefined in terms of the new pressures. It would appear that they had hoped to then relate these new versions to the original Marchetti ones in order to utilise the existing databases. However it appears that this has not yet been possible owing to the limited database of NDMT results but alternative correlations in terms of the new $K_D$ etc are being developed. It would have been useful to see direct comparisons between to the standard and redefined DMT parameters. The device widens the database of in situ tests and the NDMT may prove to be a cheap robust alternative when testing stiff soils. The authors rightly suggest further validation of the correlations will be needed in the future.

### 3.5 *SPT and DP*

In the paper by Aoki et al we return to the long established SPT test, which has been so widely used and abused over the years. The authors look at correlating measured efficiency using an instrumented rod just above the sampler with the results of static load tests on the sampler immediately after cessation of driving. Based on data from a sandy soil deposit tested above the water table, they compare the measured kinetic energy with the 'deformation' energy from the static test and show remarkably good agreement. These data are used to assess the efficiency and the authors suggest that they confirm that it is possible to evaluate the impact efficiency from the work done during the static load test.

The authors rightly stress that the approach needs to be validated on different soils and below the water table. Changing the soil type may well affect the outcome as a result of rate effects possibly, an interesting piece of work.

The measurement of energy input in the SPT test will do much to improve the data available for correlations that can be developed and it is good to see that the latest standards for the test are at long last beginning to require actual efficiency measurements for the equipment (EN ISO 22476-3.2003).

In continuing the SPT theme *Kulhawy & Chen* review the use of various Large Penetration Test devices (LPT) and the Becker Penetration test (BPT) with the SPT in Gravelly soils. They summarise the previous publications and conclude that it appears that good correlations suggest that SPT and LPT are comparable, if evaluated properly (i.e. taking all factors into account). The correlations between BPT and SPT are less well developed and require substantial additional measurement; all need to be used cautiously.

Is this again the case of trying to 'short-circuit' the development of correlations for a new device by correlating to an existing one so that it's correlations to geotechnical properties etc can be used. This approach may be valid for different sizes of the same equipment but less applicable when the actual test mechanisms are different such as with the BPT.

Continuing with dynamic penetration tests *Ampadu* looks at the dynamic cone penetrometer (DCP or DP) as a simple SI tool for simple structures. Testing was undertaken on two different soils in a mould to correlate with bearing pressure. They used hand operated light weight equipment, 10kg mass falling 460 mm with a 20mm 60$^o$ cone. Tests were undertaken on samples of sandy to silty clay compacted into moulds and showed remarkably linear increases of blow count with depth in all samples. Using basic soil parameters for the samples the authors then calculated, using Terzaghi's bearing capacity theory, likely bearing capacities for a 1.2 sq m footing and found a remarkably consistent relationship of the form:

$Q_{allow}$ = 164n -504 in kPa - for n>6

where n is the number of blows per 100mm penetration.

(Errata note. The authors state that to account for boundary effects the n values for the SL tests were 'increased' by 20% in their Figure 6, this should read 'decreased' by 20%). Whilst the correlation looks convincing it predicts much higher $q_{allow}$ than other methods. This may be a problem of developing correlations at laboratory scale with potential boundary and scale effects. More validation is needed.

Stanichevsky & Bosio look at Dynamic Penetration Cone Index (DPCI) as an alternative to CBR for the control of sub grade surface. The equipment uses an 8kg mass falling 574mm and uses a 60$^o$ 20mm dia cone. DPCI is defined as:

DPCI = 100 x number of blows/penetration in mm

The paper uses DPCI to look at compaction of a clayey sandy fill. DCPI was correlated to bearing capacity measured in plate load tests and a linear relationship found of:

Bearing capacity = 0.09 DCPI

(they rightly point out that this correlation needs to be validated for other fill types). The correlation was used to confirm variability in a fill and possible problem areas.

They then show that using DP and DPCI in the above correlation allowed them to confirm acceptable bearing pressures at the base of 10m deep excavations in clayey sands and hard clays with a saving of time and cost over load testing.

In a second example they look at DPCI tests in cemented sands and weathered sandstones at shallow depths. They show that in very strong deposits when $N_{SPT}$ exceeds 50 then DP and DPCI can be used to extend the investigation further. They correlate DPCI with the inverse of a 'Penetration Index' derived from the SPT and then use a linear extrapolation to go beyond SPT 'refusal'. In this way they say that more extensive and quicker investigations can be undertaken to look for shallow variations within surface deposits. They rightly comment that further investigations are required to validate the relationships.

Both papers are interesting applications of a simple GI tool. However, neither of the above DP configurations conform to the latest ISO/CEN standards (ENISO 22476-2.2003) or IRTP (ISSMFE, 1988) and, as such, correlations will be very much specific to the particular equipment. There is a need to develop methods whereby correlations are linked to 'work done' or values that can be quantified and linked between devices with different specifications.

## 3.6 *Plate Testing*

*Zhusupbekov et al* report Plate load testing (PLT) on loamy and loamy sandy soils to determine the optimal size for shallow foundations for structures at a new airport site. Tests were carried out on 300mm dia plates in pits and on both water saturated and unsaturated soils (being divided by saturations above or below 80%). Incremental PLTs were undertaken with different increments depending on degree of saturation. Calculated stiffnesses using Poisson's ratios of 0.35 (loamy soil) and 0.30 (sand /loamy sand) gave results that showed reducing stiffness with increasing degree of saturation. Generally their water saturated soils gave stiffnesses that were 25% less than the unsaturated soils. They also concluded that the water saturated deposits behaved non-linearly whilst the unsaturated soils could use a linear model for design to working loads. They look at the effects on ultimate loads and show that unsaturated soils fail at loads between 1.6 - 1.8 times those for saturated soils. It would be interesting to know if the effect of changing the degree of saturation was studied and what the implications might be?

*Teme & Eton* present a new portable plate load test system for testing in the Niger Delta. They needed to assess the bearing capacity of the silty-clayey lateritic deposits that had been improved by vibroflotation. Whilst it was accepted that the PLT was the most suitable test for their requirements the problem of difficult access in the delta region negated the use of more traditional PLT setups. A simple modular system was devised that was portable and easy to load by dead load. The 406mm square plate was incrementally loaded, monitoring settlements with time for each increment to a maximum load of around 2000kg giving settlements around 3.5mm. The authors then scale the maximum bearing pressure achieved during the test to that at 'a maximum' allowable settlement of 25.4mm using a simple linear factoring. Dividing this by a factor of three gave the allowable bearing pressure. The testing showed that all allowable bearing pressures exceeded the design value. (Note the loads in Figure 7 of the paper do not seem to match the text and tables). Another example of a simplified test method giving positive information in difficult locations that would have been difficult to establish cost effectively in other ways.

A third PLT related paper is that by *Souza Neto et al* who look at collapse compression prediction using an 800mm diameter PLT as a surface test in a pit and the 100mm diameter 'expandsocollapseometer' in a borehole. The expandsocollapseometer' is a modification of a plate test so that water can be introduced into a 100mm dia. plate through a central stem and then via holes in the plate to the soil beneath. In both tests the plates are loaded under constant normal stress and the water introduced and collapse compressions monitored. In the case of the 800mm plate water is introduced via the surrounding pit and maintained at a constant level with volume added being monitored. For the 'expandsocollapseometer' water is added as described above and under differing constant loads. The authors use the results of the 'expandsocollapseometer' tests to predict the collapse of the larger plate and get results that over predict by only 18%. An interesting development for plate testing that will hopefully be validated with more tests to inspire confidence in the method.

In considering soil and foundation behaviour changes due to changes in saturation, Ejjaaouani et al report work on swelling clays in Morocco. They undertook a series of laboratory based tests (oedometers, triaxial, shear box and permeameter) on samples at natural water content and varying vertical loads that were supplied with water and allowed to swell. They discuss the mechanisms taking place and show how the strength and stiffness parameters of the soil reduce significantly with increasing degree of saturation; the biggest changes occur in the early stages of wetting. Having studied mechanisms in the laboratory they undertake field tests on $1m^2$ footings. Having looked at the load settlement performance at natural water content the soils beneath the foundations were supplied with water and allowed to swell. On retesting the footings showed much greater settlements and lower capacity. The changes in performance in the field are reported to be far more dramatic than in the laboratory tests.

### 3.7 *Case Histories*

The paper by Viana de Fonseca et al gives a good example of what can be achieved in characterising a residual soil site by combining the use of a range of geophysical and geotechnical in situ techniques. Residual soils are worldwide problem, often difficult to sample and in situ/field investigation should be powerful aids to characterization. The paper is based on a vast amount of information which is only touched upon in the paper. Mapping the site they use surface and borehole seismics with electrical resistivity and GPR to initially map the 'geological' aspects of the site. All methods preformed well and they concluded that the seismic and electrical section models were very similar and the horizontal interface patterns were very consistent between the seismic stacked section the conventional refraction and GPR radargram. For the geotechnical interpretation they found that in situ testing gave consistent results with any one device but, as might be expected in unusual deposits, standard interpretation procedures had mixed success. Soil type was often identified correctly as silty/sandy with ageing/cementation etc but the engineering parameters were not so well predicted. They suggest this is a result of the soil behaving in a more cohesive way at low effective stress, while the behaviour is more frictional at higher stresses. As the individual in situ test results are generally consistent they show that soil/site specific correlations may well be possible both between devices ($q_c/N_{60}$) and between geotechnical parameter and in situ device measurement. The also show a good correlation between small strain stiffness and mean effective stress.

A second very interesting case history type paper is that by Steenfelt et al which covers some of the geotechnical and geological studies which formed an integral part of the Stage 1 investigations for alignment selection of a causeway in Kuwait. They found that using in an initial investigation a mixture of relatively few borings and CPTs then the results could be placed in a conceptual framework allowing fairly robust statements to be made concerning the identification and general properties or the different soil and rock units. Without a general geologic model, based on the geological history of the Kuwait area the site characterisation would have resulted in a perceived chaotic and anomalous subsurface condition which would have made inferences concerning foundation possibilities very difficult and uncertain. The general geology of the area is discussed and this with the above SI information allowed a geological and geotechnical model to be established. The paper discusses the general foundation problems encountered relating to soft soils in critical areas.

They discuss how basic laboratory index tests helped identify the different main soil types. Vane test results showed significant variation between boreholes, with some implying the possibility of unconsolidated soils. In contrast the CPTUs showed good consistency between locations with a steadily increasing profile with depth.

Additional laboratory tests gave information on strength and stiffness. The paper gives a clear presentation of how information from all sources were combined to allow initial decisions to be made about possible foundation solutions and to plan further investigations to gain additional information and to identify anomalies requiring further investigations. The importance of using all available information cannot be overstressed.

### 3.8 *Rock stiffness assessment*

Kovačević et al present a very interesting paper looking at the problems of determining stiffness in very hard fissured soils which fall into the category of soft rocks. Better assessments of stiffness in these deposits are being required for soil structure interaction analyses on larger structures. The authors suggest the use of rock methods of assessing stiffness might be applicable to these stiff fissured clays. Stiffness of a rock is usually determined through correlations with the Rock Mass Rating (RMR) classification because laboratory samples are not representative of the whole rock mass. They explain that whilst standard laboratory determination of soil stiffness, can greatly overestimate small soil deformations, it has been shown that the use of existing RMR correlations greatly underestimate deformations which have been measured during construction in hard fissured soils. Based on 5 cases histories of excavations in fissured soils, they produce a modified correlation between RMR and the soil stiffness where deformation measurements were taken throughout construction. A successful case of bringing experience from rock mechanics to the stiff soils area.

### 3.9 *Others*

Finally, Aitaliey et al report a study of the effects of oil and gas extraction on the geological and tectonic movements of the sea bed in the Caspian Sea. They describe a complex tectonic environment with faults and are trying to predict the likely movements as a result of man's intervention, in the form of oil and gas extraction, disturbing the equilibrium of the 'geodynamics' of the sea floor. In the work they are trying to formulate a mathematical model that will aid in their predictions. A significant challenge

## 4 CONCLUSIONS

In this session we have seen the variety of work being undertaken in the field of in situ testing. We have seen through the papers that established tests are being used in an ever wider range of deposits and improvements and developments in equipment are being made. These have resulted in:

- Improvements in test repeatability etc,
- Adaptability and simplification of tests for specific applications,
- Establishment of better databases from improved quality of testing has resulted in improvement of old/existing correlations,

- Some existing correlations being successfully validated in the new deposits whilst others have 'failed'; and as a result,
  - o New correlations being needed and developed for some deposits,
  - o Old correlations have been extended to cover wider ranges of say $I_p$,
- New modelling methods being developed to aid interpretation,
- Development of in situ techniques to define stiffness – strain decay curves,

We have also seen that:

- In some cases correlations have been applied which were inappropriate as the original derivative actually used parameters derived in a different way,
- In some cases Site specific correlations or correlations containing site specific parameters may often be the best answer,
- There is a need to know what you wish to correlate with or to e.g. vane shear strength or triaxial?
- Combination of tests can be the most powerful and give much better characterisation of soils and sites,

In conclusion we must not oversimplify our interpretation of in situ test results; we must have a quality database of information with which to validate and improve existing interpretations and develop new ones. We need to understand how the ground behaves when 'loaded by' our test devices to better understand how to interpret them. A balance between sophistication and over simplification has to be maintained. Never use in situ testing in isolation, use all data available. Unfortunately the brevity of the papers presented here often means that there is not enough hard data given for true assessment of the applicability of the data and also that its' use for present and future correlation purposes will therefore be limited. It is also unfortunate that often other published work has not been referenced or linked and compared with the work presented in the papers. However our experience has been greatly enhanced by these contributions and potential for the future identified.

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