General Report, Session 4: Soil Properties and Testing

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1. INTRODUCTION

In the first keynote address to the "Fifth Australia-New Zealand Conference on Geomechanics - *Prediction vs Performance*", Duncan (1988) reminded us that

> "The art of geotechnical engineering consists partly in deciding...what allowances should be made for the possibility that something will go wrong".

Duncan proposed nine categories of reasons why problems occur with predictions. Six of these relate directly to assumptions about soil properties and test results:

1. Unrecognised geologic details
2. (Un)disturbed samples
3. Unrepresentative samples
4. No tests, or no analyses
5. Wishful thinking
6. Variations in conditions

Identification and evaluation of risks associated with determination of soil properties is clearly an appropriate and continuing theme from the last Australia-New Zealand conference. From analysis of case histories of geotechnical problems and failures in each of these categories, Duncan concluded that

> "...Even under the most ideal circumstances we must cope with some degree of uncertainty regarding performance, due to our inevitable uncertainties regarding site conditions, and the behaviour of the materials with which we work.
>
> ...If the consequence of wrong prediction is added cost no larger than the cost to build more conservatively in the first place, the consequences of wrong prediction are minimal, and it is appropriate to risk the consequence of being wrong for the economy of being right. At the opposite extreme, where the possible consequence of being wrong is loss of human life and repair costs much higher than the costs of first construction, a conservative approach based on the worst assumptions is appropriate."

These risks relate to the consequences of <u>unconservative</u> design. Unfortunately, the decision process for a geotechnical engineer faced with providing design recommendations from a paucity of good quality data is rarely so simple. This is primarily because there is an additional risk which is often given little <u>rigorous</u> examination, yet cannot be treated superficially by the geotechnical engineer. This is the risk that through insufficient understanding of the relevant geotechnical factors a design will be overly conservative and lead to unnecessary expense.

Structures generally do not fail through conservative design. Nevertheless, the economic risk remains real. In fact it would be rare to find a geotechnical engineer who has not experienced each of three common situations:

- a professional engineer chose on a particular project to carry out all geotechnical-related design without reference to the geotechnical engineer, probably by employing very conservative design assumptions and almost certainly with minimal or no geotechnical testing

- the same professional engineer on another project accused the geotechnical engineer, with little or no basis, of being too conservative in making design recommendations

- a project which did not proceed due to the cost of actions recommended by the geotechnical engineer

Typical examples in the author's experience of where these situations occur regularly are remedial measures prior to building on unstable ground, and piled foundations for buildings. None of the three situations above has consequences which are life-threatening or might lead to expensive remedial measures at a later stage. Yet each involves risk-based decisions, which are most likely to involve a lack of detailed knowledge of soil conditions and properties. Improved understanding of soil properties therefore remains a key element of risk identification and evaluation, prior to determining solutions.

2. MEASURING SOIL PARAMETERS

In his General Report on "Geotechnical Testing" at the fifth Australia-New Zealand conference, Fahey (1988) suggested that some soil parameters cannot be reliably measured in the laboratory, although "reliably and consistently" might be a fairer qualification. Fahey highlights four groups of major parameters in particular (numbers of relevant papers in this session are noted in brackets for each category):

- coefficient of consolidation, c_v or c_h (0 papers)
- stiffness of natural sands (3)

- density, and angles of friction/dilation, of natural sands (2)
- *in situ* horizontal stress (or K_0) (0)

In the authors opinion, hydraulic conductivity should be added as a separate item to any list of this type, even though it is also incorporated in c_v and c_h. (1)

At the design stage the alternative to laboratory testing for measurement of soil parameters is *in situ* testing. There are few *in situ* tests able to quantify these soil properties with a high degree of reliability and consistency, yet at a cost reasonable for most typical projects. It is usually necessary to rely on widespread but cruder *in situ* tests. These are therefore the parameters associated with greatest uncertainty in many design situations.

Although accurate determination of soil parameters is difficult at the design stage, it is often possible to quantify these parameters quite accurately by back-analysis, when performance monitoring has been carried out.

Of the 21 papers submitted in time for review, 18 were based on laboratory testing and only three presented results of field testing. Disappointment over the comparative lack of interest in analysing and reporting *in situ* test results expressed by Fahey in his equivalent 1988 session report is reiterated here. Even more disappointing is that no paper dealt in any detail with back-analysis of soil properties from performance monitoring. Although "back-analysis *per se* is not guaranteed to provide any more accurate values of fundamental soil parameters than local point measurements" (Fahey, 1988) "this is the procedure which, in principle, leads to the most reliable assessment of geotechnical design parameters" (Jamiolkowski et al, 1985).

The usefulness of back-analysis as a tool for determining soil parameters is illustrated by analysis of settlements measured for a number of buildings in Christchurch ranging from 6 to 23 storeys high, on shallow footing or raft foundations (Traylen, 1987; Soils & Foundations, 1988-90).

Full investigation was carried out on all sites, and included testpits, boreholes, standard penetration testing, cone penetration testing, plate load testing, and extensive laboratory analysis including consolidation testing. Design settlements were estimated using a variety of methods as appropriate for each foundation type and subsurface profile. In determining appropriate values of soil compressibility for cohesive soils, greater reliance was placed on laboratory consolidation test results, but all other data were taken into account by using published correlations between secondary parameters (e.g. CPT values) and compressibility. For granular soils, established correlations were the primary means of estimating compressibility.

Settlements were monitored during and following construction by precise levelling. The range of predicted mean settlements using a number of appropriate methods in each case, and the range of actual settlements measured at different locations across each structure are shown in Table 1.

Table 1. Settlements for 3 Christchurch Buildings

BUILDING	Predicted settlement	Actual settlement	Actual / Predicted
1	60 - 100 mm	30 - 40 mm	0.4 - 0.5
2	60 - 130 mm	40 - 60 mm	0.4 - 0.6
3	30 - 50 mm	8 - 12 mm	0.25

In each case above, and for several other buildings in Christchurch not reported above, the average actual settlement measured was only quarter to half the settlement predicted using common conventional methods. Detailed back-analysis shows that mean compressibilities for Christchurch soils - alluvial gravels, sands and silts - are typically and consistently less than half the values commonly reported elsewhere for soils with similar classification, SPT, CPT resistance, etc. Further analysis indicates several factors contribute to the poor predictive performance of these existing correlations:

1. Reported correlations between compressibility and both SPT and CPT values (corrected) are generally linear whereas the correlations developed for Christchurch soils are highly non-linear. The authors suspect that this is likely to be the case for other soils, but is generally unrecognized or unreported. (A notable exception to this is Burland & Burbidge's method, which suggests a logarithmic relationship between SPT N and compressibility).

   Using non-linear relationships of similar form for different soil types, good correlations between SPT, CPT values and compressibility can be developed for all Christchurch soils from silts to gravels. Settlements can often be predicted to within ± 20-30% using SPT, CPT test results and these correlations.

2. Precise soil grading (particle size distribution), grain angularity, and other semi-geologic factors such as depositional environment probably account for the difference between established correlations and those developed for Christchurch. These are likely to be similarly important elsewhere and account for a considerable part of the unreliability often encountered when using semi-empirical methods to estimate settlements on granular soils.

   Establishment of site-specific correlations by back-analysis can eliminate much of this unreliability.

This example shows the importance of back-analysis as a technique for determining soil properties. It is unfortunate that this analysis is carried out far too rarely, probably because there is generally little direct remuneration to the practising geotechnical engineer for doing this work once construction is complete.

The benefit however is clear - and again the example above of Christchurch soil can be used to illustrate this. Many older buildings in the city have piled foundations. However since the work above was carried out by the authors, for a number of significant structures in Christchurch which in the past would have been piled, piles have not been recommended by the authors as necessary to control static settlements. Both the risk of unexpected excess settlements, and the risk of unwarranted conservatism, have been substantially reduced.

3. SESSION PAPERS

The papers received for this session may be categorised in a number of different ways: by soil property discussed, by method of soil property determination, by soil type or by areas of risk considered. The papers cover a wide range of general topics and there are few common themes. In some cases, alternative paper groupings to those used could be equally appropriate - for example, three papers dealt specifically with cemented calcareous sands. However for discussion purposes papers are loosely separated into the categories below, which combine general subject themes with the overall conference theme of risk identification, evaluation and solution. Numbers of papers are noted in brackets:

- Improvement of design predictions (3)
- *In situ* properties, correlations and natural variability (4)
- Advances in soil testing methods (6)
- Fundamental behaviour (8)

3.1 Improvement of Design Predictions

Poulos and Al-Douri carried out model tests on jacked piles in calcareous sands to study the influence of density on shaft friction under static and dynamic loading. This is a continuation of the studies relating to problems with low capacities of jacked piles used around Australia in foundations for offshore structures. These sands were from the site of the North Rankin platform. Medium-dense and dense states were investigated using two sizes of test vessel, of 300mm and 600mm diameter; boundary conditions probably affected test results for the smaller vessel.

They conclude that skin friction, elastic modulus and jacking resistance increase with soil density and overburden pressure, but that skin friction under static loading is less than that during jacking. Cyclic loading "degrades" skin friction, and the degradation becomes more severe with increasing soil density or overburden pressure, and with increasing cyclic displacement.

The results provide a significant contribution to the understanding of pile capacities in these soils.

The paper by Ervin and Kurzeme relates directly to the aspect of risk discussed above, that of balancing the risk of failure against the risk of unnecessary conservatism in design. They carried out plate load testing to estimate bearing capacities and increased settlements beneath footings on gravelly soils which were expected to experience load increases up to 30% following building refurbishment. This is an interesting application of the test, particularly as the existing building was used to provide a reaction.

A comment on the possibility that the upward reaction on the building might have reduced the existing stress beneath adjacent footings, and thereby reduced the magnitude of measured deflections, would be interesting. The authors hope that settlement monitoring was undertaken to verify the predictions of soil properties made by this method.

Vuong analyses different models for prediction of the resilient modulus, required to predict the life of granular pavement. Resilient moduli were determined for basecourse crushed rock in repeated load triaxial tests. Lives predicted using the program NONCIRCL ranged from 0.3 to 3 times the corresponding NAASRA design lives, demonstrating the risks inherent in selecting a particular model for design.

3.2 *In Situ* Properties, Correlations and Natural Variability

Particular properties of natural soils, and the risks associated with random sampling being unrepresentative of natural variability, were the subject of the four papers in this category.

Mostyn and Waters carried out detailed investigation and a comprehensive program of laboratory testing for a site in Sydney to determine the natural variability of the shrink-swell index, which is used to estimate maximum ground surface movement. They conclude that to classify a site to even a low degree of confidence at least two to three tests, together with good geotechnical logging of a borehole, are required. The inclusion of good logging may reduce the number of shrink-swell tests required to characterise a site by a factor of three or more.

This is a good example of a case where better, rather than more, testing is desirable to reduce risks associated with natural soil variability.

Natural properties of clays in the Adelaide city area are reported in companion papers by Jaksa and Kagwa, and Kagwa and Jaksa. They use data compiled from many sources for most significant investigations in the city since the mid 1960s.

In the first paper, they use reported values for *in situ* bulk density and moisture content, and assume values of specific gravity, to show that the degree of saturation generally exceeds 95%. Previously these soils have been treated as unsaturated.

It is not clear why they state reservations about the accuracy of laboratory measured values of G_s, since the test is straightforward and relatively simple. Furthermore they conclude that since the degree of saturation exceeds 95% then effective and total stresses are equal. This requires further examination before such a conclusion can be safely drawn. The suggestion that the measurement of air contents of 0 to 5% is attributable to air entering the fissure system during testing and sampling implies that the true *in situ* degree of saturation is probable 100%, although the groundwater table is known to be 20 to 30m below ground level.

In their second paper, Kaggwa and Jaksa calculate *in situ* effective stresses assuming that negative pore pressures are present, and relate undrained shear strength and elastic modulus to these. However they conclude that normalised values of s_u and E_u are within expected ranges for typical over-consolidated clays regardless of the pore pressures. They do not refute the common view that for residential buildings, soil shrinkage and swelling remain the dominant factors for footing design.

Natural soil variability is also discussed by El-Sohby et al who examined the silica/sesquioxide ratio, which is an indicator of "soil genesis and environmental setting" in Egyptian clayey deposits around the Nile valley. They note that chemical weathering significantly modifies physical and geotechnical properties.

3.3 Advances in Soil Testing Methods

Further advances in the ability to study pile capacities in calcareous sands around Australia are presented by Parkin et al who describe adaptions to the Monash double-walled calibration chamber made to accommodate the driving and cyclic load testing of model piles. Some problems in soil preparation and testing were apparent. They infer from the absence of plugging during driving that the pile clearance criterion (2D) is adequate. Crushing during pull testing probably caused shear strength to decrease substantially up the shaft, but they acknowledge that more work is required before the shear stress can be adequately predicted using CPT data.

Moore et al confirm previous findings that free-field vibration measurements may be affected by interaction between the ground and the mountings. They analyse theoretically the effects of different mountings. Results are supported in most cases by geophysical tests but some discrepancies occurred for reasons that are not understood. Embedded rods, rather than surface plates, appeared to give best estimates of ground vibrations.

The basecourse triaxial apparatus, capable of accepting samples up to 250mm diameter and 625mm high, is a useful research device but impractical for routine basecourse evaluation. An alternative, the "simple shear compactor" developed at the University of Auckland, is discussed by Pender et al. The original machine has been upgraded and automated. Various aggregates were tested and results verify the usefulness of the method, which uses a smaller quantity of material than the triaxial cell and has a very simple sample preparation procedure. The device provides a rapid means of categorising basecourse aggregates.

In companion papers, Brown and Abdel-Latif evaluate the effect of insertion on results of the screw plate test, which is used to determine soil compressibility without some of the disadvantages of the flat plate loading test. The two papers deal with the test in sand and clay using miniature laboratory testing.

In loose sand at low effective stresses, the effect of insertion appears to be minimal and to predict the soil modulus well. Scatter is greater in clays, but insertion appears to have minimal effect for overconsolidation ratios less than 5. For higher values of OCR the pressure increases beneath the screw plate during insertion, resulting in overestimation of E and c_v. A similar method for reducing disturbance when insertion is carried out with a threaded rod is proposed for both cases, but is not substantiated by further test results.

Yetton and Bell analyse the Pinhole Test, which was originally developed by Sherard et al (1976) to estimate the piping resistance of core material for dams. It has subsequently been modified and used to assess soil erodibility, particularly for Banks Peninsula loess in New Zealand. Sherard et al proposed the test result be used as a dispersion index. Although the most widespread use of the term "dispersion" indicates clay mineral deflocculation, Sherard et al redefined the term to mean colloidal erodibility.

Yetton (1986) showed that the pinhole test is a very unreliable measure of dispersion in its original and most common sense. Yetton and Bell propose that the test be retained, but used to define erodibility alone. While high erodibility may indicate the presence of dispersive clay minerals, erodibility is controlled by a number of soil properties, of which dispersion is only one. They propose a relatively quick and simple classification procedure to assign an erodibility category based on the head at which sustained erosion first occurs.

The authors suggest that rather than using the upstream head as the key indicator, it may be sensible to use the hydraulic gradient across the sample. This dimensionless form would allow use of different sized test samples, and direct comparison with field situations. Thus a soil which eroded at 100mm head in the test (i=2) would also be expected to erode rapidly in a field situation if the hydraulic gradient were close to 2 through a path of preferred flow (e.g. a fissure or interface between two soils). This would allow a factor of safety against erosion to be calculated directly, and used in design.

Dispersion and erosion are extremely important concepts, particularly in the design of water-containing earth structures. Yetton and Bell highlight a discrepancy in the interpretation of a common test which could increase the risk of failure due to a design being based on incorrect assumptions. Their suggested reinterpretation is logical and should be adopted, after consideration of the suggestion made above.

3.4 Fundamental Behaviour

Eight papers deal with fundamental behaviour and properties of soils. The first five deal with stiffness and yielding of various soils following particular loading (stress path) conditions.

Allman et al report results from the continuing studies to explain clay soil behaviour in the critical state framework. The state boundary, or yield surface (stress state) marks the transition in behaviour from high to low stiffness. The shape of this surface away from the p' axis is difficult to define accurately by conventional undrained triaxial tests since stress paths do not necessarily reach, then precisely follow, the state boundary surface.

Allman et al carried out drained testing constrained to follow specific stress paths in triaxial compression and extension. They demonstrate that, as predicted, extension tests on 1-dimensionally consolidated samples do not identify the state boundary surface, which is however closely predicted by the modified Cam-clay model.

Chu et al also consider soil yielding, in their investigation into strain-softening of dense Sydney sand. They carried out strain path triaxial and multiaxial tests and identified three principal types of strain-softening behaviour. Depending upon the test type and the strain path followed, strain-softening may occur due to non-homogeneous deformation, as a path-dependent material behaviour, or due to shear bands which occur in response to 3-dimensional loading. This study is a useful contribution to understanding the constitutive behaviour of granular media.

Three papers consider the response of soils to cyclic and dynamic loading. Zhao et al continue the theme of calcareous sand behaviour discussed earlier, investigating the effect of stress level and frequency on the number of cycles to failure. Both a stress path triaxial cell, and a conventional triaxial cell with dynamic deviator load capacity, were used and sand from the north Rankin site (see Poulos and Al-Douri, above) was artificially cemented by mixing with gypsum cement.

The cementing contributed a significant part of the static strength and rapid post-peak strength loss occurred at high cement ratios. However about 10^6 cycles were required to cause 50% strength loss. Elastic modulus, pore pressure and axial strain responses were independent of load cycle frequency, but the duration of peak load application, and hence the cyclic frequency, significantly affected the number of cycles to failure.

Although it is dangerous to generalize based on specific tests on a particular soil, it is interesting to note that it is often assumed dynamic soil properties are relatively independent of cyclic frequency. This is certainly the case in earthquake analyses, although earthquake frequencies are commonly >>1 Hz, whereas wave loadings are typically closer to 10 Hz. Without detailed behavioural information on local soils, it is necessary to use general correlations based on overseas soils when performing dynamic site response analyses.

Pender et al employ the assumption of frequency independence at earthquake frequencies in investigating the change in shear modulus and damping with strain amplitude for Auckland clays and Tauranga volcanic ash. Recent earthquake data has indicated enhanced site response may occur in soils exhibiting elastic behaviour and minimal damping over an extended range of strains. Most tests were carried out at frequencies 0.2 Hz on 75mm diameter triaxial samples. Pore pressures were measured at the sample midheight using miniature transducers.

They conclude that for Auckland clays the number of cycles has relatively little effect on the stiffness and damping behaviour. The effect of strain amplitude is adequately described by conventional models. Degradation of these soils under earthquake loading is unlikely to be significant.

The Tauranga ash soil behaved more like a sand during cyclic loading, despite having index properties similar to the Auckland clay. It experienced rapid build-up in pore-pressure during cyclic loading, with a consequent decrease in stiffness. Damping did not increase with strain amplitude.

Larkin and Chan investigated dynamic behaviour of volcanic soils from the New Zealand central volcanic plateau using the dynamic torsion test, and compared results to the "overseas" models discussed also by Pender et al. They concluded that dynamic properties - normalised shear modulus and damping - of these soils generally do not deviate significantly from trends described by the commonly used models. A number of slight differences in behaviour were noted which could be incorporated into future analyses.

Seismic response analysis is very sensitive to the dynamic properties used. It is important to evaluate properties of local soils specifically, wherever possible, and to perform sensitivity analyses to indicate the degree of reliability which can be placed on the results.

Wesley presents results of residual strength measurements carried out in the ring shear apparatus on some New Zealand soils, and also analyses data with respect to overseas correlations. Comprehensive reporting of soil classifications and index properties means that these results can be widely used, although inclusion of particle size gradings would have been beneficial.

Results for "normal" soils are consistent with those found overseas and commonly used correlations between residual friction angle, and plasticity index or clay fraction, appear reasonably valid. However the scatter in data is large, and indicates - as might be expected - that a multiple correlation with both plasticity index and clay fraction is likely to be more reliable than either alone. In this respect, it would have been useful to attempt correlation with the parameter

$$(CF)^2.LL.PI$$

which was shown by Collotta et al (1989) to predict the residual friction angle with considerably less uncertainty.

Wesley shows that residual friction angles for volcanic ash soils are considerably higher than for normal soils with comparable clay fraction and plasticity index. The alternative correlation proposed above is unlikely to perform better for these soils.

Two papers deal with soil properties unrelated to any discussed in this section. Indraratna and Husin evaluate methods for predicting the swell potential of clay shales in Thailand, which have caused instability and failure of diversion channel linings. They investigate correlations among swell potential, plasticity index, clay fraction and activity for a soil from the Mae Moh area, but considerable scatter is evident and suggests that other controlling parameters should be investigated further.

Airey assesses the effect of stress history and void ratio on hydraulic conductivity of a remoulded Sydney clay with water and kerosene. Dramatic increases in hydraulic conductivity have previously been noted when pollutants, particularly liquid hydrocarbons, contact soils. It is concluded that the relationship between hydraulic conductivity and void ratio is best determined using normally consolidated soils.

Kerosene required increased hydraulic gradient for initiation of flow, and exhibited a lower hydraulic conductivity. Possible factors affecting this behaviour are discussed, and surfactants are postulated as a cause of increased flow under certain conditions. However considerable further work is required, using a range of other permeants, before general conclusions can be drawn.

4. CONCLUSIONS

Of the twenty one papers reviewed for this session, all but two discussed work carried out in research institutions. This is probably not surprising since the laboratory facilities required to carry out research on fundamental soil behaviour are primarily located in these institutions. However it is unfortunate that greater effort is not being made by practising geotechnical engineers and geologists to investigate and report information on general soil behaviour.

As discussed earlier, laboratory testing is only one of three general methods for determining soil properties. *In situ* testing, and back-analysis of performance monitoring results, are equally valid and important. Until back analysis in particular has been used to verify assumptions made about soil behaviour, the state of the art must be concluded to have some distance to advance.