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Practitioner/academic forum

Forum professionnels/universitaires

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1 INTRODUCTION

In the early days of the International Society of Soil Mechanics and Foundation Engineering, there was considerable interaction between academics and practitioners, and in many cases, there was little distinction between the two groups. Indeed, the majority of professors acted as consultants and as a consequence, directed at least some of their research work to the solution of immediate practical problems. As with all evolving disciplines, development in soil mechanics and foundation engineering was accompanied by increasing specialization, and by the 1960's, the distinction between academics and practitioners had become quite sharp. This distinction has continued to grow, and now, in the early 21st century, 80 years after Terzaghi's pioneering text book on soil mechanics, there is a significant division between those that teach and those that practice geotechnical engineering. There are of course exceptions, with a number of professors being involved in significant projects, but it is unfortunately the case that many current academics have never practised the subject that they teach to students. As a by-product of this deficiency, there is a substantial and ever-increasing gap between research and practice. This is one of the motivations of the forum (Figure 1.1).

Figure 1.1. Motivations of the forum

From the academic's viewpoint (Figure 1.2), research is an essential component of contemporary academic life, and it is vital that they publish research findings in order to progress their career. Academic progression tends to depend more on quantity than quality of publications. Regrettably therefore, the definitive papers that graced our profession in days past are very rare indeed, and even they may not be accessible to practitioners at large because of the plethora of publications that now exist. The publications evolving out of contemporary research are often of marginal quality and originality, and may appear in obscure venues which the average practitioner will never see. Finer and finer details of research topics are explored, with the inevitable law of diminishing returns taking hold. From his/her detached viewpoint, if the academic thinks of the practitioner at all, he/she often sees the average practitioner as one who is ignorant of modern techniques of analysis and design, and who ignores what the academic considers to be improvements and advances that they have made to the understanding of a topic via the development of theory and via often-complex laboratory experiments.

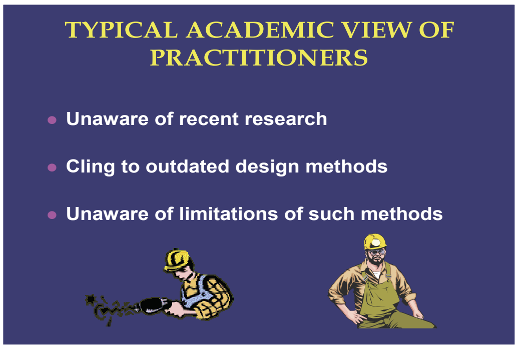


Figure 1.2. Typical academic view of practitioners

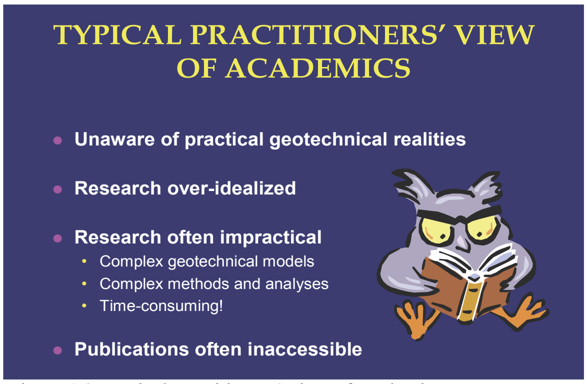


Figure 1.3. Typical practitioners' view of academics

From the practitioner's viewpoint (Figure 1.3), reliance is very frequently placed on older methods which appear in now-outdated texts, but which can be readily employed, despite their limitations, recognised or unrecognised. The time pressures and financial constraints of commercial geotechnical consulting and contracting generally preclude the possibility of learning and applying newer methods and techniques which are more complex and which may require data that cannot be easily obtained or assessed. The academic is consequently viewed as a researcher who is unaware of the practical realities of the subject and who persists in learning more and more about less and less, and who publishes morsels of knowledge in obscure locations that are only accessed by fervent fellow-academics.

The gap that has developed between academic research and practice in geotechnics does not seem to be mirrored in many other professions, certainly not, for example, in medicine, electronics or nanotechnology. There, the latest research findings are eagerly awaited and immediately put into practice as a commercial imperative. Therefore, we in the geotechnical profession must view the academic-practitioner gap with great concern and takes steps to close it. Among the measures that may be effective are the following:

1. Involving more academics in practice, and indeed, demanding that academics should not teach until they have had at least two years in professional practice. Again, the medical profession provides an example - it is inconceivable that a medical doctor can teach surgery without ever having performed an operation, yet we have many geotechnical academics who have never been involved in a site investigation or designed a foundation.
2. Having more definitive review papers published in prominent journals or conferences (and particularly in International Conferences) by experienced professionals (both in academia and in practice). Such papers should synthesise the latest research and critically review its relevance to practice.
3. Encouraging practitioners to pass on to academics the results of performance monitoring and interacting with them to assess actual performance against predicted performance.
4. Provide forums where academics and practitioners can interact and mutually benefit. Researchers can make practitioners more aware of the latest research findings, while practitioners can make academics more aware of unsolved problems that require further investigation.

It is in the spirit of this fourth measure that the Academic-Practitioner Forum has been devised. It is meant to be a first step in what hopefully may become a regular feature of International Conferences and Symposia, and which may redress the balance of conference participation which in recent years has been heavily weighted towards academics. The forum will assemble three prominent academics and three well-known practitioners.

The three participating academics are:

1. Prof. Malcolm Bolton, Cambridge University, UK.
2. Prof. Paul Mayne, Georgia Institute of Technology, USA.
3. Prof. Fumio Tatsuoka, Tokyo University of Science, Japan.

The three participating practitioners are:

1. Mr. Peter Day, Jones and Wagner, South Africa.
2. Mr. Luis Valenzuela, Arcadis, Chile.
3. Mr. Stephen Crawford, Tonkin & Taylor, New Zealand.

They will be asked to give their views on two issues that will be posed. Each issue will have an academic and a practical viewpoint, and it will be the aim of the Forum to expose these viewpoints and attempt to reconcile the gaps that may exist between them. The audience will also be asked to express its opinion on the issues discussed.

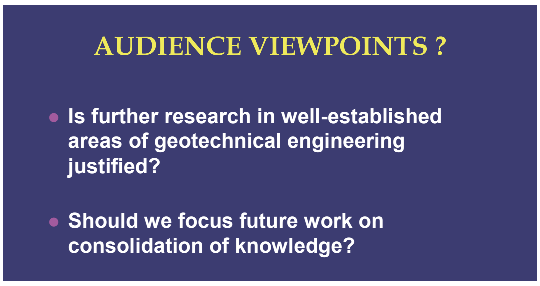


Figure 2.1. Audience viewpoints

2 PROPOSED ISSUES FOR DISCUSSION

The following two issues were discussed. In addition, the chairman raised two audience viewpoints as shown in Figure 2.1.

Issue 1:

(a) Academics – Give an example of research work that you feel has potential for practical application but has not been used extensively by practitioners.

(b) Practitioners – Give an example of a problem that you have encountered where research is perceived to be lacking and would have been of benefit in developing a solution.

Issue 2:

Should research continue on complex constitutive laws for soil behaviour when we are unable to adequately assess parameters for simpler soil models, or should we focus on better evaluation of the simpler soil model parameters?

3 PROPOSED RULES OF THE FORUM

The following rules were implemented during the forum.

1. For each issue, each of the 3 academics and 3 practitioners will be given a maximum of 7 minutes to give their response to the issue.
2. On Day 1, limited contributions from the floor will be invited to express their views after the Academics & Practitioners have given their presentations. The audience will be asked to express an opinion on whether the academic-practitioner gap is too large and if so, whether forums such as this one can assist in closing the gap.
3. On Day 2, after the issue has been discussed, the audience will be asked to indicate their opinion on the issue via a show of hands. At the end of the session, the Chairman will attempt to summarize briefly the conclusions reached.
4. At the end of the forum, the Chairman will attempt to sum up and identify areas where research is required and areas in which the results of research should be implemented in practice. It is hoped that this will suggest an agenda for action with respect to both categories.

4 SUMMARY OF PRESENTATION BY FIRST PRACTITIONER (DAY, P.) ON ISSUE 1: LONG TERM SETTLEMENT OF GRANULAR FILLS

4.1 *Background*

Granular fills occur in various guises ranging from uncompacted backfill in open pit mines to engineered fills behind bridge abutments or around buried structures. The long term settlement of these fills plays a crutial role in the development potential of the land and the settlement of structures founded on the fill.

Considerable work has been done on the prediction of settlement of natural soils including secondary settlement of clays and creep settlement of footings placed on sands. However, little work apprears to have been done on granular fills.

4.2 *Nature of the problem*

On a natural soil profile, settlement resulting from the self weight of the soil has already occurred and the profile is in equilibrium with its surroundings. The settlement of foundations placed on a natural soil profile is thus due solely to the load imposed by the foundation. Normal settlement (immediate plus consolidation) is generally the dominant contributor to foundation movement and creep plays a lesser role. Such settlement and the time period over which it occurs can be predicted with reasonable accuracy using classical soil mechanics principles if the stress-strain relationship for the soil and the consolidation characteristics are known.

In the case of fill materials, settlement due the self weight of the soil may still be occurring at the time development takes place. As the weight of the fill is generally far in excess of any foundation loading, self weight settlement of fills can exceed the normal settlement of foundations constructed on the fill, sometimes by many orders of magnitude. Significant settlements are often recorded even in the absence of any external loading on the fill.

Two factors are thought to contribute to ongoing settlement of granular fills. The first is creep movement under conditions of constant load and constant moisture content. The second is "collapse" settlement due to a change in the moisture content of the fill material. The latter is likely to play a significant role in the case of uncompacted fills.

Materials used in engineered fills may be amenable to laboratory testing. However, most fills comprising mining or construction spoil cannot be tested in the laboratory due to the large particle sizes involved. Thus, neither the compressibility of the fill material nor its susceptibility to collapse on wetting can be established in this manner.

4.3 *Examples*

Example 1: Railway line on coal mining spoils. In this case, a mine railway line was constructed over 30m of recently placed spoil in an opencast coal mine. The load imposed by the railway line is negligible compared to the self weight of the fill. The principle components of the long term settlement were assessed to be creep settlement of the fill under its own weight, localized collapse settlement at shallow depth in areas of poor surface drainage and general collapse settlement as the water table reestablishes at the end of the mine's life. The fill consists of blasted fragments of shale and sandstone up to 2m across in a matrix of rock fines. On the basis of experience and published case histories, collapse settlement was estimated to be 4% of the height of rise of the water table. This has yet to be verified as the mine is still in operation. Local collapse has, however, proved to be a problem in cuttings and has lead to high maintenance costs. The mine has accepted these costs as they are considerably less than the cost of soil improvement below the railway line.

Example 2: Settlement of structures on deep compacted sand fill. This case involves a 23m deep, engineered fill around a buried power plant structure. The fill is to be constructed using the excavated soil (a non-plastic fine to medium grained sand) placed under carefully controlled conditions. Rigid concrete structures essential to the operation of the plant are to be founded on the fill with construction commencing within weeks of completion of fill placement. In this case, creep settlement of the fill under its own weight is expected to dominate and a limited amount of collapse settlement is expected as the dewatering system is switched off and the water table allowed to re-establish at 5m below the top of the fill. This project has yet to be built and various methods are being considered to assist in assessing the long term settlement potential of the fill.

Example 3: Settlement of structures on shallow aged fill. This case involves a single storey car park built on a 100 year old uncompacted cut-to-fill terrace previously used as a school playground. Due to the age of the fill, it was assumed that no further self-weight settlement of the fill would occur and that only normal settlement of the foundations need be considered. This was taken care of by re-compacting the material to a depth of 1,5B below the foundations. Despite these precautions and age of the fill, gradual settlements of up to 120mm occurred over a period of 5 years before the structure was underpinned. This is far greater than one would expect below a lightly loaded structure. We suspect that, although the fill had achieved equilibrium under exposed conditions where evaporation exceeds precipitation (the area is semi-arid), covering the fill with an impermeable floor slab has lead to an increase in the moisture content of the uncompacted fill material initiating further creep movements. The design engineer was held liable for the cost of the repairs.

4.4 *Where to now?*

Probably the most logical starting point is to distinguish between compacted and uncompacted fills and between materials that can be tested in the laboratory and those that cannot.

As far as uncompacted fills are concerned, it would be beneficial to compile a database of case histories recording parameters such as type of material, method of placement, depth of fill, monitoring period and moisture regime. Many of the mining companies involved in open cast mining operations have records of the settlement of beacons or performance of structures placed on the pit backfill. In addition, there is an increasing tendency to return undermined land to beneficial use. Such developments are often well instrumented and monitored.

For soils that can be tested in the laboratory (sands and fine gravelly sands), one could compare the consolidation characteristics of the compacted soils with those of natural soils. Our experience has been that it is more difficult to distinguish between primary and secondary consolidation for compacted fills than for natural soils. It would also be beneficial to investigate the dependence of creep rate and collapse potential on the degrees of compaction.

In the interim, practitioners will continue either to avoid founding on fill materials or will attempt to adapt theory developed for natural soils, generally clays, to assist in predicting the performance of granular fills.

5 SUMMARY OF PRESENTATION BY SECOND PRACTITIONER (VALENZUELA, L.) ON ISSUE 1: RELIABLE ESTIMATE OF GROUNDWATER FLOW AND SOLUTES TRANSPORT

Heterogeneity of natural ground plays a fundamental role in groundwater flow, and consequently in the transport of solutes and the flow characteristics of leach solutions. Solute transport problems are not only dependent on effective permeability (depending in turn on the scale of the problem and boundary conditions) but to an important extent on the factors governing both macro and porous dispersion. The real nature of the dispersion phenomena is closely related to the characteristics of the porosity of the ground and to the presence of highly permeable zones such as macropores, open channels, root holes and fractures.

If no major discontinuities are present it is possible to apply a stochastic approach using geo-statistical methods and Montecarlo simulations in order to consider several possible scenarios. This can serve as a first approximation to an iterative process that will require monitoring and new calculations to calibrate the model to the actual situation. The difficulty is that real solute transport problems can be of a dual nature: deterministic, with part of the flow concentrated along a few discontinuities, and, additionally stochastic in terms of the flow through the porous portion of the natural ground.

One way to answer the question arose by Issue 1 is to try to make an estimate of the current state of development in the main areas related to groundwater flow and solute transport problems. The following is the author`s overview:

a) Conceptual Models- reasonably well developed and accepted;

b) Mathematical Models- generally well developed although some aspects require more investigation;

c) Numerical Models- commercially available in various programs, where there is a need to improve some of the methods, mainly in terms of efficiency:

d) Laboratory and Field Determination of Ground Properties- this is by far the main cause of most of the problems with groundwater flow and solute transport studies since the present state of the art in laboratory and field tests do not allow to obtain a reliable detailed characterization of an aquifer and the values of the permeability and hydraulic conductivity of its different zones.

Meanwhile, practitioners must go on providing solutions to groundwater flow and solute transport problems within reasonable timeframes and subject to the limitations on resources common in most practical problems. Solutions usually consider the following: i) a preliminary physical model of hydrogeology built based on a geological model, supported by observations on the ground surface and through a limited number of borings, combined with permeability and hydraulic conductivity tests; b) mechanical dispersion, usually treated as diffusion, by adopting a certain variable coefficient intended to represent the possible range of variation; c) use of numerical models based on finite differences or finite elements, and d) refining of the model by collecting new data through the installation of monitoring wells, and eventually introducing modifications to the original project if necessary (for instance by adding extracting wells at specific locations). Two examples of practical applications are presented: one with a great degree of geological complexity, where an iterative process carried out over almost 20 years enabled a final solution to be reached; and a second with a much more simple geology supported by plentiful data, allowing an acceptable solution to be found in a straightforward manner through the application of numerical models.

Major areas for additional research include:

1) achievement of substantial improvements in the methods and technologies used to determine basic parameters of ground such as permeability coefficient and hydraulic conductivity;

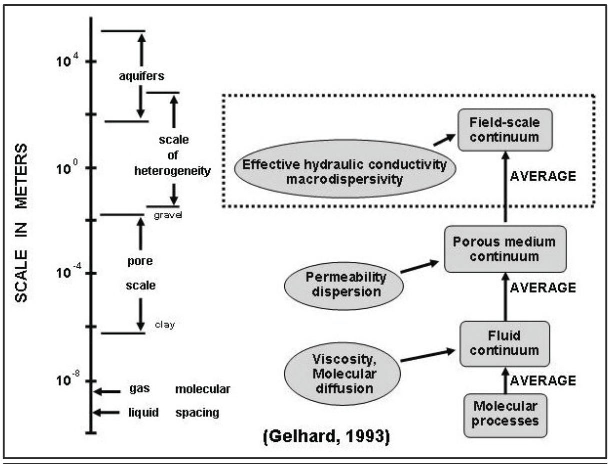
2) large scale field tests to check on the existing capabilities of the available models for handling groundwater flow and solute transport problems, such as the Borden field test run by the University of Waterloo, for instance;

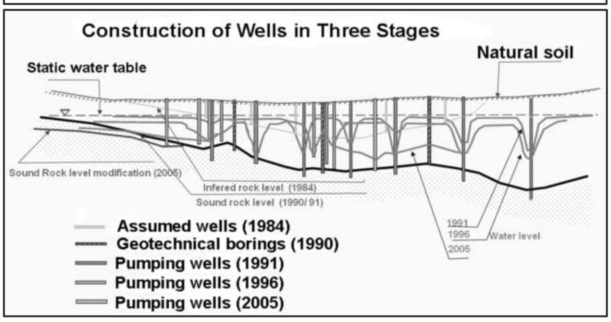
3) improvements in the present capabilities of exploration and monitoring instruments to yield quick and reliable information; and

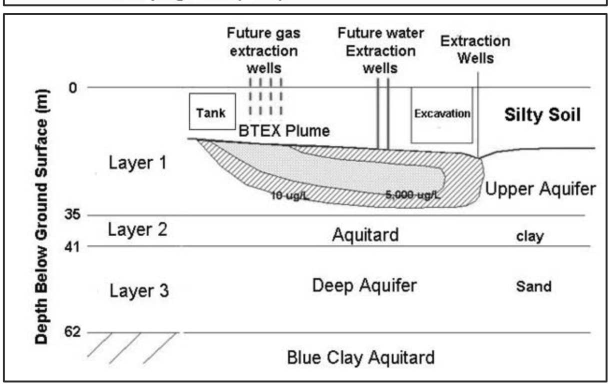
4) achievement of higher levels of standarization of methods and procedures for practical use, considering the potential public and legal implications of the outcome of many groundwater and solute transport studies..

It is rather difficult to single out the Academy as the one to be solely responsible for the important investigation effort now required to achieve substantial improvements in present capabilities to solve practical problems of groundwater flow and solute transport in a satisfactory way while yielding meaningful and workable results. More likely, a joint Academy-Industry-Government Agencies- Practitioners effort would be the best way to tackle this important problem.

This presentation was prepared with the collaboration of Dr. Carlos Espinoza, Universidad de Chile and Prof. Paulo Ivo, from ITA, Brazil. The author acknowledges the contribution and comments from Dr. Sat Sansar, Mrs Cecilia Riveros,. Mr.Edgar Bard and Mr. Jorge Proust from Arcadis Geotecnica, Chile and Prof. José Muñoz, Universidad Católica, Chile.







6 SUMMARY OF PRESENTATION BY THIRD PRACTITIONER (CRAWFORD, S.) ON ISSUE 1: GEOTECHNICAL PERFORMANCE OF VOLCANIC PUMICE DEPOSITS

Pumice deposits are associated with explosive volcanic activity, which leaves the material highly vesicular and lightweight. It is deposited in ash showers with pumice material particle sizes being largely dependent on distance from the volcanic source. The materials may then be re-deposited by fluvial and/or aeolian processes. The distribution of pumice around the world is linked to the occurrence of volcanism and includes more than 50 countries on the 'Pacific Rim of Fire', the eastern Mediterranean, SE Asia and others.

The performance of pumiceous (rhyolitic-volcanic ash shower) deposits and re-deposited pumiceous alluvial materials has long been known to behave significantly better than predicted by traditional soil mechanics, in terms of strength and stiffness. For example drained cut shapes are traditionally cut very steep to near vertical; predicted settlement beneath shallow footings or slab foundations is nearly always much higher than measured; pavement subgrades (subject to repetitive loading) are very often significantly stiffer than predicted and perform much longer than expected.

It is noted that it is often difficult to obtain and return undisturbed samples of these materials to a laboratory situation. Research on behaviour of pumiceous deposits has not fully explained such differences. Some recent NZ work [Wesley, Pender et al (1999), Wesley (2002, 2003 & 2004)] has identified that commonly used field tests including CPT (SPT & other penetrometer tests) do not consistently measure the expected better properties. Controlled CPT tests on quartz and pumiceous sands in the laboratory show significantly different behaviour, in particular:

- The peak strength of pumice is similar to quartz sand, but the strain to failure is 2 to 3 times greater for pumice. The post peak behaviour is also very different (Fig. 6.1).
- Pumice sand is much more compressible than quartz sand, whether loose or dense.
- The ultimate strength of loose and dense pumice sand is typically similar to that of ***dense*** quartz sand and higher than loose quartz sand.
- Controlled CPT tests indicate that there is little difference in CPT resistance between loose and dense pumice sands - both have similar response as ***loose*** quartz sands (Fig. 6.2).

This lack of understanding often results in conservative cut slope design (and associated land take and earthworks volumes), conservative measures to remedy predicted excessive settlements (e.g. foundation treatment, preload surcharges/ time delays or piled foundation designs), or deeper pavements using expensive metal courses.

Generally, practitioners with local knowledge and experience will empirically upgrade the strength and or stiffness of pumiceous materials and apply local rules of thumb. However when it comes to liquefaction, empirical assessments are not available. Conservative assessment often means liquefaction is assumed where it probably will not occur.

Also as a side issue, there is the timing of liquefaction with respect to peak earthquake load. It is generally acknowledged that pore pressures take time to build up in an earthquake so that by the time liquefaction fully develops (e.g. around a bridge piled foundation), the maximum earthquake load has passed. One local empirical approach was to design for a full earthquake load plus only 20% of the liquefaction strength loss, or 20% of the full earthquake load and full liquefaction strength loss (whichever design load is higher). This 20% factor was somewhat arbitrary. Further research on this aspect also would be welcomed.

Some suggestions for further investigation are the influences of: CPT skin friction on strength of pumice deposits and liquefaction/settlement assessment; crushing of sand/silt grains under high strain/point load and low strain/distributed load; the varying mixes of quartz-pumice sands; weathering/slaking of different pumice particle sizes; compare mapped liquefaction with CPT tests; develop affordable low-mid strain measurement of pumice properties.

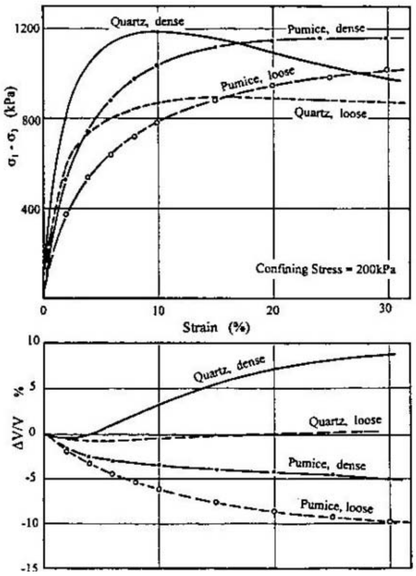


Fig. 6.1. Typical Triaxial Behaviour of Pumice & Quartz Sands

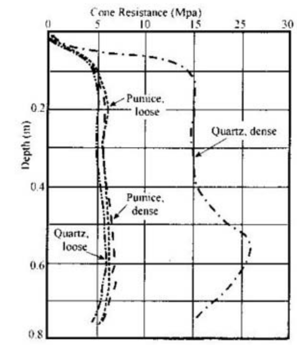


Fig. 6.2. Typical CPT Results for Pumice & Quartz Sands

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7 SUMMARY OF PRESENTATION BY FIRST ACADEMIC (MAYNE, P.) ON ISSUE 1

Amazingly, geotechnical practice has still not adopted the framework of critical-state soil mechanics (CSSM), yet the supporting research and clear evidence have been available for over a half-century (e.g., Hvorslev, 1960). A review of geotechnical textbooks used in the USA, for example, reveals that the most popular and best-selling books do not discuss or even mention CSSM, the exceptions being those by Budhu (2000) and Lancellotta (1995). Of course, there are several excellent books specific to CSSM (Schofield & Wroth, 1968; Atkinson, 1981; Wood, 1990), however, for the most part are either no longer available or otherwise not utilized as undergraduate (nor graduate) textbooks in geotechnical education.

With a lack of background in CSSM, the result is that many practitioners still cling to the nebulous notion of running a set of strength tests on triplet laboratory specimens to produce three Mohr's circles with corresponding total stress c and ϕ parameters. For clays, the idea of "$\phi = 0$" analysis still prevails in practice, yet all soils (clays, silts, sands, gravels) are frictional materials. As a consequence, the term "cohesion" in often used without clear meaning, in some instances referring to the undrained shear strength ($c = c_u = s_u$) yet in other circumstances to mean the effective cohesion intercept (c'). The latter is obtained by force-fitting of a straight line ($y = mx + b$) to represent the Mohr-Coulomb strength criterion ($\tau = \sigma' \tan\phi' + c'$) from laboratory strength data. In fact, the strength envelope is more complex and best described by a frictional envelope (ϕ') having a superimposed three-dimensional curved yield surface that is governed by the preconsolidation stress ($P_c' = \sigma_{vmax}' = \sigma_P'$). The shape, size, features, and movement of the yield surface distinguishes one constitutive model from another, yet this facet is not necessary in order to convey the overall simplicity and elegance of CSSM (e.g., Lancellotta, 1995).

With respect to strength characteristics, CSSM is a valuable framework to interrelate concepts of frictional strength and consolidation, normally-consolidated and overconsolidated behavior, contractive vs. dilative response, undrained vs. drained strengths, porewater pressure generation, and other matters. In the most simplistic version involving saturated soils (e.g., Schofield & Wroth, 1968), only three soil properties are considered (ϕ', C_c, C_s) in addition to the initial state (e_0, σ_{vo}', and OCR $= \sigma_P'/\sigma_{vo}'$). An infinite number of stress paths can be imagined for each soil element, ranging from drained to undrained, semi-drained to partly undrained. The condition called "undrained loading" is merely one particular stress path which occurs at constant volume ($\Delta V/V_0 = 0$). Therefore, there is little reason to measure the undrained shear strength using the UU (useless, unreliable) test on "undisturbed" samples, or unconfined compression, or series of three triaxial specimens, or other. At any depth, the undrained strength can be reliably calculated for intact clays [$s_u = (s_u/\sigma_{vo}') \cdot \sigma_{vo}'$] where the normalized ratio for simple shear conditions is given by (Wroth, 1984; Kulhawy & Mayne, 1990):

$$(s_u/\sigma_{vo}')_{DSS} = \tfrac{1}{2}(\sin\phi')\ OCR^{(1-Cs/Cc)}$$

As clay strength is anisotropic, simple shear is an overall representative mode for embankment stability, excavations, and foundation behavior (e.g., Ladd, 1991). Recent data on clay strength from a number of well-documented sites shows that CSSM does in fact provide sound reasonable results. Figure 7.1 provides the normalized undrained strength ratio (OCR = 1) for the direct simple shear (DSS) mode for intact natural clays versus the $\sin\phi'$ over the full range of observed frictional characteristics from $18° < \phi' < 43°$ (Diaz-Rodriguez, Leroueil, & Aleman, 1992). For overconsolidated clays, Figure 7.2 presents the combined effects of stress history and friction. Yet CSSM does not include all aspects of soil behavior, however, and the influence of fissuring can be significant with a corresponding reduction in the strength, as illustrated by the data on London clay.

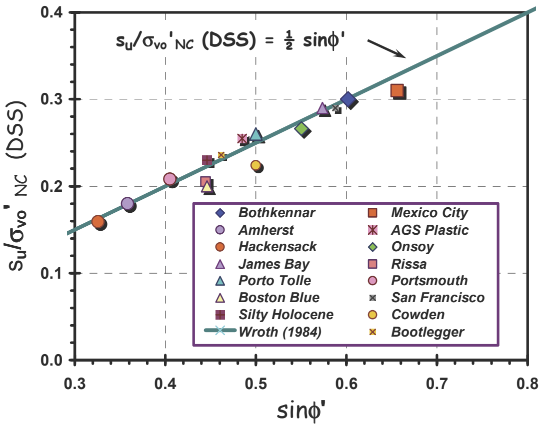

Figure 7.1. CSSM-evaluated undrained strength at OCR = 1.

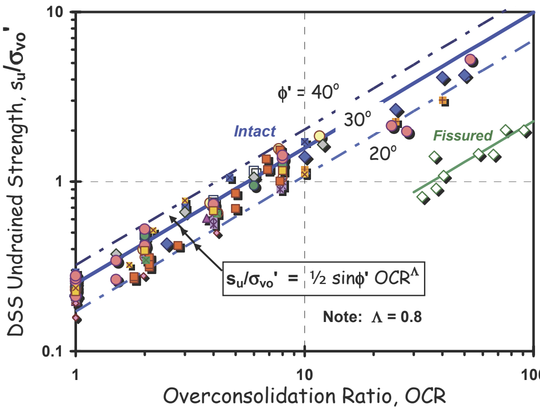

Figure 7.2. CSSM undrained strengths for OC clays (Mayne 2005).

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8 SUMMARY OF PRESENTATION BY SECOND ACADEMIC (BOLTON, M.) ON ISSUE 1

Practitioners are not generally aware that some simple but surprisingly accurate calculations are available to predict ground movements due to construction on or in clays, as first illustrated by Bolton (1993a). This has developed into a new approach analogous to the limit equilibrium method, but made applicable both to ultimate failure and to ground displacements under working conditions, through the adoption of geo-structural mechanisms. Such mechanisms feature compatible strain and displacement fields derived from closely monitored field studies, from analogous centrifuge model tests, or from non-linear finite element simulations. The essential insight is that the geometry of such deformation mechanisms depends hardly at all on the particular stress-strain behavior of the soil (only that it will be broadly non-linear) or, within wide bounds, on its strength or stiffness profile with depth. Plastic deformation mechanisms can be used in the spirit of upper bound plasticity solutions to assess the equilibrium of the ground, but they also provide clear guidance on the compatibility of the ground movement with that of the structure. Ground displacements are normalized by the significant structural dimension in the case being considered, so that the shear strain inside the mechanism is related via a constant compatibility factor M_c to the settlement/width ratio of a footing, or to the lateral displacement/height ratio of a retaining wall, for example.

Predictions are based on a stress-strain curve for an undisturbed core, or from a pressuremeter test, from a location which is chosen to be representative of the average for the zone of deformation. Typically, this will be the centroid of the plastic mechanism. The test axes can be normalized for application at field scale through an equilibrium factor N_e applied to the stress axis, and the compatibility factor M_c applied to the strain axis, both derived from the assumed plastic deformation mechanism. In this way, a stress-strain test can be converted into a full scale load-deformation prediction.

Practicing engineers already understand equilibrium scaling factors for foundations, for example, in terms of bearing capacity factor N_c. However, they wrongly assume that this can only be applied at the ultimate limit state (ULS). In fact, the same equilibrium factor $N_e = N_c$ also represents the ratio between a working bearing pressure and the average mobilized shear strength in the soil supporting the footing. So this can be the first step in calculations to limit ground movements at a serviceability limit state (SLS). Given a representative stress-strain curve, the designer can deduce the average mobilized shear strains in the zone of plastic deformation, and this can be converted into a structural distortion using the compatibility factor M_c. This approach has recently been validated for shallow circular or square foundations on clay (Osman and Bolton, 2005).

Engineers do not presently have much confidence in predicting ground displacements, because the choice that has confronted them has been either to treat the soil as quasi-linear-elastic, or to face the task of validating and applying complex constitutive models in finite element analyses. The new approach described here is called the Mobilizable Strength Design (MSD) method. It is set out in more detail in Osman and Bolton (2005b), and provides an approximate but straightforward calculation method which might be regarded as the foundation engineer's equivalent of structural engineer's beam theory. Rapid progress is being made in extending the repertoire of boundary value problems for which plastic deformation mechanisms have been derived and for which MSD has been validated: Osman and Bolton (2004, 2005a, 2005c).

In principle, geotechnical engineers should be happy to find that they can estimate the influence of construction effects in clays – the settlement of footings and rafts, the permissible load on piles, the rotation of retaining walls, or the settlement trough behind a propped bulkhead subject to excavation – all on the basis of an undrained triaxial test and few simple calculations. It means that performance-based design for limiting ground movements is much closer to fulfillment than they might have expected. But in practice, they may feel constrained to follow Codes of Practice that demand conventional factors of safety against collapse (or equally arbitrary partial factors on soil strength). In a 1993 keynote lecture to the TC23 Conference on Limit State Design, the author (Bolton, 1993b) posed the question "What are partial factors for?". The argument that was put forward was that the large majority of geotechnical works displace excessively before they collapse, and that the superstructure would effectively be ruined and dangerous before the soil was mobilizing its full strength. Criteria for structural damage imply that the neighboring soil should not strain differentially in excess of about 0.1%. The most useful design approach to preventing structural damage or collapse would therefore be to prevent excessive ground strain of about this order. Unfortunately, this argument found little support amongst the Eurocode committees who had decided, pragmatically, that European harmonization of design rules must be based on ULS criteria, not SLS criteria. Perhaps a new generation of engineers, building owners and insurance companies will be more attracted to SLS-based design using MSD.

The author does not, however, expect unqualified approval for the MSD method, with its predictions based on hand calculations or a simple spread sheet. Academic experts on ground movements in clay have generally used finite element analysis based on mathematical constitutive models of soil behavior. Such experts may well criticize MSD on the grounds that it does not explicitly account for heterogeneity, anisotropy or sensitivity for example. Regarding heterogeneity, critics may need to be reminded that while collapse tends to be an extreme value problem, deformations prior to collapse are better understood using the statistics of averages. A contribution of MSD is to define the region within which the average soil stiffness is required. Regarding the finer points of soil constitutive behavior, critics may need to be reminded that very few (if any) practical designs are done in which a full set of soil tests is commissioned to provide values for the dozens of parameters used in a complex soil model. At least MSD clarifies, for example, that it is the average of compression and extension triaxial test data which may best predict the performance of shallow foundations, and that simple shear test data may be preferred for the effects of deep excavations.

Engineers need mechanistic models such as MSD to take decisions and make designs. MSD teaches us, first, that our previous obsession with the peak strength of soil has been misplaced – it is not mobilizable without incurring severe structural damage. Second, it teaches us that we can get a good estimate of ground displacements at working load from a single, high-quality stress-strain test. Third, it shows that a good location for the sample is the centroid of the plastic deformation mechanism.

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9 SUMMARY OF PRESENTATION BY THIRD ACADEMIC (TATSUOKA, F.) ON ISSUE 1

It seems that "*particle size effects on the stability of soil mass consisting of densely compacted granular material (e.g., issues of slope stability, earth pressure bearing capacity of footing …..), linked to post-peak strain softening associated with shear banding,"* have potential for practical application but has not been used extensively by practitioners. Apparently, many engineers and researchers recognize that a soil mass becomes more stable with an increase in the particle size under otherwise the same conditions. However, this factor is usually not taken into account in the limit-equilibrium based stability analysis. Rather, the angle of internal friction decreases with an increase in the particle size in drained triaxial compression tests. This dilemma basically results from the assumption of isotropic rigid-perfectly plastic stress-strain behaviour with a zero-thickness of shear band of soil, by which the peak strength becomes constant irrespective of strain and all the local peak strengths are mobilized simultaneously along the whole shear band(s) that control(s) the collapse load. On the other hand, the actual soil exhibits anisotropic pre-peak stiffness and strength with strain-softening with a shear band thickness rather proportional to the particle size. In particular, the peak-to-residual strength ratio is controlled by the dry density, ρ_d, and the uniformity coefficient, U_c (among others), while the strain-softening rate is controlled by the ratio of particle size to specimen size. Due to these factors, the failure of a soil mass is more-or-less progressive, becoming more with an increase in: a) the total rotation angle of shear band; b) the ratio of shear band length to particle size; and c) the soil compressibility.

The related long-lasting argument is whether the design soil shear strength should be the peak or residual strength or another in between. As the residual strength is independent of ρ_d and insensitive to U_c, its use cannot properly take into account compaction efforts and the effects of soil type. On the other hand, the peak strength increases with ρ_d, of which the rate becomes higher with U_c, depending on soil conditions and soil type. However, its use in the limit equilibrium-based stability analysis is on the unsafe side, becoming more as the magnitude and rate of strain-softening increases. For these reasons, a new practical method using both peak and residual strengths while taking into account the strain-softening rate and associated effects of progressive failure (affected by particle size among others) is needed. To this end, it is necessary to understand the shear banding characteristics. The results from drained plane strain compression (PSC) tests on a wide variety of granular material observing shear banding reveled that the shear deformation of shear band until the residual state increases with particle size (as represented by the mean diameter, D_{50}) (Fig. 9.1). This means that failure energy increases with an increase in not only the dry density but also the particle size. Okuyama et al. (2003) showed that seismic shear deformation along the critical failure plane in a rockfill dam evaluated by a modified Newmark method taking into account the post-peak strain-softening property decreases with an increase in not only the peak strength but also the particle size.

Further, the particle size effects on the bearing capacity coefficient $N\gamma$ for a strip footing on a granular material are significant. A realistic FEM simulation is possible only when taking into account the shear banding property and associated particle size effects (e.g., Tatsuoka et al., 1997; Siddiquee et al., 1999). Fig. 2a shows the relationships between the $N\gamma$ value from plane strain model tests and the angle of internal friction, $\varphi_{(PSC,\delta=90\,\mathrm{deg.})}$, from drained PSC tests in which the σ_1 direction was the same as the pluviation direction during specimen preparation. Three types of granular material having different D_{50}s were used. A typical classical theory (assuming isotropic perfectly-plastic soil property) is also presented in Fig. 9.2a. Fig. 9.2b summarises the relationships between the ratio of the measured $N\gamma$ value to the theoretical value for the same $\varphi_{(PSC,\delta=90\,\mathrm{deg.})}$ and the particle size ratio, D_{50}/B_0, from a series of 1g and centrifugal model tests. The scatter of data for the same D_{50}/B_0 is due to the pressure level effects. A general trend that the ratio of the measured $N\gamma$ value to the theoretical value decreases with a decrease in D_{50}/B_0 (i.e., particle size effects) is obvious. In usual full-scale cases, D_{50} is much smaller than B_0. When D_{50} is not much smaller than B_0, any failure mechanism with shear bands may not be formed, resulting into $N\gamma$ values larger than the theoretical value.

In summary, the classical stability theories simplify, perhaps overly, the actual stress-strain behaviour of soil, ignoring the particle size effects associated with shear banding and strain-softening in particular. On the other hand, the soil mass becomes generally more stable with an increase in the particle size, not because of an increase in the peak shear strength but because of a decrease in the post-peak strain-softening rate. A new practical method using both peak and residual strengths while taking into account particle size effects is necessary.

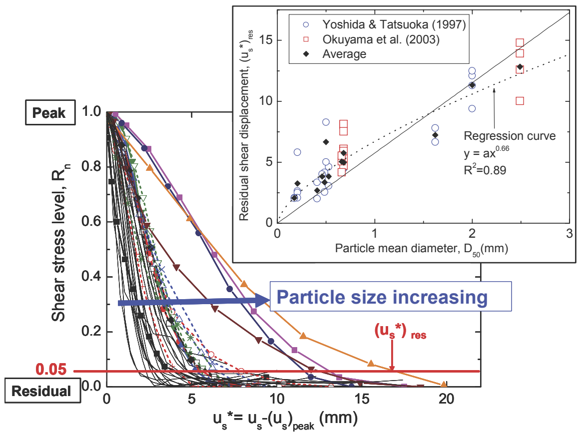

Fig. 9.1. Effects of particle size on the shear deformation of shear band in drained PSC tests (Okuyama et al., 2003).

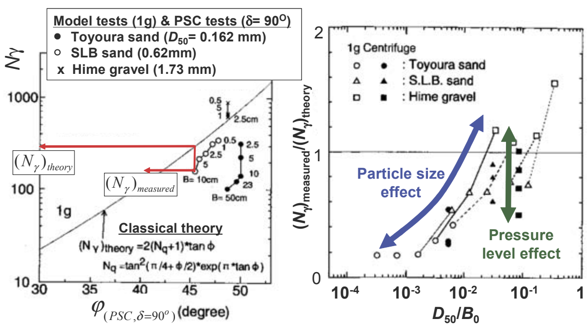

Fig. 9.2. Particle size effects on $N\gamma$ for a strip footing on a granular material (Tatsuoka et al., 1997).

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10 SUMMARY OF INVITED DISCUSSIONS ON ISSUE 1

10.1 *Discussion by Prof. Charles Ng (The Hong Kong University of Science and Technology)*

10.1.1 *Introduction*

There are two areas of geotechnical research that have not been used by practionsers extensively. The first one is in the area of theoretical development of unsaturated soil mechanics and the second one is the use of a geotechnical centrifuge as a physical modelling tool for assisting in engineering designs.

10.1.2 *Theory of unsaturated soil mechanics*

During the first day of the Practitioner/Academic Forum, Mr Peter Day from South Africa presented a number of practical examples that could employ the theory of unsaturated soil mechanics. One was moisture-induced collapse settlement problem, the solution of which requires the theory of unsaturated soil mechanics.

In the last two decades, significant progress has been made in understanding the fundamental behaviour of unsaturated soils and various theoretical frameworks and constitutive models have been developed to address different aspects of mechanical and flow characteristics of unsaturated soils (Alonso et al., 1990; Fredlund & Rahardjo, 1993; Thomas & He, 1994; Wheeler & Sivakumar, 1995; Cui & Delage, 1996; Chiu & Ng, 2003; Alonso & Navarro, 2005). Advances in laboratory testing methods (Fredlund, 2002; Fredlund & Rahardjo, 1993; Ng et al. 2002a; Hoyos et al. 2005; Hoffmann et al. 2005; Ng & Chen, 2005), field measurement techniques (Alonso et al. 2005; Marinho et al. 2005; Tarantino et al. 2005), in-situ monitoring programmes (ENRESA, 2000; Ng et al. 2003; Alonso et al. 2005), and physical modelling with centrifuge facilities (Take et al. 2004; Ng, 2005) make it possible to utilise the theory of unsaturated soil mechanics for engineering designs.

In order to gain confidence in using unsaturated soil mechanics in engineering practice initially, it may be best for practitioners to apply it for the designs of temporary works and for forensic studies such as in understanding the causes of unsaturated soil slope failures. Moreover, the theory of unsaturated soil mechanics can be used in the designs of covers and liners for landfills, investigations of transient flows of pollutants in unsaturated soils, designs of underground nuclear radioactive waste isolation system, shallow foundations and railway embankments when the main ground water table is always low.

10.1.3 *Geotechnical centrifuge modelling*

Although the principles and limitations of geotechnical centrifuge modeling are generally well-understood (Schofield, 1980), the geotechnical centrifuge has not been widely used as a common design tool in most countries except for Japan and the UK. With significant advances in robotics technology (see Fig. 10.1.1) for in-flight simulations of various construction activities in an increasing number of centrifuge centres such as at the University of Western Australia, LCPC and HKUST (Gaudin et al. 2002; Ng et al. 2002b, 2005); and substantial progress made in one-directional and bi-axial shaking capabilities at RPI, the University of California, Davis, TIT and HKUST (see Fig. 10.1.2) for in-flight simulation of earthquake-induced problems (Ng et al. 2002b; Ng et al. 2004), it is believed that geotechnical centrifuge modelling should now be useful in practical design problems.

Fig. 10.1.1. The 4-axis robotic manipulator at HKUST (Ng et al. 2002b)

Fig. 10.1.2. The bi-axial shaking table at HKUST (Ng et al. 2004)

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10.2 *Discussion by Prof. Carlo Viggiani (University of Napoli, Italy)*

Current design practice for piled foundations is based on the assumption that the foundation behaves as a pile group with the cap clear of the ground; the design requisite is to ensure that the piles as a whole (and, in some instances, each pile individually) guarantee a proper factor of safety against failure.

In fact, available experimental evidence clearly shows that the cap transmits directly to the soil a significant fraction of the load acting on the foundation. Fig. 10.2.1 (Mandolini et al., 2005) shows that, for piles uniformly spread over the whole foundation area, such a fraction ranges between 20% and 70%.

It is possible to properly account for such interaction with simple methods of analysis. Under service loads the use of codes based on BEM is widespread and can be considered part of the daily design practice; in any case, reasonable predictions may be obtained by the simple PDR method (Poulos, 2000). At failure, the factor of safety FS_{PR} of a piled raft may be expressed as follows:

$$FS_{PR} = \xi_{PR}\left(FS_R + FS_P\right)$$

where FS_R is the factor of safety of the unpiled raft, FS_P is the factor of safety of the uncapped pile group and ξ_{PR} is a coefficient. De Sanctis & Mandolini (2005) suggest that:

$$\xi_{PR} \geq 80\%$$

It is claimed that there is no reason to keep the unnecessarily conservative assumptions that all the load is taken by the piles. Removing such an assumption, that at present is the basis of most current codes and regulations, would allow significant savings in the foundation cost while keeping a satisfactory behaviour.

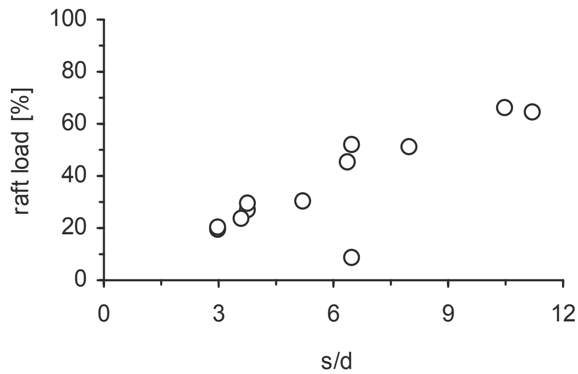

Fig. 10.2.1. Experimental evidence of load sharing between raft and piles. 11 case histories with piles uniformly spread over the whole foundation area

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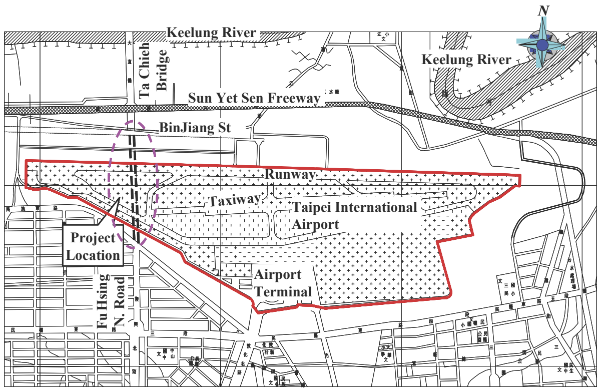
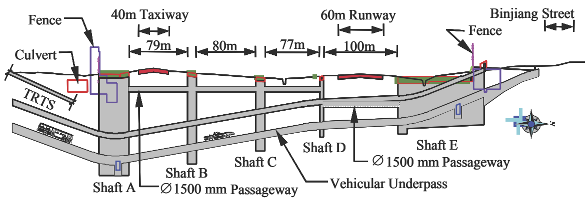
10.3 *Discussion by Dr. Za-Chieh Moh (Moh and Associates, Taiwan)*

To ease the local traffic congestions an underpass has been constructed to extend a major thoroughfare in the Taipei City to connect to the northern part of the city. A major portion of this extension has to pass underneath the only runway and taxiway of the Taipei International Airport which carries more than 300 flights a day for both military and civilian air traffic (Fig. 10.3.1). Due to the importance of the airport to the Taipei Metropolis, air traffic must be maintained all the time. Construction of the underpass within the airport boundary had to be carried out at night and maximum settlement of the runway during construction of the tunnel must be controlled within 25mm. At the time of construction of this 520m long tunnel, it was the first underground tunnel built under a runway with undisrupted operation.



(a) General layout



(b) Longitudinal section

Fig. 10.3.1. Underpass beneath Taipei International Airport

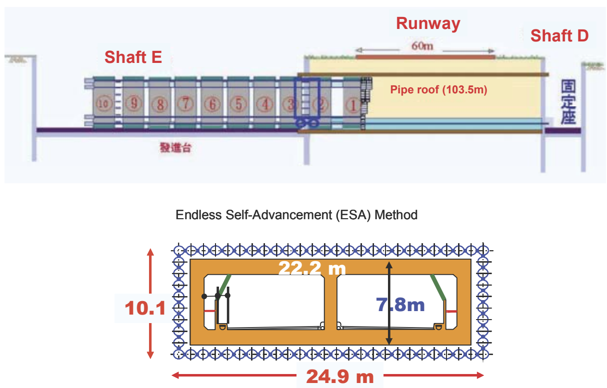


Fig. 10.3.2. Construction sequence

The tunnel, a twin-cell precast R.C. box, 22.20m wide by 7.80m high, was constructed by the combination of pipe-roof and Endless Self Advancing Method. Extensive ground improvement work by means of high pressure as well as low pressure grouting was necessary to strength and to stabilizing the very soft subsoils and to cut off seepages (Fig. 10.3.2).

To portray the effect of construction sequences, two-dimensional as well as three-dimensional finite difference analyses were carried out. Comparing the 3-D analytical results in Fig. 10.3.3 with the actual field measurements of the settlement in Fig. 10.3.4 indicate that results of numerical modeling did give an order of magnitude prediction although they did not exactly match the actual behavior. Due to the complexity of the construction sequence which involved ground improvement, excavation by sections, etc., physically modeling, such as centrifuge test, might be of help.

The Discusser mentioned several observations about cooperation between academics and practitioners, including:

(i) lack of understanding of the actual construction by the academic,

(ii) difficulty in soil characterization modeling,

(iii) the academic has difficulty in appreciating the time constrains of practical engineering projects, and

(iv) too much reliance on students' work effort.

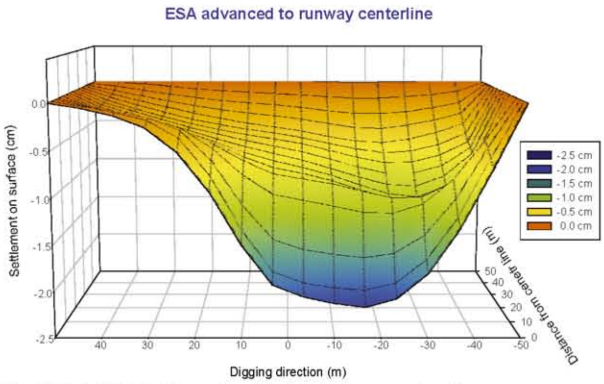


Fig. 10.3.3. 3-D Settlement prediction before construction

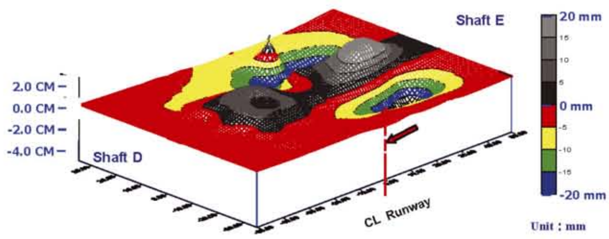


Fig. 10.3.4. Measured ground surface settlement after completion of the ESA construction under runway

10.4 *Discussion by Dr. Luiz G. De Mello (Vecttor, Brasil)*

In our perspective a very important topic in NATM soil tunnelling is that of the time and length delay allowed before invert closure is imposed to the contractors (Fig. 10.4.1).

At what maximum distance should invert closure occur?

The answer to this question, in our point of view, needs to consider two important aspects:

- Calculation of stability requirements,
- Construction requirements, as size of equipment in use at the tunnel face, lateral range of excavators, roadheaders, presence and dimension of utilities, etc.

From a theoretical perspective, some close form solutions are available, most of them based on the lower and upper bound plasticity theorems (Mulhaus, 1985; Atkinson and Potts, 1997a,b; Leca and Domiex, 1990; Leca et al., 2000; Negro et al. 1992; Negro, 1994 etc.). Unfortunatelly these close form solutions have strong limitations, like those of a representative geometry of the tunnel face and of a representative soil stratigraphy. Soil strata as occur in nature, with its heterogeneity (Fig. 10.4.2), different stress-strain-time behaviour, cannot be taken in account. The resulting estimated factors of safety vary over a wide range.

Many jobsites end by defining the referred distance between the recently excavated tunnel face and the closed invert (usually by shotcrete) as a function of the tunnel diameter (1 to 2 tunnel diameters) based on past experience. And, when Contractors have strong arguments to ask for flexibility with relation to the previously defined invert closure distance, designers have loose arguments to present.

In our understanding this is a topic needing attention and research of the Academicians, who should provide the profession with easy to use, flexible formulations that represent site conditions.

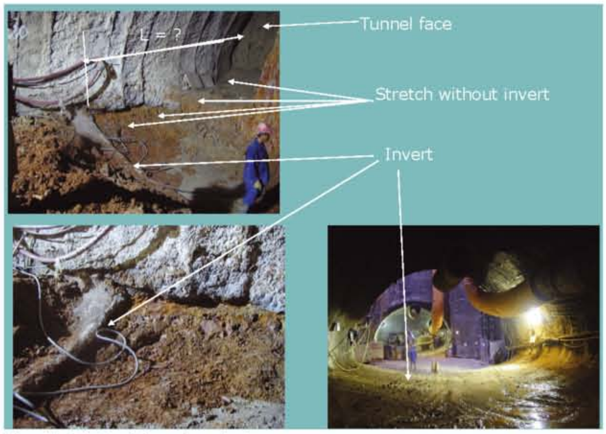


Fig. 10.4.1. Problem description

Fig. 10.4.2. Heterogeneous soils

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11 SUMMARY OF PRESENTATION BY FIRST PRACTITIONER (DAY, P.) ON ISSUE 2

Let me come straight to the point, it is not the prerogative of any grouping within the profession to dictate the course of research. There will always be those researchers who are intent on refining analysis tools and much good can flow from their efforts. However, as with foresters fighting a bush fire, there must be those who go ahead and those who follow some distance behind consolidating the gains made by the front-runners.

11.1 *The path from research to practice*

This forum involves two groups of professionals, the researchers on one hand who develop the theory and the practitioners who apply it to practical problems. However, in the case of numerical analysis, there is a third group involved in the process, namely the software developers.

Constitutive models are developed by researchers who know a great deal about the theoretical behavior of various types of soils, often local materials that are either well researched or behave in a predictable manner. They develop the mathematical models that best describe the behavior of the soil.

For these models find their way into the toolbox of the practicing engineer, they need to be incorporated into various computer algorithms (finite difference, finite elements, etc) by the software developer. The software developer is more concerned about the user-friendliness of his program and the consistency of the results it produces than about the behavior of the soil. Although the manual that accompanies the software will usually provide guidance on the parameters to be used in the program, many of the "niceties" that were the focus of the researcher are lost in the process.

We then come to the design engineer who has purchased the software package and will now use it to obtain solutions to practical problems, often under severe time restraints and inadequate test data. All too often, he will scan the manual until he finds the definition of the input parameters required to run the program and proceed without any reference to the developmental work undertaken by the researchers or the relevance of the solution to the problem at hand. The more complex the computer model, the less likely the average design engineer is to understand the relevance of the input parameters or to assess the validity of the results.

Unfortunately, it is not only the complex models that give rise to problems. Even the most simple and intuitive models are abused as illustrated below.

11.2 *Even simple models abused*

Probably the simplest model for analyzing soil-structure interaction is the subgrade reaction model. It involves a single parameter for modeling the behavior of the soil, namely "k" - the modulus of subgrade reaction.

While any competent researcher will know (a) that this is a gross over-simplification, and (b) that the modulus of subgrade reaction is not a unique property of the soil, there are any number of practicing engineers who will pull a typical value of "k" out of a handbook and use it blindly. In fact, many designers will become quite annoyed when the geotechnical engineer informs them that the modulus of subgrade reaction is not a unique value for a particular soil but depends on a number of factors including the elastic modulus of the soil, its variation with depth, the size and shape of the loaded area. They become even more annoyed when geotechnical engineer tells them that there is a difference between the modulus of subgrade reaction to be used for dynamic analysis of a footing subject to uniform compression to that for the same footing subject to a rocking mode of excitation.

This is a typical case of the designer applying a convenient method of analysis without any understanding underlying theory or of the limitations involved.

11.3 *How can researchers assist in resolving such problems?*

I would like to re-phrase the question from a practitioner's point of view. My query is more along these lines: "What can researchers do to assist practitioners in the every-day analysis of practical problems using numerical methods?".

Firstly, I would like to see research effort being focused on a critical appraisal of the various methods of numerical analysis in common use, including the constitutive models they employ, with a view to identifying their limitations and fields of application.

Secondly, in much the same way as many researchers have compared the results from various methods of bearing capacity or settlement prediction methods with observed performance in the field, it would be of benefit to practitioners to have a comparison of the predictions of performance from various methods of numerical analysis.

Finally, it would be of great assistance to practitioners to be provided with clear guidance on the determination of input parameters for the various constitutive models and the tests best suited to the provision of such parameters.

11.4 *Conclusion*

While researchers may derive great satisfaction from advancing the frontiers of knowledge, there is a desperate need in the profession for a "winnowing" of available knowledge. The methods used in practice will always lag the latest research findings. Few practicing engineers are able to keep abreast with the latest developments and many will spend their entire career using the methods they became familiar with when they still had the time and the inclination to explore new methods of analysis. When new innovations come onto the market (and that is what it is from a practitioner's perspective), it is difficult for the practicing engineer to discern those that will stand the test of time and that are destined to be sidelined.

Yes, the geotechnical industry needs research into new methods of analysis and advanced constitutive models. Equally, it needs an objective appraisal by the research community of the available methods to assist practitioners is assessing which methods of analysis are best suited to the problem at hand. Perhaps most importantly, it needs people with both the knowledge and experience to come from behind and "cull" those methods of analysis which, despite their widespread use, should be deleted from our hard drives.

12 SUMMARY OF PRESENTATION BY SECOND PRACTITIONER (VALENZUELA, L.) ON ISSUE 2

The sole question posed in Issue 2 gives an idea of the general concern of the geotechnical community regarding the many new and complex constitutive laws developed to describe soil behaviour in the last three decades. If the various models proposed in that period are examined, it is evident that the capability of the models has been gained at the cost of their simplicity and with serious difficulties in defining adequate values to a largely increased number of parameters. The reasons why the new, more complex models are not widely used are several: complex mathematical relations difficult to understand; increased number of material parameters, many of them without physical meaning and with problems in assessing proper values to them; time consuming process; lack of well documented information on the use of many of the more complex models in real soil-structures problems. Actually in the geotechnical practice in general the most widely used models are the simple ones having only four to five material parameters with clear physical meaning, e.g. the Elastic Perfectly Plastic model, with Mohr Coulomb failure criteria, and the Non Linear Elastic or Hyperbolic models and less frequently the Cam Clay model. Also the commercially available programs are generally limited to these more "simple" models. But even with these "simple" models the evaluation of model parameters is still a debatable issue that requires more research.

Constitutive laws are an important part of the soil "modelling" process to finally solve practical geotechnical problems, as commented by Burland (1987) and others. But many of the new, complex soil models have been calibrated using particular and homogeneous soil samples with controlled border conditions in laboratory, but not with real soil-structures problems. In a certain way one could say that these models are still part of an "uncompleted" research. Well documented cases of the use of these new complex models in real problems, including the evaluation of the material parameters, would allow a wider and a proper use of them by the geotechnical community.

The question arises whether such complex models can be justified. The more simple soil models have many advantages for the geotechnical practice: limited number of parameters; availability in commercial computer programs with reasonable processing effort; generally good understanding of them by the practice with well documented information of their use in real soil-structures problems. However their use is in fact valid only for some calibrated stress paths. On the other hand the more complex models are aimed at a more general application, at the cost of including many parameters that are difficult to evaluate, as well as complex and time consuming calculation processes.

However there are some cases in which simpler models do not provide the right answer and the use of the more complex soil models could be justified. This use would require considerable effort and time, not always available in common practice, but once such exercises are completed they can constitute valuable examples for the geotechnical community. A good example of the use of a complex model is the application of Lade´s model (Lade, 1979) to the case of the excavation of an urban tunnel in a porous and slightly over-consolidated clay by applying the method developed at the University of Coimbra by Almeida e Souza (1998). The results obtained with this model which uses 14 parameters and considerable processing effort, matched the field data obtained for vertical displacements above the tunnel extremely well, while the results obtained for the same case applying a simple Elastic Perfectly Plastic model with Mohr Coulomb failure criteria provided misleading results (Almeida e Souza et al, 2005).

The answer to the original question presented in this issue 2 is not black and white in nature, but rather a one of compromise:

- An urgent need definitely exist for a substantial research effort to improve the tools used to evaluate soil model parameters, starting with initial emphasis in the simpler models
- The research on more complex soil models should be completed with applications of these models to real soil-structures problems, and only then should they be considered "available" for use on specific problems in geotechnical practice. It is not enough to publish a paper on a new model "calibrated" by a few laboratory tests on a particular soil sample.
- There is a potential role for learned societies such as ISSMGE on furthering this effort: to orient the geotechnical community on the selection and use of the different soil models. This is also important for liability aspects.

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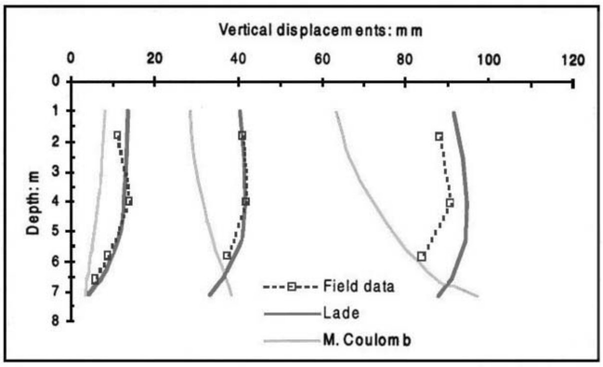

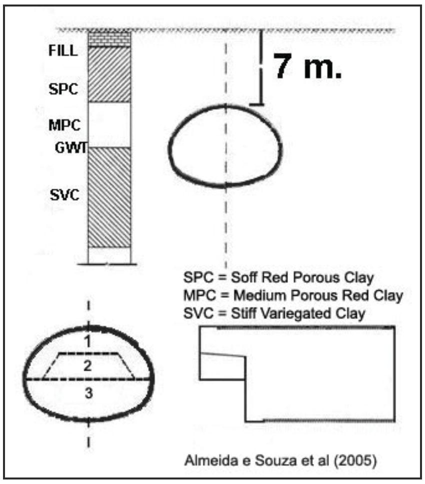

13 SUMMARY OF PRESENTATION BY THIRD PRACTITIONER (CRAWFORD, S.) ON ISSUE 2

Should research continue on complex constitutive laws for soil behaviour when we are unable to adequately assess parameters for simpler models, or should we focus on better evaluation of the simpler soil model parameters?

Response:

This is a loaded question and I welcome it. Yes! We should put more effort into parameter determination. As a practitioner I generally use the c, ø approach for strength. I prefer to use an elastic or elasto-plastic approach for soil-structure interaction problems, but am often governed by the Structural Engineer's desire for a subgrade reaction co-efficient approach (because of their software). Given the wide variation in settlement prediction, I generally use 1-D consolidation theory as well as 2-D elastic/consolidation plus empirical techniques. I am embarrassed to say that I can vaguely remember Cam Clay models and critical state soil mechanics at University. [Even tried to use it once.] And Drucker-Prager, Tresca, ….. well I admit my ignorance.

I have over the years developed a healthy distrust of analytical models that are too complicated for problems that involve a ground model with a wide variation of material types, spatial distributions and properties. As editor of NZ Geomechanics News I took the opportunity to run national surveys on practitioners' approaches to slope stability and competitions on settlement predictions. Both produced widely varying sets of answers. In fact the settlement competition was won by an engineering geologist who used an empirical technique to be just 1mm off "the answer". The other predictions ranged almost between two orders of magnitude.

I recognize that there is always an element of research that is a venture into the unknown. If we solely assess the benefits of research on their direct benefit then this may limit the opportunity for innovation.

I suggest the ease, relevance and economics with which parameters can be determined should be the governing criteria for the need for further research into more complex soil models. Another target should be to develop simple design methods from the complex constitutive models, which require only a few input parameters that are already in general use.

As a practitioner the tools I use, serve me well. So with a resounding "yes", I would like to see more focus on better, more cost-effective ways of determining parameters for the widely accepted simpler soil models.

More than this issue though, is the need to put more effort /focus into geological and geotechnical models. The best "bang for the geotechnical buck" often comes from staged targeted investigations.

Too often, in my experience, the geotechnical investigation is seen as a preliminary stand-alone package, put out to tender well in advance of detailed and sometimes conceptual design. I suggest the Engineering Geologist should control the first stage of investigation, progressing from big picture (regional) geology to site-specific data. In my experience, this is where risk minimization, understanding of geotechnical models, innovation and cost benefits are optimised. Subsequent investigations are very often reduced in scope as a result and are much more cost effective. This is often said to clients who focus on minimising up-front costs, in case the project doesn't fly or gets delayed.

However, over the past two years, I have been privileged to be involved in a Motorway "Alliance" Project where the Constructor, Designer/Planner and Client join forces to complete the project for an agreed budget, scope and time frame; no extras. In this environment, contractual issues, bureaucracy and paperwork are minimised and the focus is on smart ideas, design and methods of construction. An unforeseen problem is everyone's problem and the primary focus is to solve it, not wrangle over whose fault it is or which party is to pay for it.

It is enlightening to see how influential the understanding of the regional geology *and* the site-specific ground model is to achieving significant cost and timesavings. For example, the project has benefited by millions of dollars from the joint effort of geotechnical and geometric designers and planners in adopting various late options:

- tunnels instead of cuts in excess of 60m deep
- reinforced slopes combined with shear piles instead of tied back concrete walls
- steepened cuts in better materials as exposed in trial cuts.

No amount of complex constitutive modelling or even better definition of investigation of soil input parameters could have achieved the same effect (e.g. trial cuts to cause failures for backanalysis, or even better definition of where "rock" is). This sounds so obvious, I wondered whether it was wise to point it out. However, most projects suffer from this very issue: the stand-alone investigation up-front, divorced from the actual conditions uncovered during construction. It is also worth pondering how much money is "lost" on other/previous projects.

Part of the problem is also practitioners often have limited time and budget available and also have to get work approved by regulatory authorities/councils or peer reviewers. While it is admirable that we advance the 'state of the art', what is often needed is for the practicing community to lift its game, i.e. set minimum investigation design and/or reporting standards. In these days of government and corporate drives for efficiencies and 'user pays', the development of engineering standards and guidelines is often voluntary, and so has suffered from lack of interest. Would funding have made any difference to the output of the ISSMGE Technical Committees?

Finally, what about research into those who practise? Are they adopting at least minimum standards of investigation, analysis and design and why not introduce (simple) geotechnical competitions as a standard feature of conferences (e.g. predicting slope failures, pile capacity, settlement or deformation of excavations or foundations). These could be sent out with the conference bulletins and the results collated and published to conference attendees. Why not even invite submissions from those who can't attend to extend the participation of ISSMGE members.

Here then is a chance for the academic who wants to prove a new analytical model, means of determining soil properties or a method of design is a valuable way forward for the profession. These results would provide a source of data for research and information (or a measure) for practitioners on both preferred and accurate analytical methods, as well as a summary of how parameters can be evaluated or inferred from raw data.

14 SUMMARY OF PRESENTATION BY FIRST ACADEMIC (MAYNE, P.) ON ISSUE 2

Higher-order constitutive models are needed in geotechnical engineering in order to properly represent the complete and varied nuances of soil behavior. Soils are very complex geomaterials that have quite complicated origins, constituency, initial states, and fabric, coupled with long histories of exposure to varied environments, weathering, cementation, and ageing. Thus, their mechanical behavior to loading is most difficult to characterize and quantify by simple tests. The stress-strain-strength response of soils is highly nonlinear from the small-strain to intermediate- to large strain range at peak strength, followed by strain softening or critical-state region, and eventual residual strength conditions. Adding to this are the notions of flow characteristics such as permeability, conductivity, and rate effects, such that we must distinguish between undrained and drained, fine-grained and coarse-grained, plastic vs. nonplastic, and additional descriptors including: contractive, dilative, sensitive, brittle, structured, dispersive, collapsible, and liquefiable. All of these facets make the task of geotechnical site characterization a formidable and challenging effort.

Initial efforts to represent soil response started with simple elastic and plastic models (e.g., Ko & Sture, 1981) which gave some order of magnitude in the representation of stress-strain response. Here, soil strength was represented by simple linear approximations, such as the Mohr-Coulomb criterion. However, the advent of critical-state soil mechanics (CSSM) has shown that effective stress state controls soil stress paths, interrelating undrained and drained conditions, virgin to overconsolidated states, and static to cyclic loading, with corresponding porewater pressure generation. The simplest CSSM model predicts only an isotropic triaxial compression type undrained strength (e.g., Mayne 1980). Yet, a hierarchy of undrained strength modes can be captured to represent initial stress state (K_0), direction of loading (compression vs. extension), intermediate principal stress (plane strain vs. triaxial), as well as simple shearing (torsional vs. simple shear). For example, using the rotated bullet yield surface, Ohta et al. (1985) have shown a suite of undrained shear strengths for a given clay with minimal number of soil properties (see Figure 14.1).

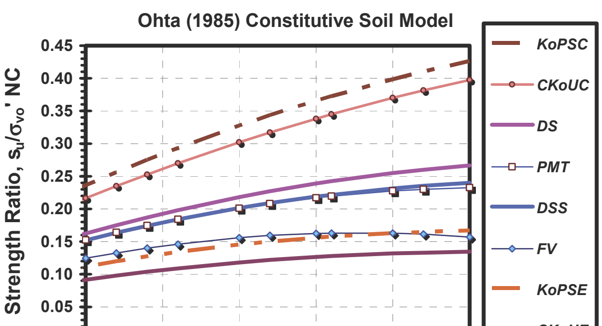

Figure 14.1. Hierarchy of undrained strength modes for NC clays using CSSM model with rotated bullet yield surface (after Ohta, et al. 1985).

Experimental data on natural clays have shown that the failure strength criterion is complex and consists of a frictional envelope and superimposed rotated elliptical yield surface (e.g., Diaz-Rodriguez, et al. 1992). Not only are the magnitudes of strength affected by direction of loading, anisotropic stress state, rate of loading, and strain history, the deformational characteristics and stiffness response of soils are significantly changed (Tatsuoka, et al. 1997). Of particular significance over the past two decades in European and Asian centers, the importance of the initial small-strain shear modulus ($G_{max} = G_0$) as the beginning and start of all stress-strain curves has been documented (Burland, 1989). Sophisticated constitutive models with up to 15 parameters have been introduced which can address small- to intermediate- to large-strain behavior (e.g., Whittle & Kavvadas, 1994; Pestana & Whittle, 1999).

Additional efforts are now needed to model the full suite of in-situ tests that are utilized for geotechnical site characterization, as each test method follows a unique stress path during insertion, testing direction, and has a different strain rate (see Figure 14.2). These numerical simulations would help fulfill the mission to obtain a consistent interpretation of different field and laboratory results, as well as provide the means for comparing different types of foundation systems, construction procedures involving ground modification, tunneling, and excavation in a systematic and rational framework.

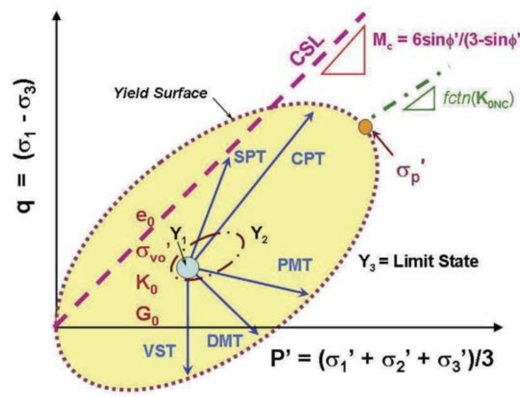

Figure 14.2. Conceptual Stress Paths for Various In-Situ Tests

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15 SUMMARY OF PRESENTATION BY SECOND ACADEMIC (BOLTON, M.) ON ISSUE 2

The Author should immediately declare an interest. He is Chair of TC 35 Geomechanics of Particulate Materials, and is therefore the advocate of further research on soil behavior. However, this need not imply that "complex constitutive models" are to be derived. Sometimes, research leads to simplification.

Constitutive modelers assume an unlimited capacity to recover identical undisturbed samples and perform appropriate laboratory tests. They take as their task the creation of a mathematical framework which can absorb the test data by the fitting of modeling parameters (perhaps 20 variables in a "complex" model) so that state paths that are qualitatively similar to those that have been investigated experimentally can be made available to a routine to calculate the incremental stiffness of each finite element in an FE program.

In contrast, the Author's statement in respect of Issue 1, above, makes clear that it should be possible to make acceptable deformation predictions in undrained clays from a single well-chosen stress-strain test by using plastic deformation mechanisms in the Mobilizable Strength Design method. In general, it would be more helpful to practicing engineers if academics made it their mission not to mathematically replicate all possible stress-path tests, but to answer the question "What single stress-strain test can be performed that will most enhance the designer's ability to control ground movements due to the intended construction activity on this site?".

If academics adopted this decision-oriented research agenda they would be investigating the higher-level problem of the sensitivity of deformation predictions to the location and type of soil tests that might be conducted. Although the final answer may not be given in the classical framework of constitutive modeling (it may propose a pressuremeter test at a particular location, with a rebound loop and a pore pressure dissipation phase, with these raw data used in conjunction with a geostructural mechanism) it is very likely that the validation of the answer for different boundary value problems and soil conditions would include FE analyses using realistic "complex" soil models. It is the Author's opinion that such higher level harvesting of the results of FE analyses is required if the gulf between research and practice is to be bridged. The appropriate measure of the success of this change of agenda could be the proportion of geotechnical designs in which at least one high quality stress-strain test was conducted; we currently start close to zero.

Even if some high-quality stress-strain tests do start to be conducted for practical design purposes, much of geotechnical decision-making will remain empirical. Academics can be useful here, also. Empirical correlations require that the soils concerned be characterized. If scatter is found in our correlations, it signifies the existence of undiscovered additional parameters that have varied from site to site without having been recognized explicitly. In these circumstances it is generally helpful to form dimensionless groups of parameters, and to study the influence of each dimensionless group on the behavior of interest.

What is the difficulty with characterizing soils? First there is a sociological problem in the definition of "soil type". Words such as "granular", "cohesive", "plastic", "clay", "silt" and "sand" have been defined with specific meanings that may not relate to a modern scientific understanding. Then there is a difficulty with the definition of "behavior". Terms such as "normal consolidation", "primary consolidation", "secondary consolidation", "apparent over-consolidation" and "liquefaction" are used to loosely convey a behavioral concept, but they can be mystifying and misleading even to specialists. Because all these terms were coined before soil mechanics was well understood, it is almost certain that simpler, more physically meaningful definitions can be made on the basis of actual granular behavior, whether or not the grains can be seen without the aid of a microscope.

The geometry of a sand grain can be conveyed by a Fourier series, of which the first term represents its mean diameter d and higher order terms convey shape characteristics from sphericity through to roughness: Bowman et al (2001). Since clay particles are known to agglomerate due to interparticle attractions during sedimentation, it might be presumed that clay agglomerates might equally be treated as porous, compliant "grains". The basic three mechanical responses of a grain are: elastic deformability, frictional sliding, and crushing strength. So the three material parameters that control granular deformation are: an elastic modulus k, a coefficient of friction μ, and a fracture strength Σ. The existence of only three granular deformation parameters is a considerable encouragement to those who model with Discrete Elements (DEM). The non-linear behavior of soils and their ultimate instability, emerges naturally from the self-organization of the assemblies of these very simple grains.

In addition, there are complications such as the reduction of coefficient of friction with reduced velocity which may promote creep, and the increased probability of grain fracture with increased time of loading which may also promote creep. Furthermore, fluid viscosity η and surface tension T exert their influence on flow rates and suction effects through the variation of void channel sizes controlled chiefly by the statistical distribution of d values, including the sizes of fluid droplets or gas bubbles. The volumetric proportions of voids e, gas S_r, and grain sizes d form the backbone of geotechnical engineers' current characterization of soils, and understandably so.

It follows that the majority of parameters used in advanced constitutive models must represent the effects of d, k, μ and Σ across the whole population of grains, in relation to the rearrangements of the fabric. Geotechnical engineers are currently unaware of insights on soil behavior that can be gained by greater familiarity with k, μ and Σ. However, research conducted by members of TC 35 is providing direction. For example, Nakata et al (2005) relate normal compression and critical state lines to grain characteristics, and Cheng et al (2005) explain why crushing irreversibly alters critical voids ratios.

This understanding of grain breakage may have far greater implications. Since clays and sands share a normal compression and critical state framework, and sands have been shown to achieve this by grain breakage, the simplest hypothesis for clay compressibility in the absence of tensile strength would be to adopt the same terminology. Ultimately, this understanding of clastic mechanics should make it possible to envisage a common framework encompassing all soils from rock-fill to clay, with a reduction in the number of arbitrary parameters, and a focusing of currently confusing concepts.

If so, soil characterization will have been sharpened through research, and engineers should make fewer basic errors (confusing cohesion and friction calculations in clay, for example) that lead to catastrophes. Of course, those who enjoy constructing constitutive models will equally be free to make better ones. But a more creative and practical contribution to the profession of geotechnical engineering will come from a fresh focus on fundamental granular mechanics, with an application to the characterization of soils and soil behavior.

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16 SUMMARY OF PRESENTATION BY THIRD ACADEMIC (TATSUOKA, F.) ON ISSUE 2

It is true that *all the time the development of a simple-enough constitutive model for soil using a smaller number of determinable parameters should be sought, but research should continue to develop a more soil-like model as needed*. Shear banding and coupled effects of ageing and loading rate are only two factors among many others that need more research.

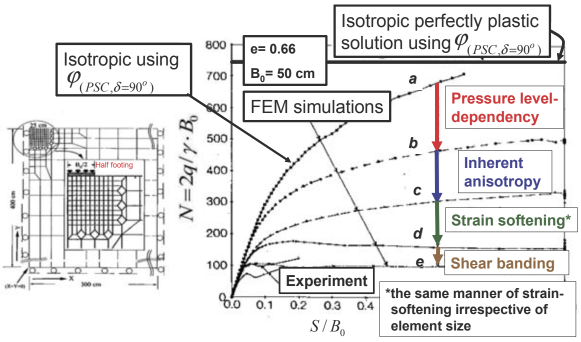

Fig.16.1. Effects of assumptions in the constitutive model for sand on the plane strain FEM analysis of the bearing capacity of a strip footing on Toyoura sand (Siddiquee et al., 1999).

Firstly, for realistic FEM analysis of the failure of a soil mass, it should be taken into account the fact that the shear deformation of shear band during strain-softening toward the residual state increases with particle size. That is, for the same dimension of FEM element, the average post-peak stress-strain relation becomes slower for a larger particle. For example, Fig. 16.1 shows that the bearing capacity of a strip footing on sand can be predicted by the FEM analysis only when based on a more soil-like constitutive model.

Table 16.1. Definitions of ageing and loading rate effects (Tatsuoka et al., 2003; Kongsukprasert & Tatsuoka, 2005).

	Phenomenon	Mechanism/material property	Parameter	
e.g., cement-mixed soil	Ageing effect	Time-dependent changes in the material property+	Time with the fixed origin (t_c)	
	Apparent ageing Loading rate effect (creep, stress relaxation, etc.)	A rate-dependent response of material due to viscous property	Strain rate, $\dot{\varepsilon}^{ir}$	e.g., air-dried Sand*

+ Positive ageing: e.g., cementation
Negative ageing: e.g., weathering

* excluding geological effects

Secondly, ageing and loading rate effects are caused by different mechanisms (Table 16.1). For a soil specimen subjected to different loading histories (1)-(5) in drained triaxial compression (TC) performed along the same stress path (Fig. 16.2a), a unique stress-strain curve (like curve (1) in Fig. 16.2b) is predicted by an elasto-plastic model without effects of loading rate and ageing, whether the constitutive law is complex or simple. For an elasto-viscoplastic model without ageing effects, different stress-strain curves due to different loading rate effects are obtained (Fig. 16.2b). Apparent ageing effect due to the viscous property is observed when monotonic loading (ML) is restarted after creep a-b for loading history (3) (Fig. 16.2b). The same stress-strain curve is obtained for loading histories (1) & (2). With ageing effects, different stress-strain curves due to different effects of ageing and loading rate are obtained (Fig. 16.2c). For loading history (3), after creep a-b, the stress-strain behaviour becomes very stiff for a large stress range. Without coupling between the effects of ageing and loading rate, the same

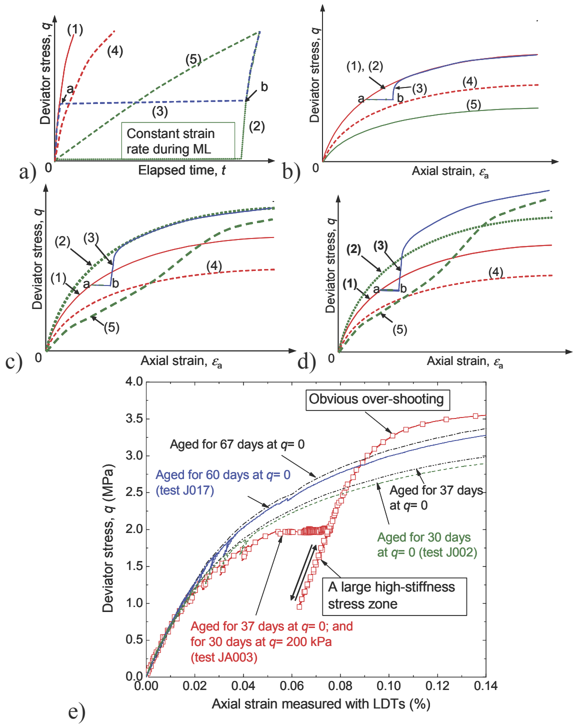

Fig. 16.2 .a) Various loading histories in drained TC; and stress-strain curves for elasto-viscoplastic models; b) without ageing; c) with ageing (no coupling); and d) with ageing (positive coupling); and e) drained TC tests on moist cement-mixed gravelly soil (σ'_c= 19.7 kPa & $d\varepsilon_v/dt$= 0.03 %/min) (Kongsukprasert & Tatsuoka, 2005).

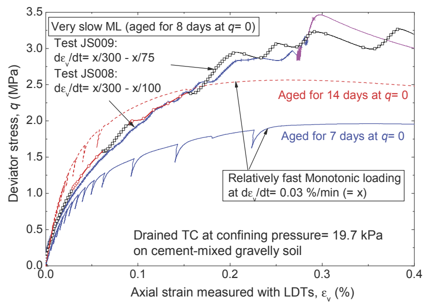

Fig. 16.3. Effects of ageing during very slow ML TC tests (Kongsukprasert & Tatsuoka, 2005).

stress state is obtained for different loading histories when the irreversible strain rate and ageing period are the same (for the isotach viscosity, Fig. 16.2c). With positive coupling between the ageing and loading rate effects (Fig. 16.2d), the ultimate strength for the same irreversible strain rate and ageing period becomes larger as aged longer at higher deviator stresses. The test results presented in Fig. 16.2e indicate that the cement-mixed soil backfill may respond only elastically to load perturbation (such as traffic load) after aged at a shear stress. This result also explains why old natural soil deposits are very stiff for a large stress range. For loading history (5) (at a very low strain rate), the tangent modulus may become larger during ML and a high ultimate strength may be obtained due to ageing effects (Figs. 16.2d & 16.3).

In summary, more research is necessary to develop more soil-like constitutive models as needed, which should be simple enough to be used in practice.

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17 SUMMARY OF INVITED DISCUSSIONS ON ISSUE 2

17.1 *Discussion by Dr. Suzanne Lacasse (Norwegian Geotechnical Institute)*

Based on about 25 years as a practitioner and 10 years as an academician, one does not have to choose between "either" complex "or" simple constitutive laws. Engineering is making the judgement on <u>when</u> to use the one or the other, and when using the models to <u>critically select</u> the parameters and <u>critically assess</u> the results; and to even do this more critically than done in practice today.

Much is to be said for using a model as simple as possible. We can today evaluate reliable parameters for both simple and complex constitutive laws. A proof of this is that the profession can predict (beforehand) the results of model tests and prototypes with measured or interpreted soil parameters, and using simple or complex soil models. We can also do this for simple and complex loadings, e.g. bearing capacity problems, slope instability and offshore installations subjected to cyclic loading. We need though to evaluate in a more objective manner the reliability of the different constitutive laws and analysis models. To achieve this requires benchmark databases of results of model tests and prototypes recognised by the profession for their quality, and the validation of the different models with these data.

Academics and practitioners complement each other. Research is important to maintain technical competence and readiness for design, to bring breakthrough technologies to practice and to promote future growth. Good research develops through the interaction of several persons with different backgrounds and qualifications. There should be a 'revolving door' for industry and academia working in a learning partnership: industry to define needs, give feedback and provide lifelong learning; academia to give perspective, reflect on future needs and also provide lifelong learning. Solutions are optimum when research and practice strengthen each other.

17.2 *Discussion by Prof. Poul Lade (Catholic University of America, USA)*

Simple models such as Hooke's law and the elasto-perfectly plastic Mohr-Coulomb model can be used with numerical methods to determine immediate deformation due to loading and evaluate conditions of failure of geotechnical structures. But what are the soil behavior phenomena that require the employment of advanced constitutive models? The most important issues included in advanced constitutive models are:

Pre-failure: Nonlinear soil stress-strain behavior
Effect of confining pressure, σ_3
Effect of intermediate principal stress, σ_2
Volume contraction and expansion (dilation)
Pore pressures
Behavior during large stress reversals
Anisotropic behavior
Effects of stress rotation
Time effects

Failure & other instabilities:
Peak failure (realistic 3D failure criterion required) & drained softening
Shear banding in dilating soils
Instability and liquefaction of contracting soils

Most realistic, practically useful, advanced constitutive models have been developed with the requirement that parameter determination be performed on the basis of

3 Triaxial compression tests &
1 Isotropic compression test

Procedures for parameter determination for simple and advanced constitutive models have been discussed in detail in Yamamuro and Kaliakin (2005). This publication also contains an overview of models (Lade 2005) in which 31 well-known simple and advanced constitutive models have been evaluated based on 7 criteria:

1) Model includes a theoretically sound framework
2) Model is sufficiently transparent and accessible to anticipate and evaluate its performance
3) Model includes effect of confining pressure
4) Model can handle 3D conditions
5) Parameter determination is straightforward
6) Parameters may be determined from conventional experiments
7) Model exhibits overall high quality of fit with the observed behavior

The real limitations in employment of advanced constitutive models are caused by:

Education: Personnel may not have sufficient knowledge about constitutive models and their performance
Expense: Required testing of intact soil samples can be expensive

It should be noted that determination of soil parameters from test results represents a small effort compared with the effort and expense of obtaining the experimental results. The availability of computer programs often simplifies the determination of soil parameters. Thus, whether models are simple or advanced does not substantially change the difficulty in parameter determination. Both simple and advanced models are useful for each their separate applications. Research should therefore continue to improve the advanced constitutive models for soil behavior under various loading conditions.

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18 GENERAL CONCLUSIONS OF ACADEMIC-PRACTITIONER FORUMS (POULOS, H.G.)

18.1 *Session 1*

In this session, the academics were asked to give examples of research that had a potential application to practice, but which had not yet been realized. The aspects discussed by the academics included:

1. Critical state soil mechanics (Prof. Mayne);
2. Simple calculations to estimate deformations including non-linearity effects (Prof. Bolton);
3. The effects of particle shape on the behaviour of granular soils (Prof. Tatsuoka).

The practitioners were the asked to indicate projects where research would have been useful in developing a solution, but was lacking. The projects mentioned included:

1. Time effects for granular fills, including creep and hydro-compression (Mr. Day);
2. Assessment of geotechnical and hydrogeological properties for groundwater and solute transport in heterogeneous ground conditions (Mr. Valenzuela);
3. The properties of volcanic pumice deposits (Mr. Crawford).

In addition, brief discussions were invited from another two academics and two practitioners. From the academic side, Prof. Charles Ng outlined the importance of unsaturated soils and the usefulness of centrifuge testing while Prof. Carlo Viggiani discussed the uses of piled raft foundations for economical deep foundation systems. From the practitioner side, Dr. Za-Chieh Moh pointed out that there has indeed been cooperation between academics and practitioners in solving complex problems in Taiwan, and questioned the extent to which numerical and physical modeling could help practitioners. Dr. Luiz G. de Mello discussed problems relating to the effects of delayed invert installation in tunneling and the effects of heterogeneous ground conditions.

The contributions of the panelists and the invited discussers have been documented elsewhere in this report of the forum.

Based on a show of hands, the audience was composed of approximately 60-65% academics and 35-40% practitioners.

After the presentations, the audience was invited to participate in expressing their views on two issues:

1. Is further research in well-established areas of geotechnical engineering justified?
2. Should we focus at least some future research on consolidation and assimilation of existing knowledge?

Approximately 20% of the audience was of the opinion that research should continue in well-established areas. There appeared to be a view that, despite the fact that we may think we understand such areas, research may lead to new knowledge and improved methods of design and analysis.

About 75% of the audience agreed with the proposition that some research effort should be devoted to consolidating and assimilating existing knowledge. Clearly, this was a view that was shared by both practitioners and academics. It is unfortunate that in academia, so much emphasis is placed on publishing small fragments of new research at the expense of publishing authoritative papers which summarize and interpret, in a dispassionate manner, existing knowledge.

18.2 *Session 2*

The topic of this second session was more focused than in the first session. The following question was put to the panel:

Should research continue on complex constitutive laws for soil behaviour, or should we focus on better evaluation of simpler soil models?

The practitioners responded first, and expressed the following views:

1. Even the simplest of parameters can be misinterpreted in practice (modulus of subgrade reaction) by those who do not have a proper understanding of the limitations of simple models. Also, practitioners need assistance and guidance in selecting appropriate parameters for much of the commercially-available software (Mr. Day).
2. Research should continue on both fronts, but complex models should be applied to real problems to test their applicability, rather than being considered as things-in-themselves (Mr. Valenzuela).
3. An understanding of regional and site geology is likely to be more influential than whether complex or simple models are used. Prediction events are a good way of assessing our ability to analyze geotechnical problems, and should become a standard part of many conferences (Mr. Crawford).

In reply, the academics offered the following views:

1. There is a need to better model the suite of in-situ tests that are now in use in order to obtain a more consistent interpretation and provide the means for comparing alternative foundation systems and treatments (Prof. Mayne).
2. Simpler models can often suffice, but there should be a fresh focus on granular mechanics to better understand real soil behaviour (Prof. Bolton).
3. The development of “simple-enough” soil models using a small number of parameters should continue, but at the same time, research should continue to develop more “soil-like” models which incorporate such realities as shear-banding and coupled effects of ageing and loading rates (Prof. Tatsuoka).

Two invited discussers also presented their viewpoints. The practitioner, Dr. Suzanne Lacasse, outlined an example problem of a retrogressive sliding slope that could only be solved by using a relatively complex soil model. She was also of the opinion that there should be a “revolving door” relationship between academia and industry in a learning partnership. The academic, Professor Poul Lade, expressed the opinion that there were many important aspects of soil behaviour that required an advanced soil model, but that there were some limitations in using such models, in particular educating personnel that needed to know about such models and the expense of the required testing of intact soil samples.

The audience had a similar composition to that for the first session, i.e. about 60% academics and 40% practitioners. When the question of whether research on constitutive models should continue was raised, about 30% of the audience agreed that it should. However, the vast majority of the audience shared the view that research should continue on both fronts, i.e. on developing complex models but at the same time seeking to obtain better estimates of parameters for simpler models. There appeared to be a widespread view that, in order to develop better simple models, one should first develop more complex models and then determine the extent to which simplification could and should be undertaken. This was a view that had been expressed by Professor Tatsuoka, and is encapsulated in Figure 18.1 which was supplied by Professor J-L. Briaud. This figure indicates that we should approach simplicity from complexity so that we understand the limits to which simplicity can be taken.

A feature of this session was the number of people that indicated a desire to discuss the issue but could not because of time limitations. There was also a view expressed by some of the audience that this second session was more fruitful as it was more focused than the first, which was broad-ranging. On the basis of this experience, there would appear to be at least two lessons that could be learned and applied for future academic-practitioner forums:

1. Discussion topics should be focused on specific topics of mutual interest;
2. There should be time allowed for discussion from the floor.

Judging from comments provided to the convenor after the sessions, a floor discussion could have provided both illumination and entertainment for the audience and the panelists alike. Nevertheless, the inaugural academic-practitioner forums appeared to be appreciated and enjoyed by the large audience that attended and participated.

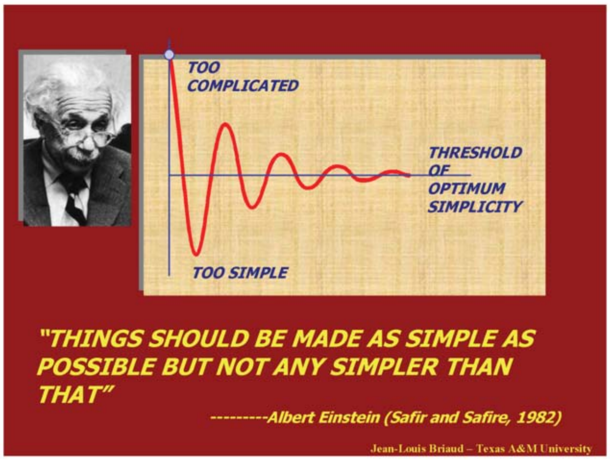

Figure 18.1.Philosophical Approach to Complexity (after Prof. J-L. Briaud)