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Micro-geomechanics

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ABSTRACT: The paper discusses the neglect of micro-mechanics in soil mechanics, and seeks to establish a role that will benefit both the research worker and the practitioner. In support of the mathematical construct of "plasticity", micro-mechanics introduces observations of grain crushing and re-arrangement. Not only does this help to explain the dimensionally inconsistent concept of the normal compression line, it goes some way to unifying our understanding of sands and clays. Indeed, bridging the grain-continuum duality is the key to raising the confidence of practitioners both in the meaningfulness of certain constitutive modelling parameters and in the scaling rules applied to the behaviour of small scale physical models.

1 INTRODUCTION

A gulf has arisen between research and practice in geotechnical engineering. Practitioners need to take decisions about real geotechnical phenomena. They have at their disposal a large body of empirical correlations and rules of thumb with which to filter observations they make on site and in the laboratory. They also have the principles and routines of soil mechanics set out by Terzaghi and his followers, which they learned at University. Research workers, on the other hand, have developed the art of creating elaborate geotechnical simulations, both numerically using finite element packages and physically in centrifuge models. Given the required effort to fix parameters in the mathematical models, or to create representative soil profiles in centrifuge models, devotees of each technology may feel it is possible to predict what will happen in the field. The contrast in research ideology between the mathematical and the physical model is the subject of this Workshop. But we should not forget the credibility gap between the pair of them and engineering practice. Practitioners are often deeply sceptical about the capacity of any simulation technique to recreate the essential features of the behaviour of real soil profiles. The reasons for this are instructive.

The fundamental mechanical behaviour of soils has largely been investigated through the testing of homogeneous reconstituted samples of uniform grain size and mineralogy. Soils with a wide dispersion of particle sizes are empirically known to behave quite differently from uniform soils, but repeatable samples with dispersed particle sizes are very difficult to create in the laboratory. Dispersed granulometries have generally been avoided. This is as true of the databases of compressibility and strength of sands in triaxial tests used to create mathematical models, as it is of centrifuge models used to represent real soil strata. In almost every case, sands are "clean" and clays are "pure". And yet we know that the phenomena of internal erosion, piping and subsidence which afflict gap-graded soils, for example, are of great practical significance in the design of earthworks. We should reflect that they are also precisely the same class of phenomenon as the segregation of grain sizes which deflects us from using dispersed granulometries in our research. This is obviously perverse. It is also characteristic of a wider carelessness of soil fabric amongst research workers that surprises and disappoints the practitioner.

Although every practitioner has been taught the importance of grain size ratios for uniformity coefficient and filter criteria, they find that constitutive models ignore particulate classifications and employ unfamiliar abstract parameters instead, sometimes by the dozen. Real soil profiles are generally defined using a distribution of grading curves, water contents, plastic and liquid limits, and Standard Penetration Tests or Cone Penetration Tests. Having defined some soil layers using these "empirical" index tests, there follows an obscure process leading to the selection of certain modelling parameters representing the mechanical behaviour of each of stratum. Often, these modelling parameters conform to certain features on the non-linear responses of triaxial tests conducted on a relatively sparse distribution of "undisturbed" tube samples taken from within the various strata. Enlightened modellers may seek to cross-correlate modelling parameters with index test values in order to maximise the use of this plentiful empirical information. This laudable objective is presently hampered by ignorance of the physical meaning both of our standard index tests, and of mathematical modelling parameters: these numbers come from universes of knowledge that barely overlap for most engineers.

In other branches of material science, the link between material characterisation and engineering behaviour was made through microscopy. Solids are easy to section and view, in electron microscopes for example, so that idiosyncrasies in macroscopic behaviour could be related to micro-structural features. Soils are much more difficult to "fix" under the microscope so their micro-structural responses to changes in stress, or vice versa, are largely unknown. Soil mechanics, and especially constitutive modelling, has therefore developed in the absence of an understanding of the micro-mechanisms which control macro-behaviour - whether in index tests or in research-quality triaxial tests. International Society of Soil Mechanics and Geotechnical Engineering Technical Committee (ITC35) aims to foster this understanding.

2 UNDERSTANDING COMPRESSIBILITY

Compressibility is conventionally represented on plots of voids ratio versus the logarithm of effective stress using functions that are not even dimensionally consistent (McDowell and Bolton 1998). These familiar bi-linear plots have an "elastic" gradient C_s (using logarithms to base 10) or κ (using natural logarithms) and corresponding "plastic" gradients C_c or λ. An example for dry Dog's Bay carbonate sand is shown in Figure 1.

The relationship between this data and the mathematical models taught to engineers is revealing. There are two distinct phases of *virgin* compression, with 1 MPa as the transition stress. The first phase is far from linear on this logarithmic plot, but much more nearly linear when stress is plotted on a linear scale, see Figure 2. However, it is also almost completely irrecoverable, or "plastic" in nature. The second phase is linear on a log plot, and equally irrecoverable.

The unload-reload loops (not unique lines) are comparatively recoverable, but not exactly so, as the expanded version in Figure 3 demonstrates. Each unload path is more nearly linear against log stress, while the reload path is more linear versus stress. The re-invigoration of plastic irrecoverable strain is seen to occur before the soil reaches its previous maximum stress, and this is seen to create on-going compaction when the cycle is repeated.

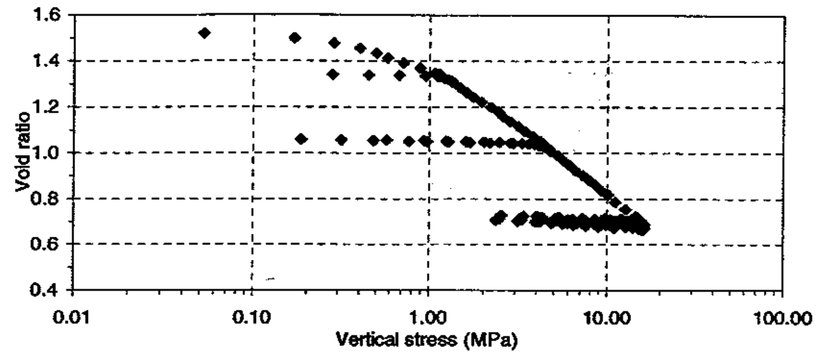

Figure 1. One-dimensional compression curves for Dog's Bay sand

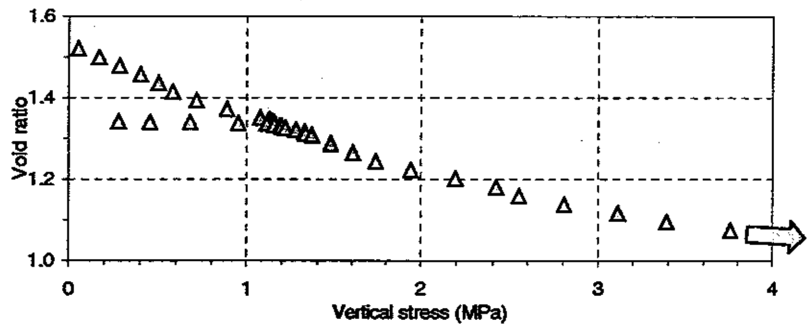

Figure 2. Virgin compression: initially linear, and irrecoverable

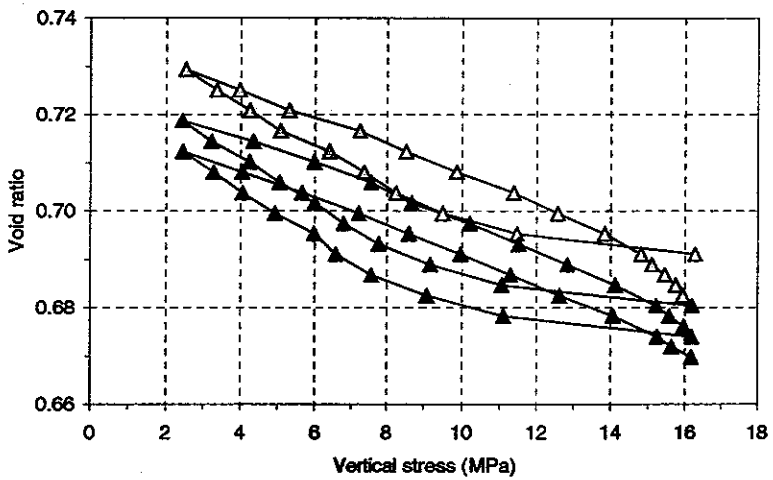

Figure 3. Hysteretic compaction due to unload-reload cycles

Some of the discrepancies between the data and the conventional teaching of geotechnical engineers is understandable in that specialists, at least, would be aware of them. Some discrepancies would seem strange even to engineers who had taken a specialist graduate course, however; this would include the two-phase virgin compression and the question of whether to use gradients based on natural or logarithmic scales. In these circumstances the engineer might look back to start of this section and blame the trouble on the fact that the data is of carbonate sand. That is how we have developed the geologist's habit of assuming that every ground type (carbonate soils, residual soils, quick clays etc.) requires its own specialists and conferences. The agenda of this paper is that we can discover micro-mechanical interpretations of data which should have wider currency than a particular region, stratum, mineralogy or particle size.

The one-dimensional tests reported above were conducted in a mini-oedometer with a glass platen through which photographs were taken with a digital camera - see Figure 4. These images, with Figures 1 to 3, show different regimes of micro-mechanical behaviour to set against the different regimes of macroscopic data. Figure 5 shows the progression of microstructures. At zero stress there is an ambiguity about the definition of the top surface of the sand sample, which is not yet in contact with the glass; there is a significant compression required on the first application of load as projecting particles rotate to permit the glass to bear on a greater number. As stress increases in phase 1 of virgin compression there is an occasional breakage of a few grains, accompanied by rotations and rearrangements of many other grains. Voids are reduced in size mainly by particle rearrangements. Beyond 1 MPa, however, it becomes clear that compression is occurring by the fracture of grains, including those that are already broken. Larger voids close as split grains rearrange; as this continues, the smaller voids themselves are filled by the readjustment of even smaller fragments. This self-similar process is qualitatively the same as that first described by McDowell et al. (1996), where it was pointed out that the mechanism possesses natural hardening due to the increased brittle strength of finer fragments. The data of Figures 1 to 3 can be made much more intelligible through this application of clastic mechanics - the recognition that soils comprise brittle grains that can crush, rearrange and deform elastically.

Images such as Figure 5, when taken together with particle size analyses showing statistical evidence of the consequences of grain crushing, can raise confidence in the data of Figure 1 so as to support the ultimate selection of compressibility parameters. Instead of relying on some mathematical model that was fitted blindly to some particular range of stress applied to some different soil, the selection of plots and parameters can be referenced against the observed response of the microstructure. The compressibility of sand is determined by the degree to which the grains can rearrange with and without fracture. On an unload-reload cycle, as observed in Figure 5, there is very little observable change in structure and correspondingly very little change in voids ratio. Nevertheless, the small cyclic compression seen in Figure 3 may have very significant engineering effects. If a similar change in structure were to occur at constant volume, there would have to be a strong reduction in effective stress. One possible cause of these small changes on cyclic loading is demonstrated in Figure 6 which focuses on the response of a single carbonate grain. It is seen to fragment up to a stress of 5.6 MPa, and the fragments then remain fixed in number on unloading and reloading, while one fragment in particular finds it can rotate and readjust its position. Perhaps these slight rearrangements of fine fragments explain the cyclic degradation of sands, which give rise to liquefaction.

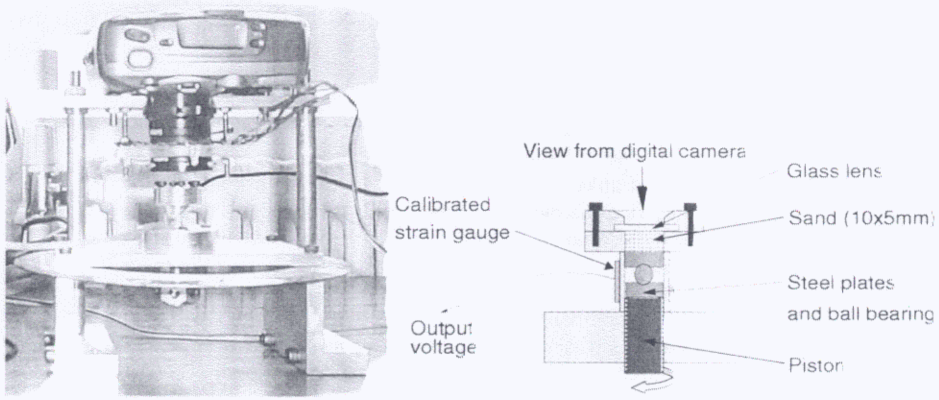

Figure 4. Mini-oedometer for capturing images of microstructure during 1D compression

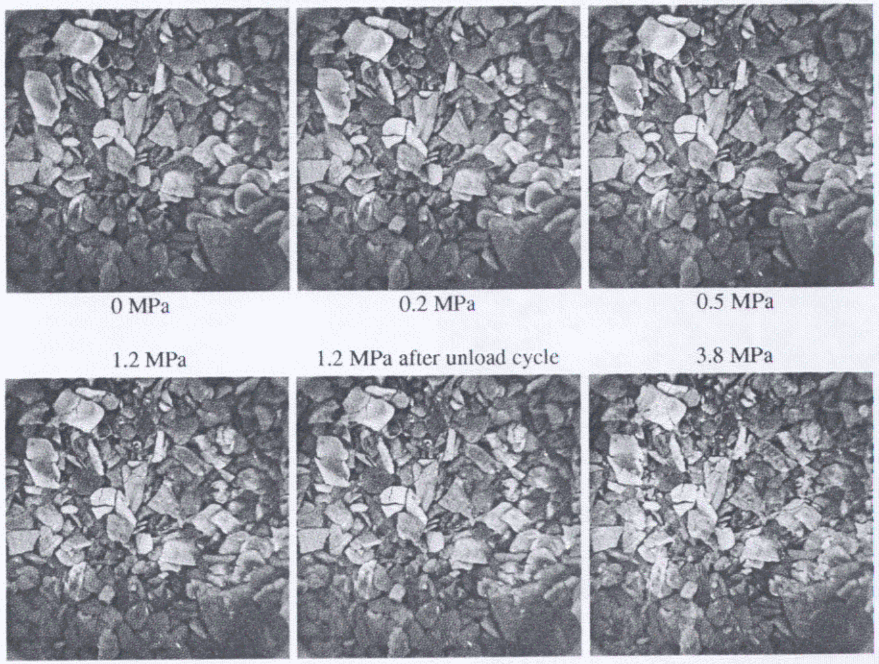

Figure 5. Platy carbonate sand seen through a platen loaded at advancing stress levels

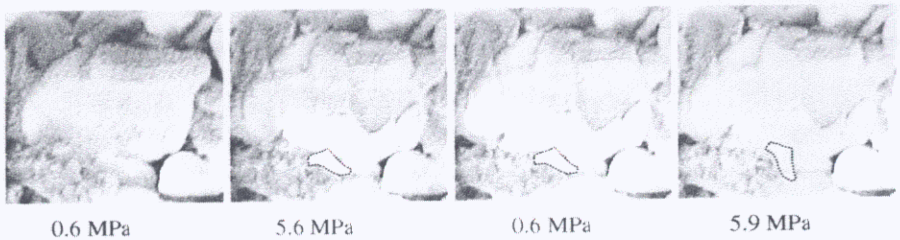

Figure 6. The migration of a small fragment during an over-consolidation cycle

It immediately becomes obvious that platy, shelly sands such as Dog's Bay sand will probably behave quite differently from sands comprising rotund silica grains. This point is proved in Figure 7, which shows the result of compression in a uniform sub-rounded silica sand. After some initial rearrangement as the grains successively carry strong contact forces, and then rotate and release them, the aggregate stiffens up. A very interesting crushing mechanism is then observed. One grain crushes to small fragments against the glass platen; then its neighbours are overloaded and similarly crush. Eventually, the majority of grains at that level are crushed into a "snow" of fines which can be seen to fall downwards into the cavernous voids which remain between the original grains below the elevation at which crushing first occurred. While the fines are percolating downwards, the soil seems rather compressible. It then tightens up in possible preparation for another "snowfall".

The data of Figure 8 clearly show that while the silica sand is stronger than the previous carbonate sand, its "yielding" is more sudden, and its eventual "normal compression line" is much steeper and occasionally unstable. There are a few occasions when the stress actually falls; this is possible since the apparatus is displacement controlled. At points of instability, a heavily loaded grain crushes into a myriad of fragments and the sample unloads since neighbouring grains fail to take up the lost contact force. The easy migration of tiny fragments of silica sand into relatively large parent voids contrasts with the relative immobility of the split grains of carbonate sand; hence the relative large plastic compressibility of the former. The observation of microstructure has not only explained the scatter evident in Figure 8 but has provided a rich stream of ideas relevant to soil crushability and compressibility in general.

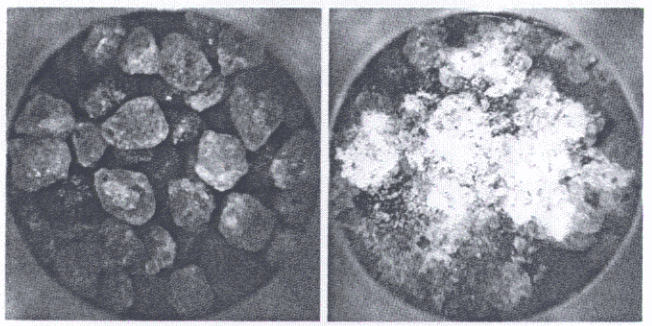

Figure 7. Severe fragmentation during compression of a silica sand

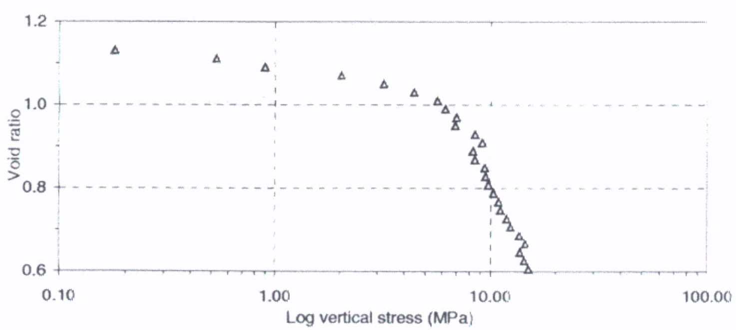

Figure 8. Compression curve for the silica sand

3 UNDERSTANDING PENETRATION RESISTANCE

Friction and dilatancy are understood in terms of grains sliding, rotating and over-riding at points of contact. Engineers broadly recognise that dilation in dense, interlocked soils can produce double the shear strength measured in stress-controlled element tests on loose soil, which does not dilate. The volumetric expansion causes more work to be done against confining pressure at the boundaries. It is also understood that the penetration resistance (CPT, SPT) of dense sands can be greater by an order of magnitude, due to the additional stresses induced around the penetrometer by suppressed dilation. Engineers should also follow the author (Bolton 1986) in understanding that dilation caused by interlocking is very large at small stresses, and reduces towards zero as stresses increase leading to the progressive crushing of interlocking asperities and grains.

Such an understanding in the context of penetrometer data, for example, is of immense value in correlating CPT or SPT data with more fundamental parameters. Klotz & Coop (2001) show that "state" incorporating both density and stress, is required for good correlations of centrifuge model pile test results, and Jamiolkowski et al. (1988) show, from calibration chamber tests, that increased compressibility gives reduced penetration resistance. We are unable to distinguish independently, from SPT or CPT data alone, the effects of reduced sand density and those of increased compressibility. However, one might have thought that we could up-scale from CPT to pile penetration resistance simply using area ratios, since the same mechanisms would be at play. This is not the case, however.

Two factors of uncertain origin emerge from test data. Firstly, a reduction factor of up to 2 or 3 must be applied to q_c from a CPT to obtain the tip resistance q_b of driven piles. Chow (1997) attributes this to the ratio in diameter between the two, explained by the enhanced dilation in shear bands around the smaller penetrometer. White (2002) reassesses the available data and shows that no simple scaling law emerges when grain size is taken into account. Secondly, Heerema (1980) coined the term "friction fatigue" to describe the reduction in skin friction as piles are driven. A variety of field and centrifuge model test data chart the loss of skin friction at a given point in space as a pile is driven past it. Randolph et al. (1994) explained this as a loss of radial effective stress due to local volume reductions accompanying the rearrangement of grains on the surface of a rough pile.

White's micro-mechanical observations of the penetration process provide a rather clearer explanation, and go some way to resolving the contrast between observations made at different scales, and in different soils. Figure 9 indicates the various zones of sand influenced by penetration seen through the window of a calibration chamber. As the model pile advances, the stress beneath it increases beyond that required to crush grains. A "nose cone" of crushed and compacted sand is driven ahead of the blunt pile, creating a quasi-penetrometer. Sand crushes as it is displaced laterally by the nose-cone, and this ends up coating the skin of the pile with fine fragments, once the tip has gone past. These fines can easily migrate into the voids of the neighbouring parent material in the flanks: see Figure 10. When this occurs, the fines effectively disappear, permitting the sand at the flanks to relax towards the pile. One might imagine the asperities on the pile shaft shaking the fines as they travel past, encouraging them to diffuse away. The further the pile is driven, the more relaxation there is, up to the point when the fines have been absorbed entirely. Not only does this micro-mechanical observation support the tenor of Randolph's argument, it also suggests that the granulometry of the parent soils and the manner of its grain fragmentation (i.e. Figure 7 compared with Figure 5) will dominate the rate at which fines can migrate, and at which lateral stresses can relax. Broadly graded sands, comprising grains that split rather than crumble, might be expected to display slower frictional degradation than the rest. This hypothesis needs validation, of course.

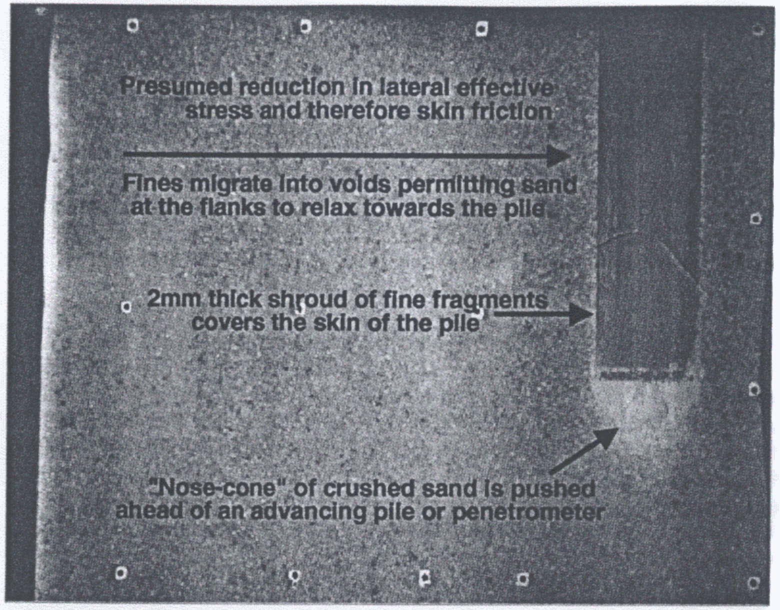

Figure 9. Photograph of sand penetration in a calibration chamber: White (2002)

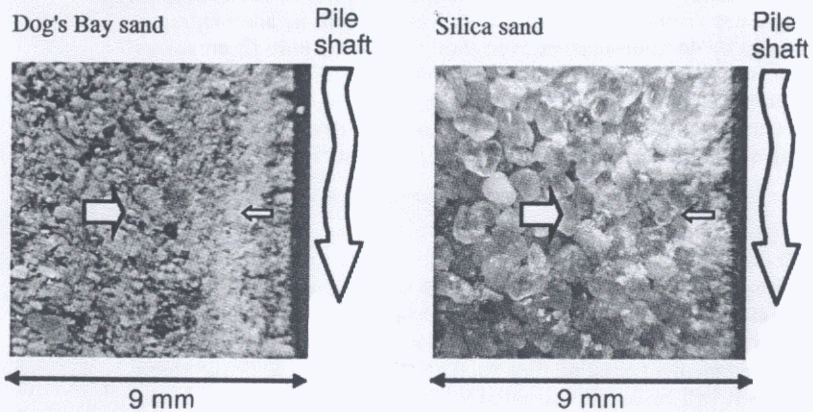

Figure 10. Close-up of fines rearranging to permit relaxation of sand near pile: White (2002)

Micro-mechanics provides an interesting new commentary on research priorities. It suggests, for example, that it will be impossible *in principle* to obtain a good prediction of pile penetration resistance in sand from a finite element simulation, even when finite deformations have been allowed for. Instead, it points to the availability of physically meaningful correlations between soil grain characteristics, CPT data and pile performance, raising confidence both in elementary mechanical models and in the meaning of empirical data-bases.

4 UNDERSTANDING PLASTICITY

There is no better example of confused characterisation than in the understanding of soil plasticity. For geotechnical engineers, the term relates to the range of water contents at which a clayey soil can be moulded, specifically between Atterberg's liquid and plastic limits. The drop-cone test provides an arbitrary "liquid limit" as the water content of a paste that displays a shear strength of about 1 kPa "measured" via the depth of penetration of an 80 gram steel cone. The meaning of the "plastic limit" is more obscure. The test requires that a clay thread must crumble in tension as it is rolled and remoulded. One may imagine that the mechanism must depend on the creation of indirect tension, as in the Brazilian test for concrete, but clay cracking is not really a "solids" phenomenon. The clay particles are utterly cracked and separated to begin with. The separation of two pieces of a rolling thread is solely due to the separation of the once continuous fluid phase. Separation of fluid implies either cavitation or air entry, and the air entry suction for clays is about 100 to 1000 kPa. If the clay were not completely saturated, then gassing would be likely to occur at 100 kPa suction in any event. If the minor effective stress in saturated clay at air entry is 100 to 1000 kPa, then the major effective stress would be 230 to 2300 kPa, if the angle of internal friction was about 23°, typical of clays. This provides a range of 65 to 650 kPa shear strength for clay at air entry and, by implication, at the plastic limit. It is interesting that the shear strength of clay at the plastic limit is quoted in the same range, and increases with decreasing particle size: (Wood 1990).

University research workers in the constitutive relations of soils have a rather different perspective: recoverable strains are called "elastic" whereas irrecoverable strains are called "plastic". Many constitutive modellers base their idealised relationships on the concepts of plasticity theory, created to describe the behaviour of ductile metals and then adapted by Schofield & Wroth (1968) to describe some of the key features of saturated soils - sand or clay, drained or undrained, normally consolidated or over-consolidated - in models which were typified by Cam Clay. The fundamental plasticity concepts that were used to derive Cam Clay were:

- the decomposition of total measurable strains into elastic and plastic components,
- the existence of a closed yield surface in effective stress space which delineates purely elastic from elastic-plastic behaviour,
- a flow rule which fixes the proportions of plastic strain increments induced at yield (in Cam Clay, strain increments to be normal to the yield surface, when consistent definitions of stress and strain are used),
- a hardening rule governing the expansion or contraction of the yield surface (in Cam Clay, simply the current voids ratio).

The Cam Clay concept of soil plasticity was connected with the engineer's plasticity index only through the adoption of an empirical plastic compressibility. As their title suggests, Schofield & Wroth's book, Critical State Soil Mechanics, invoked a critical state line as a unique path of voids compressing according to the logarithm of mean effective stress, as the soil is continually sheared. They could as easily have introduced a line of normal compression. Cam Clay's first prediction is that the existence of one must imply the existence of the other, parallel to it on a log stress plot.

Let us not forget, however, that plasticity to the driller performing ground investigations, or for that matter to the sculptor, simply refers to the "cohesive" mouldability of fine-grained soils. All that is necessary is that the soil can trap water in suction up to about 100 kPa, and that the corresponding effective stresses are not lost on remoulding. Sands can neither trap nor retain such suctions. Their pore sizes are too large for surface tension to produce more than a few kPa of pore suction. Quartz silts may be able to hold suctions but, like sands, their mean effective stress can be annihilated by gentle shaking, or strongly reinforced by remoulding. When confined at constant water content, the "undrained" strength of sand or silt is variable. The driller places a lump of fine-grained soil, "silt" or "clay", on the back of his hand and shakes it: if it liquefies into a puddle with a bright top surface, it is "silt", if not it is probably "clay". He then steadily squeezes the lump: if the once liquefied soil now cracks, the well-trained driller declares that it is simultaneously at its liquid and plastic limits and confirms the diagnosis "non-plastic fine soil".

The stereotypical driller of the last paragraph has, in a sense, set the agenda for soil constitutive relations for the last 25 years. Constitutive modellers have been looking for a more universal soil model, that will replicate many features of Cam Clay, but which will also offer ambiguous values of the undrained strength of saturated sandy soils, leading to phenomena such as liquefaction. Models have certainly been produced, but based simply on curves with more than a dozen parameters fitted over the complex non-linear responses of certain triaxial stress-path tests. Decision-makers generally express doubts about the meaning and value of the many curve-fitting parameters, and designers generally ignore such complex models.

It is proposed here that micro-mechanics offers the way out of the impasse that constitutive modelling has got itself into. The first step is to attempt to place a micro-mechanism against each facet of soil behaviour, and then to relate these mechanisms to constitutive equations. This adventure has hardly begun. Consider the Environmental Scanning Electron Microscope (ESEM) pictures of kaolin mud, shown at three different scales in Figure 11. One must surely been drawn by the similarity with carbonate sand seen in Figures 5 and 6. In each case there is a jumble of apparently broken and breakable pieces of different sizes. And of course, the shape of the compression curve of Speswhite kaolin (Al Tabbaa 1987) shown in Figure 12 bears a striking resemblance to that shown in Figure 1 for Dog's Bay sand. The best fit λ value is almost identical, κ is about twice as large, but the ratchetting compaction after an over-consolidation cycle is also similar. Perhaps there is no qualitative difference between the mechanical behaviour of a platy carbonate sand and a platy kaolin clay. Quantitatively, of course, the clay is about 10 times weaker at the same voids ratio as the sand, and many times less permeable due to its different void size.

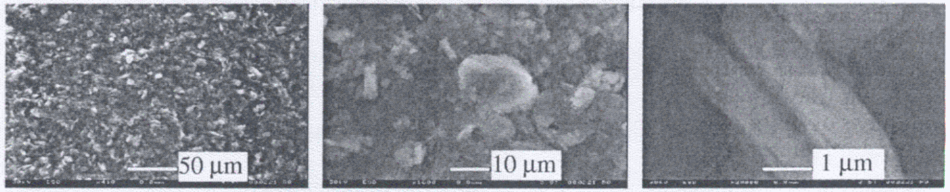

Figure 11. ESEM pictures of kaolin mud

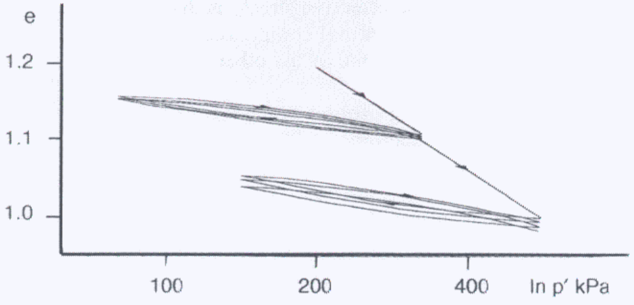

Figure 12. Compression of kaolin (Al Tabbaa 1987)

Where does this leave the engineer's concept of plasticity as defined by Atterberg's limits, or by Critical State Soil Mechanics? Consider the following thought-experiment. If a fluid were found with 10^4 times the surface tension of water, and used as a pore fluid for Dog's Bay sand in liquid and plastic limit tests qualitatively similar to Atterberg's but scaled up in accordance with the greater grain size and strength, would the sand appear to be "plastic"? Instead of having an air entry value of about 10 kPa, the sand saturated with this hyper-water would have an air entry value of 100 MPa. So left out on the bench to evaporate, a "wet" sand would shrink, crush and rearrange its grains after the fashion of Figures 5 and 6 and compress after the fashion of Figure 1, whilst remaining saturated. If one continually remoulded it by rolling it like dough for a French loaf, would it not display a "plastic limit" with a "cohesion" of about 100 MPa? And if, on the other hand, one performed a drop test with a cone of 80 kg mass, might one not define an arbitrary "liquid limit" for a shear strength of 1MPa? Figure 1 clearly shows that the voids ratios at the plastic and liquid limits defined in this way would be about 1.35 and 0.30. The corresponding hyper-water contents would be 50% and 11%, and the plasticity index would be 39%. This would match very reasonably with the observed plastic compression index $C_c = 0.53$, or $\lambda = 0.23$, which are values appropriate not only to Dog's Bay sand but also to kaolin clay. Perhaps the key question is whether the sand saturated with hyper-water would show stable "cohesion" like a clay. Casagrande's original test protocol for the liquid limit called for a groove to be cut in the soil, and then shaken vertically until it closed. Perhaps it would simply be necessary to for the hyper-fluid also to have 10^4 times the viscosity of water, to slow down particle rearrangements, if the sand were to appear "plastic" in Atterberg's sense.

The general concepts of plastic yielding - used in modern constitutive models - are completely unclear with regard to the relative influence of grain crushing and grain rearrangement. Constitutive modellers have seen nothing wrong in defining a yield stress that they can not normalise in terms of any inherent material characteristic of the grains. They rely on the concept of "stress history" without asking what manner of "book" a soil uses to record its autobiography. They expect to be taken seriously by decision-makers even though they can only fit curves using mathematical expressions, in the absence of physical understanding. There are some advances now being made in computer simulation of granular materials by Discrete Element Models. DEM simulations of crushable grains (Robertson & Bolton 2001) may well prove to be crucial in resolving outstanding questions about the true nature of soil plasticity.

5 CONCLUSIONS

A particulate approach to soil behaviour is advocated by ITC35 with the aim of drawing together two contrasting streams of thought - the empirical approach to soil classification, and the continuum approach to constitutive modelling. It will be essential to obtain a new data stream of micro-structural information to complement existing data such as CPT resistance, or the stress-strain data of triaxial stress paths. It will be equally essential to develop micro-mechanical models corresponding to the main classes of macro-behaviour (compressibility, hypo-elasticity, stiffness degradation, strength, dilation, creep, phase transformation, liquefaction etc.) so that the influences of particle morphology, fabric and grain strength can be understood. This should also help physical modellers convince engineers that they have selected appropriate soil types to represent the full-scale field profile, and that the influence of larger relative particle sizes can be accounted for properly.

Some promising developments have been shown. A view through the platen of a one-dimensional compression apparatus has demonstrated that the initial virgin loading of a carbonate sand induces strong rearrangement of the grains, with occasional grain fractures that indicate local stress concentrations. As the stress increases, more grains break. As the virgin compression becomes quasi linear versus the logarithm of stress, one sees the fracture even of previously broken fragments, with a corresponding reduction of volume due to the increased efficiency of packing. On the other hand, unloading and reloading induces a quasi-elastic response, though closer attention reveals a hysteresis loop which does not actually close after each cycle. Further work is required to elucidate whether cyclic compaction (tantamount to loss of effective stress in a cyclic undrained test) is due to irreversible grain displacements, or to further grain breakage. Equally, there is the question of whether a view through a "window", which is analogous to a very large particle, is representative of internal micro-mechanisms. Nevertheless, the growing evidence that plastic hardening is due to grain breakage must be of great future value, and is certainly provocative for those who wish to include the progressive crushing of clay agglomerates in their constitutive understanding of clays.

Evidence has also been presented of the different roles of grain crushing and re-arrangement in the penetration of sands by piles and penetrometers. It is well-known, of course, that cavity expansion analysis provides a reasonable justification for the penetration resistance of sands; the precise kinematics accordingly may have seemed less significant. However, it has now been demonstrated that a model pile becomes coated in fine, crushed sand as it is driven. The apparent diffusion of these fines laterally, into the voids of the parent soil, provides an explanation for the tendency of this flanking soil to fall towards the shaft as this drives past. This lateral soil extension can in turn explain the very large reduction in lateral stress at a given point in space as the pile tip advances a few further diameters. If it were not for this very strong relaxation, the skin friction of driven piles would be enormous. Engineers use empirical factors to predict this reduction, which can almost eliminate skin friction in carbonate sands. However, the precise factors which should influence this reduction factor have not been understood. Observation of the micro-mechanics now suggests that "fretting" is a more accurate description than "friction fatigue" which has been used heretofore, and offers a rational basis for selecting influence factors in a database.

There has been a corresponding success in the modelling of a soil fabric using Discrete Elements, in the shape of agglomerates formed of bonded micro-spheres. The crushability of these grains has been shown to transform the behaviour of the soil element they comprise. Realistic virgin compression and over-consolidated soil behaviour have been shown to depend on grain breakage. This occurs at external stress levels orders of magnitude smaller than would have been expected considering the strength of the individual grains, due to the strong deviations in contact forces from place to place. The precise role of grain crushing (and therefore grain shape and mineralogy) in the gamut of soil constitutive behaviour is now much more open to scientific investigation. There is even the strong indication that concepts such as soil plasticity will be clarified in a way that will unify our understanding of soils and strengthen the ability of practising engineers to make rational judgements about soil properties.

REFERENCES

Al-Tabbaa, A. 1987. *Permeability and stress-strain response of Speswhite kaolin.* PhD dissertation, University of Cambridge.

Bolton, M.D. 1986. The strength and dilatancy of sands. *Géotechnique* 36(1): 65-78.

Chow F.C. 1997. Investigations into the behaviour of displacement piles for offshore foundations. PhD dissertation, University of London (Imperial College).

Heerema, E.P. 1980. Predicting pile driveability: Heather as an illustration of the friction fatigue theory. *Ground Engineering* 13(Apr.): 15-37.

Jamiolkowski, M., Ghionna, V.N., Lancellota, R. & Pasqualini, E. 1988. New correlations of penetration tests for design practice. *1st Int. Symposium on Penetration Testing, Orlando,* 1:263-296.

Klotz, E.U. & Coop, M.R. 2001. An investigation of the effect of soil state on the capacity of driven piles in sands. *Géotechnique* 51(9): 733-751.

McDowell, G.R., Bolton, M.D. & Robertson D. 1996. The fractal crushing of granular materials. *International Journal of the Mechanics and Physics of Solids* 44(12): 2079-2102.

McDowell, G.R. & Bolton, M.D. 1998. On the micro-mechanics of crushable aggregates. *Géotechnique* 48(5): 667-679.

Randolph, M.F., Dolwin, J. & Beck, R. 1994. Design of driven piles in sand. Géotechnique 44(3): 427-448.

Robertson, D. & Bolton, M.D. 2001. DEM simulations of crushable grains and soils. *4th Int. Conference on Micromechanics of Granular Media, Sendai*: 623-626.

Schofield, A.N. & Wroth, C.P. 1968. *Critical State Soil Mechanics.* New York: McGraw-Hill.

White, D.J. 2002. *An investigation into the behaviour of pressed-in piles.* PhD dissertation, Cambridge University.

Wood, D.M. 1990. *Soil behaviour and critical state soil mechanics.* Cambridge: Cambridge University Press.

DISCUSSION

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Sarah Springman commented that David Muir Wood had proposed continuum modelling as the preferred mode for practice, whereas Malcolm Bolton had advanced the discrete element modelling (DEM) cause and Cino Viggiani had worried about localisation effects. Viggiani questioned the DEM numerical results in that a monogranular material had been investigated, with particles of the same diameter (uniform grading) and spherical shapes. He criticised these severe limitations, which would hide some phenomena, and in particular, shear banding. He wondered how strong the limitations would be in terms of shape and grading. Bolton replied that, to his knowledge, this was the first attempt to simulate real crushable soil. He noted that it would be necessary to model very dispersed sizes to get anything looking like soil behaviour, and this research had considered dispersed sizes in the sense that there were grains within the larger aggregates. However he accepted that it would be a significant improvement if the grains themselves had an initial fractal distribution of sizes, and if each of those grains contained micro-grains that could permit that grain, however large, to break. He felt that an even bigger restriction was the roughness of the grains modelled as agglomerates of bonded spheres, but held that the representation of grain crushability was paramount.

Ivo Herle asked about the similarity between clay and sand and disagreed that the only difference was the physico-chemical bond and that these were weak for clays. He pointed out that these bonds were extremely strong, and they depended on the distance between particles, which was much smaller for clay particles. He felt that making these bonds weaker could not explain the similarity in behaviour between sand and clay because the physical background was totally different! Bolton replied that some weak breakable bonds should be present to reproduce the characteristics of a crushable material and that clay particles could be modelled as agglomerates with with strong bonds almost everywhere and weak bonds in some places. He noted that this would be similar to modelling in rock mechanics with fissures, joints and bedding planes, with micro-cracks developing from various directions and points in the agglomerates. Bolton thought that current modelling of clay as breakable agglomerates had captured most aspects of mono-