EFFECT OF PARTICLE SIZE DISTRIBUTION ON PILE TIP RESISTANCE IN CALCAREOUS SAND IN THE GEOTECHNICAL CENTRIFUGE

G.R. McDowell* and M.D. Bolton†

ABSTRACT

Until recently, the micro mechanical origins of soil behaviour have remained illusive, but it is now known that that the constitutive behaviour of a soil is largely determined by its particle size distribution. This paper examines the specific boundary problem associated with the penetration of a model pile into two different gradings of dry calcareous sand in a geotechnical centrifuge, in order to establish the effect of the inclusion of fine particles on the pile end bearing resistance. The first grading of sand comprised particles smaller than 0.5 mm; the second grading contained particles of nominal size d such that 0.15 mm < d < 0.5 mm. Each test was performed on each of two samples of each grading. Tip resistance was observed to rise to a peak at shallow depths, and then fall; a micro mechanical explanation is presented for this instability. Following the centrifuge tests, particles were retrieved from the centres of the soil samples, where the pile had previously been driven, for subsequent particle size analysis. It was found that insignificant crushing had occurred in the sand retrieved from depths less than the depth of peak resistance, but that significant crushing had occurred in the sand retrieved from greater depths. The peak in tip resistance was a factor of two larger for the well-graded sand, but the ultimate tip resistance at greater depths was found to be approximately independent of the initial particle size distribution for all four tests. A micro mechanical explanation is also proposed for this observation.

* Lecturer, School of Civil Engineering, University of Nottingham, NG7 2RD, UK.
† Reader, Department of Engineering, University of Cambridge, CB2 1PZ, UK.

Keywords: pile, centrifuge, micro mechanics, fracture, particle size distribution

1. INTRODUCTION

Geotechnical engineering problems are usually analysed using soil parameters which have been measured in-situ or in the laboratory, and then applied in calculations as though they were always constant. However, soil parameters are strongly dependent on factors such as stress history and the type of pore fluid present, and the engineer therefore desires "reliable" parameters: those for which the physical origins are understood. In general, the micro mechanics of soil behaviour has remained a mystery to geotechnical engineers. Until recently [1], even the micro mechanics of plastic compression in soil has remained illusive. However, it is now well known that the constitutive behaviour of a soil is very dependent on the distribution of particle sizes present.

McDowell and Bolton [1] proposed that for the one-dimensional compression of an aggregate of uniform grains, the yield stress is proportional to the average grain tensile strength σ_o, and showed that beyond yield, a fractal distribution of particle sizes evolves under increasing stress, in agreement with available data. They proposed a mechanism of "clastic" hardening in soil, whereby the smallest grains continue to fracture under increasing stress levels, and protect the larger grains in the soil matrix, so that a proportion of the original grains remains. They derived for an initially uniform aggregate, an expression in terms of fundamental particle parameters, for the plastic compressibility index λ when voids ratio is plotted against the logarithm of effective stress. McDowell and Bolton [1] proposed that although the yield strengths of different uniformly graded sands should vary, the plastic compressibility at high stress levels might be a fundamental soil constant. They showed that if linear elastic fracture mechanics can be applied, and if a fractal distribution of particle sizes emerges, an energy equation predicts that the soil compressibility is a constant if the particle surface energy is a certain function of grain size. Their approach relates to the compression of an aggregate of initially uniform grains, but highlights the important role of the smallest grains in determining the behaviour of the aggregate. It would be anticipated that the yield strength of a well-graded sample would be far greater than a uniform sample of the same maximum particle size, but the values of plastic compressibility index at high stress levels might be equal. The yield strength and plastic compressibility of a given soil will certainly affect the response of a pile driven into it, and this paper explores the specific boundary value problem concerned with the influence of the initial particle size distribution in a crushable sand on the tip resistance of a model pile driven into it in a geotechnical centrifuge.

The question of the resistance of a driven pile, while important in itself, is made even more significant by the fact that penetrometers, which may be regarded as small diameter piles, are used to characterise ground profiles. In the case of sands, which are exceedingly difficult to core, engineers must rely almost exclusively on penetrometer probings. Many guides exist for the correlation of penetrometer data with fundamental soil properties, especially relative density: see Meigh [2] for example. It is invariably assumed that the penetration resistance of sands is related to some weighted average of the relative density of the sand around the advancing tip. We will show that this basic assumption is wrong.

The end bearing pressure q_b of a pile or penetrometer may be expressed in terms of the pre-existing effective vertical stress σ'_v and the bearing capacity factor N_q:

$$q_b = N_q \sigma'_v \qquad (1)$$

The most commonly used values of N_q are those quoted by Berezantzev et. al. [3] where N_q is a function of the angle of shearing resistance ϕ' and the depth/diameter ratio for the pile. However, at great depths it is known that considerable particle crushing occurs, and the angle of shearing resistance of the soil at failure decreases with stress level according to Bolton [4]:

$$\phi' = \phi'_{crit} + 3\left[I_D \ln\left(p_c/p'\right) - 1\right] \qquad (2)$$

where I_D is the relative density, p' is the mean effective stress at failure, p_c is a measure of the crushing strength of the soil grains, and ϕ'_{crit} is the critical state angle of friction for constant volume shearing. Lee [5] crushed single grains of sand diametrically between flat platens, and measured the tensile strength of a grain as the applied force at failure divided by the square of the particle diameter, following the work of Jaeger [6] on the tensile strength of rocks. Recently, McDowell and Bolton [1] showed that for triaxial tests on approximately uniformly graded aggregates, data by Lee [5] suggested that equation (2) would be better written

$$\phi' = \phi'_{crit} + 3\left[I_D \ln\left(B\sigma_o/p'\right) - 1\right] \qquad (3)$$

where σ_o is the average grain tensile strength, as measured by diametral compression between flat platens, and B is a scalar multiplier. Fleming et.al. [7] proposed an iterative method which used an average mean effective stress

$$p' \approx \sqrt{N_q}\sigma'_v \tag{4}$$

to determine the angle of friction in (3), and hence the corresponding Berezantzev [3] bearing capacity factor, which is then re-input to (4) etc. The definition of p' in (4) has been found to give observed values of N_q, when used in Bolton's correlation (3) to determine the angle of friction as a function of relative density and stress level. Thus the rate of increase of end bearing resistance with depth reduces as depth increases, due to the influence of soil compressibility induced by particle crushing. Vesic [8, 9] also accounted for the compressibility of the soil using a cavity expansion approach to determine the bearing capacity factor, but as yet the micro mechanics of soil behaviour during pile penetration has been poorly understood. This paper explores the micro mechanics of soil behaviour during pile penetration in attempt to explain pile tip resistance as a function of depth.

Centrifuge tests were conducted to determine the effect of the grain size distribution in dry calcareous sand on the tip resistance of a model pile. The sand used was Quiou sand from Brittany, France, for which extensive test data can be found in Golightly [10]. A 10 mm diameter pile with a 60° conical tip was advanced at 1 mm/s in a centrifugal acceleration field of 70g into dense Quiou sand. The main purpose of centrifuging is to recapture full-scale stresses in a geometrically similar small-scale model, tested with a correspondingly enhanced body force. From the perspective of dimensional analysis, similarity is established with a conceptual "prototype" in earth's gravity, for which every significant dimension is increased by the scaling factor. Thus a 10 mm diameter pile driven at 1 mm/s at 70g behaves like a 0.7 m diameter pile driven at 7 cm/s into the soil in earth's gravity.

Two alternative grain size distributions were used: the first soil contained particles smaller than 0.5 mm; the second sample was scalped of fines so that it contained particles of nominal size d such that 0.15 mm $< d <$ 0.5 mm. Bolton et. al. [11] showed that the for particle size effects to be insignificant in centrifuge tests on pile penetrometers, the diameter of the pile should be a factor of at least 20 greater than the mean particle size d_{50}, which is satisfied by the chosen geometry. The distance of travel of the pile was approximately 240 mm. Each experiment was performed twice. Following the experiments, the model pile was replaced by a hollow tube attached to a vacuum pump. The tube was then advanced into the soil in order to retrieve the particles that were adjacent to the pile during the centrifuge test. The distribution of particle sizes was then analysed. A micro mechanical explanation is proposed for the results of the experiments.

2. EXPERIMENTAL SET-UP

Four samples of soil were tested in total: two of each grading of the calcareous Quiou sand. The soil samples were placed in plastic tubs of internal diameter 190 mm and four plastic tubs were located in a 850 mm diameter tub which fits onto the Cambridge beam centrifuge. The operation of the Cambridge 8.25m diameter beam centrifuge is described in detail by Schofield [12]. The set-up of the four plastic tubs inside the 850 mm tub is shown in Figure 1. The use of four small plastic tubs enabled the testing of four samples of sand without having to remove the samples from the beam centrifuge, whilst using much less sand than if the whole 850 mm tub had been filled. The model pile used was of 10 mm diameter with a 60° conical tip.

Parkin and Lunne [13] studied the penetration of a model pile in calibration chamber tests. In such tests, a model pile is driven into soil subjected to a given vertical effective stress level by stresses applied at the top and bottom soil boundaries. The test therefore considers pile penetration at a given stress level only. Gui [14] showed that for calibration chamber tests, the presence of the top platen prevents surface heave of the soil at shallow penetrations. In addition, the calibration chamber test cannot easily predict pile penetration resistance as a function of depth for full-scale piles in the field. However, in the geotechnical centrifuge, the model correctly replicates the field case of initial stress increasing proportional to depth. Furthermore, the top free surface in the model permits soil heave as the model pile penetrates into the soil. Thus the centrifuge is seen to be a very powerful means of studying the penetration of full-scale piles driven into sand in the field.

Parkin and Lunne [13] suggested that for their calibration chamber tests, boundary effects were negligible for loose sand (relative density < 30%). However, they showed that the effect of the container walls becomes more pronounced as the relative density of the sand increases. They suggested that a chamber to pile diameter ratio of 50 was desirable in order to eliminate boundary effects. Gui [14] suggested that for centrifuge tests on silica Fontainbleau sand, the soil container boundary effects should be negligible if the diameter of the container is greater than that of the pile by a factor of 40 or more. However, a closer look at the data given by Gui [14] in Figure 2, which shows normalised tip resistance $Q = (q_c - \sigma_v)/\sigma_v$ as a function of the ratio of the container diameter D to the pile diameter B, reveals that a ratio of 20 should be acceptable at a relative density of 76%. Calcareous sand is considerably more compressible than silica sand, and the effect of the soil chamber boundary should therefore be even less than is suggested by Figure 2. Thus the use of 190 mm diameter plastic tubs with a 10 mm diameter pile was deemed acceptable for the dense samples tested here.

Figure 1 also shows the location of the linear actuator used to drive the model pile into the soil samples. The displacement of the actuator was measured by a linear potentiometer attached to the actuator, and the pile tip resistance was measured by a calibrated load cell in the pile penetrometer tip. The potentiometer and penetrometer were connected to a standard junction box on the 850 mm tub (Figure 1), and the output fed to Labtech Notebook in the Centrifuge Control Room via the beam centrifuge slip rings. The actuator was servo-controlled, and once the 850 mm tub had fully swung up, and a gravitational acceleration of 70g obtained, was enabled by switching on the servo-controller in the Centrifuge Control Room.

2.1 Sample Preparation

The soil samples were prepared at maximum relative density in the plastic tubs by vibration under a 5kg weight (i.e. a compaction stress of about 2 kPa). After compaction, the height of each soil sample was approximately 260 mm. The final densities are given in Table 1 for each of the four tested soil samples, together with the appropriate gradings. Table 2 shows the maximum and minimum densities and the specific gravity for the two gradings of sand, as determined by British Standard methods [15]. The maximum density was obtained by vibratory compaction in a standard 1 litre compaction mould. Comparing Tables 1 and 2, it can be seen that maximum density was achieved for all four centrifuge tests. The initial particle size distributions for the two soils are given in Figure 3.

2.2 Experimental procedure

Following the swing-up of the 850 mm tub on the beam centrifuge, the acceleration field was increased to 70g. The servo-controlled actuator was then driven at 1mm/s into the centre of one of the soil samples in a 190 mm diameter plastic tub. The initial height of the cone was set to 275 mm above the base of the plastic tub, and the travel limited to 235 mm. This ensured that the pile penetration stopped at a distance of 40 mm above the base of the tub. The model pile tip load cell data and displacement were measured as a function of time and recorded using Labtech Notebook for subsequent input to Microsoft Excel. Once the full penetration depth was reached, the pile was withdrawn, the centrifuge stopped, and the plastic tubs switched to test the next soil sample.

Following the completion of all four tests, an effort was made to retrieve particles from the centrifuged samples for particle size analysis. The model pile was replaced by a hollow tube of outer diameter 10 mm and inner diameter 8mm, and attached to a sand trap connected to a vacuum pump. This is shown schematically in Figure 4. The tube was then advanced into the soil, under vacuum, in order to retrieve the particles which were adjacent to the pile during the centrifuge test. Samples were successfully retrieved for tests 1 and 2 and the final particle size distributions subsequently determined and compared with the initial grain size distributions.

3. RESULTS

The tip resistance q_c for the model pile was determined by dividing the load measured by the load cell in the conical tip by the end cross-sectional area. The values of q_c as a function of depth are given in Figure 5 for each of the four tests. In each case the tip resistance reached a maximum and then fell as penetration continued. It is not clear whether the skin friction on the pile would also drop; only the end resistance was measured in these tests. Nevertheless it seems likely from the severe reduction in resistance after the maximum end bearing force was reached, that this would result in a peak in the total axial load in these displacement-controlled tests, which would coincide with a point of instability if the tests had been load-controlled.

4. ANALYSIS OF RESULTS

Following the withdrawal of the pile during flight, the hole created by the pile collapsed. Thus particles at the centres of the plastic tubs were those particles adjacent to the pile during flight. It was therefore necessary to retrieve these particles using the hollow tube vacuum method described. Based on the density of sample 1, ρ = 1656 kg/m^3, it was anticipated that the tube might create a 12 mm diameter cylindrical hole in the soil, thus corresponding to a retrieved mass of sand of 10 g for every 60 mm travel of the tube. For each of the four plastic tubs, particles were first retrieved from the soil at depths less than the depth of instability. i.e. the tube was advanced into the soil until the depth corresponding to peak tip resistance in Figure 5 was reached. The retrieved sand was then removed from the sand-trap (see Figure 4) and the tube advanced until the bottom of the tube was 10 mm from the base of the tub. The masses retrieved from samples 1 and 2 were found to be consistent with the expected 10g/60mm travel of the tube. Figure 6 shows the hole created by the advancing tube: following withdrawal of the tube, the sand was "locked": at the centre of the sample, the soil experiences vertical effective stress, but no horizontal effective stress. For the more uniformly sized soils (tests 3 and 4), no reliable data was achieved: on retrieving grains from sample 4, the tube blocked, whilst on retrieving particles from sample 3, grains were too easily retrieved by the vacuum pump, so that large samples were obtained, providing inconclusive data. The particles retrieved from samples 1 and 2 did, however, show a consistent trend. Figure 7(a) shows the particle size distribution for sample 1 at shallow depths (i.e. before the depth of instability) together with the grading for depths greater than the instability depth. The initial particle size distribution is also shown. Figure 7(b) shows the results for sample 2. The figures also show the total masses of the sieved samples, which can be seen to be consistent with the expected 10g per 60mm travel of the vacuum tube. It is clear that there has been insignificant particle crushing in the soil retrieved from depths less than the instability depth, corresponding to the soil which was adjacent to the pile at shallow depths during flight. However, significant particle fracture has occurred in the soil retrieved from depths greater than this.

4.1 Ambient mean effective stress

According to Fleming et. al. [7], the average mean effective stress p' around the tip of a pile can be calculated according to equation (4). This definition of p' has been found to give observed values of N_q, when used in Bolton's correlation (3) to determine the angle of friction as a function of relative density and stress level.

Values of p' around the pile tip were calculated for each of the four tests as a function of depth, using equation (2). The results are given in Figure 8.

5. DISCUSSION

It can be seen from Figures 5 and 8 that the results of the pile penetration test are certainly repeatable. However, the peak in tip resistance is surprising since the end bearing resistance of piles is normally found to increase monotonically with depth [7]. Tables 1 and 2 show that the sand had been compacted to its maximum density, so there can have been no loose pockets to explain the drop from peak resistance. Parkin and Lunne [13] noted that data by Last [16] for pile tip resistance as a function of depth in calibration chamber tests did exhibit a peak value. They attributed this to wall friction mobilised on the rigid side walls of the calibration chamber. Wall friction around the test cylinder would have had the effect of reducing vertical effective stresses in the soil, which could reduce penetration resistance. However, even if wall friction causes a reduction in vertical stress in the soil, it is expected that vertical stress would still be an increasing function of depth, so one might anticipate q_c to nevertheless increase monotonically with depth. Any sudden reduction in stresses around the pile tip would tend to reduce horizontal stresses acting on the chamber wall, thus reducing the wall friction. It seems unlikely, therefore that the sustained drop in pile tip resistance could be caused by wall friction. Furthermore, existing evidence [14, 17] suggests that a chamber / pile diameter ratio of 19 should have been adequate to remove boundary effects even in quartz sands – and crushable sands should be even less affected by the chamber since soil expansion will be reduced.

In practice, penetration probings are used to search for, and characterise, "loose spots" within otherwise dense strata. Moreover, piles are generally used in large groups, typically at spacings of only 2 to 4 diameters. In each of these cases the overall geometry around each penetration, taking symmetry in account, resembles the scenario used in all centrifuge or chamber tests of a pile entering a confined zone. Even if the unusual result is due to chamber effects, therefore, its practical significance is not diminished.

If chamber effects are not considered a good enough explanation, micro-mechanical hypotheses might be advanced for the apparent instability. Figure 7 shows insignificant crushing of the soil retrieved from depths shallower than the instability, but observable crushing of the soil retrieved from greater depths. Figure 9 shows the results of simple high-pressure oedometer tests on the two initial gradings of dry soil. It can be seen that the well-graded sample does not show such a well-defined yield region as the more uniform soil, but both apparently are compressing plastically in the vicinity of $\sigma'_v = 2$ MPa. Visually, the soil beyond this point appeared to be both crushed and "locked" on removal from the oedometer; see Figure 10. The soil compact could hold its shape, just as the cylindrical hole left by an extracted pile remained open if the self-weight of the soil was not too great. In both cases, however, the over-consolidated sand could be crumbled to a powder in the fingers. Figure 11 shows a scanning electron micrograph of soil retrieved from below the depth of instability for test 2: it can be seen that the larger particles seem to be coated with many small particles. This is evident to a much lesser extent in Figure 12, which shows a micrograph for soil retrieved from sample 3. The micrograph in Figure 11 for the well-graded soil resembles those often obtained for clays [18], and this might suggest the presence of electrostatic forces between particles. Nevertheless, such a microstructure permits a high degree of particle interlocking, which is observed in the hole produced by the vacuum tube following the centrifuge tests (Figure 6) and in the soil observed following the oedometer tests (Figure 10).

For sand which is being compressed one-dimensionally in an oedometer, we can use Jaky's formula [19]

$$K_o = \sigma'_h / \sigma'_v \approx 1 - \sin\phi' \tag{5}$$

to deduce a typical value of the coefficient of earth pressure at rest K_o. Muir Wood [20] states that for dense sands, the value of ϕ' in (5) should relate to the initial structure of the sand. i.e. the peak angle of friction ϕ'_{max} should be used. If we accept a $\phi' \approx 45^\circ$ (i.e. $K_o \approx 0.3$) for the compressing sand (consistent with peak angles of friction observed by Golightly [10] in triaxial tests on dense well-graded medium to coarse Quiou sand at low stress levels), we find that $\sigma'_v = 3$ MPa in oedometer tests may induce $\sigma'_h = 0.9$ MPa giving $p' \approx 1.5$ MPa, which also corresponds approximately with the plateau of average p' found in Figure 8. It seems as though the plateau corresponds with a stress level at which significant crushing and compression would take place all around the penetrometer tip. If soil is compressing as it shears it must mobilise a stress ratio inferior to the critical state strength. Suppose we use the maximum shear stress at clastic yield in the oedometer $\tau_{max} = (3-0.9)/2 \approx 1$ MPa, to calculate the penetration resistance as though every element of soil mobilised the same stress. Then, as with the resistance of piles in clay [7], $q_c \approx 9\tau_{max} \approx 9$ MPa. It can be seen that this qualitative argument produces a result commensurate with the observed ultimate resistance at depth. The slight increase in p' at the end of tests 2, 3 and 4 (consistent with the observed values of q_c in Figure 5) is most likely due to the presence of the base of the plastic tub. Gui [14] showed that for a sand at maximum relative density, a base boundary effect would be detected when the pile tip reached a depth 10 pile diameters from the base boundary. Thus, in the centrifuge tests conducted here, with samples of approximate depth 260mm, it would be expected that the base boundary effect would be detected at a depth of about 160mm. This is entirely consistent with the results in Figures 5 and 8.

Although the argument above may explain the plateau in pile tip resistance at great depths, it hardly seems to justify the peak tip resistance of about 20 MPa for the well-graded soil tests at shallower penetrations. The explanation must lie with the nature of non-uniform soil properties and stress distributions created by the process of penetration. In other words the mechanism of penetration in the pre-peak profile must be different from that in the post-peak profile. It may be that the influence of friction from the pile-shaft creates a wider zone of vertical compression at greater depths, which might eliminate expansion completely so that the pile continually enters an "imploding" soil zone. Alternatively, the pile may initially create an intensely compressed soil plug ahead of the pile tip. This would effectively make the pile longer and enhance its resistance, until the plug eventually shears or fractures, leading to a rapid drop in tip resistance. This would also be consistent with the difference in the particle size distributions for the soil retrieved from above and below the depth of instability following the centrifuge tests. If an intensely compressed plug is created by the crushing and compaction of soil grains and driven ahead of the pile tip at shallow depths, the soil adjacent to the pile at shallow depths should suffer little crushing. This was observed in the retrieved samples. If significant crushing then occurs all around the pile tip beyond the depth of instability, it would be anticipated that the soil adjacent to the pile at great depths would show evidence of grain fracture. This was found to be true, as shown in Figure 7. It seems therefore, that the soil adjacent to the pile at shallow depths suffers little crushing as soil is crushed and driven ahead of the pile. At the depth of instability, crushing occurs all around the pile tip, and this was also shown to be consistent with the observed plateau in tip resistance.

Further work is being carried out to observe deformation mechanisms around advancing piles, and to perform numerical simulations with realistic penetration sequences and life-like soil constitutive relations. This work clearly has implications for CPT interpretations and pile design in compressible soils.

6. CONCLUSIONS

Centrifuge tests were performed to establish the tip resistance of a model pile driven into calcareous Quiou sand. In each of four tests, a peak tip resistance was observed, the value of which was about 20 MPa for the well-graded soil and 10 MPa for the uniform soil. The peak resistance is therefore a strong function of the initial particle size distribution, which has also been shown by oedometer tests to affect the yield strength and compressibility of the soil. Particle size analysis for the well-graded soil showed no evidence of crushed grains for particles retrieved above the depth of peak resistance. However crushing was observed below that depth. Although shaft friction was not measured, the sharp drop in tip resistance following the peak value would imply that the peak in tip resistance would give rise to a point of instability in load-controlled tests.

No simple explanation of the observed peak can be offered. Hypotheses must be based on different shallow and deep penetration mechanisms, influenced by particle crushing and soil compressibility. Conventional explanations of aberrantly low penetration resistance focus attention on the role of chamber boundary friction reducing the pre-existing vertical stresses at depth. However, some additional argument is necessary to explain a *fall* in resistance with depth. Possible candidates include the influence of pile shaft friction eliminating volumetric expansion completely, so that the pile enters an imploding soil zone, or alternatively, the creation of an intensely compressed plug of soil ahead of the pile tip, which eventually fractures. Beyond the peak in tip resistance, the average mean effective stress p' is approximately constant at about 1.5 MPa, and is approximately independent of the initial particle size distribution. This value of p' is consistent with the stress level required in oedometer tests to cause significant crushing and compression. The observed ultimate tip resistance was observed to be about 9 MPa in each of the four tests, and this has been shown to be consistent with the end bearing resistance calculated on the assumption that every element of soil mobilises the same shear stress.

Whichever of these arguments is preferred, the qualitative conclusion remains the same. The penetration resistance of sands depends not only on the initial condition (e.g. relative density and pre-existing stress level) of the sand being penetrated, but also on a complex function of penetration geometry and soil compressibility.

REFERENCES

1. G.R. McDowell & M.D. Bolton, Géotechnique 48 (1998) 667.
2. A.C. Meigh, Cone Penetration Testing methods and interpretation, Ciria (1987).
3. V.C. Berezantzev, V. Khristoforov & V. Gulubkov, Proc. 5th Int. Conf. on Soil Mech. and Found. Eng. 2 (1961) 11.
4. M.D. Bolton, Géotechnique 36 (1986) 65.
5. D.M. Lee, Ph.D. dissertation, University of Cambridge (1992).
6. J.C. Jaeger, Int. J. Rock. Min. Sci. 4 (1967) 219.
7. W.G.K. Fleming, A.J. Weltman, M.F. Randolph, and W.K. Elson, Piling Engineering. Blackie (1992).
8. A.S. Vesic, J. of the Soil Mechanics and Foundations Division ASCE 9 (1972) 265.
9. A.S. Vesic, National Co-operative Highway Research program 42 (1977).
10. C.R. Golightly, Ph.D. dissertation, Bradford University (1990).
11. M.D. Bolton, M.W. Gui, & R. Philips, 11th South East Asia Geotechnical Conf. Singapore (1993) 85.
12. A.N. Schofield, Géotechnique 30 (1980) 227.
13. A.K. Parkin & T. Lunne, 2nd Symposium on Penetration Testing, Amsterdam (1982) 761.
14. M.W. Gui, Ph.D. dissertation, University of Cambridge (1995).
15. BS 1377, Methods of test for soils for civil engineering purposes. British Standards Institution (1991).
16. N. Last, NGI Report No. 52108-8 (1979).

17. M.W. Gui, M.D. Bolton, J. Garnier, J.F. Corte, G. Bagge, J. Laue & R. Renzi, Centrifuge '98, Tokyo 1 (1998) 155.

18. P. Smart & N.K. Tovey, Electron microscopy of soils and sediments: examples. Clarendon Press, Oxford (1981).

19. Jâky, Magyar Mérnök és Epitész-Egylet Közlönye [J. Union Hungarian Engrs Architects (1944) 355.

20. D. Muir Wood, Soil Behaviour and Critical State Soil Mechanics. Cambridge University Press (1990).

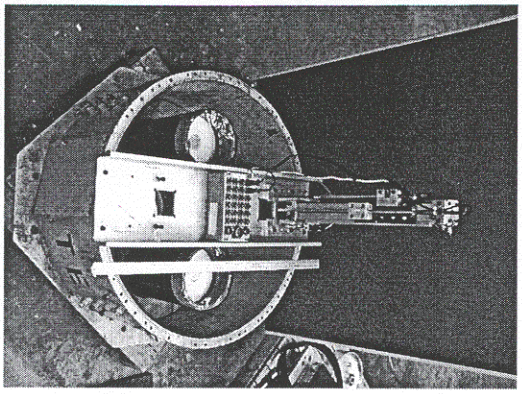


Figure 1. 850 mm diameter steel tub containing four 190 mm diameter plastic tubs.

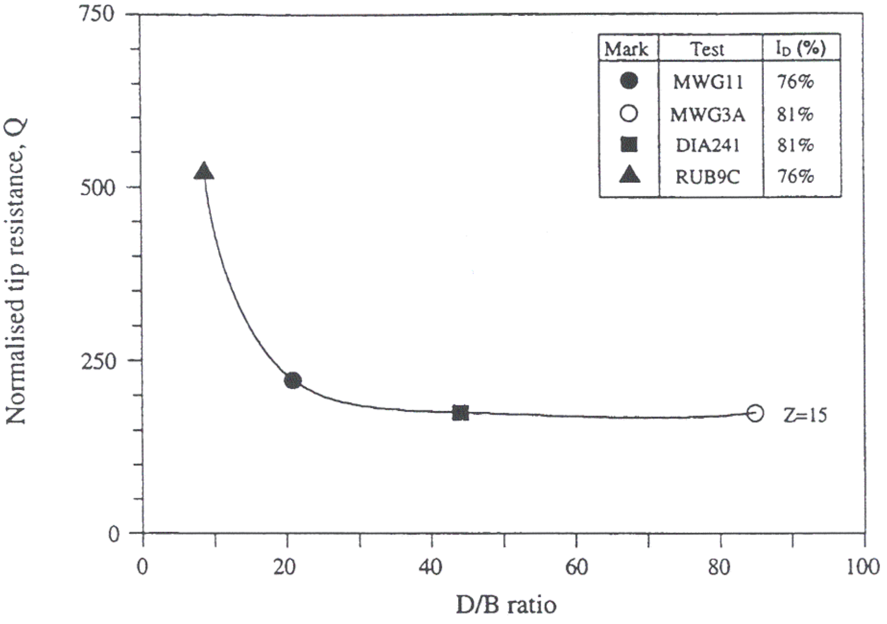


Figure 2. Effects of diameter ratio D/B on normalised tip resistance (Gui [14]).

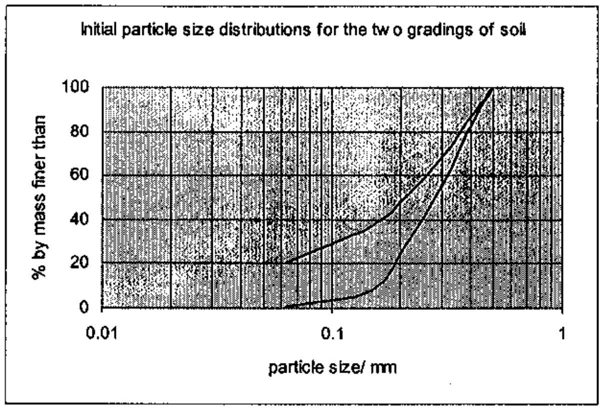


Figure 3. Initial grading curves for tested Quiou sand.

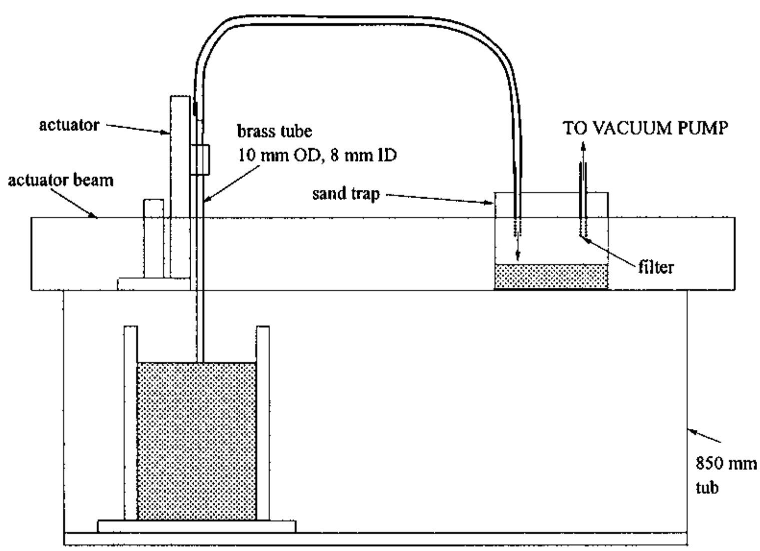


Figure 4. Vacuum method of retrieving particles located adjacent to the pile during the centrifuge test.

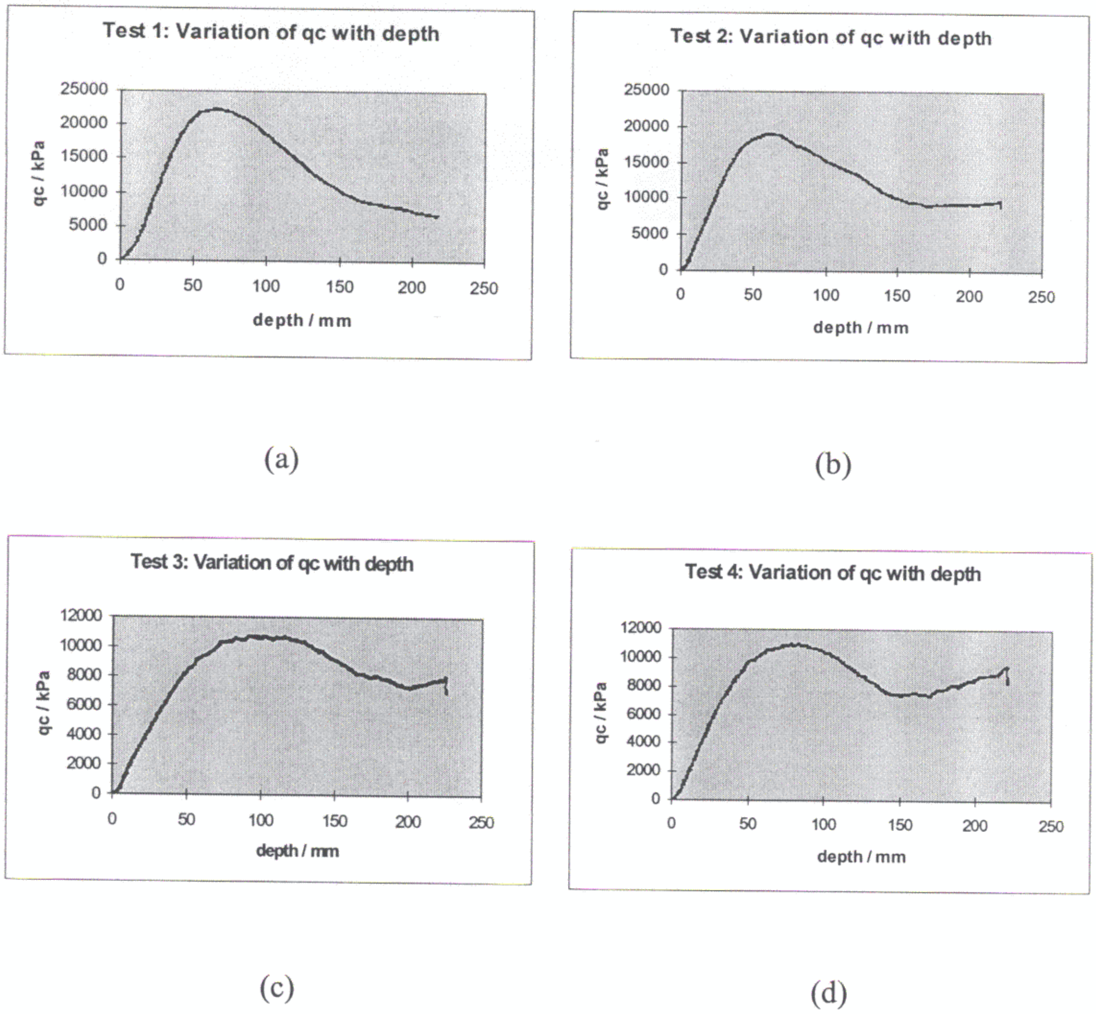


Figure 5. Plots of tip resistance as a function of depth for each of the soil samples.

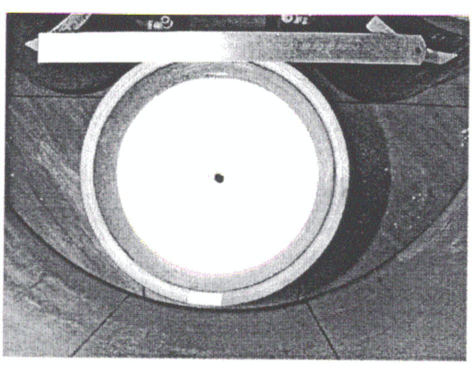
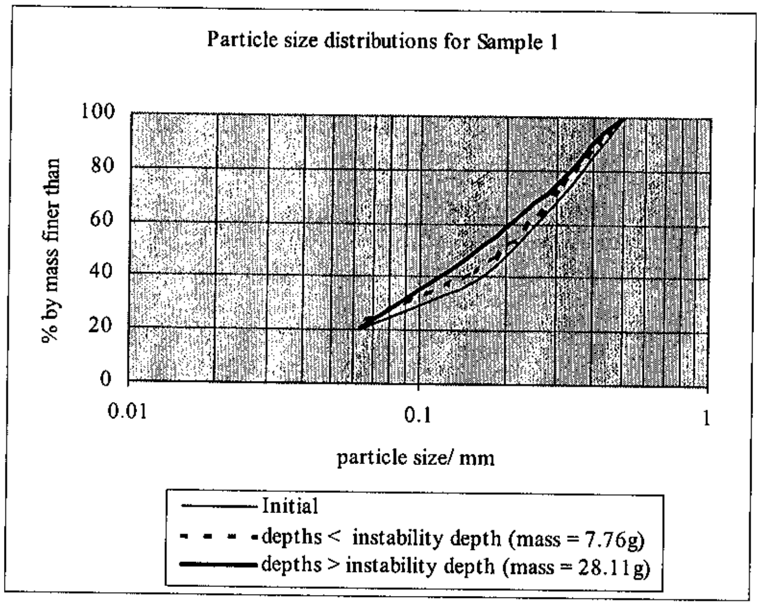
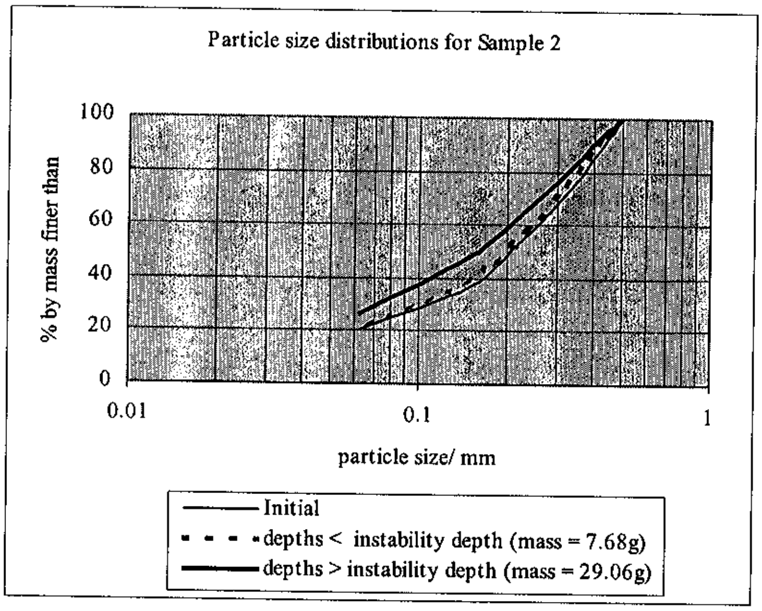


Figure 6. Hole created by advancing sampling tube.

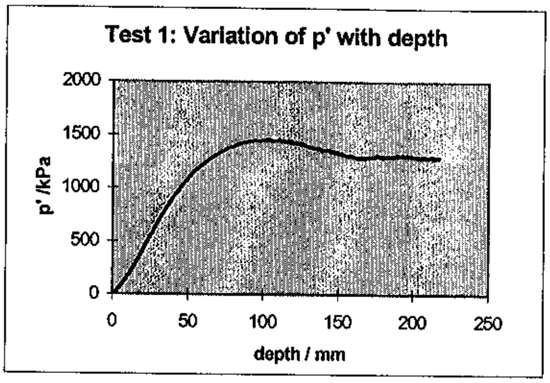


(a)

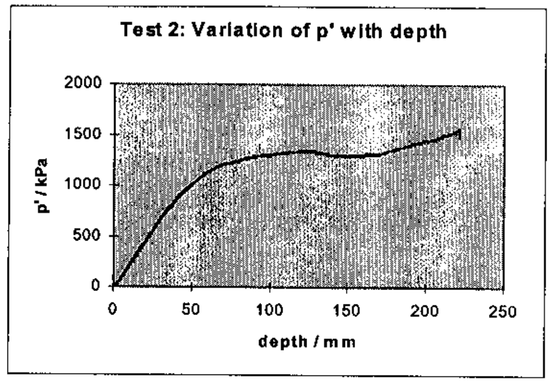


(b)

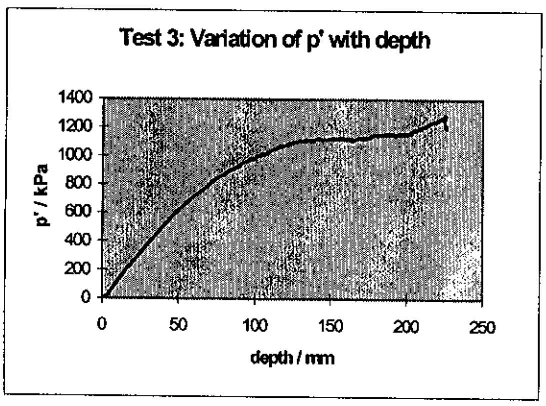
Figure 7. Particle size distributions for samples 1 and 2 at depths less than and greater than the instability depth.



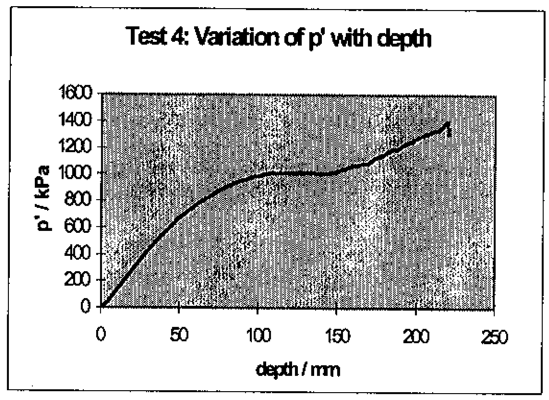
(a)



(b)



(c)



(d)

Figure 8. Mean effective stress at pile tip as a function of depth.

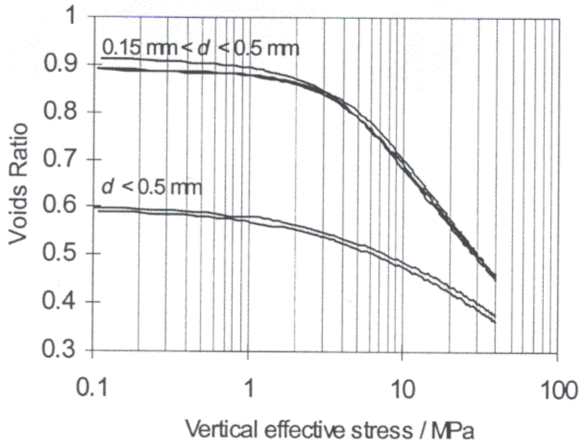


Figure 9. Results of oedometer tests on the uniformly-graded and well-graded soils.

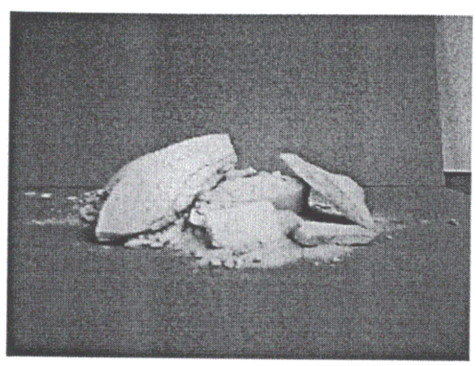


Figure 10. Blocks of locked sand after oedometer test.

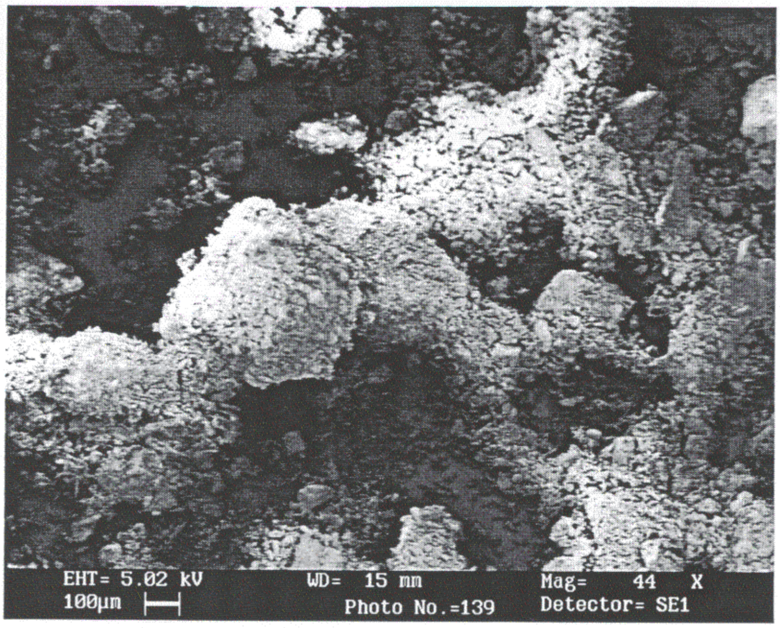


Figure 11. Scanning electron micrograph of sand retrieved from below the instability depth after test 2, showing clay-like microstructure.

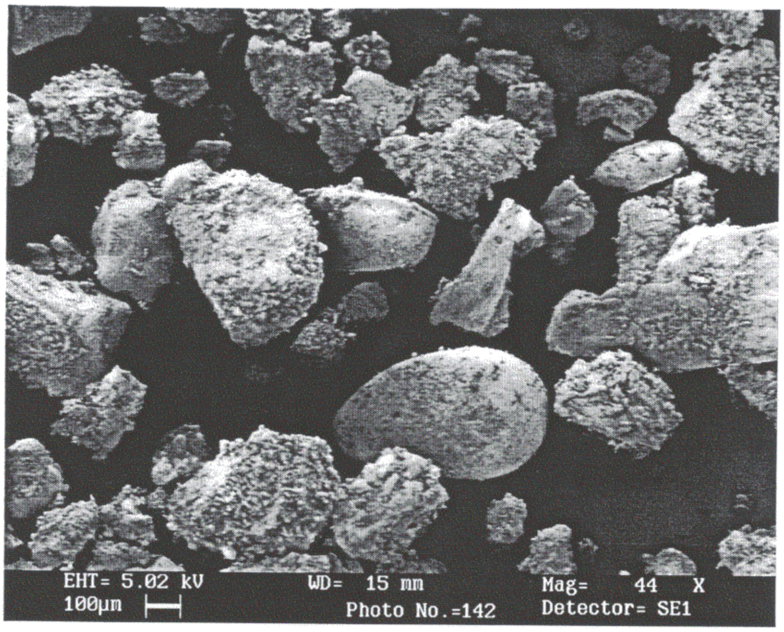


Figure 12. Scanning electron micrograph of soil retrieved from sample 3.

	Test 1	Test 2	Test 3	Test 4
Soil grading	$d < 500$ µm	$d < 500$ µm	150 µm < d < 500 µm	150 µm < d < 500 µm
Density after compaction / kg/m^3	1645	1615	1335	1364

Table 1. Densities of the sand specimens after compaction.

Soil grading	$d < 500$ µm	150 µm < d < 500 µm
Maximum density / kg/m^3	1656	1332
Minimum density / kg/m^3	1199	1052
Specific gravity	2.72	2.72

Table 2. Properties of the two gradings of Quiou sand.