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The Effect of Layering on Cone Resistance: Calibration Chamber Tests.

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Summary: The analytical study of Vreugdenhil quantified the long-recognised effect of adjacent soil layers of different stiffness on cone resistance, q_c. The solution, obtained under the large assumption of linear elastic behaviour was well supported by the three sets of experimental results found in the literature. However, it was clear that further experiments were required before the elastic results could be adopted with confidence in practice. This paper reports a set of calibration chamber experiments carried out with a mini-cone (1 cm^2). Four tests, all on three-layer systems of sand, gave experimental response within 10 percent of the elastic solution when the thin included layer was denser than the surrounding thick layers and 20 percent when the thin layer was looser. These results suggest that after further verification on a wider range of soils and soil states, the simple elastic solution should be able to be employed usefully in practice. They also provide a reference for the verification of more complete analytical solutions. Furthermore, the good agreement between the elastic solution and the recorded penetration resistance implies that the far-field elastic response has an important influence on the resistance offered to the cone by the immediate zone of plastic deformation. Presumably, this is due to the influence of the confining stresses imparted by the elastic region to the plastic zone.

INTRODUCTION

It has long been known (Cacot and Kerisel, 1966; Sanglerat, 1972; Meigh, 1987, Lunne et al, 1997) that as a cone penetrometer enters a new layer, the cone resistance q_c does not immediately jump to the characteristic value for that layer. Instead, the penetrometer must travel some distance into the layer for q_c to reach a steady-state value. The problem is illustrated by the solid line in Figure 1, showing the results of a penetration test carried out in a calibration chamber containing two uniform layers of sand separated by a sharp, plane boundary.

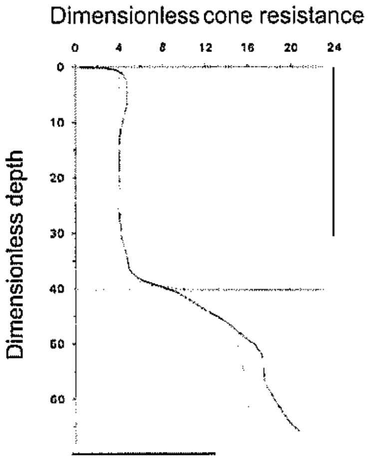

Figure 1. Cone resistance q_c from a calibration chamber test on two homogeneous layers of sand (Canou, 1989). Dotted line shows prediction by Vreugdenhil *et al.* (1994)

While the cone is still in the loose upper layer, it begins to sense the dense layer some distance ahead of the boundary. Once in the dense layer, a constant cone resistance is not reached until the penetrometer has traveled several cone diameters into that layer. For thick layers such as these, a steady-state value of q_c is attained, which characterises the state of the layer. However, a difficulty arises with layers that are too thin for the *characteristic* *value* of q_c to be reached. When the thin layer is denser than the surrounding layers, the characteristic value of q_c may not be attained. Similarly, in a thin loose layer q_c may be overestimated. This effect has serious implications for stability analyses based on cone resistance. Previous work, described by the authors cited above, generally focused on methods for finding the bearing capacity of piles, where tip resistance is estimated from an average of the strength of the bearing layer and its neighbours. While the problem of estimating the characteristic value of q_c of the thin layer itself has been recognised implicitly in previous work, it does not seem to have been addressed explicitly. However, knowing the characteristic value is of prime importance in many problems, especially those that depend on the weakest link in the soil system, such as slope stability and liquefaction analyses.

In an attempt to quantify the layering effect more precisely, Vreugdenhil et al. (1994) made a simple elastic analysis of cone resistance in multi-layered soil systems. Although the analysis was an elastic one, it reproduced test results remarkably well. Vreugdenhil's prediction for the two-layer calibration chamber result is shown by the dotted line in Figure 1. Although the prediction reproduces the general form of the test trace well, it does not follow it very closely near the interface; therefore, it may not model thin layers well. At that time, no experimental data was available for thin layers and the method could not be tested further. Subsequently, Joer *et al.* (1995) carried out tests in three-layer systems of artificially cemented calcareous sand using 3 and 10 mm model penetrometers. They found that although the elastic analysis did not agree exactly with the test results, it did capture the main effects of layering well. The aim of the present work is to test three-layer systems of quartz sand, with the broader objective of examining the elastic analysis as a means of correcting cone resistance measured in thin strata.

A set of four tests was carried out in the small calibration chamber of the CERMES group at the Ecole des Ponts et Chaussees (ENPC), Paris, using a miniature cone of 11.3 mm diameter (1.0 cm^2). Fontainebleau sand was used in all tests, and layers were formed by varying the density of the soil. Thickness of the included layer varied from 4 to 9 cone diameters.

APPARATUS AND PREPARATION OF SPECIMENS

The Fontainebleau sand used in the tests is a naturally-occurring fine quartz sand with a D_{50} of about 0.1mm. All tests were carried out on *dry* sand, deposited by pluviation in air. Figure *2* shows the pluviation apparatus in operation. Dry sand falls through a grill at the base of the hopper onto a travelling diffuser, which is lifted by a motorised winch at a pre-selected speed chosen to maintain a constant height of deposition of 100 mm. The rate of deposition was controlled by changing the grill at the base of the hopper. This, in turn, controlled the density of the deposited sand. Layers were formed by overfilling and then gently planing back the surface using a vacuum cleaner nozzle held at a fixed height. Trials with various combinations of hole size and spacing in the hopper grill gave the relationship between grill configuration, deposition rate and density shown in Table 1.

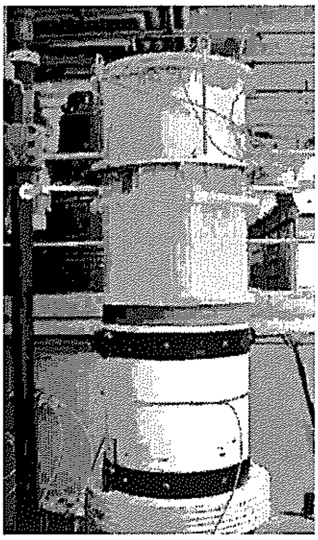

Figure *2*. The CERMES sand raining apparatus in operation. The diffuser, out of sight in the specimen mould, is being lifted at the rate of deposition by the cable going off to the left of the photograph.

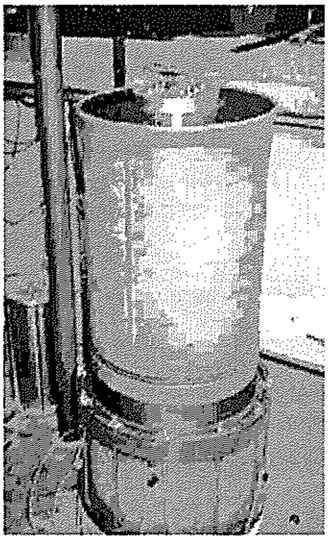

Figure 3. A completed specimen, supported by vacuum, resting on the pressure-controlled bottom platen of the chamber. The rubber membrane and the rigid top platen can be seen.

Table 1. Relation between properties of the hopper grill and density of dry pluviated Fontainbleau sand.

Test No.	Grill Properties (on diagonal grid)	Flow Rate (mmls)	Density (t/m^3)	Void Ratio,
1a	48 mm at 20 mm c/c	16.8	1.547	0.713
1b	Ditto	16.8	1.544	0.716
2	43 mm at 40 mm c/c	1.18	1.69	0.566
3	ϕ5 mm at 40 mm c/c	3.90	1.67	0.590
4	43 mm at 20 mm c/c	7.71	1.60	0.659

The closeness of the densities obtained in tests 1a and 1b suggest that the raining process is repeatable. We chose to use the soil states 1 and 2 for the subsequent tests on layered systems, to have as large a contrast as possible between layers.

Calibration chambers are, in effect, large triaxial cells. The CERMES cell takes a specimen 520mm in diameter by 700 mm (Figure 3). Axial load is applied by a hydraulically-driven steel piston forming the lower platen, the top platen being fixed. Radial confining pressure is applied through rubber membranes. A homogeneous confining pressure of 100kPa was used for all tests.

The 1.0cm^2 miniature cone penetrometer used in the tests was made by FUGRO and is a scaled down version of the standard 10cm^2 CPT cone. It is described further by Canou (1989). It was driven into the specimen at 5mm/s by a hydraulic ram. (Canou has shown that in *dry* sand q_c is independent of penetration rate.) Point resistance q_c and displacement, measured by LVDT, were sampled by a Hewlett Packard data acquisition system.

RESULTS FROM THE CALIBRATION CHAMBER

Four three-layer systems were tested, three with a thin dense layer between two thick loose layers, and one with a loose layer between dense layers. Three further tests were made on single- or two-layered specimens to establish characteristic values of q,. Configuration of the test specimens is given in Table 2.

Table 2. Configuration of the test specimens.

Test No.	Thickness of central layer, h (mm)	Dimension-less layer thickness, hld	Depth to interface(s) (mm)	Soil configuration (loose = soil1; dense = soil 2; medium = soil 4, in Table 1)		
				Layer 1	Layer 2	Layer 3
3	Two layers		349	Medium	dense	
4	104	9.2	312, 416	Loose	dense	loose
5	69	6.1	345.414	Loose	dense	loose
7	41	3.6	347, 388	Loose	dense	loose
8	Single layer			loose		
9	Single layer			dense		

Results from one of the thick-layer tests (No. 8) are shown in Figure 4 and those from the three-layer tests in Figure 5. It is clear from Figure 5 that layer thickness generally has an inverse effect on the observed peak value of q, , as predicted by Vreugdenhil *et al.* (1994). To see how well the experimental curves are reproduced by the Vreugdenhil model requires characteristic values of q, for the loose and dense sand layers.

Characteristic values

We should be able to read characteristic values for each density-class of soil from the thick-layer regions of the various tests. This can be done for the lower half of the specimens, where steady values of q_c are attained. However, as exemplified by test **8** in Figure 4, in the upper half of the specimen q_c does not reach a steady state. Rather, after an initial rapid rise in the frst 50 mm of penetration, it increases slowly and levels off only at a depth of 350 to 400 mm. This is believed to be caused by an inhomogeneous confining stress field in the upper part of the specimen due to cavities that were built into the top platen of the cell, for previous tests on groups of model piles. The central 30mm diameter hole, used to admit the penetrometer, is surrounded by 12 similar holes, within a radius of 100 mm. Although these holes were plugged, the lower face of the steel plugs stood several millimetres above the face of the platen. A sheet of 3mm thick fibreboard was glued to the platen to cover these holes. However, subsequent tests have shown that it was not stiff enough to prevent some reduction of confining stress in the upper central zone of the specimens.

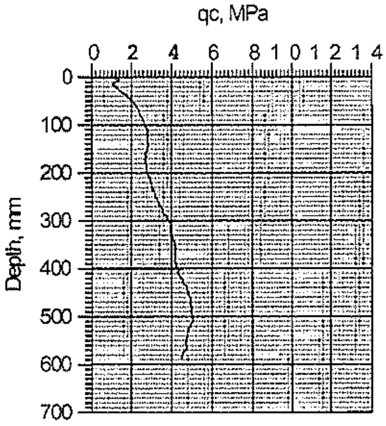

Figure 4. Test No 8: Uniformly deposited, Loose Sand.

A further complication arises from an error in the procedure used in the first two tests. Generally, an isotropic confining pressure of 100 kPa was used in all tests and it was intended to place the soils in a normally-consolidated state. However, in tests 3 and 4, the soil was overconsolidated by inadvertently retaining the full vacuum (of about 90 Wa, used to support the specimen during assembly of the cell) while the cell pressure was applied. Hence, these specimens were overconsolidated during cell set-up. However, they were tested at 100 kPa. In subsequent tests, the vacuum was reduced as the cell pressure was applied, so that the total lateral confining pressure did not exceed the final value of 100 Wa. Therefore, we require characteristic values of q, for the loose and dense sands, under both normally-consolidated and over-consolidated conditions.

Characteristic values of q_c for the four soil states are indicated by the seven tests listed in Table 3. These values were read from the lower half of the q_c traces, where uniform cone resistance was attained. Their mean values are taken as the characteristic values of q_c for the thin layers, in calculating the analytical cone response by the elastic model, plotted in Figure 5.

Test No.	Characteristic q_c (MPa), for soil type:			
	Loose, n.c.	Loose, o.c.	Dense, n.c.	Dense, o.c.
5	5.2			
6			10.2	
7	5.0			
8	(4.0)-5.0			
9			10.9	
Mean	5.1	5.1	10.6	12.1

COMPARISON OF TEST RESULTS WITH ELASTIC MODEL

From Figure 5, we see that while the elastic model somewhat underestimates the extreme value in the thin layer, by about 10 to 20 percent, it does capture the general features of the actual cone response. We note also that for the case of a dense included layer, it substantially underestimates the width of the peak.

Table 4 summarises the observed peak values, denoted by q_c^*, together with peak values predicted by the elastic model, as well as the corresponding characteristic values. For the case of a dense thin layer, the elastic analysis underestimates the observed peak by 6 to 10 %. For the case of the loose included layer, the analysis again underestimates q_c, but in this case, the difference is much larger, at 22 percent, with the analytical value closer to the characteristic one.

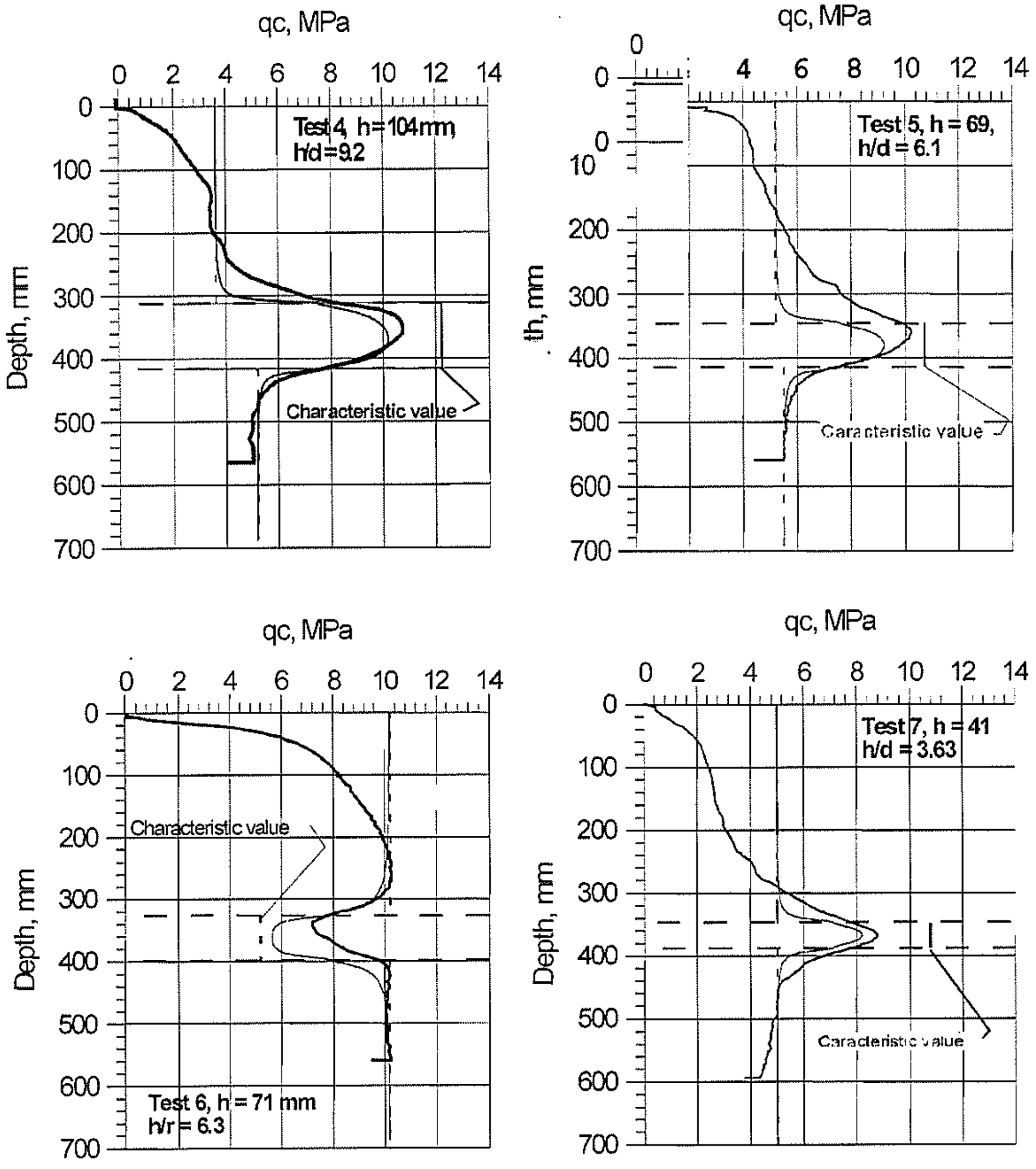

Figure 5. Test Results (irregular, heavy lines) compared with Vreugdenhil's solution (smooth, thin lines).

Table 4. Comparison of peak values predicted by the elastic model with experimental ones.

Test no.	**h/d**	**Characteristic q_c (MPa)**	**Observed peak, q_c^* (MPa)**	**Predicted peak q_c, (MPa)**	**Difference,%**
4	9.2	12.2	10.8	10.2	-5.6%
5	6.1	10.7	10.2	9.21	-9.7%
6	6.3	5.2	7.6	5.95	21.7%
7	3.6	10.7	8.8	8.21	-6.7%

CONCLUSIONS

1. In general, although the penetration problem clearly involves plastic deformation, the elastic solution captures the general features of the layering effect well. This suggests that the stress state imposed on the zone of plastic deformation immediately around the cone by the surrounding elastic region is of paramount importance.
2. For a *dense*, thin included layer, all tests show a reduction in q_c that is close to but less than that predicted by the elastic analysis; in all three tests the experimental value was within 10% of the analytical one.
3. In the single test made with a *loose* thin layer, the difference between the experimental and analytical results was greater, at 22%.
4. Clearly, further proving on a wider range of soil types, densities and geometries is required before Vreugdenhil's analytical method could be used routinely and with confidence. However, the fairly consistent results for the dense-layer case, together with the generally similar good agreement found by Joer *et al.*(1995) with a quite different, calcareous, sand, suggest that Vreugedenhil's solution has promise as the basis for a correction procedure. It does appear that it would be worthwhile to undertake additional testing.

REFERENCES

J. Canou (1989), "Piezocone et liquefaction des sables: rapport de synthese des travaux realises au CERMES (1986-1988)". *Research Report*, CERMES/ENPC, Paris, 176 pp.

H.A. Joer, M.F Randolph and Y.H. Liew (1995), 'Interpretation of Cone resistance in Layered Soils', *Proceedings. 7th ANZ Conference on Geomechanics*, Adelaide, pp 92-101.

T. Lunne, P.K. Robertson and J.J.M. Powell (1997), *Cone Penetration Testing in Geotechnical Practice*. Blackie.

A.C. Meigh (1987), *Cone Penetration Testing : Methods and Interpretation*. Butterworths.

Guy Sanglerat (1972), *The Penetrometer and Soil Exploration : Interpretation of Penetration Diagrams - Theory and Practice*. Elsevier.

R.A. Vreugdenhil, R.O. Davis and J.B. Berrill (1994) "Interpretation of Cone Penetration Results in Multilayered Soils", *International Journal Numerical and Analytical Methods in Geomechics*, Vol. 18, pp. 585-599.