Geotechnical Site Characterization, Robertson & Mayne (eds) © 1998 Balkema, Rotterdam, ISBN 90 5410 939 4

Geometry and scale effects in CPT and pile design

M.W.Gui & M.D.Bolton
Engineering Department, Cambridge University, UK

ABSTRACT: A study of geometry, particle size and stress effects in the cone penetration test (CPT) is presented. The influence of the penetration distance to mobilise full resistance is considered in relation to some CPT penetrations and model anchor extractions in centrifuge tests. Some implications for the use of CPT results in the design of long and short piled foundations are then discussed.

1 INTRODUCTION

The cone penetration test (CPT) is often chosen to characterise cohesionless material. Values are generally taken to apply to soil conditions at a point, or in a local zone, through some emprical correlation obtained from calibration tests. The nature of calibration chambers makes it difficult to discriminate between geometrical, size and stress-level effects. The application of CPTs in geotechnical and foundation design therefore relies on empirical factors which are not fully understood.

Commenting on the failures of the Nerlerk berm, Been and Crooks (1988) suggested that some of the correlations derived from calibration chamber tests could be erroneous. Erbrich (1994) who modeled the foundations of an offshore structure on a sand sea-bed, pointed out that it is impossible to reproduce the 20 m deep field tip resistance of 60 MPa in a calibration chamber. These seem to question the direct applicability of calibration chamber data.

The launch of the 'Seascout' system by Fugro U.K. Ltd. which adopts a 10 mm cone, as compared to the conventional 38 mm cone, has further encouraged the study of geometry effects in the CPT.

2 INTERPRETATION OF RESULTS

A detailed description of CPTs in the centrifuge has been presented elsewhere (Gui, 1995) and will not be repeated here. Dimensional analysis has also been recommended (Bolton et al, 1993) and used to interpret the results. In general, the tip resistance q_c is normalized with respect to overburden pressure σ'_v, and the penetration depth z is normalized with respect to cone diameter B. Normalized tip resistance Q and normalized penetration depth Z are given as:

$$Q = \frac{q_c - \sigma_v}{\sigma'_v} \quad (1)$$

$$Z = \frac{z}{B} \quad (2)$$

where σ_v and σ'_v are the total and effective stresses respectively.

3 GEOMETRY AND SCALE EFFECTS

3.1 *Initial penetration Δz/B effect*

It has been observed that a CPT will not be able to register the absolute tip resistance at the instant when it penetrates into a new soil layer. Fig 1 shows two tip resistance profiles, the observed and the ideal profiles. After some distance, the observed tip resistance profile starts to deviate from the ideal profile before entering the hard soil layer because the cone is capable of detecting the hard boundary at a few cone diameters away. Once it enters the hard soil, "development" takes place prior to registering the full resistance of the hard soil. Thereafter, the observed profile falls drastically before it enters the soft soil lying beneath it.

An approximate analysis based on a modified Boussinesq's solution (Vreugdenhil et al 1994) also demonstrates the effect. This has a significant impact if the resolution of a thin soil layer is important to the design. It is therefore important, at least qualitatively, to study the effect of the penetration depth (Δz) required to develop the resistance of a new soil layer.

A set of unusual cone uplift tests (Gui, 1995) sheds further light on the displacement Δz required for development of the full penetration resistance. Fig 3 clearly shows the slow mobilization of resistance as a buried cone develops resistance in the sand ahead of itself. Comparitive CPT data fell just above the envelope created by the uplift tests. There is no layering in this case. It simply takes about 5 cone diameters of displacement to "develop" a uniformly graded sand ahead of an advancing probe, whichever direction it is travelling in. Well-graded materials have been observed to require smaller development distances, perhaps as small as 1 or 2 diameters of penetration. A displacement pile will develop the soil ahead of its tip by crushing particles as the tip approaches. A wider grading is produced which leads to voids reduction, permitting the pile to advance. The crushing strength of broken fragments always exceeds that of the original particles; this is proposed by Bolton and McDowell (1997) as the origin for "plastic hardening" during "normal consolidation" of soils.

A driven pile may therefore be capable of carrying almost the complete CPT resistance of the formation, but only if the greater "development" distance of pile is allowed for in the competent soil layer, and if no incompetent layer is in the zone of influence beneath the tip . A cast-in-place pile or anchor will not benefit from any prior development, and will suffer excessive displacements if it is asked to carry more than a small fraction (e.g. 10 to 20%) of the CPT resistance.

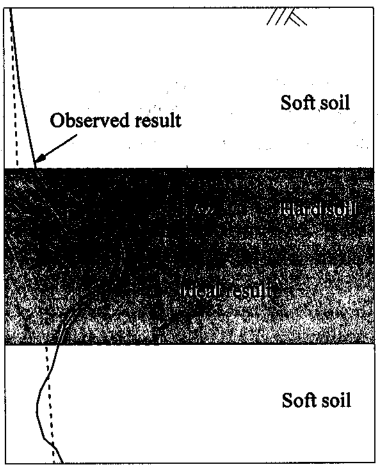

Fig 1: Effects of development on CPT profiles.

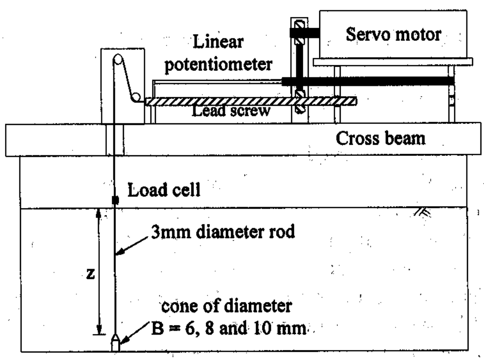

Fig 2: Test set-up for upward-pointing cone tests.

3.2 *Grain size B/d_{50} effect*

The effect of the ratio of cone diameter to mean grain size (B/d_{50}) was studied for Leighton Buzzard sand by Lee (1990). For fine sand at a single relative density, normalized tip resistance Q is plotted against

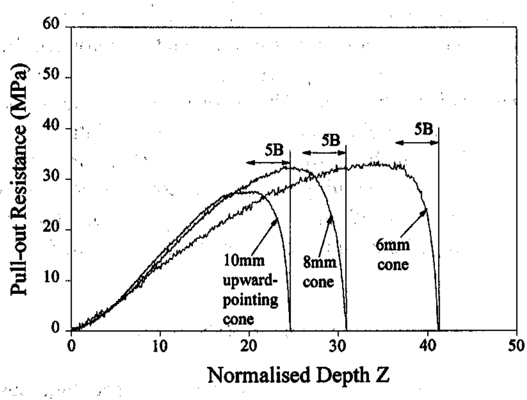

Fig 3: Mobilization of upward pointing cones.

normalized depth Z for cones of different diameter in Fig 4(a). Tests have been carried out in different gravity fields (21g, 40g and 63g) because it is necessary to preserve a constant stress level σ'_v for the different cones, in relation to the constant aggregate crushing strength p_c. Now

$$\sigma'_v = \rho_{dry} \cdot g \cdot z \cdot N \qquad (3)$$

which can also be written as

$$\sigma'_v = \rho_{dry} \cdot g \cdot \frac{z}{B} \cdot N \cdot B \qquad (4)$$

Hence, we must keep "$N \cdot B$" constant in order to preserve a constant σ'_v for each value of Z. Each test therefore models a single prototype cone, of 0.4 m diameter in this case.

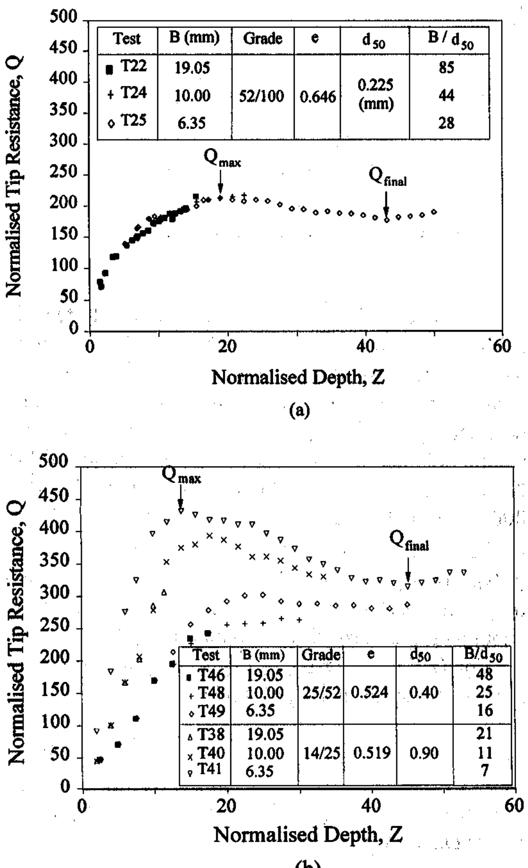

Fig 4: Grain size effects (a): fine particle; (b) medium and coarse particles.

Fig 4(a) shows that the data from this modelling-of-models trial superimposed nicely until each cone approached the base of the test container. This proves that the soil particle size does not affect the result for the ratio B/d_{50} in the range of 85 to 28.

Fig 4(b) repeats the same plot for medium and coarse Leighton Buzzard sand. Treating each soil separately, the plots for the medium sand merge reasonably well for B/d_{50}=48 and 25, but there is a suggestion of a small amount of extra resistance at B/d_{50}=16. For coarse sand, all the data are somewhat higher and while there is insufficient evidence of distortion in reducing B/d_{50} from 21 to 11, it can be seen that a further reduction to 7 does raise resistance especially at shallow depths.

Table 1: Effects of effective cone diameter

No.	B (mm)	d_{50} (mm)	B'	Q_{max}	Q_{final}	$\left(\frac{B'}{B}\right)^2$	$\frac{Q_{max}}{Q_{final}}$
T40	10.0	0.9	10.9	395	320	1.19	1.20
T41	6.35	0.9	7.25	434	324	1.30	1.34
T49	6.35	0.4	7.25	304	278	1.13	1.09

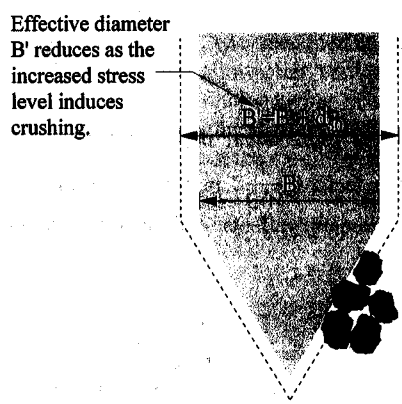

Fig 5: Particle characteristic: degree of freedom.

There may be more than one effect causing the differences in Fig 4. Particle angularity or roughness (which always vary with sand sizes) may be significant dimensional parameters in addition to relative density. However, particle size does also seem to be involved explicitly. Fig 5 demonstrates an empirical definition of an effective diameter of the cone B′ increased by one particle diameter ($B'=B+d_{50}$). The extra resistance of initially large soil grains can be perceived in terms of the increase in the ratio $\frac{Q_{max}}{Q_{final}}$, where Q_{max} is the peak normalized tip resistance and Q_{final} is the final normalized tip resistance before the base boundary effect is detected, by which stage particle crushing would have transformed the native soil. Table 1 shows that normalized tip resistance ratio $\frac{Q_{max}}{Q_{final}}$ is almost exactly equal to the area ratio $\left(\frac{B'}{B}\right)^2$, which shows that B′ can eliminate the peak effect in Fig 4 if other effects can explain the rest.

4 EFFECTS IN PILE DESIGN

The application of CPT in pile design has been very common among practicing engineers on the grounds that the cone penetrometer can be treated as a model pile. Meyerhof (1976) suggested that when the pile point is above some critical depth in the bearing stratum, the unit tip resistance of a shallow pile q_{pile} should be reduced below the tip resistance q_c, in proportion to the embedment ratio $(z/B)_{pile}$ in this stratum:

$$q_{pile} = \frac{q_c}{10} \cdot \left(\frac{z}{B}\right)_{pile} \tag{5}$$

where z is the embedded depth of the pile. Presumably the factor "$\frac{z}{10B}$" in eqn. (5) is to take care of the geometry effects and also the stress history effect.

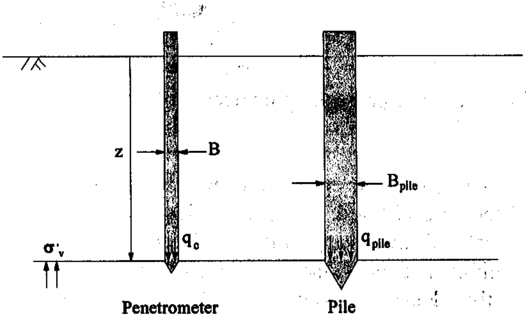

Fig 6: Geometry effects between a penetrometer and a pile.

Fig 6 shows that for a particular stress level, (z/B) ratio for a penetrometer is very much greater than the $(z/B)_{pile}$ ratio for a pile. For example, at a depth of 10 m, the (z/B) ratio for a cone is about 280; whereas for a 500 mm diameter pile, the $(z/B)_{pile}$ ratio is only 20. The results obtained by Lee (1990) were replotted in Fig 7. Due to the geometrical effects, for a particular stress level, probes with smaller diameter would have a higher tip resistance. Therefore, the CPT result can not have a one to one correlation with the tip resistance of a pile.

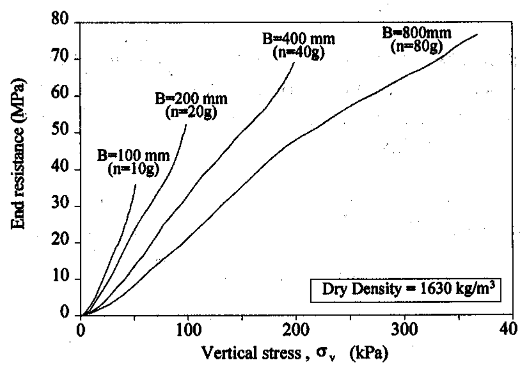

Fig 7: Tip resistance vs vertical stress.

Jamiolkowski et al (1985) demonstrated in calibration chamber tests that q_c is proportional to $(\sigma'_v)^{0.5}$. This should correspond to piles deeper than their critical depth. To illustrate the relationship between q_c, $(\sigma'_v)^{0.5}$ and (z/B) for all depths, centrifuge results obtained by Kokturk (1993) are plotted in Fig 8. Prototype diameters of his probes were 452, 791, 800, 1400 and 1412 mm. All the results were plotted in the fashion of $(q_c/\sqrt{\sigma'_v})$ versus normalized depth (z/B). For a particular relative density and regardless of the diameter of the probes, all the results seem to adopt a unique form similar to that in Fig 9.

For $(z/B)_{pile} > (z/B)_{crit}$, Fig 9, it is reasonable to assume the tip resistance of a pile to be:

$$q_{pile} = q_c \tag{6}$$

For $(z/B)_{pile} < (z/B)_{crit}$, Fig 9, the tip resistance of a pile is taken to be:

$$q_{pile} = \alpha \cdot \sqrt{\sigma'_v} \cdot \left(\frac{z}{B}\right)_{pile} \tag{7}$$

Taking $\sigma_v' \approx \gamma' z$, and knowing that

$$q_c = \alpha \cdot \sqrt{\sigma'_{v,crit}} \cdot \left(\frac{z}{B}\right)_{crit} \tag{8}$$

We obtain, for $(z/B)_{pile} < (z/B)_{crit}$:

$$\frac{q_{pile}}{q_c} = \left(\frac{z}{z_{crit}}\right)^{1.5} \tag{9}$$

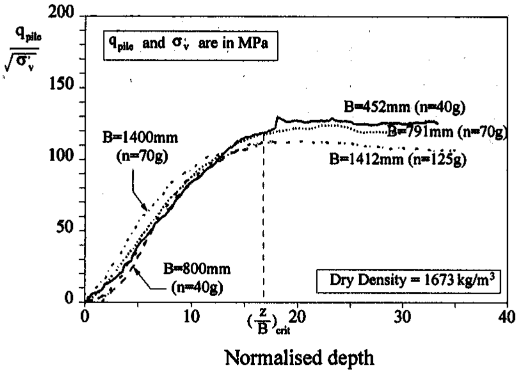

Fig 8: Plot of $\frac{q_{pile}}{\sqrt{\sigma'_v}}$ versus normalized depth Z.

Depending on the relative density, $(z/B)_{crit}$ may lie between 5 to 20. Compared to dense sand, loose sand will have a smaller value of $(z/B)_{crit}$. Thus the equation proposed by Meyerhof (1976) might well underestimate the tip resistance of a pile in loose sand. On the other hand, it would overestimate the tip resistance of a pile in dense sand, if Fig 8 is considered relevant.

5 DISCUSSION

It is clear that there are two phases of behaviour depending on the critical penetration depth ratio Z_{crit}. Shallower than some critical ratio $[(z/B)_{crit} = 20$ in Fig 4(a)] the coefficient Q increases with depth ratio in the fashion of shallow foundations. At depths greater than this critical depth, the coefficient seems to hold steady, or to fall slightly, characteristic of deep foundations. The dichotomy was observed by Meyerhof (1983) in his 1g tests on model piles. This effect is highly significant in model tests with CPT probes which behave more like prototype driven piles, and less like field CPTs.

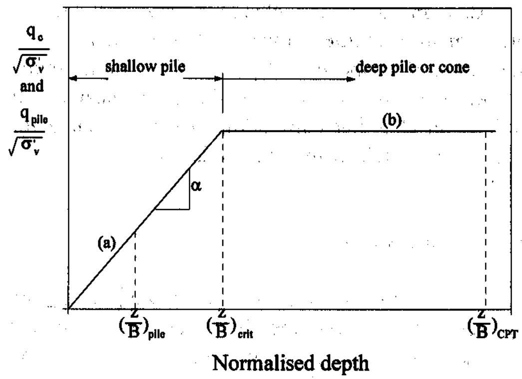

Fig 9: Idealized plot of $\frac{q_{pile}}{\sqrt{\sigma'_v}}$ versus normalized depth Z.

A shallow mechanism is associated with surface heave while a deep mechanism is associated with local penetration. When the probe penetrates into the soil, cavity expansion occurs around the cone. As the probe penetrates further downwards, it will generate a succession of cavities, Fig 10(a) and (b) at increasing pressure. This phenomenon is introduced and defined here as the development of stress history. In a calibration chamber, the specimen is covered by a rigid platen which allows the vertical stress to be applied, Fig 10(a). Since heaving of soil around the probe is not possible, the shallow mechanism can not be developed as it is in the case of centrifuge or field tests, Fig 10(b).

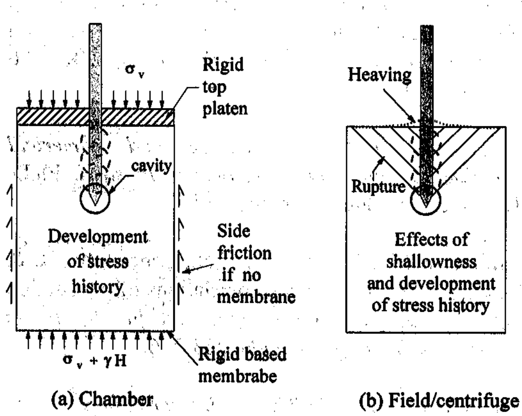

Fig 10: Penetration mechanism in (a) calibration chamber; and (b) field/centrifuge.

Also, in calibration chamber tests, the shallow mechanism is omitted since most of the data points are recorded at the mid depth of the sample, much deeper than the critical depth. This means that calibration chamber correlations are not suitable for direct use in shallow foundation design.

6 CONCLUSION

A CPT can be regarded as a scaled-down pile, and its tip resistance offers the best starting point for designers of full-scale driven piles. However, the bearing capacity of short piles must be reduced in relation to the critical depth of the formation, which can readily be explored in simple centrifuge tests.

REFERENCES

Been, K. & Crook, J.H.A. (1988) A critical appraisal of CPT calibration chamber tests. *Proc. Int. Symp. on Penetration Testing, Orlando,* Vol. 2, 651-660.

Bolton, M.D., Gui, M.W. & Phillips, R. (1993) Review of miniature soil probes for model tests. *Proc. 11th South East Asia Geotechnical Conf., Singapore*, 85-91.

Bolton, M.D. and McDowell, G.R. (1997) Clastic mechanics. *Proc. IUTAM Symposium on Mechanics of Granular and Porous Materials, Cambridge*, Kluwer Academic Publishers, 35-46.

Erbrich, C.T. (1994) Pileless jacket foundations: A New Era. *Offshore Engineering Society, British Geotechnical Society and Ground Engineering Joint Meeting*, Institute of Civil Engineers, UK., 30 November'94.

Gui, M.W. (1995) *Centrifuge and numerical modeling of pile and penetrometer in sand.* Ph.D. Thesis, Cambridge University, U.K.

Jamiolkowski, M., Ghionna, V.N., Lancellotta, R. & Pasqualini, E. (1985) New correlations of penetration tests for design practice. *Proc. 1st Int. Symp. on Penetration Testing, Orlando*, 263-296.

Kokturk, H. (1993) *Interpretation von Drucksondierungen in Zentrifugen-modellversuch.* Diplomarbeit, Ruhr Universitat Bochum, Germany.

Lee, S.Y. (1990) *Centrifuge modeling of cone penetration testing in cohesionless soils.* Ph.D. Thesis, Cambridge University, U.K.

Meyerhof, G.G. (1976) Bearing capacity and settlement of pile foundations. *J. of Geotechnical Engineering Division ASCE*, 102(3), 197-228.

Meyerhof, G.G. (1983) Scale effects on ultimate pile capacity. *J. of Geotechnical Engineering Division ASCE*, 109(6), 797-806.

Vreugdenhil, R., Davis, R. & Berrill, J. (1994) Interpretation of cone penetration results in multilayered soils. *Int. J. for Numerical and Analytical Methods in Geomechanics*, Vol. 18, 585-599.