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ENGINEERING PROPERTIES OF ALLUVIAL SOILS IN NEWCASTLE USING CONE PENETRATION TESTING

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ABSTRACT: The modern cone penetration test provides a rapid and reliable method of profiling soil conditions, and is particularly suited to alluvial sediments such as encountered in many parts of the Newcastle area. Numerous empirical and theoretical methods have been developed to relate measured cone parameters to various engineering properties. This paper presents the results of selected engineering soil parameters derived from cone penetration tests carried out in the Newcastle region. Where possible, the parameters are compared to those measured from other laboratory and in situ tests.

INTRODUCTION

The cone penetration test (CPT) is widely used throughout the world in geotechnical investigations to assess subsurface conditions and estimate the engineering properties of the soil. The modern CPT dates back to the 1960s, however recent advances in technology have enabled more instruments to be placed in the cone and these are monitored by digital data acquisition systems. CPT is generally confined to use in deposited sediments comprising alluvial soils or man-made fill, with the depth capability generally limited by the available thrust of the pushing device.

The tests discussed in this paper were obtained using several types of cone (see Table 1) at various sites in the Newcastle area. In each case the 35mm diameter, 60° apex instrumented cone was hydraulically thrust into the soil at a rate of 2cm/s from a purpose-built truck mounted rig which has a maximum pushing force of 13t. This thrust is generally sufficient to reach bedrock. The instruments in the cone were monitored and processed using software developed by the author. The measurements were recorded at 20mm depth intervals, thereby providing a virtually continuous profile.

A further type of cone which has also been used extensively in Newcastle is the conductivity (or resistivity) cone, which measures the bulk electrical conductivity of the soil-water system. The conductivity cone is primarily used in environmental investigations to profile contaminant plumes, and is not discussed further in this paper.

All cones also include inclinometers which monitor and record any deviation from verticality as the test proceeds; hence the depths can be adjusted to true vertical depths as required, although this is generally not necessary.

The results from several sites in the Newcastle area are used to illustrate the derivation of selected engineering properties from CPT results.

Table 1. Cone types.

Cone Type	Parameters Measured
Standard cone	cone resistance, sleeve friction
Seismic cone	cone resistance, sleeve friction, shear wave velocity
Piezocone	cone resistance, sleeve friction, dynamic pore pressure

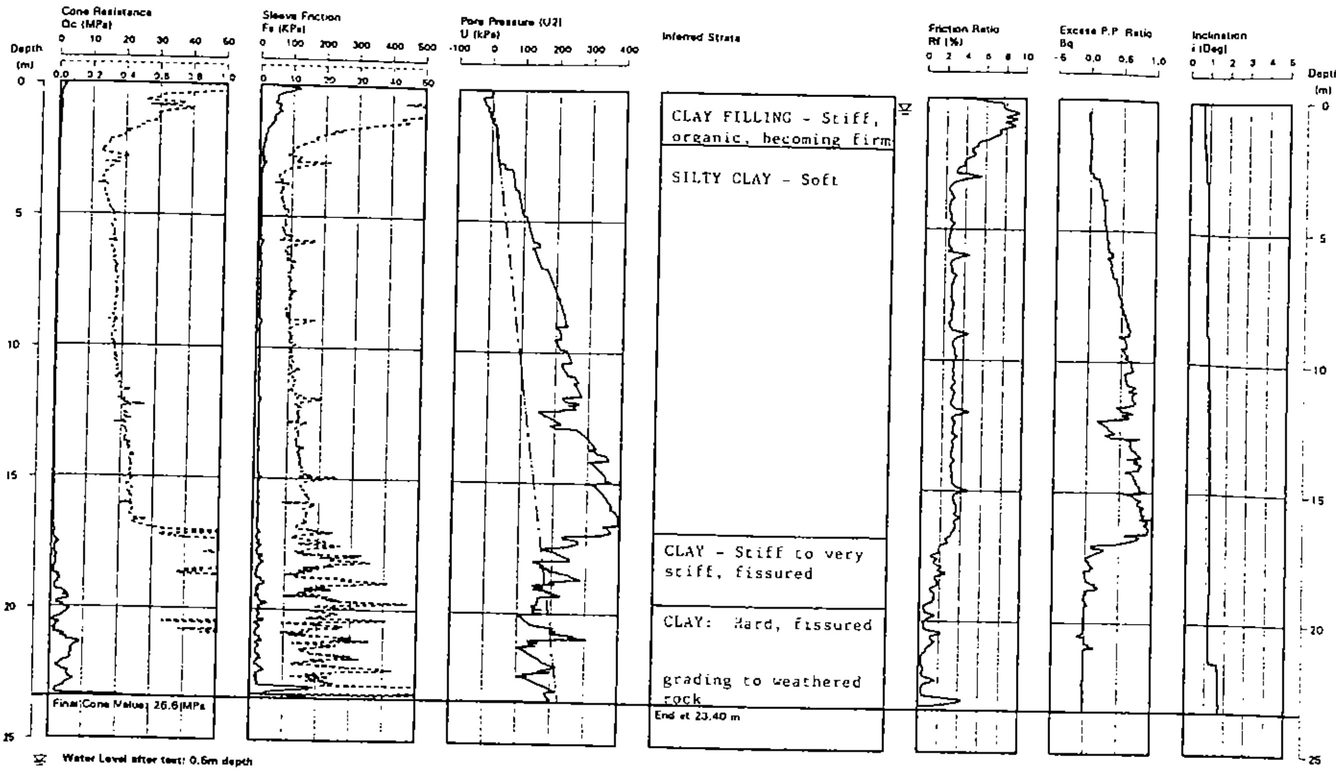

Figure 1. Typical piezocone test at Hexham Swamp.

UNDRAINED SHEAR STRENGTH

The undrained shear strength of clays, particularly soft alluvial sediments, is of great interest in geotechnical design. The most widely used relationship between CPT parameters and shear strength is:

$$s_u = (q_t - \sigma_v)/N_{kt} \qquad \dots (1)$$

where q_t is the total cone resistance, corrected for pore water pressure effects (requires piezocone, otherwise assumed to equal measured cone resistance q_c), σ_v is the total vertical overburden stress and N_{kt} is the cone bearing factor. N_{kt} has been shown to be a function of over-consolidation ratio and rigidity index, which is defined as the ratio of shear modulus to undrained shear strength (G/s_u). Hence the selection of an appropriate value of N_{kt} is not straightforward. Expressions for N_{kt} have been derived from cavity expansion theory, which are also functions of rigidity index. For normally consolidated marine or estuarine clays, previous studies indicate a typical range for N_{kt} of 12 to 18, with a value of 15 commonly used.

Piezocone results from a clay site on the edge of Hexham swamp are shown in Figure 1. The estimated shear strengths using Equation 1 are compared to field values measured in an adjacent borehole using a shear vane in Figure 3. The shear vane results were corrected using Bjerrum's correction factor μ, whereby

$$s_{u(field)} = s_{u(vane)} \cdot \mu = s_{u(vane)} \cdot \mu_R \mu_A \qquad \dots (2)$$

The factor μ_R takes account of the strain-rate effect of testing, which is a function of the plasticity index I_p and the time to failure. In field tests, failure generally occurs within about one minute, and the corresponding empirical expression for μ_R is:

$$\mu_R = 1.05 - \sqrt{I_p} \qquad \dots (3)$$

At this site the plasticity index was measured to range from 56% to 66% (average 60%), resulting in $\mu_R = 0.82$. The factor μ_A accounts for anisotropic effects and is relatively independent of I_p. Results reported by Chandler (1988) suggest a value of about 0.95 for μ_A, resulting in $\mu = 0.78$.

By plotting $s_{u(field)}$ against net cone resistance ($q_t - \sigma_v$) as shown in Figure 2, the slope of the line of best fit represents the average backfigured value for N_{kt} at this site, which equals 13.7, with a correlation coefficient r = 0.89. There is some scatter however, with backfigured values of N_{kt} in the range 6.8 to 24.4. These extreme values possibly relate to variations in over-consolidation ratio and/or test inaccuracies. It is satisfying however that the backfigured average value of N_{kt} for this site is within the accepted range.

OVER-CONSOLIDATION RATIO (OCR)

The over-consolidation ratio is defined as the ratio of the maximum past effective over-burden pressure to present effective overburden pressure (σ_p'/σ_v'). For normally consolidated soils OCR = 1.

For clays, following the relationships can be deduced from Wroth (1984), Robertson and Campanella (1984) and Mayne (1979):

$$s_u / \sigma_v' = 0.3\ \mathrm{OCR}^{0.8} \qquad \ldots (4)$$

Combining Equations 1 and 4, and rearranging leads to:

$$\mathrm{OCR} = 10^{\left\{1.25\log\left[\frac{q_t - \sigma_v}{0.3\sigma_v' N_{kt}}\right]\right\}} \qquad \ldots (5)$$

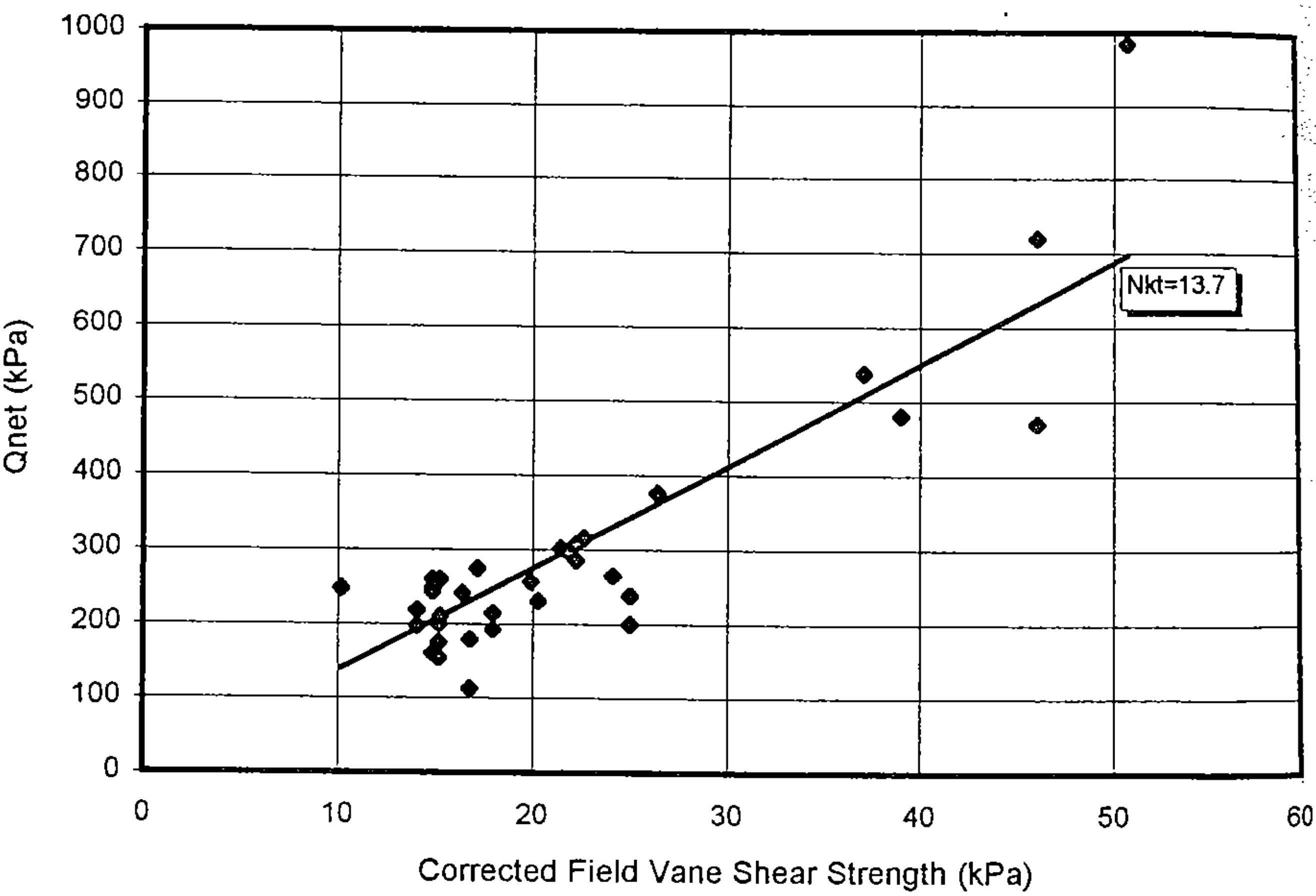

Figure 2. Net cone resistance vs. field vane shear strength.

Mayne & Chen (1994) derived the following model using piezocone and the measured pore pressure behind the tip (u_2):

$$\mathrm{OCR} = \left[\frac{1}{1.95M + 1} \cdot \frac{(q_t - u_2)}{\sigma_v'}\right] \qquad \ldots (6)$$

where $M = 6\sin\phi'/(3 - \sin\phi')$, and hence requires a value for the drained friction angle ϕ' for the clay; in the absence of test data $\phi' = 30°$ has been assumed. The estimated OCR is not particularly sensitive to ϕ', which for most clays ranges from 20° to 40° (Mayne and Chen, 1994).

Six consolidation tests on piston samples taken from the same bore at Hexham swamp were used to determine the effective preconsolidation pressure and hence OCR. Figure 3 shows these values plotted against Equations 5 and 6 using the companion piezocone test, and using $N_{kt} = 13.7$ as previously backfigured.

These results show excellent agreement between the laboratory estimates of OCR and Equation 5. The Mayne and Chen model appears to over-estimate OCR at this site. Given the interdependence between OCR, s_u and N_{kt}, the development of an iterative routine would seem appropriate to achieve compatibility.

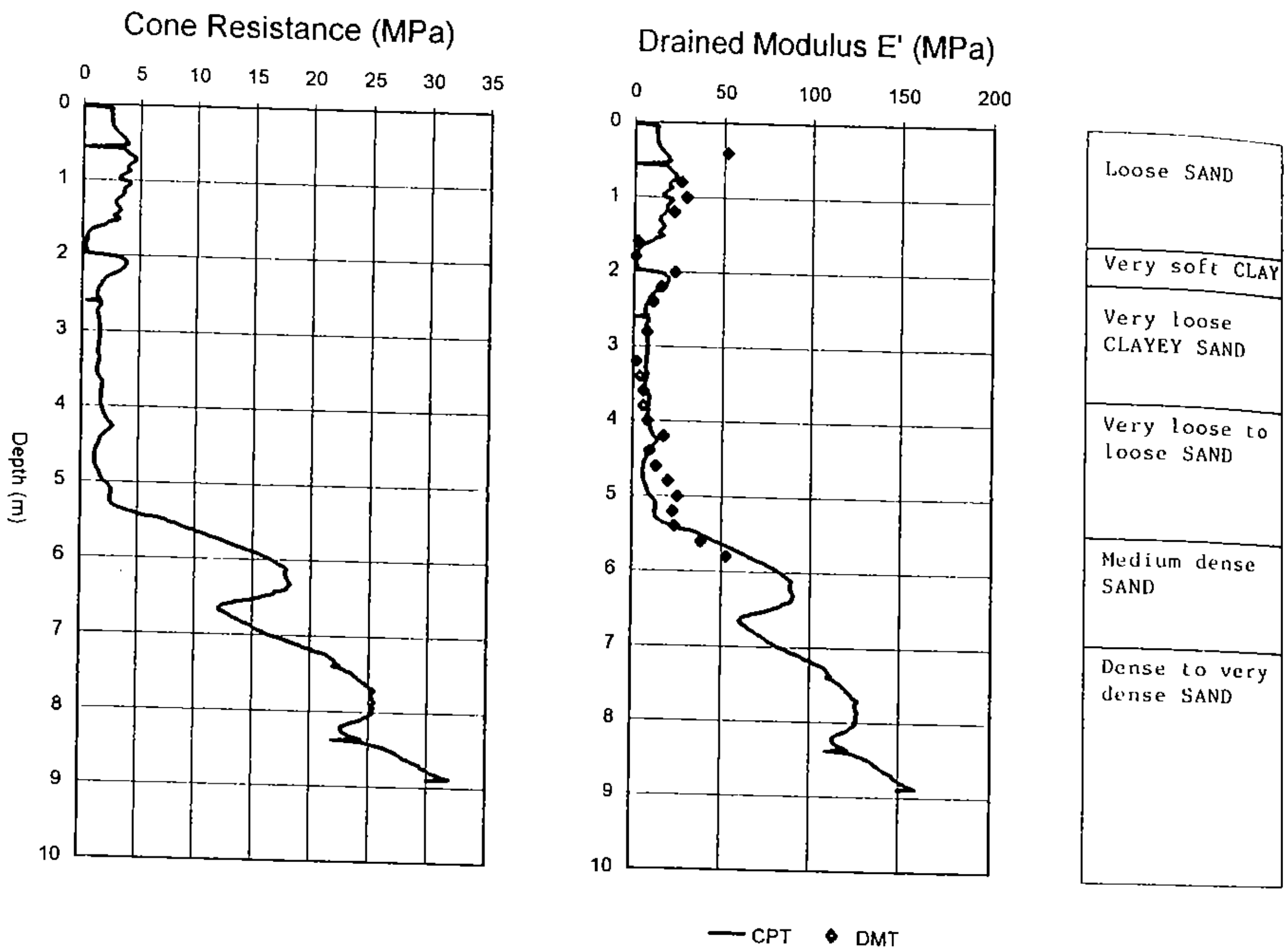

Figure 4. Drained modulus at Williamtown.

Considerable scatter in backfigured α values was obtained however, and this highlights the difficulty in accurately measuring modulus values. Laboratory calibration chamber tests to calibrate local sands with both CPT and DMT would be highly desirable. (A calibration chamber test program for DMT has recently commenced at Newcastle University).

COEFFICIENT OF CONSOLIDATION

By pausing the penetration of a piezocone in a layer of interest, the ensuing dissipation of pore pressure can be monitored to provide estimates of consolidation parameters and permeability.

Several methods have been proposed for the analysis of dissipation data to determine the coefficient of consolidation. Dissipation is governed primarily by horizontal permeability (Teh and Houlsby, 1989) and hence analysis provides an estimate of c_h, the horizontal coefficient of consolidation. The excess pore pressure in sandy soils dissipates almost immediately, while low permeability clays can take several hours to dissipate.

The results are processed using a root-time corrected plot to estimate the time for 50% dissipation of excess pore pressure (t_{50}). This method is described by Sully and Campanella (1994), and the results obtained require a particular treatment to arrive at c_h, as discussed below.

Dissipation records measured behind the cone tip (u_2) often show an initial rise in pore pressure after stopping the cone, particularly in overconsolidated soils. This is primarily due to large pressure gradients around the cone tip and the redistribution of these pore pressures. It can also be associated with an initial drop in pore pressure as the cone is unloaded and subsequent redistribution of pore pressures.

The straight line portion of the root-time dissipation curve is used to determine the effective initial pore pressure (u_i). The final pore pressure is assumed to approximate the hydrostatic pore pressure (u_o), and hence t_{50} can be estimated.

Estimation of c_h may then be made using the following relationship derived by Teh & Houlsby, 1989:

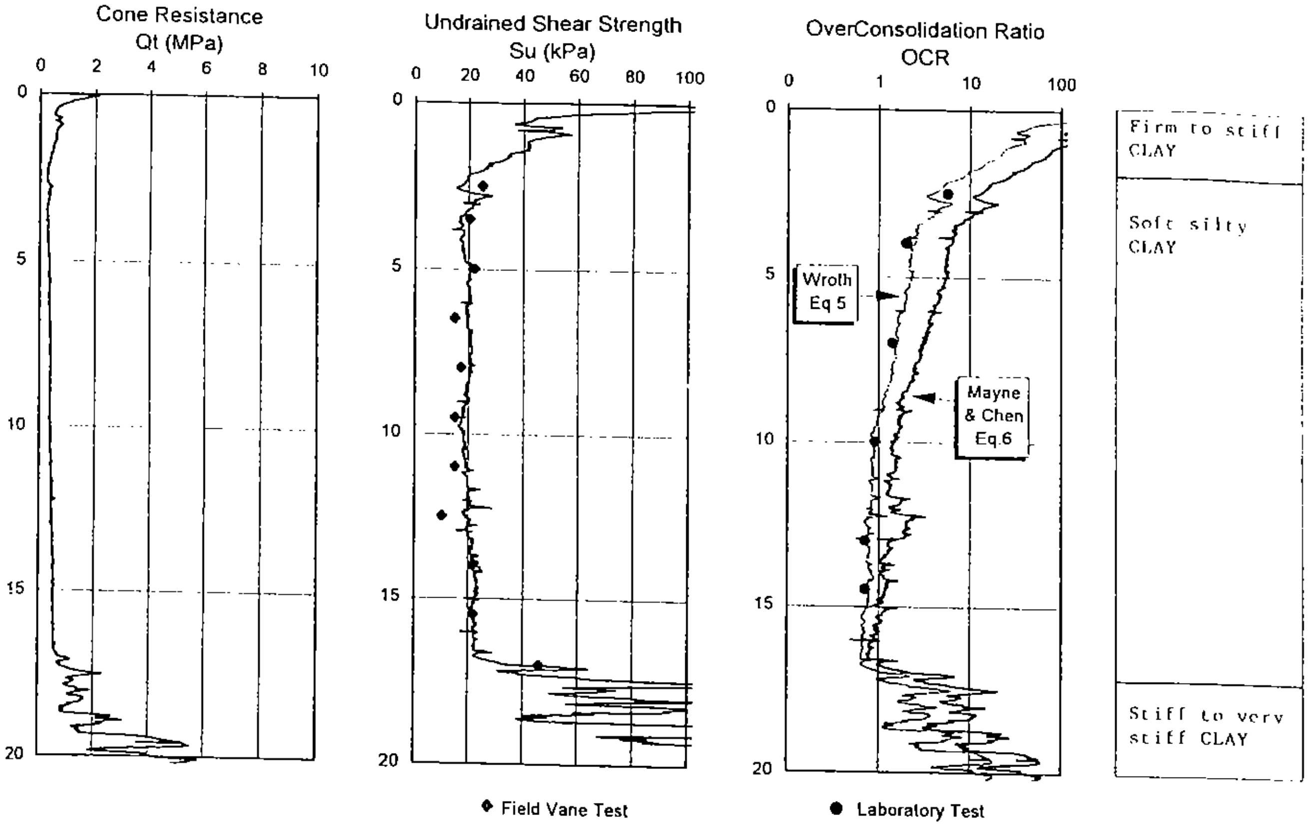

Figure 3. Comparison of shear strength and OCR results.

COMPRESSIBILITY

The drained modulus (E'), undrained modulus (E_u) and the coefficient of compressibility (m_v) may be estimated directly from the cone resistance, using an empirically derived linear factor α. The following relationships are used:

Coefficient of compressibility:

$$m_v = 1/\alpha q_t \qquad \ldots (7)$$

Drained modulus:

$$E' = \frac{(1+v')(1-2v')}{(1-v')m_v} = \frac{(1+v')(1-2v')\alpha q_t}{(1-v')} \qquad \ldots (8)$$

Undrained modulus:

$$E_u = \frac{3E'}{2(1+v')} \qquad \ldots (9)$$

where v' = drained Poisson's ratio.

The factor α varies with soil type, degree of overconsolidation and plasticity (for clays). For normally consolidated soils, typical values are: α = 3–11 for sands, and α = 2–8 for clays (Wiesner, 1985). Hence modulus values can only be approximated from CPT unless local correlations with soil type enable appropriate α factors to be established. As laboratory calibration testing of local soils has not been undertaken, it is only possible to compare modulus values determined by two different field methods. At a site in Williamtown, four piezocone tests were carried out in tandem with Marchetti Dilatometer (DMT); and the results from one of these tests are compared in Figure 4. Review of all four pairs of tests has indicated that the best fit between CPT and DMT results at this site was provided by α = 8 for sand and α = 4 for clay. The modulus profile in Figure 4 is shown for α = 8, as the predominant soil is sand.

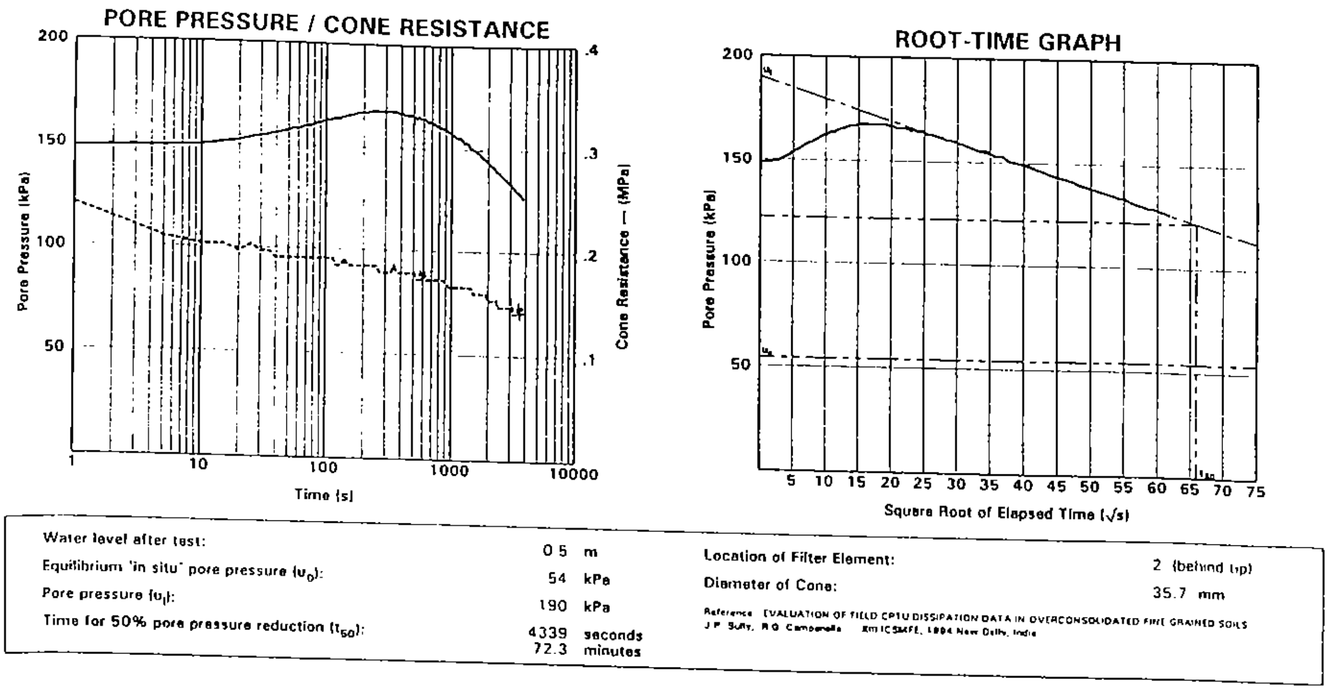

Figure 5. Pore pressure dissipation test in a soft marine clay in Newcastle.

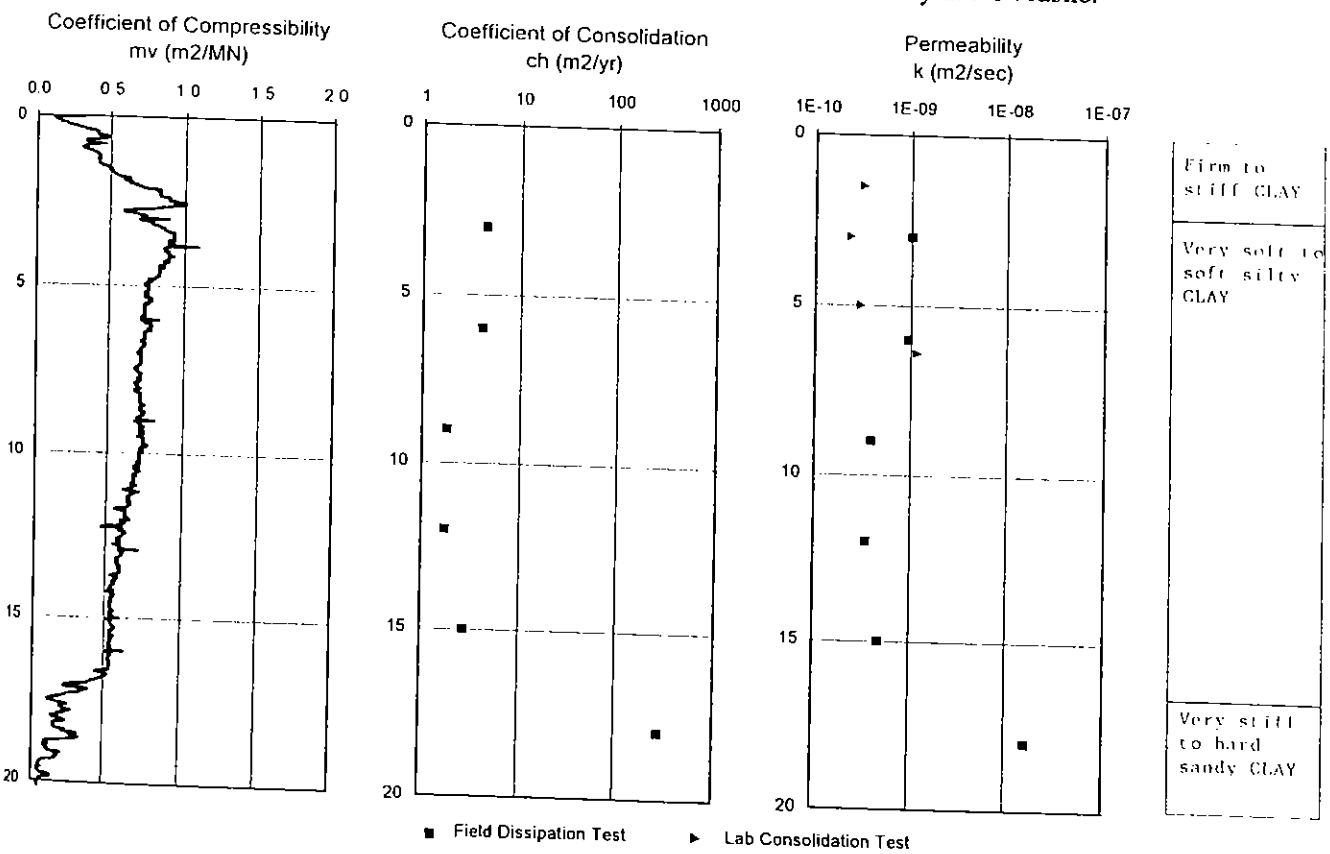

Figure 6. Coefficients of compressibility, consolidation and permeability — marine clay, Newcastle.

$$c_h = \frac{T^* a^2 \sqrt{I_r}}{t} \qquad \ldots (10)$$

where

- T^* is a modified time factor
- a is the cone radius (17.84 mm)
- I_r is the rigidity index = $E_u/3s_u$
- t is measured time

The modified time factor T^* varies with degree of dissipation and position of the filter element. For 50% dissipation using a filter element behind the tip (u_2), the value of T^*_{50} is 0.245.

Given the value of c_h, the coefficient of permeability (k) can be estimated from:

$$k = \gamma_w \, m_v \, c_h \qquad \ldots (11)$$

which assumes the soil is isotropic. For anisotropic soils, an estimate of the ratio c_h/c_v or m_v/m_h is required to estimate k_v or k_h. Hence, estimates of permeability from piezocone dissipation tests tend to be very approximate and provide an initial guide to the order of magnitude only.

A typical dissipation record from a soft marine silty clay in Newcastle is shown in Figure 5. A total of six dissipation tests were carried out during this particular piezocone test, and the results may be compared with four laboratory consolidation tests on samples taken from a nearby bore (Figure 6).

FRICTION ANGLE — SANDS

Since the early 1970s several methods have been proposed for estimating the friction angle of sands, which can be broadly divided into three main groups: those based on bearing capacity theory, those based on cavity expansion, and purely empirical correlations. The bearing capacity approach appears to be most widely used, particularly that of Durgunoglu & Mitchell which has been found to compare favourably with both the cavity expansion models and laboratory calibration chamber tests.

The Durgunoglu & Mitchell method (Puppala *et al.*, 1993) is based on bearing capacity theory for cohesionless soil, from which:

$$q_c = \gamma B \, N\gamma_q \, \xi\gamma_q \qquad \dots (12)$$

where

- γ = soil density
- B = cone diameter
- $N\gamma_q$ = bearing capacity factor for wedge penetration (plain strain)
- $\xi\gamma_q$ = shape factor to convert wedge factors to cone factors.

The combined cone factor $N\gamma_q \, \xi\gamma_q$ is then related to the friction angle ϕ' via base roughness δ/ϕ', relative depth of penetration D/B and lateral earth pressure coefficient K_0.

More recently, Masood & Mitchell (1993) have suggested a method using sleeve friction to estimate in situ lateral stress. The sleeve friction during penetration can be expressed as:

$$f_s = c_a + k_s . \sigma_v' . \tan\delta \qquad \dots (13)$$

where

- c_a = unit adhesion between the soil and the sleeve
- k_s = lateral earth pressure coefficient during penetration
- δ = angle of friction between the soil and the sleeve.

By assuming that c_a is negligible, $\delta = \phi'/3$ and $k_s = k_p$ (passive earth pressure coefficient), Equation 13 becomes:

$$f_s = \sigma_v' . \tan^2 \left[45 + \frac{\phi'}{2}\right] . \tan \left[\frac{\phi'}{3}\right] \qquad \dots (14)$$

which enables ϕ' to be solved directly from the measured sleeve friction.

The above two methods are compared in Figure 7, using a test carried out at Williamtown, with very good agreement achieved. It is however necessary to check these methods against controlled laboratory calibration chamber tests to verify the accuracy of the results in local sands.

SHEAR WAVE VELOCITY AND DYNAMIC SHEAR MODULUS

The incorporation of tri-axial accelerometers within the cone assembly enables in situ downhole measurements of shear wave and compression wave velocities. Known as the seismic cone, this method enables rapid profiling of dynamic soil properties, with depth, at the same time as determining stratification from cone resistance and sleeve friction measurements. Following the December 1989 earthquake, this technique was used at several sites in Newcastle to aid geotechnical assessment of the dynamic response of the soil profile, and amplification of bedrock motion through the alluvium.

The shear wave velocity, V_s, is measured at successive depth increments by stopping penetration and generating a shear wave at the ground surface. The dynamic shear modulus G_d (low strain, maximum shear modulus) may than be estimated from the relationship:

$$G_d = \rho V_s^2 \qquad \dots (15)$$

where ρ is the soil unit density γ/g, g is acceleration due to gravity.

Figure 8 shows a typical test in the Newcastle CBD area. From a compilation of a number of tests in central Newcastle, typical ranges of V_s and G_d are shown in Table 2 as a function of soil type.

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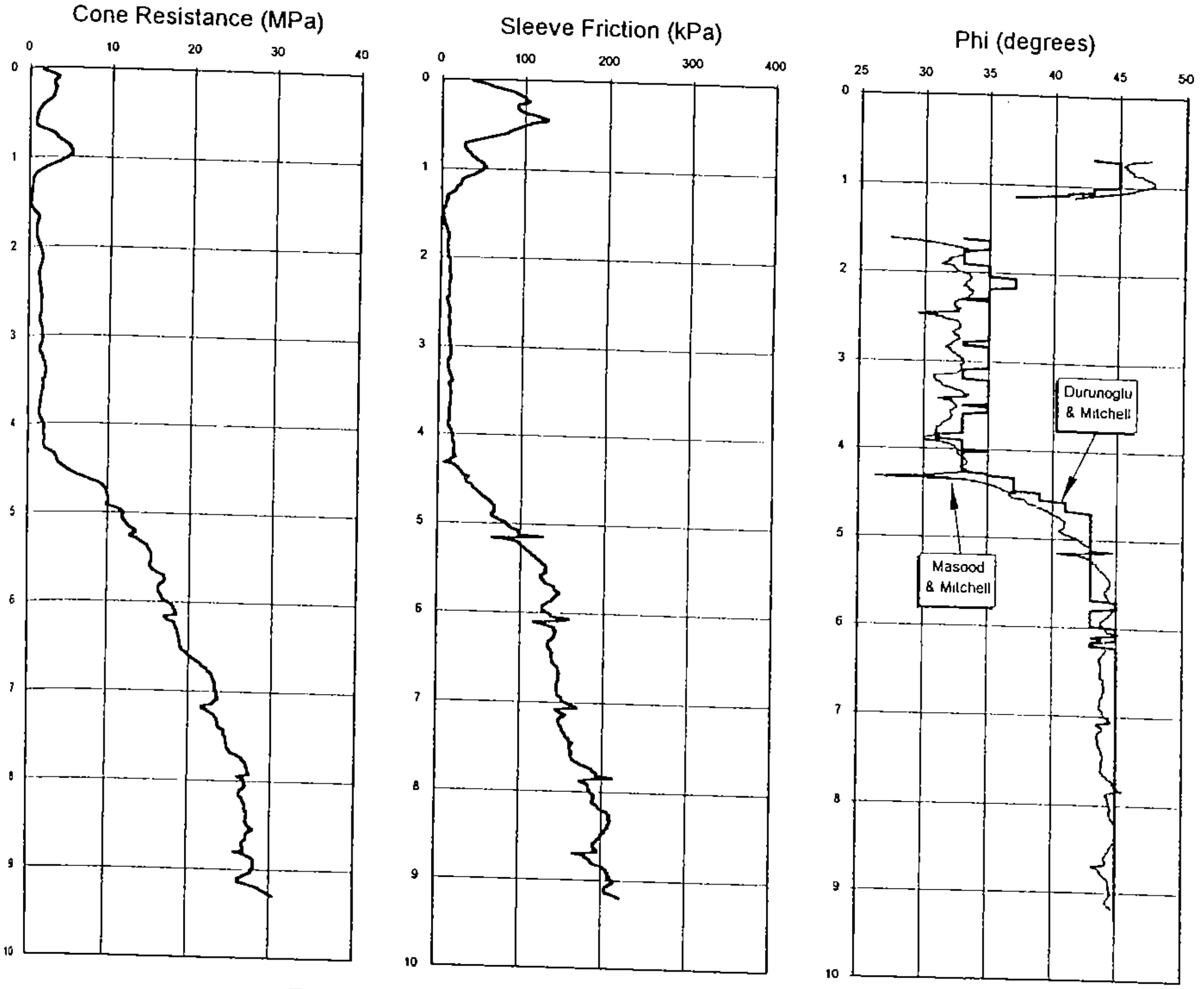

Figure 7. Estimation of friction angle — Williamtown sand.

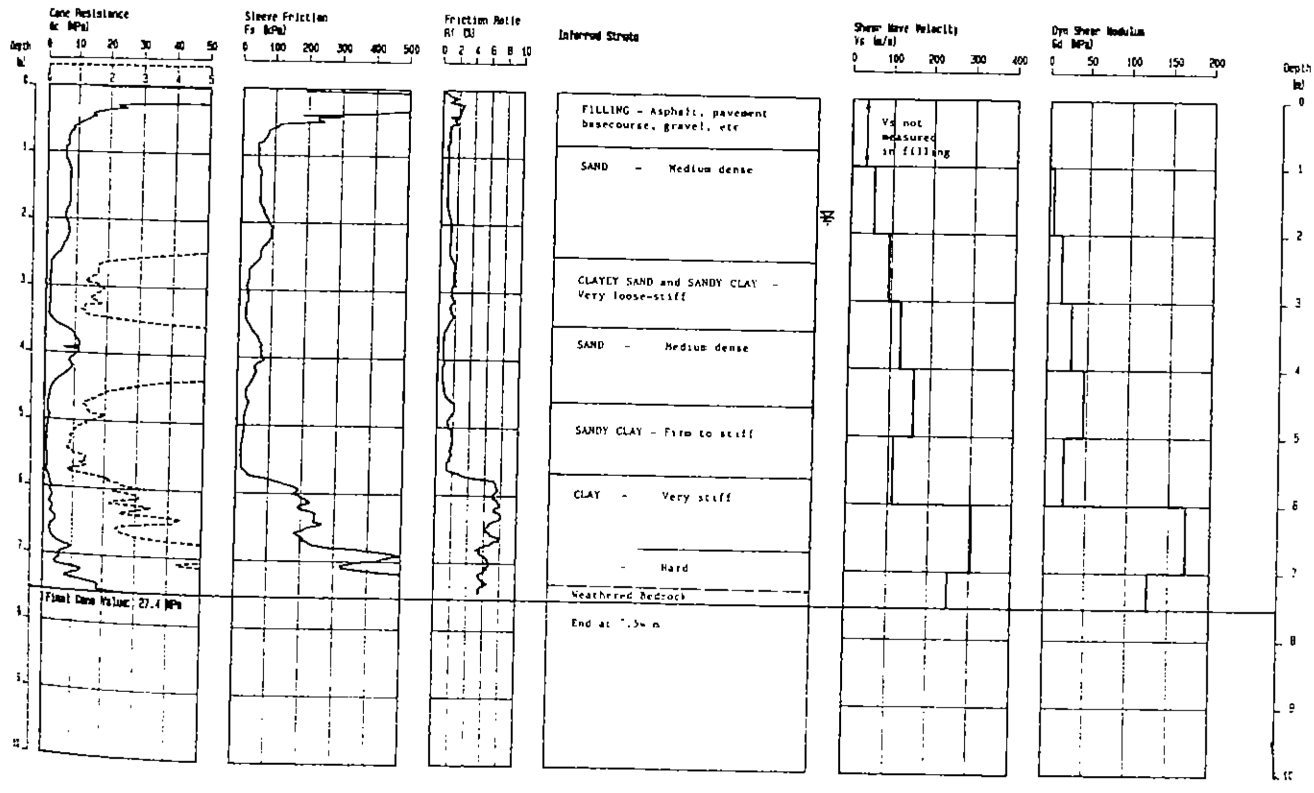

Figure 8. Seismic cone test in Newcastle CBD.

Table 2. Typical shear wave velocity and shear modulus values, Newcastle CBD.

Soil Type	Shear Wave Velocity V_s (m/s)	Dynamic Shear Modulus G_d (MPa)
SAND - very loose to loose	60 - 120	5 - 25
- medium dense	80 - 180	10 - 60
- dense to very dense	200 - 500	80 - 500
CLAY - firm to stiff	110 - 200	22 - 75
- very stiff to hard	190 - 250	70 - 120
SAND/CLAY/SILT mixtures - generally low strength	80 - 110	10 - 22

The range of results obtained indicates the inherent variability in dynamic soil properties. Although some researchers, such as Imai and Yokota (1982) and Mayne and Rix (1993) have developed relationships between SPT N or CPT q_c values and V_s or G_d, there are clearly other influences, such as layering, cementation, fissuring and over consolidation ratio. Site specific measurements of V_s and G_d are therefore preferable to using empirical correlations.

CONCLUSION

The state-of-the-art CPT is a powerful and rapid form of in situ testing which can provide reasonably reliable first-order estimates of many engineering parameters, generally in continuous form with depth. CPT data should preferably be correlated with companion bore and laboratory test data to calibrate the relationships. However, companion data is not always available, and the results reported herein has indicated that the relationships for Newcastle alluvial soils are consistent with world-wide research into such relationships.

ACKNOWLEDGMENTS

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