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Use of Piezometer Cone Data

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Introduction

Cone penetration testing with pore pressure measurements has become increasingly more popular in recent years. The piezometer cone test is now standard practice for all major soil investigations in the Norwegian Sector of the North Sea (Lunne and Lacasse, 1985). The piezometer cone is generally regarded as the most efficient tool for stratigraphic logging of soft soils. A thin pore pressure element (5 mm thick) can sense layers a few centimeters thick. However, there remains some uncertainty with regard to the interpretation and use of the piezometer cone data for engineering applications.

This paper summarizes some of the experience with piezometer cone testing and its use in the Vancouver, B.C., Canada area.

Equipment

Piezometer cone testing has been carried out by the In-situ Testing Group at the University of British Columbia (UBC) over the last seven years. Tests have been performed using Fugro, Hogentogler, Geotech and UBC developed piezometer cones. Full details of the cones are given by Zuidberg et al. (1982), Auxt and Nolan (1985), Jefferies and Funegard (1983) and Campanella et al. (1983). Pore pressures have been measured at several locations on the cone but predominately immediately behind the tip and midway along the face of the tip. A cone has recently been developed at UBC that can simultaneously measure the pore pressure behind the friction sleeve and either behind the tip or on the face of the tip.

Recent work (Campanella et al. 1982) has shown that the pore pressures generated during cone penetration influence the measured cone resistance, q_c, and sleeve friction f_s. Because water pressure acts on an area immediately behind the cone tip, q_c should be corrected for the effect of pore pressure using;

$$q_T = q_c + u(1-a) \quad (1)$$

where q_T = correct total tip resistance
u = pore pressure measured immediately behind tip
a = net area ratio

This correction cannot be eliminated except with a unitized, jointless cone design. Cone data should be corrected to allow comparison of data from cones of different design. A similar correction is required for sleeve friction data. However, information is required of the pore pressures at both ends of the friction sleeve. The importance of the sleeve friction correction can be reduced using a cone design with an equal end area friction sleeve.

Several cone users who use cones that record the pore pressure on the face of the cone tip have suggested correction factors to correct the measured pore pressures to those that are assumed to exist immediately behind the tip. The assumed ratio of the pore pressure on the face to the pore pressure behind the tip is generally taken to be about 1.2 (i.e. the pore pressure on the face is assumed to be 20% larger than that immediately behind the tip). Measurements made by many cone users around the world (Campanella et al. 1985, Jamiolkowski et al., 1985, Lunne 1985) have shown that the ratio of 1.2 is generally only true for soft, normally consolidated clays. A summary of some data illustrating pore pressures measured at different locations on the cone is shown in Fig. 1.

Figure 1 is an update of the data presented by Campanella et al. (1985). Figure 1 clearly shows that in stiff, overconsolidated, cemented or sensitive clays, the pore pressure on the face of the tip can be many times larger than that immediately behind the tip. Therefore, to correct the cone bearing to q_T, the pore pressure must be measured behind the tip. It is also interesting to note that pore pressures generally do not vary significantly along the friction sleeve. Therefore, equal end area friction sleeves should record approximately the correct stress.

Sites in the Vancouver, B.C. area

Much of the data presented in this paper was obtained at sites near Vancouver, B.C. The geology of the Vancouver area is such that most of the softer soils suitable for cone penetration testing are deltaic sands, silts and clays or glaciomarine clays and silty clays. The glaciomarine clays have subsequently been leached and have sensitivities of up to 20. Details of the deltaic sediments are given by Campanella et al. (1983).

Table 1 summarizes index properties of fine grained soils from four of the main research sites near Vancouver, B.C. Undrained shear strength and sensitivity measurements were obtained using Nilcon and Geonor field vane equipment. The In-situ Testing Group at UBC has performed piezometer cone testing (CPTU) at many sites near Vancouver but data from these four sites will be used to illustrate the general conclusions. Data has also be reviewed from piezometer cone testing performed throughout the world to evaluate some of the points raised in this paper.

Soil Classification

Traditionally, soil classification from CPT data has been related to cone bearing, q_c, and friction ratio, FR = $(f_s \div q_T)100\%$. Several charts have been developed that use this basic CPT data. All the charts are similar in that sandy soils generally have high cone bearings and low friction ratios whereas, clayey soils generally have low cone bearings and high friction ratios. However, the measurement of sleeve friction is sometimes less accurate and reliable than the cone resistance. Also cones of different designs will often produce variable friction sleeve measurements. This can be caused by variations in mechanical and electrical design features of the friction sleeve as well as unequal end areas.

To overcome the problems associated with sleeve friction measurements, several classification charts have been proposed based on q_T and pore pressures (Jones and Rust, 1982; Baligh et al. 1980; Senneset and Janbu, 1984). The chart by Senneset and Janbu (1984) uses the pore pressure parameter ratio, B_q, defined as;

$$B_q = \frac{\Delta u}{q_T - \sigma_{vo}} \qquad (2)$$

where Δu = excess pore pressure
q_T = cone resistance corrected for pore pressure effects
σ_{vo} = total overburden stress.

The original chart by Senneset and Janbu (1984) uses q_c. However, it is generally agreed that the chart and B_q should use the corrected cone bearing, q_T. The correction is usually only significant in soft, fine grained soils where q_c can be small and Δu can be very large.

The chart proposed by Senneset and Janbu (1984) and modified to use q_T is shown in Fig. 2. The chart is based on pore pressures measured immediately behind the cone tip, as shown in Fig. 2. Recent experience has shown that the measured pore pressures are influenced by factors, such as, stress history, sensitivity and stiffness to strength ratio (G/s_u). Experience has also shown that it is possible to record pore pressures behind the tip that are less than the static equilibrium pressure (u_o) in some overconsolidated or dilative soils. Therefore Δu can be negative in some soils. The possibility of a negative Δu was incorporated into the classification chart proposed by Jones and Rust (1982), which is reproduced in Fig. 3. It is interesting to note that the slope of the boundaries defining the zones of various soil types in Fig. 3 are comprised of the same parameters used to define B_q.

The authors have found from their experience that it is not always possible to clearly identify a soil type based soley on the q_T and Δu data. Sometimes changes in the friction ratio have been able to more clearly define changes in soil type. Therefore, the authors recommend and use all three pieces of data (q_T, u, f_s) in the form of q_T, B_q and FR to define soil behaviour type. A first attempt at defining such a system is shown in Fig. 4.

The charts in Figure 4 can be used as a guide to define soil behaviour type based on CPTU data. Factors such as changes in, stress history, sensitivity, stiffness and void ratio will influence the classification using either the FR or the B_q chart. Occasionally soils will fall within different zones on each chart, in these cases judgment is required to correctly classify the soil behaviour type. Often the rate and manner in which the excess pore pressures dissipate during a pause in the cone penetration will aid in the classification. For example, a soil may have the following CPTU parameters; q_T = 10 bars (1 MPa), FR = 4%, B_q = 0.1. It would classify as a clay on the FR chart and as a clayey silt to silty clay on the B_q chart. However, if the rate of pore pressure dissipation were very slow this would add confidence to the classification of a clay. If the dissipation were rapid (t_{50} < 60 secs) the soil may be more like a clayey silt or possibly a clayey sand. The manner of the dissipation can

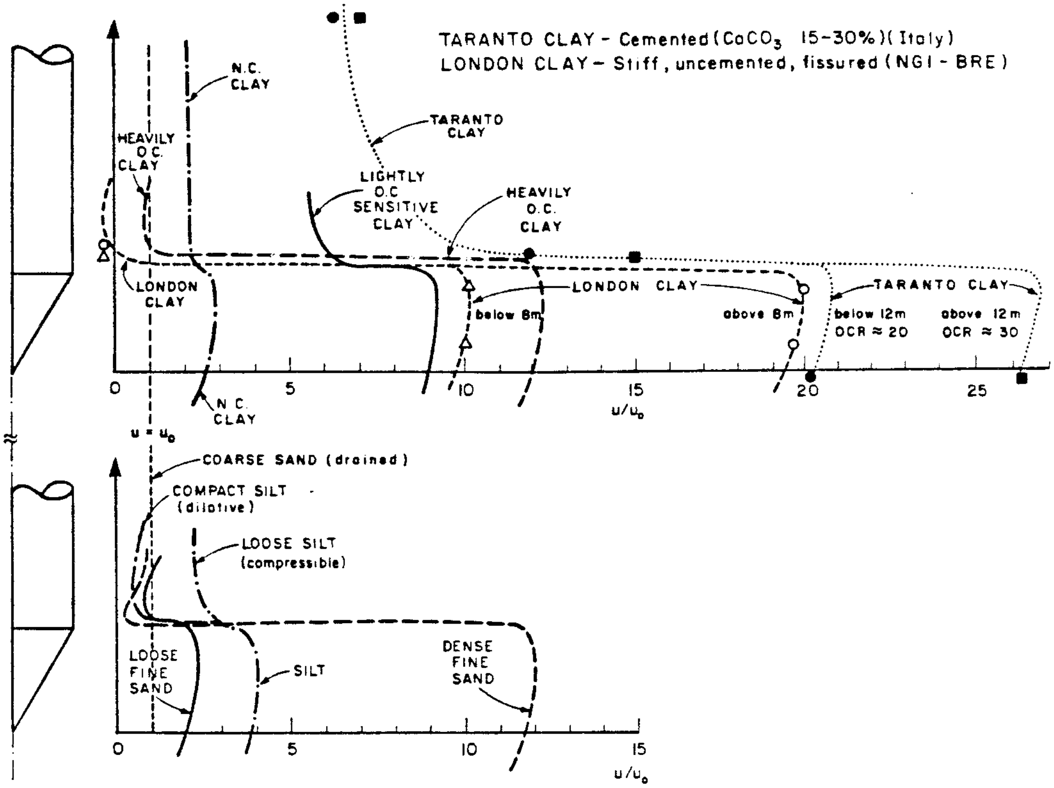


Fig. 1. Conceptual pore pressure distribution in saturated soil during CPTU based on field measurements.

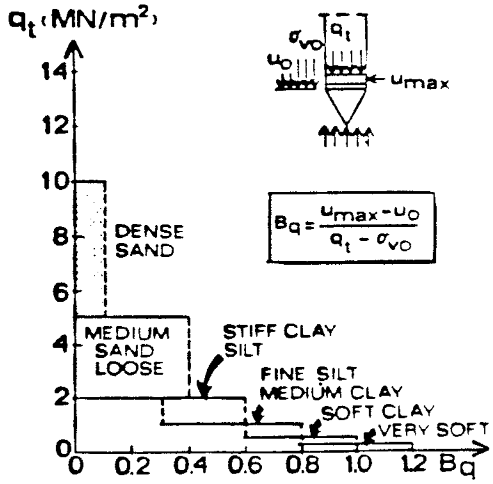


Fig. 2. Soil classification chart from CPTU data proposed by Senneset and Janbu (1984).

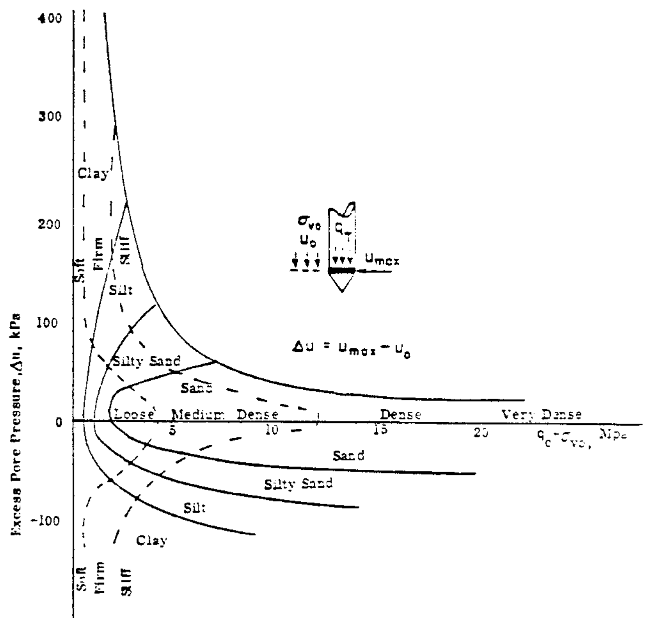
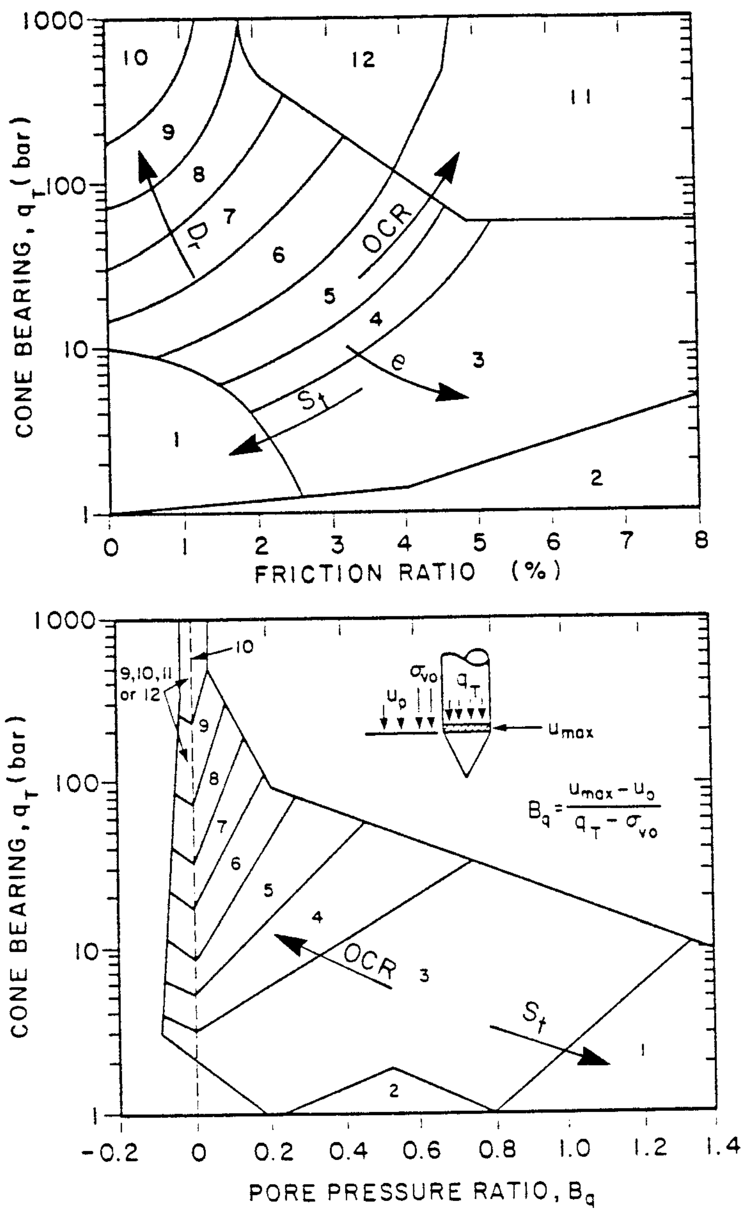


Fig. 3. Soil classification chart from excess pore pressure and cone resistance data proposed by Jones and Rust (1982).



Zone	Soil Behaviour Type
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained*
12	sand to clayey sand*

* overconsolidated or cemented.

Fig. 4. Proposed soil behaviour type classification system from CPTU data.

also be important. In stiff, overconsolidated clay soils, the pore pressure behind the tip can be very low in comparison to the high pore pore pressures on the face. When penetration is stopped, pore pressures recorded immediately behind the tip may initially rise before dropping to the equilibrium pressure. The rise is caused by local equilization of the high pore pressures on the nearby cone face.

A further problem associated with existing CPT classification charts is that soils can gradually change in their apparent classification as cone penetration increases in depth. This is due to the fact that q_T, u and f_s all tend to increase with increasing overburden pressure. For example, in a thick deposit of normally consolidated clay, the cone bearing will increase linearly with depth resulting in an apparent change in CPT classification. Existing classification charts are based predominantly on data obtained from CPT profiles extending to a depth of less than 30 m. Therefore, for CPT data obtained at depths significantly greater than 30 m some error can be expected when using the standard global CPT classification charts.

Attempts have been made to account for this by normalizing the cone data in sand with the effective overburden stress, σ'_{vo} (Robertson and Campanella 1985, Olsen, 1984; Douglas, 1985). However, it is not clear how CPT data in general should be normalized. Testing in large calibration chambers has shown that for sands the cone resistance can be normalized as follows,

$$q_{T1} = q_T(\sigma'_{vo})^{-n} \tag{3}$$

where q_{T1} = normalized cone bearing,
n = stress exponent.

Where n varies from 0.60 to 0.86 (Baldi et al. 1985) with an average of about 0.7. The chamber test results also show that the friction sleeve stress, f_s, can be normalized in a similar manner. However, for clays and clayey soils, the technique for normalization is less clear.

In theory, any normalization to account for increasing stress should also account for changes in horizontal stresses. This could be achieved by using a parameter such as the octahedral stress, σ'_m;

$$\text{where,} \quad \sigma'_m = \frac{1}{3}(\sigma'_1 + \sigma'_2 + \sigma'_3) \tag{4}$$

However, this has little practical benefit without a prior knowledge of the in-situ horizontal stresses.

Normalization of the CPT data would also avoid some of the problems associated with variations in q_T with soil density. At present, a very loose clean sand may be classified as a sandy silt to silty sand because of the low q_T.

Until a consistent method for normalization is adopted, the authors use and recommend the charts shown in Figure 4. However, some caution is suggested if the cone data extends beyond a depth of about 30 m below existing ground surface. The authors would also suggest that a general normalization using equation 3, where n = 0.7, should be adopted as a first attempt.

It is often important to realize that the classification charts are generalized global charts that provide a guide to soil behaviour type. The charts cannot be expected to provide accurate prediction of soil type for all soil conditions. However, in specific geological areas, the charts can be adjusted for local experience to provide excellent local correlations.

Undrained Shear Strength (S_u)

In the past, the undrained shear strength, S_u, has been estimated from the measured cone bearing, q_c, using the following bearing capacity equation;

$$S_u = \frac{q_c - \sigma_o}{N_k} \tag{4}$$

where N_k is an empirical cone factor and σ_o is generally taken to be the total overburden pressure (σ_{vo}). With the corrected cone resistance, q_T, the cone factor has been expressed (Lunne et al. 1985) as:

$$N_{KT} = \frac{q_T - \sigma_{vo}}{S_u} \tag{5}$$

Figure 5 shows how the N_{KT} factor varies with B_q for the soils in the Vancouver area. Clearly, no clear correlation exists between N_{KT} and B_q.

Senneset et al. (1982) have suggested the use of the effective cone resistance, q_E, to determine S_u. Where q_E is defined as follows;

$$q_E = q_c - u \tag{6}$$

and u = total pore pressure measured immediately behind the cone tip.

Robertson and Campanella (1983) redefined the effective cone bearing using the corrected cone resistance, q_T. The undrained shear strength can then be determined as follows,

$$S_u = \frac{q_E}{N_{KE}} = \frac{q_T - u}{N_{KE}} \tag{7}$$

Senneset et al. (1982) proposed that $N_{KE} = 9$ with a likely variation of ± 3. Lunne et al. (1985) showed that N_{KE} varied from 2 to 12 and appeared to correlate with B_q.

Figure 6 shows N_{KE} and B_q data obtained from the Vancouver area. Also included in Fig. 6 is the range of data obtained by Lunne et al. (1985) from 5 sites in the North Sea. Data from the Vancouver area show the same basic trend as that from the North Sea but with considerably more scatter. It is interesting to note that the data presented by Lunne et al. (1985) incorporates q_T values based on pore pressures measured on the face of the cone tip but reduced by 25% to obtain the assumed pore pressures

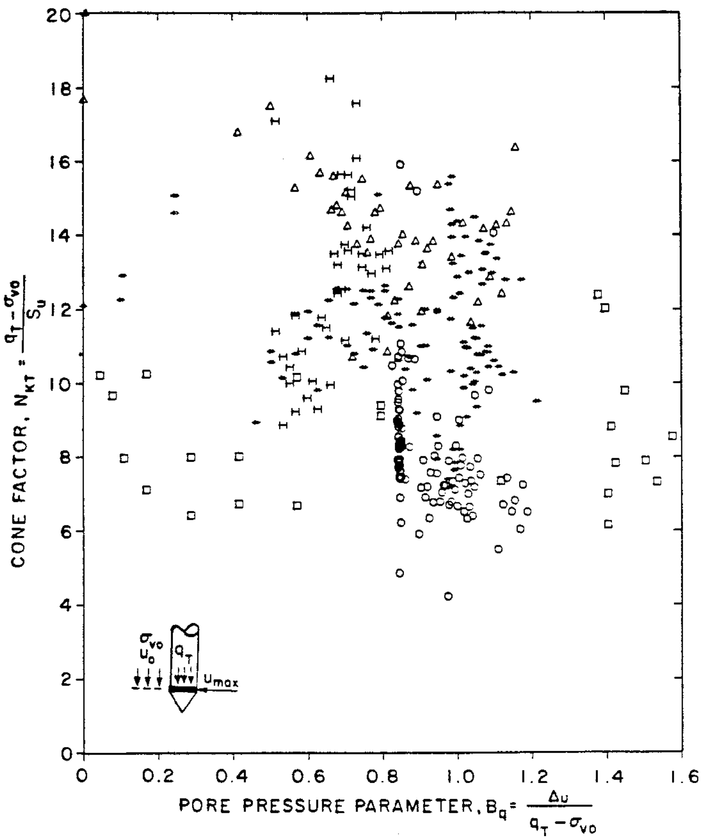


Fig. 5. Cone factor N_{KT} versus pore pressure parameter B_q.

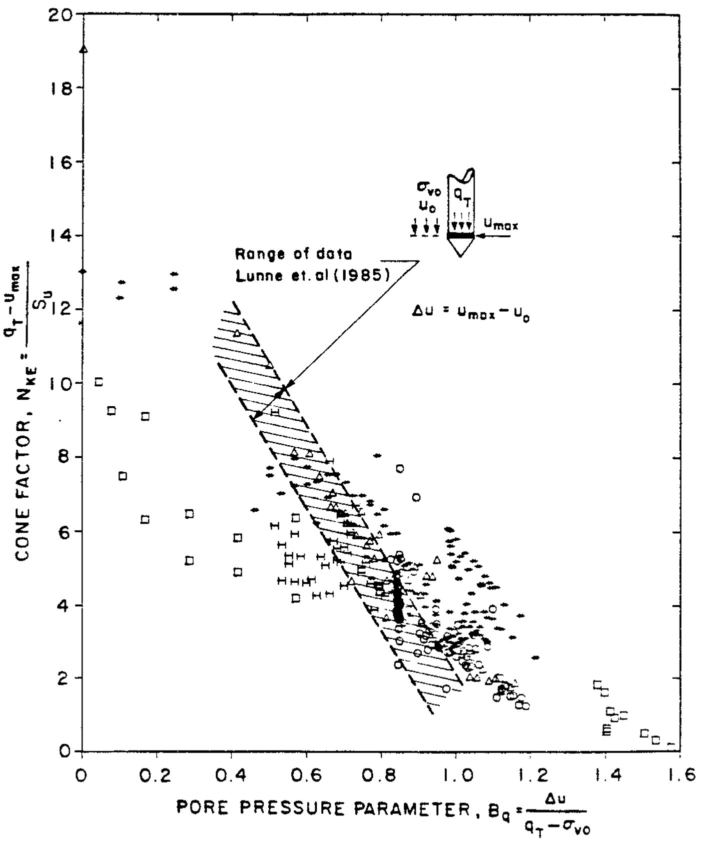


Fig. 6. Cone factor N_{KE} versus pore pressure parameter B_q.

immediately behind the tip. Based on Fig. 1, this may have introduced some error.

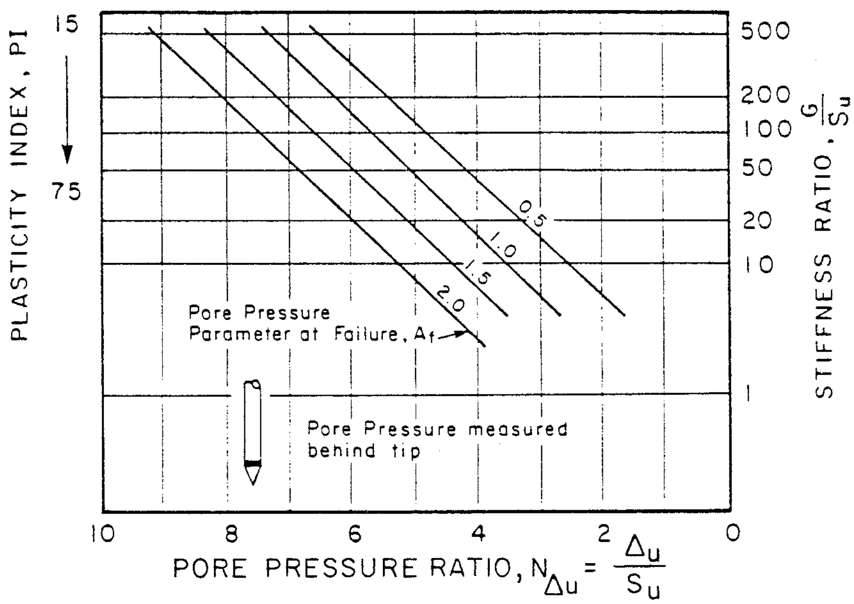
One major drawback using the effective cone resistance, q_E, is the reliability to which q_E can be determined. In soft normally consolidated clays, the total pore pressure, u, generated immediately behind the tip during cone penetration is often approximately 90 percent or more of the measured cone resistance, q_c. Even when q_c is corrected to q_T, the difference between q_T and u is often very small. Thus, q_E is often an extremely small quantity and is therefore sensitive to small errors in q_c measurements.

The problems of accuracy and pore pressure effects associated with the measured cone resistance in soft clays may explain some of the large scatter in published data concerning the cone factors N_K and N_{KE}. An alternative approach for the determination of S_u was suggested by Campanella et al. (1985) using the excess pore pressure (Δu) generated during cone penetration. In soft clays, the cone resistance can be very small and typically the cone tip load cell may be required to record loads less than 1% of rated capacity with an associated inaccuracy of up to 50% of the measured values. However, in soft clays, the pore pressures generated can be very large and the pressure transducer may record pressures up to 80% of its rated capacity with an associated accuracy of better than 1% of the measured value. Therefore, estimates of S_u in soft clays will inherently be more accurate using pore pressure data, as opposed to the tip resistance.

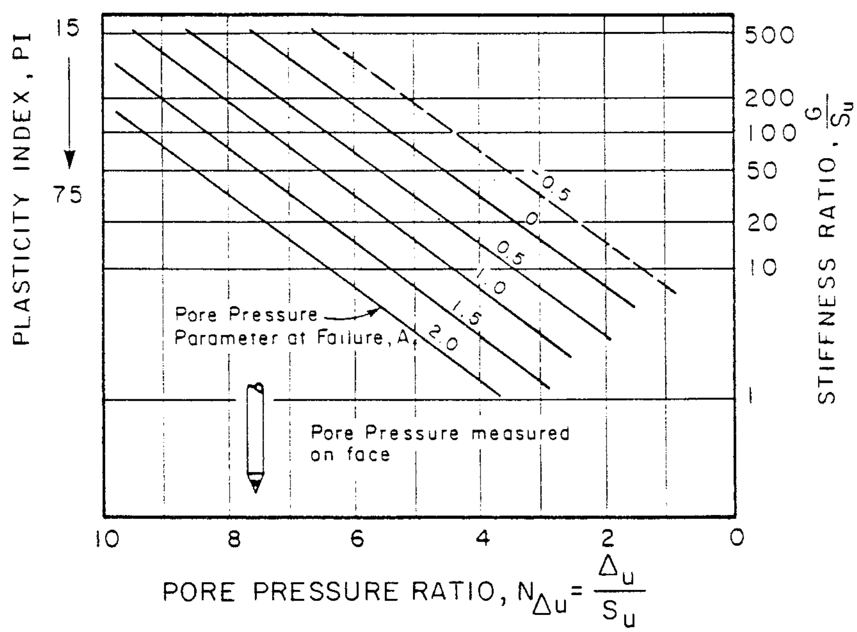
The excess pore pressures generated during cone penetration into fine grained soils will be dependent on the stress history, sensitivity and stiffness ratio. Low values of stiffness ratio generally apply to highly plastic clays (plasticity index, PI $>$ 80) which tend to generate low pore pressures. High values of stiffness ratio generally apply to low plastic clays and silts (PI $\leq$ 15) which tend to generate high pore pressures. The excess pore pressures also tend to increase with increasing soil sensitivity and decrease with increasing overconsolidation ratio (stress history).

A semi-empirical solution was proposed by Massarch and Broms (1981) based on cavity expansion theories but including the effects of overconsolidation and sensitivity by using Skempton's pore pressure parameter at failure (A_f). Charts illustrating this approach are given in Fig. 7. Approximate values for A_f can be estimated from Table 2. Clearly a knowledge of the shear modulus (G) or plasticity index (PI) would assist in the estimate of S_u. The addition of shear wave velocity measurements during seismic CPTU is a promising method to obtain an independent measure of the shear modulus.

If pore pressures are measured immediately behind the cone tip, the measured values may not have reached the true cylindrical cavity expansion value. Therefore S_u estimated from the chart with the pore pressures behind the tip may be slightly over estimated. Also because of the tendency for low or negative pore pressures measured behind the tip in insensitive, overconsolidated clays (see Fig. 1), the chart in Fig. 7 is not recommended for highly overconsolidated clays ($-0.5 < A_f < 0$).



(a) Pore pressures measured behind tip.



(b) Pore pressures measured on face of tip.

Fig. 7. Proposed charts to obtain S_u from excess pore pressure measured during CPTU. (After Campanella et al., 1985)

Table 1. Summary of Soil Index Parameters for Fine Grained Soils from Four Sites Near Vancouver, B.C.

Site	Water Content w, %	Liquid Limit w_L, %	Plasticity Index PI, %	Sensitivity S_t	Overconsolidation Ratio, OCR
McDonald's Farm	23-40	25-42	3-20	2-7	1-2
B.C. Hydro Rail	27-53	32-59	16-34	7-10	1-3
232nd Street	42-47	40	19	2-19	1-10
Haney	40-45	44	18	3-13	2-10

Table 2. Estimates of Skempton's Pore Pressure Parameter, A_f.

Saturated Clays	A_f
Very sensitive to quick	1.5-3.0
Normally consolidated	0.7-1.3
Lightly consolidated	0.3-0.7
Highly consolidated	-0.5-0.0

Although the charts in Fig. 7 are based on cavity expansion theories, they are basically semi-empirical in nature. The advantage in using the charts is that they provide some rational guide to the correct selection of the cone factor, $N_{\Delta u}$. The charts clearly show how the factor $N_{\Delta u}$ will vary with OCR, sensitivity and stiffness.

Figure 8 presents data from the Vancouver area showing how the cone factor $N_{\Delta u}$ varies with the pore pressure parameter B_q. Also included on Fig. 8 is the range of North Sea data presented by Lunne et al. (1985). Again the Vancouver area data show the same basic trend but with more scatter. When the data presented in Fig. 8 is reviewed a little more closely, it is apparent that trends in the data can be defined. Soils with high OCR have low B_q and low $N_{\Delta u}$ values. Soils with the same OCR but increasing sensitivity (S_t) show a marked increase in $N_{\Delta u}$ and a smaller increase in B_q. Unfortunately, the data shown is for soils of predominantly similar plasticity index (PI) and no clear trend with changing PI can be seen. The trend lines for OCR and S_t have been included on Fig. 8 as a guide. These same trends in $N_{\Delta u}$ can be obtained from the chart shown in Fig. 7(a). The data shown in Fig. 8 would indicate that increasing sensitivity (S_t) can increase $N_{\Delta u}$ to as high as 18, compared to the maximum value of about 10 shown in Fig. 7(a).

Based on the data shown in Figs. 7 and 8, the authors suggest the following method for determining the undrained shear strength from CPTU data;

1. Using the CPTU profile estimate the OCR and sensitivity (S_t). Methods to estimate OCR and S_t are discussed in a later section.

2. Estimate appropriate value of A_f from Table 2.

3. Use Fig. 7 to estimate $N_{\Delta u}$.

4. Calculate B_q and use Fig. 8 to estimate $N_{\Delta u}$, again use estimated OCR and S_t.

5. Compare $N_{\Delta u}$ values from Figs. 7 and 8 and use average value to calculate S_u.

6. Using calculated value of S_u re-evaluate OCR using S_u/σ'_{vo} (Robertson and Campanella, 1983).

7. Iterate from 1 to 6 until consistent value of S_u is derived.

The data presented in Figs. 5 to 8 clearly show that no simple unique relationship exists between CPTU data and undrained shear strength, S_u, for all clay type soils. Therefore procedures, such as outlined above, are required to more realistically evaluate S_u for all possible clay soils. However, simple relationships are possible for site specific soils, as shown on Figs. 5, 6 and 8.

Stress History

Several methods have been suggested to correlate pore pressure parameters, such as B_q, to OCR. However, Robertson and Campanella (1983) postulated that any relationship between pore pressure and OCR would be influenced by variations in soil stiffness and sensitivity since the excess pore pressure is also a function of stiffness ratio and sensitivity.

Fig. 9 presents the pore pressure parameter B_q against the best estimate of the in-situ overconsolidation ratio (OCR). Generally, the OCR has been estimated from relationships between undrained shear strength, S_u, plasticity index, PI, and OCR. Admittedly, there are significant uncertainties in the values of OCR determined this way. However, the data shown in Fig. 9 show that B_q generally decreases with increasing OCR. Also for the same OCR, B_q increases with increasing sensitivity, S_t. Again the trends and variation in data shown in Fig. 9 could have been predicted using the chart in Fig. 7a.

Figure 9 clearly shows that there is no unique correlation between B_q and OCR for all clay soils. However, reasonable estimates of OCR can be made using B_q if the influence of sensitivity and stiffness are taken into account. Fig. 9 can therefore be used to estimate OCR and then applied to the estimate of S_u, as discussed in the previous section.

Unfortunately, all the data shown in Fig. 9 and presented by Lunne et al. (1985) was for soils with low plasticity (PI < 30%). Therefore, the influence of increasing plasticity is unclear. However, review of isolated data from other areas in the world do indicate that increases in PI will cause slight decreases in B_q.

Sensitivity

The sensitivity (S_t) of a clay which is the ratio of undisturbed strength to totally remolded strength can be estimated from the friction ratio (FR%) using,

$$S_t = \frac{N_s}{FR\%}$$

Schmertmann (1978) suggested a value of $N_s = 15$ for mechanical CPT data. Robertson and Campanella (1983) tentatively suggested $N_s = 10$ for electronic CPT data. Recent data (Greig, 1985) collected in the Vancouver area suggest an average of $N_s = 6$. The authors therefore suggest using an average N_s of 6 for an initial estimate of S_t.

It has been recognised for many years that the sleeve friction stress, f_s, is approximately equal to the remolded undrained shear strength, S_{ur}. Fig. 10 presents a summary of the sleeve friction measurements, f_s, and the remolded undrained shear strength, S_{ur} for the soils in the Vancouver area. The remolded strength was obtained from field vane shear tests. Data presented in Fig. 10 show that the friction sleeve stress is generally close to the remolded strength. However, the friction sleeve values are very small and the variation in results are probably due to the inherent difficulty of measuring small sleeve frictions. The data shown in Fig. 10 was obtained using the UBC tension type cones with equal end area friction sleeves.

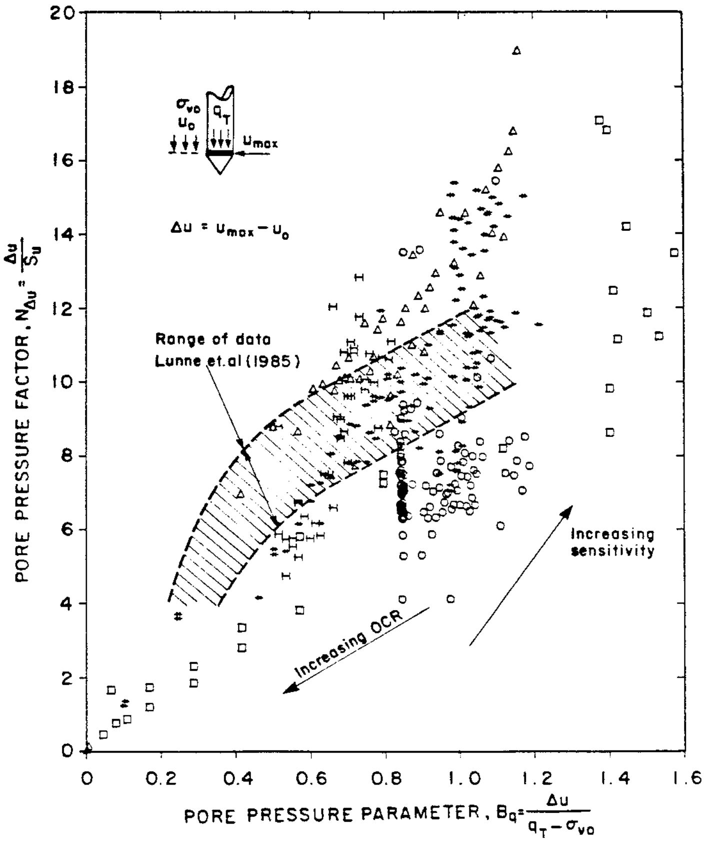


Fig. 8. Pore pressure factor $N_{\Delta u}$ versus pore pressure parameter B_q.

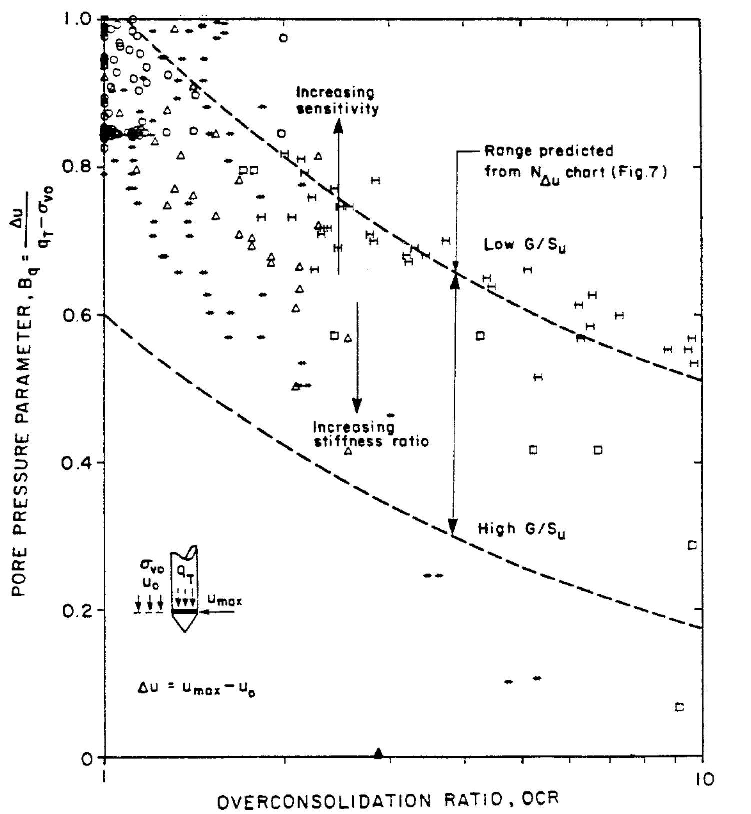


Fig. 9. Pore pressure parameter B_q versus overconsolidation ratio, OCR.

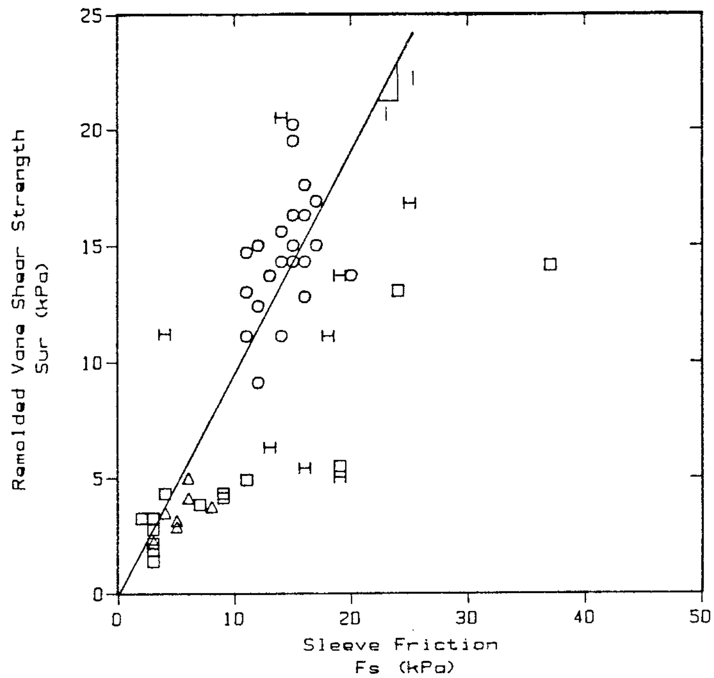


Fig. 10. Field vane remolded undrained shear strength versus CPT sleeve friction stress.

The observation that soils with a high sensitivity have very low sleeve frictions is also reflected in the FR classification chart in Fig. 4.

Summary

Experience with interpretation and use of CPTU data from the Vancouver area has been presented. Cone readings have been corrected for pore pressure effects using the pore pressure measured immediately behind the tip. For soil classification, it is recommended that all cone data (q_T, f_s, u) plus pore pressure dissipation data be used to define soil behaviour type. For deep cone soundings, the cone data should be normalized to account for the affect of increasing overburden pressure.

For the determination of undrained shear strength from CPTU data, the cone factors N_{KE} and $N_{\Delta u}$ vary systematically with the pore pressure parameter B_q. However, the estimate of S_u is strongly influenced by variables such as stress history, sensitivity and stiffness. There appears to be no unique relationship between CPTU data and S_u for all soil types. An iterative approach has been recommended that provides a rational guide to the selection of the cone factor $N_{\Delta u}$.

Essential to the estimate of S_u from cone data is the confidence and accuracy in the CPTU data. It is important to understand the limitations of the instrument and procedures. Use of the pore pressure data can be subject to problems in saturation procedures. Also use of the pore pressure parameter B_q can be difficult at shallow depth and close to the groundwater level where Δu may be very small and therefore less accurate than at greater depth.

No unique relationship exists between B_q and OCR for all clays since sensitivity has a significant influence on the measured pore pressure.

The remolded undrained shear strength can be assumed to be close to the friction sleeve stress for most clay soils.

The dissipation data obtained during pauses in the cone penetration can be used to improve soil classification and to provide an index on the soils permeability and consolidation characteristics.

New combined instruments like the seismic CPTU can provide useful data to assist in the selection of stiffness ratio (G/S_u) for determination of S_u.

Acknowledgements

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