Soil classification using the cone penetration test

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Several charts exist for evaluating soil type from electric cone penetration test (CPT) data. A new system is proposed based on normalized CPT data. The new charts are based on extensive data available from published and unpublished experience worldwide. The new charts are evaluated using data from a 300 m deep borehole with wire-line CPT. Good agreement was obtained between samples and the CPT data using the new normalized charts. Recommendations are provided concerning the location at which to measure pore pressures during cone penetration.

Key words: soil classification, cone penetration test, *in situ*, case history.

Il existe plusieurs abaques pour identifier le type de sol en partant des données d'essais de pénétration au cône (« CPT »). L'on propose un nouveau système basé sur des données CPT normalisées. Les nouveaux abaques sont établis en partant d'une quantité importante de données provenant de l'expérience publiée et non publiée à travers le monde. Les nouveaux abaques ont été vérifiés en utilisant les données obtenues dans un forage de 300 m de profondeur avec un CPT à câble. Une bonne concordance a été obtenue entre les échantillons et les données de CPT utilisant les nouveaux abaques. L'on présente des recommandations quant à la position du point de mesure de la pression interstitielle durant la pénétration au cône.

Mots clés : classification du sol, essai de pénétration au cône, *in situ*, histoire de cas.

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Introduction

One of the primary applications of the cone penetration test (CPT) is for stratigraphic profiling. Considerable experience exists concerning the identification and classification of soil types from CPT data. Several soil classification charts exist for CPT and for cone penetration testing with pore pressure measurements (CPTU).

In this paper the limitations of existing CPT and CPTU classification charts are discussed and a new system is proposed based on normalized measurements. A discussion is also presented regarding the recommended position of measurement of pore pressure during cone penetration.

Soil classification

Some of the most comprehensive recent work on soil classification using electric cone penetrometer data was presented by Douglas and Olsen (1981). One important distinc-

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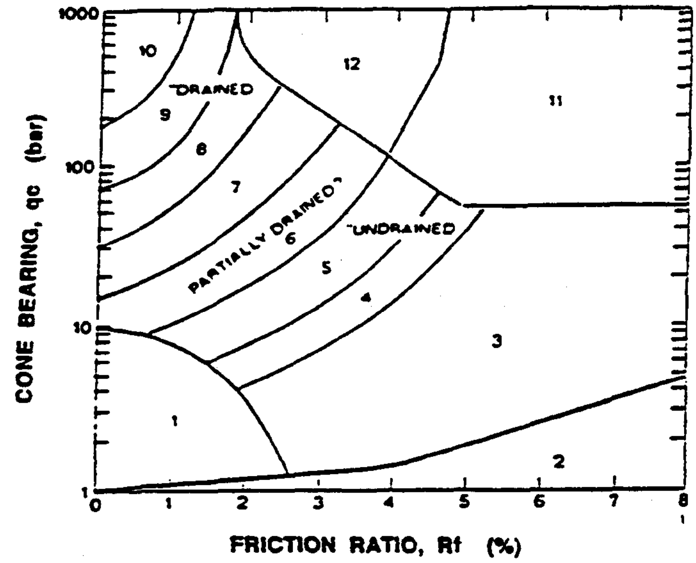

ZONE	SOIL BEHAVIOUR TYPE
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

(*) overconsolidated or cemented

FIG. 1. Simplified soil behaviour type classification for standard electric friction cone (Robertson *et al.* 1986). 1 bar = 100 kPa.

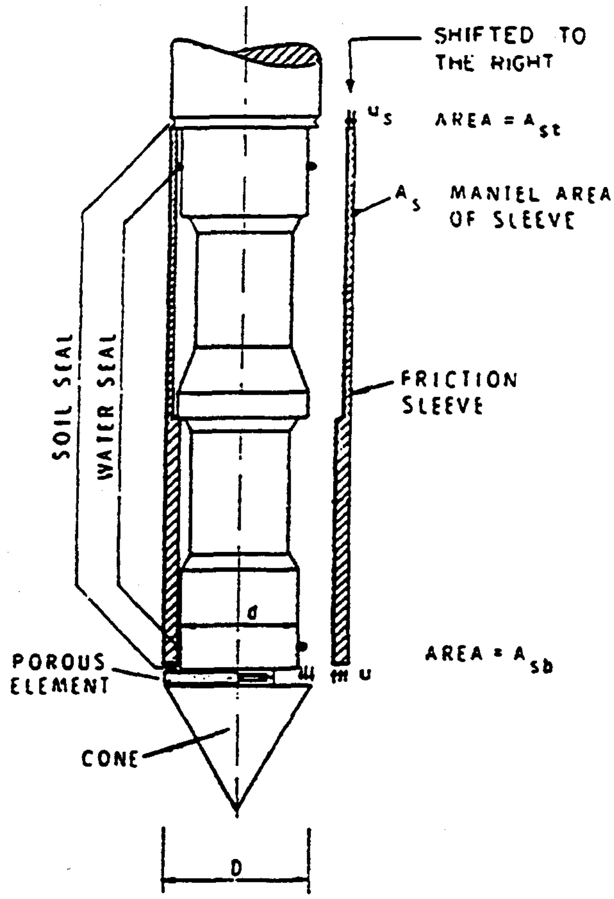

FIG. 2. Schematic representation of piezo-friction-cone penetrometer (adapted from Konrad 1987).

tion made by them was that CPT classification charts cannot be expected to provide accurate predictions of soil type based on grain size distribution but can provide a guide to soil behaviour type. The CPT data provide a repeatable index of the aggregate behaviour of the *in situ* soil in the immediate area of the probe.

In recent years soil classification charts have been adapted and improved from an expanded data base (Robertson 1986; Olsen and Farr 1986). An example of such a soil classification chart for electric CPT data is shown in Fig. 1. The chart in Fig. 1 is based on data obtained predominantly at depths less than 30 m and is global in nature. Therefore, some overlap in zones should be expected.

Most classification charts, such as the one shown in Fig. 1, use the cone penetration ressitance, q_c, and friction ratio, R_f, where

$$[1] \quad R_f = \frac{f_s}{q_c} \times 100\%$$

f_s is sleeve friction.

Recent research has illustrated the importance of cone design and the effect that water pressures have on the measured penetration resistance and sleeve friction because of unequal end areas (Campanella *et al.* 1982; Baligh *et al.* 1981). Thus, cones of slightly different designs, but conforming to the international standard (ISSMFE 1977) and reference test procedure (ISOPT 1988), will give slightly different values of q_c and f_s, especially in soft clays and silts.

For electric cones that record pore pressures (Fig. 2), corrections can be made to account for unequal end area effects. Baligh *et al.* (1981) and Campanella *et al.* (1982) proposed that the cone resistance, q_c, could be corrected to a total cone resistance, q_t, using the following expression:

$$[2] \quad q_t = q_c + (1 - a)u$$

where u is pore pressure measured between the cone tip and the friction sleeve and a is net area ratio.

It is often assumed that the net area ratio is given by

$$[3] \quad a = \frac{d^2}{D^2}$$

where d is diameter of load cell support and D is diameter of cone. However, this provides only an approximation of the net area ratio, since additional friction forces are developed due to distortion of the water seal O-ring. Therefore, it is recommended that the net area ratio should always be determined in a small calibration vessel (Battaglio and Maniscalco 1983; Campanella and Robertson 1988).

A similar correction can also be applied to the sleeve friction (Lunne *et al.* 1986; Konrad 1987). Konrad (1987) suggested the following expression for the total stress sleeve friction, f_t:

$$[4] \quad f_t = f_s - (1 - \beta b)cu$$

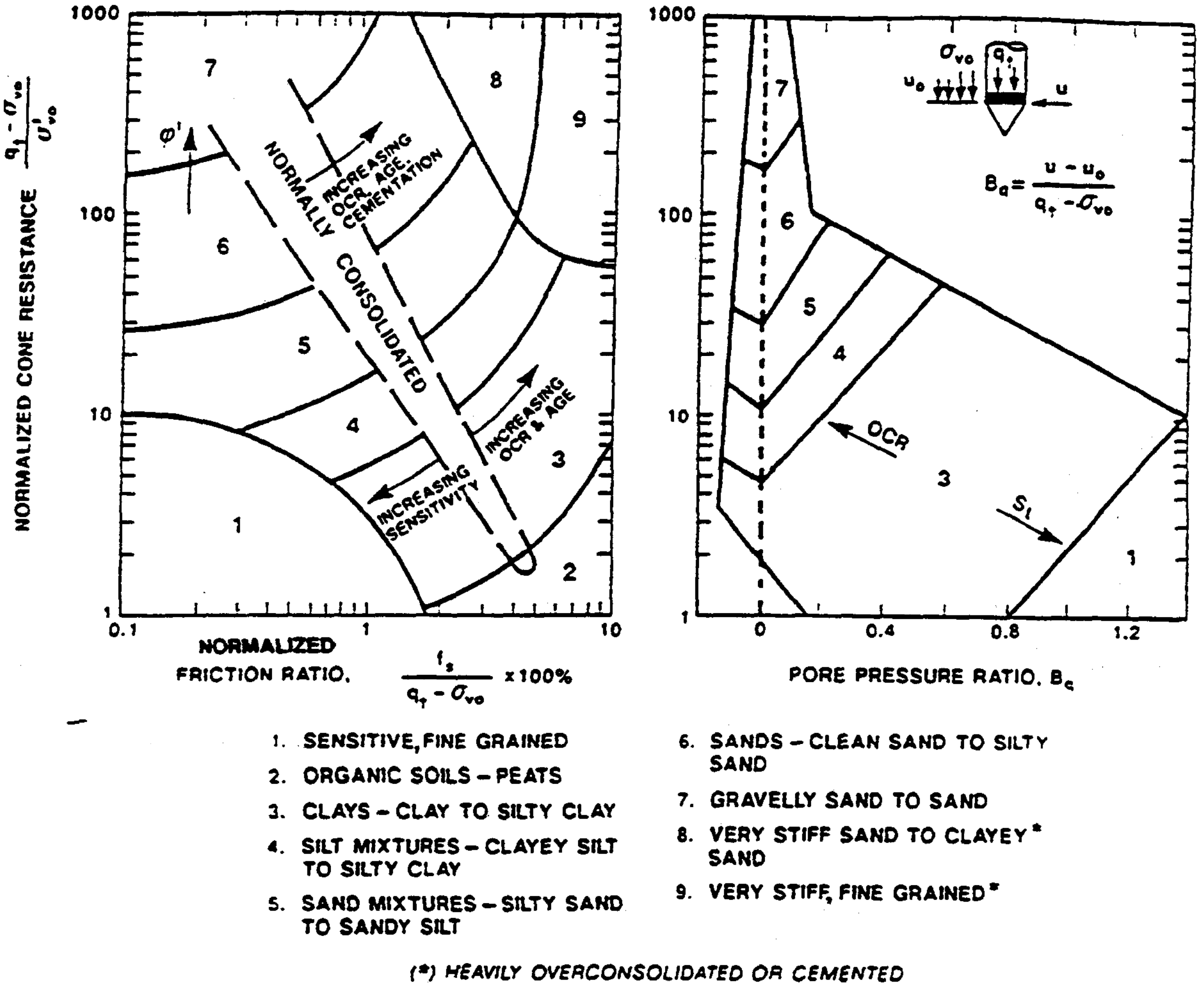

FIG. 3. Proposed soil behaviour type classification chart based on normalized CPT and CPTU data.

where

$$b = \frac{A_{st}}{A_{sb}}; \qquad c = \frac{A_{sb}}{A_s}; \qquad \beta = \frac{u_s}{u}$$

A_{st} is end area of friction sleeve at top, A_{sb} is end area of friction sleeve at bottom, A_s is outside surface area of friction sleeve, and u_s is pore pressure at top of friction sleeve.

However, to apply this correction, pore pressure data are required at both ends of the friction sleeve. Konrad (1987) showed that this correction could be more than 30% of the measured f_s for some cones. However, the correction can be significantly reduced for cones with an equal end area friction sleeve (i.e., $b = 1.0$).

The corrections in [2] and [4] are only important in soft clays and silts where high pore pressure and low cone resistance occur. The corrections are negligible in cohesionless soils where penetration is generally drained and cone resistance is generally large. The author believes that the correction to the sleeve friction is generally unnecessary provided the cone has an equal end area friction sleeve. Hence, classification charts use uncorrected f_s.

Recent studies have shown that even with careful procedures and corrections for pore pressure effects the measurement of sleeve friction is often less accurate and reliable than that of tip resistance (Lunne *et al.* 1986; Gillespie 1989). Cones of different designs will often produce variable friction sleeve measurements. This can be caused by small variations in mechanical and electrical design features as well as small variations in tolerances.

To overcome 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

$$[5] \quad B_q = \frac{u - u_0}{q_t - \sigma_{vo}}$$

where u is pore pressure measured between the cone tip and the friction sleeve, u_0 is equilibrium pore pressure, and σ_{vo} is 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 q_t.

Experience has shown that, although the sleeve friction measurements are not as accurate as q_t and u, generally more reliable soil classification can be made using all three pieces of data (i.e., q_t, f_s, and u). A first attempt at defining a system that uses all three pieces of data was proposed by Robertson *et al.* (1986) and used q_t, B_q, and R_f.

Normalized CPT data

A problem that has been recognized for some time with soil classification charts that use q_t and R_f is that soils can change in their apparent classification as cone penetration

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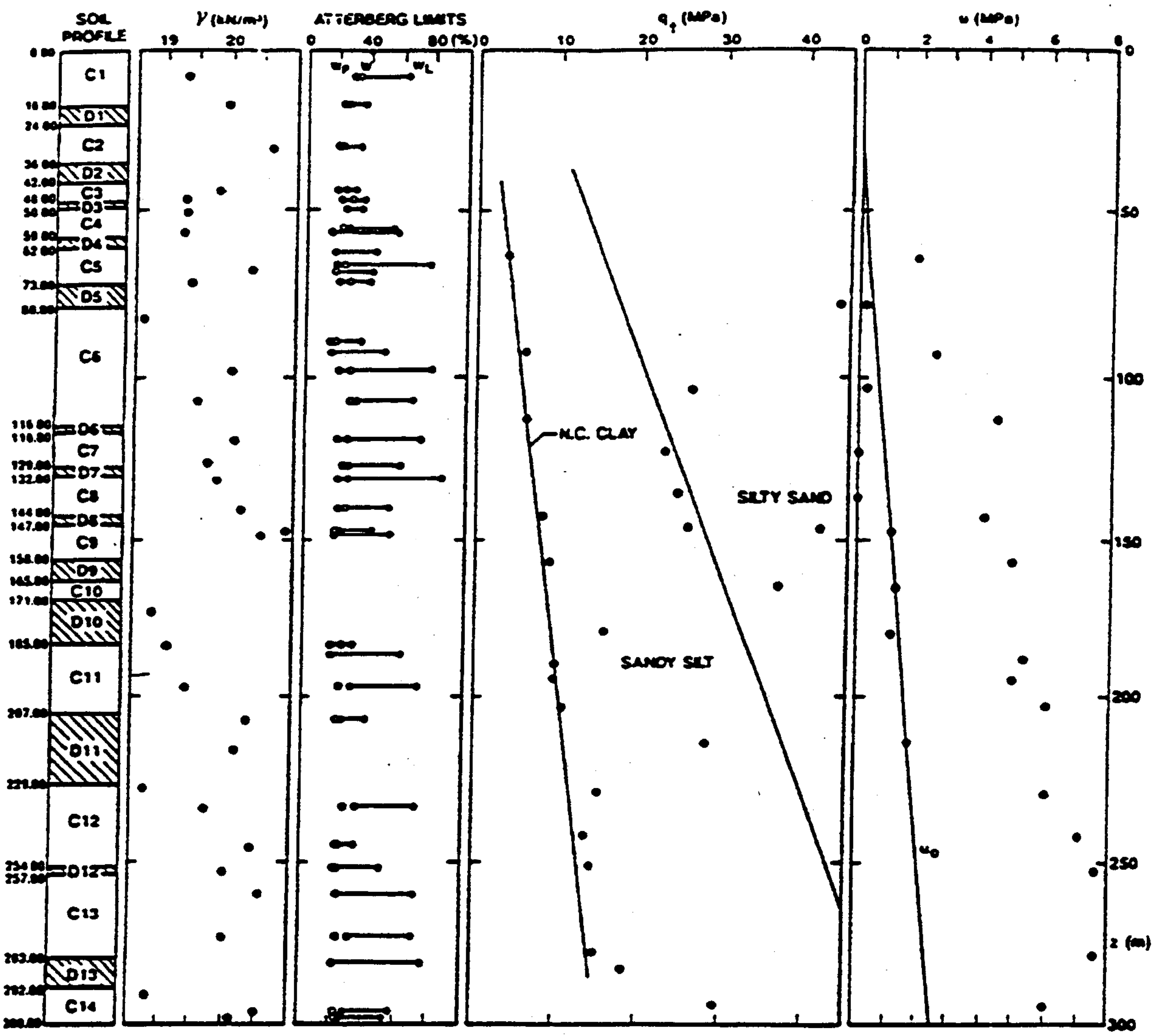

FIG. 4. Summary of soil profile and geotechnical characteristics from 300 m deep borehole (after Belfiore *et al.* 1989).

resistance increases with increasing depth. This is due to the fact that q_t, f_s, and u all tend to increase with increasing overburden stress. For example, in a thick deposit of normally consolidated clay the cone resistance, q_c, will increase linearly with depth, resulting in an apparent change in CPT classification for large changes in depth. 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 significantly greater depths, some error can be expected using existing CPT classification charts that are based on q_t (or q_c) and R_f.

Attempts have been made to account for the influence of overburden stress by normalizing the cone data (Olsen 1984; Douglas *et al.* 1985; Olsen and Farr 1986). These existing approaches require different normalization methods for different soil types, which produces a somewhat complex iterative interpretation procedure that requires a computer program.

Conceptually, any normalization to account for increasing stress should also account for changes in horizontal stresses, since penetration resistance is influenced in a major way by the horizontal effective stresses (Jamiolkowski and Robertson 1988). However, at present, without prior detailed knowledge of the *in situ* horizontal stresses, this has little practical benefit. Even normalization using only vertical effective stress requires some input of soil unit weights and groundwater conditions.

Wroth (1984) and Houlsby (1988) suggested that CPT data should be normalized using the following parameters:

(1) Normalized cone resistance:

$$[6] \quad Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$$

(2) Normalized friction ratio:

$$[7] \quad F_R = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

(3) Pore pressure ratio:

$$[8] \quad B_q = \frac{u - u_0}{q_t - \sigma_{vo}} = \frac{\Delta u}{q_t - \sigma_{vo}}$$

Using these normalized parameters and the extensive CPTU data base now available in published and unpublished sources, modified soil behaviour type classification charts have been developed and are shown in Fig. 3.

The two charts shown in Fig. 3 represent a three-dimensional classification system that incorporates all three pieces of CPTU data. For basic CPT data where only q_c

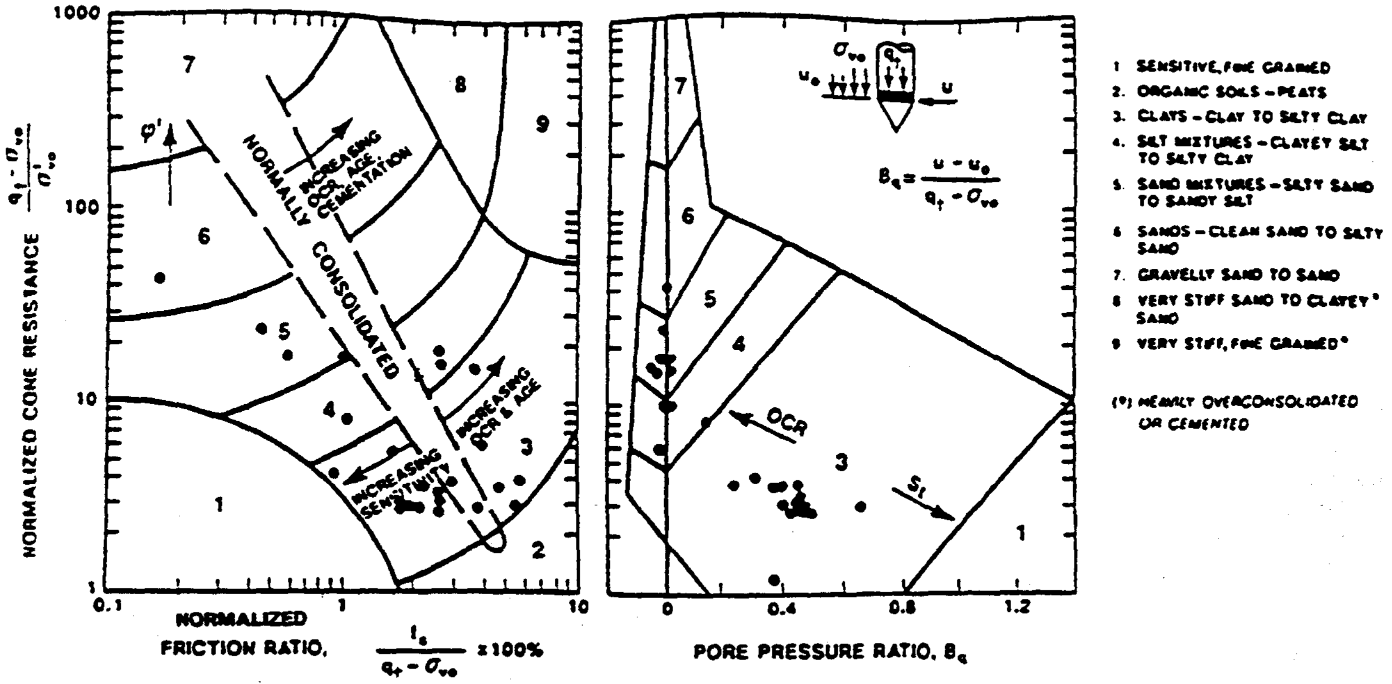

FIG. 5. CPT and CPTU data from the deep borehole plotted on the proposed normalized soil behaviour type classification charts.

and f_s are available, the left-hand chart (Fig. 3) can be used. The error in using uncorrected q_c data will generally only influence the data in the lower part of the chart where normalized cone resistance is less than about 10. This part of the chart is for soft, fine-grained soils where q_c can be small and u can be large.

Included in the normalized soil behaviour type classification charts is a zone that represents approximately normally consolidated soil behaviour. A guide is also provided to indicate the variation of normalized CPT and CPTU data for changes in (1) overconsolidation ratio (OCR), age, and sensitivity (S_t) for fined-grained soils, where cone penetration is generally undrained, and (2) OCR, age, cementation, and friction angle (ϕ') for cohesionless soils, where cone penetration is generally drained.

Generally, soils that fall in zones 6 and 7 represent approximately drained penetration, whereas soils in zones 1, 2, 3, and 4 represent approximately undrained penetration. Soils in zones 5, 8, and 9 may represent partially drained penetration. An advantage of measuring pore pressures during cone penetration is the ability to evaluate drainage conditions more directly.

The charts in Fig. 3 are still global in nature and should be used as a guide for defining soil behaviour type based on CPT and CPTU data. Factors such as changes in stress history, *in situ* stresses, sensitivity, stiffness, macrofabric, and void ratio will also influence the classification.

Occasionally, soils will fall within different zones in each chart; in these cases judgement is required to correctly classify the soil behaviour type. Often, the rate and manner in which the excess pore pressure dissipates during a pause in the cone penetration will significantly aid in the classification. For example, a soil may have the following CPTU parameters: $q_t = 0.9$ MPa, $f_s = 40$ kPa, and $\Delta u = 72$ kPa at a depth where $\sigma_{vo} = 180$ kPa and $\sigma'_{vo} = 90$ kPa. Hence, the normalized CPTU parameters are $Q_t = (q_t - \sigma_{vo})/\sigma'_{vo} = 8$, $F_R = [f_s/(q_t - \sigma_{vo})] \times 100 = 5.6\%$, and $B_q = \Delta u/(q_t - \sigma_{vo}) = 0.1$. Using these normalized parameters the soil would be classified as a slightly overconsolidated clay (clay to silty clay) on the normalized friction ratio chart and as a silt mixture (clayey silt to silty clay) on the normalized pore pressure ratio chart. However, if the rate of pore pressure dissipation during a pause in penetration were very slow, this would add confidence to the classification as a clay. If the dissipation were more rapid, say 50% dissipation in 2–4 min (2 min $< t_{50}$ $<$ 4 min), the soil is more likely to be a clayey silt.

The manner in which the dissipation occurs can also be important. In stiff, overconsolidated clay soils, the pore pressure behind the tip can be very low and sometimes less than the equilibrium pore pressure, u_0, whereas on the face of the cone the pore pressure can be very large due to the large increase in normal stresses created by the cone penetration. When penetration is stopped in overconsolidated clays, pore pressures recorded behind the tip may initially increase before decreasing to the equilibrium pore pressure. The rise can be caused by local equalization of the high pore pressure gradient around the cone tip (Campanella *et al.* 1986).

Case history

To illustrate the advantage of using normalized data, a case history involving a deep borehole with wire-line CPT will be briefly presented. The deep (300 m) borehole was performed as part of a research program to study the land subsidence of Bologna in Italy (Belfiore *et al.* 1989). A hydraulic drill rig equipped with a wire-line system was used for sampling and cone penetration testing. During the boring 30 undisturbed samples were taken and 27 static penetration tests were performed, using both electric CPT and CPTU. At suitable elevations, dissipation tests were carried out with the CPTU to measure equilibrium pore pressures and the rate of dissipation of the excess pore pressures. Geophysical data were also obtained, including electrical, seismic, and radioactivity logs. Full details of the test program are given by Belfiore *et al.* (1989).

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A summary of the soil profile and the CPTU data are presented in Fig. 4. From the results of the boring, a total of 14 well-defined compressible layers were identified and are marked by a C in Fig. 4. The compressible layers consist of approximately normally consolidated clayey silt and silty clay, of medium to high plasticity. A total of 13 cohesionless drainage layers were also identified and marked by a D in Fig. 4.

It can be seen from Fig. 4 that the points of minimum q_t represent the compressible layers and lie approximately on a straight line corresponding to a normalized cone resistance of about 2.8. The corrected q_t range from 3.7 MPa (37 bars) to 15 MPa (150 bars) at depths of about 65–280 m. The calculated friction ratio values (R_f) vary from 3.3 to 1.3%. Hence, the predicted soil behaviour type using the classification chart in Fig. 1 would change with increasing depth from a clayey silt to a sand. However, using normalized cone data and the proposed normalized charts, the compressible layers (C) are more correctly classified as a clay soil behaviour type throughout the depth range investigated. A summary of the CPT and CPTU data from the deep borehole plotted on the normalized charts is shown in Fig. 5.

It is interesting to note that the excess pore pressures during cone penetration ($\Delta u = u - u_0$) have high positive values in clay layers, negative values in silty layers, and values close to zero (i.e., equilibrium pore pressures) in coarse-grained layers.

The proposed charts in Fig. 3 were developed before the data from Bologna were available. Belfiore *et al.* (1989) found that the proposed classification chart (Fig. 3) based on normalized CPTU data showed good agreement with the samples and other field data.

The Bologna data represent a somewhat extreme example of a deep CPT sounding. Generally, most onshore CPT's are performed to a depth of less than 30 m and existing charts using nonnormalized data, such as the one shown in Fig. 1, often provide reasonably good evaluations of soil behaviour type.

A disadvantage of the charts shown in Fig. 3 is that an estimate is required of the soil unit weights and equilibrium pore pressures to calculate σ_{vo} and σ'_{vo}. However, charts using normalized CPT data are conceptually more correct than previous charts such as the one shown in Fig. 1.

It is likely that the simplified chart in Fig. 1 will continue to be used because of its simplicity and because the basic field data can be applied without complex normalization. However, with the increasing use of field computers, normalized charts such as that presented in Fig. 3 should become more frequently used.

Pore pressure element location for CPTU

The pore pressure ratio shown in Fig. 3 is based on pore pressures measured immediately behind the cone tip and in front of the friction sleeve. Much has been published in recent years concerning the locations for recording cone penetration pore pressures (Roy *et al.* 1982; Smits 1982; Campanella *et al.* 1982; Battaglio *et al.* 1986). Recommendations concerning the location of the piezometer element have generally been based on considerations of either equipment and procedures or interpretation methods. On the basis of a review of existing experience, the following comments can be made about pore pressure measurements during cone penetration.

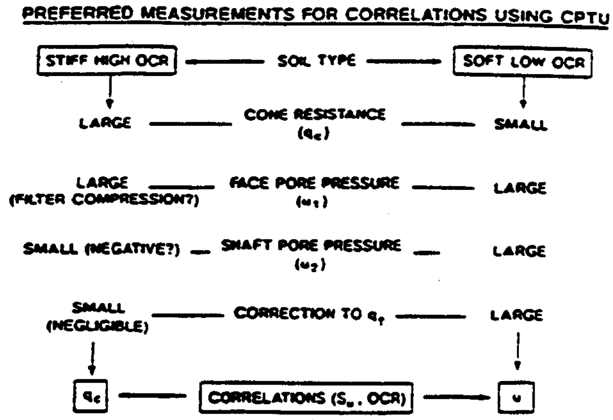

FIG. 6. Preferred measurements for correlations using CPTU.

In terms of equipment design and test procedures there has been a trend towards placing the pore pressure element behind the cone tip, usually in front of the friction sleeve. This location has the advantages of good protection from damage due to abrasion and smearing and generally easier saturation procedures. The location behind the tip is also the correct location to adjust the measured penetration resistance (q_c) to total resistance (q_t) to account for unequal areas.

In terms of interpretation it is generally agreed that pore pressures measured on the face of the cone tip produce the maximum values and provide excellent stratigraphic detail, provided problems with filter element compression, load transfer, abrasion, and smearing have been removed.

Interpretation of cone penetration pore pressures is generally limited to fine-grained soils in which penetration is essentially undrained and is generally directed towards the evaluation of undrained shear strength (s_u) and stress history (OCR, σ'_p). To identify the preferred measurement parameter (q_c or u) to be used for interpretation, it is necessary to distinguish between soft, uncemented fine-grained soils and stiff, fine-grained soils with high OCR. Figure 6 presents a summary of the main differences in measurement parameters between soft, low-OCR and stiff, high-OCR soils.

For cone penetration in soft, uncemented fine-grained soils the measured q_c is generally small, whereas, the pore pressures on the face (u_1) and behind the tip, on the shaft (u_2) are both large. Generally, for cone penetration in soft soils, the pore pressure u_2 is approximately 80% of the pore pressure u_1. However, both pore pressure locations (u_1 and u_2) provide large pore pressures and good stratigraphic detail. The pore pressures further up the shaft away from the tip tend to be somewhat smaller and provide a less detailed response to changes in stratigraphy. Because q_c is generally small in soft, low-OCR, fine-grained soils and the pore pressures are large the correction to q_t is generally significant. Hence, it is generally important to record the pore pressure just behind the tip (u_2) so that the correct pore pressure can be applied to obtain q_t using [2]. Because of a generally decreased accuracy in recording the small q_c values and the need to make significant corrections because of unequal area effects, the preferred measurement for use in interpretation in soft soils is the penetration pore pressure. Because of equipment and procedural considerations (saturation), the preferred location for the pore pressure measurement is just behind the cone tip (i.e., to give u_2).

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For cone penetration in stiff, high-OCR, fine-grained soils the measured q_c is generally large. The pore pressure u_1 is also generally large, but problems with filter compression are frequently encountered and pore pressures may be unreliable (Battaglio *et al.* 1986). However, the pore pressure u_2 is often small and can sometimes be less than the equilibrium pore pressure. An exception to this can occur in cemented and (or) sensitive stiff clays where large u_2 pore pressures can be recorded due to the collapse of the soil structure. Because the q_c values are generally large and the u_2 pore pressures are generally small, the correction to q_t is often small and negligible. Hence, the penetration resistance (q_c) is often a more reliable measurement than the penetration pore pressure and is preferred for interpretation when penetrating stiff, high-OCR, fine-grained soils.

During a stop in the penetration, any excess pore pressure starts to dissipate and the rate of dissipation can be interpreted to evaluate consolidation characteristics of the surrounding soil (Tortensson 1977). In soft, low-OCR soils the pore pressure dissipation data are generally good for pore pressure element locations both on the face and behind the tip. However, in stiff, high-OCR soils the dissipation behind the tip can suffer from local equalization with the much higher pore pressures on the face of the tip and interpretation can be difficult.

From the above observations it is clear that there is no single location for pore pressure measurements that meets all requirements for all soil types. Hence, the preference is to record pore pressures at two or more locations simultaneously (to give u_1, u_2, etc). Cones presently exist that can record pore pressures at two or more locations but saturation procedures are often complex. To avoid increased complexities with equipment and saturation procedures it is recommended to have flexibility in cone design so that pore pressures can be measured either on the face of the cone tip or just behind it. Many cone designs already exist that enable the filter location to be easily changed in the field.

For general piezocone testing it is therefore recommended to measure the pore pressure just behind the tip for the following reasons: (1) good protection from damage, (2) easy saturation, (3) generally good stratigraphic detail, (4) generally good dissipation data, and (5) right location to correct q_c. However, if a stiff, high-OCR, clay deposit is encountered and measured pore pressures behind the tip become very small, it is recommended to change the location (in the field) and record pore pressures on the face of the tip. For quantitative interpretation of the pore pressures measured on the face of the tip during penetration in stiff soils it is important to avoid, or be aware of, potential errors due to filter compression.

Summary

A new soil behaviour type classification system has been presented using normalized cone penetration test parameters. The new charts represent a three-dimensional classification system incorporating all three pieces of data from a CPTU. The charts are global in nature and can be used to define soil behaviour type. Factors such as changes in stress history, *in situ* stresses, sensitivity, stiffness, macrofabric, and void ratio will also influence the classification. A guide to the influence some of these variables have on the classification has been included on the charts.

Occasionally soil will fall within different zones on each chart. In these cases the rate and manner in which the excess pore pressures dissipate during a pause in the penetration can significantly aid in the classification. A case history involving wire-line CPTU data from a 300 m deep borehole has been presented to illustrate the usefulness of applying normalized data for soil classification.

A discussion has also been presented regarding the recommended position to measure pore pressures during cone penetration. No single location for pore pressure measurements meets all requirements for all soils. Hence, the ideal situation is to record pore pressures at two or more locations simultaneously. However, to avoid increased complexities with equipment and saturation procedures it is recommended to have flexibility in cone design so that pore pressures can be measured either on the face of the cone tip or just behind it. For penetration into granular soils and soft cohesive soils it is recommended to measure the pore pressures just behind the cone tip. For penetration into stiff, high-OCR clay or silt deposits it is recommended to change the location (in the field) and record pore presures on the face of the cone tip. However, for quantitative interpretation of pore pressures measured on the face of the tip during penetration in stiff soils it is important to avoid, or be aware of, potential errors due to filter element compression.

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BALIGH, M.M., VIVATRAT, V., and LADD, C.C. 1980. Cone penetration in soil profiling. ASCE Journal of the Geotechnical Engineering Division, 106: 447–461.

BALIGH, M.M., AZZOUZ, A.D., WISSA, A.Z.E., MARTIN, R.T., and MORRISON, M.H. 1981. The piezocone penetrometer. Symposium on Cone Penetration Testing and Experience, ASCE, Geotechnical Engineering Division, St. Louis, pp. 247–263.

BATTAGLIO, M., and MANISCALCO, R. 1983. Il peizocone esecuzione ed interpretatzione. Scienza della Construzioni Politecnico di Torino, No. 607.

BATTAGLIO M., BRUZZI, D., JAMIOLKOWSKI, M., and LANCELLOTTA, R. 1986. Interpretation of CPT's and CPTU-s—undrained penetration of saturated clays. Proceedings, 4th International Geotechnical Seminar, Singapore.

BELFIORE, F., COLOMBO, P.F., PEZZELLI, G., and VILLANI, B. 1989. A contribution to the study of the subsidence of Bologna. 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro.

CAMPANELLA, R.G., and ROBERTSON, P.K. 1988. Current status of the piezocone test. Proceedings, 1st International Symposium on Penetration Testing, ISOPT I, vol. 1, pp. 93–116.

CAMPANELLA, R.G., GILLESPIE, D., and ROBERTSON, P.K. 1982. Pore pressures during cone penetration testing. Proceedings, 2nd European Symposium on Penetration Testing, ESOPT II, pp. 507–512.

——— 1986. Factors affecting the pore water pressures and its measurement around a penetrating cone. Proceedings, 39th Canadian Geotechnical Conference, Ottawa.

DOUGLAS, B.J., and OLSEN, R.S. 1981. Soil classification using electric cone penetrometer. Symposium on Cone Penetration Testing and Experience, Geotechnical Engineering Division, ASCE, St. Louis, pp. 209–227.

DOUGLAS, B.J., STRUTYNSKY, A.I., MAHAR, L.J., and WEAVER, J. 1985. Soil strength determinations from the cone penetration test. Proceedings, Civil Engineering in the Arctic Offshore, San Francisco.

GILLESPIE, D.G. 1989. Evaluating velocity and pore pressure data from the cone penetration test. Ph.D. thesis, Department of Civil Engineering, University of British Columbia, Vancouver.

HOULSBY, G. 1988. Discussion session contribution. Penetration Testing in the U.K., Birmingham.

ISOPT. International Symposium on Penetration Testing. 1988. Report of the ISSMFE Technical Committee on Penetration Testing. *In* Working party, vol. 1, pp. 27–51.

ISSMFE. International Society for Soil Mechanics and Foundation Engineering. 1977. Report of the Subcommittee on Standardization of Penetration Testing in Europe. Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, vol. 3, Appendix 5, pp. 95–152.

JAMIOLKOWSKI, M., and ROBERTSON, P.K. 1988. Future trends for penetration testing, Closing adress. Proceedings, Penetration Testing in the U.K., Birmingham.

JONES, G.A., and RUST, E.A. 1982. Piezometer penetration testing CUPT. Proceedings, 2nd European Symposium on Penetration Testing, ESOPT II, Amsterdam, vol. 2, pp. 607–613.

KONRAD, J.-M. 1987. Piezo-friction-cone penetrometer testing in soft clays. Canadian Geotechnical Journal, 24: 645–652.

LUNNE, T., EIDSMOEN, T., GILLESPIE, D., and HOWLAND, J.D. 1986. Laboratory and field evaluation of cone penetrometers. Proceedings, *In-situ* '86, ASCE Specialty Conference, Blacksburg, VA.

OLSEN, R.S. 1984. Liquefaction analysis using the cone penetration test. Proceedings, 8th World Conference on Earthquake Engineering, San Francisco.

OLSEN, R.S., and FARR, J.V. 1986. Site characterization using the cone penetration test. Proceedings, *In-situ* '86, ASCE Specialty Conference, Blacksburg, VA.

ROBERTSON, P.K. 1986. *In situ* testing and its application to foundation engineering. Canadian Geotechnical Journal, 23: 573–594.

ROBERTSON, P.K., CAMPANELLA, R.G., GILLESPIE, D., and GRIEG, J. 1986. Use of piezometer cone data. Proceedings, *In-situ* '86, ASCE Specialty Conference, Blacksburg, VA.

ROY, M., TREMBLAY, M., TAVENAS, F., and LA ROCHELLE, P. 1982. Development of pore pressures in quasi-static penetration tests in sensitive clay. Canadian Geotechnical Journal, 19: 124–138.

SENNESET, K., and JANBU, N. 1984. Shear strength parameters obtained from static cone penetration tests. American Society for Testing and Materials, Special Technical Publication 883.

SMITS, F.P. 1982. Penetration pore pressure measured with piezometer cones. Proceedings, 2nd European Symposium on Penetration Testing, ESOPT II, Amsterdam, vol. 2, pp. 871–876.

TORTENSSON, B.A. 1977. The pore pressure probe. Nordiske Geotekniske Mote, Oslo, Paper 34, pp. 1.15–34.15.

WROTH, C.P. 1984. Interpretation of *In situ* soil test, 24th Rankine Lecture. Géotechnique, 34: 449–489.

BACK