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SUMMARY

The cone penetrometer allows for the soil type to be determined from the measured values of cone resistance and sleeve friction. As the cone penetrometer progressed from the mechanical cone to the electrical cone to the piezocone, the reliability of the determination of the soil type also improved. The paper references several published methods of soil profiling. All but two of these apply cone resistance plotted against the friction ratio. However, the friction ratio includes the cone resistance and this manner of data presentation violates the principle of not plotting a variable against itself. The paper presents two soil profiling methods based on the piezocone and compare them against three specific cases containing sand, normally consolidated clay, and overconsolidated clay. Both methods result in an accurate soil type determination.

INTRODUCTION

In-situ sounding by standardized penetrometers and execution methods came along early in the development of geotechnical engineering. For example, the Swedish weight-sounding device (Swedish State Railways Geotechnical Commission, 1922), which still is in common use. The cone resistance obtained by this device and other early penetrometers included the influence of soil friction along the rod surface. In the 1930's, a "mechanical cone penetrometer" was developed in the Netherlands where the rods to the cone point were placed inside an outer tubing, separating the cone rods from the soil (Begemann, 1963). The mechanical penetrometer was advanced by first pushing the entire system to obtain the combined resistance. Intermittently, every even metre or so, the cone point was advanced a small distance while the outer tubing was held immobile, thus obtaining the cone resistance separately. The difference was the total shaft resistance. Begemann (1953) introduced a short section of tubing, a sleeve, immediately above the cone point. The sleeve arrangement enabled measuring the "sleeve friction" near the cone. Later, sensors were placed in the cone and sleeve to measure the cone resistance and sleeve friction directly (Begemann, 1963). This penetrometer became known as the "electrical cone penetrometer". In the early 1980's, piezometer elements were incorporated with the electrical cone penetrometer, leading to the modern cone version, "the piezocone", which provides values of cone resistance, sleeve friction, and pore pressure at close distances, usually every 25 mm. The sleeve friction is regarded as a measure of the undrained shear strength—of a sort—the value is recognized as not being accurate (e. g., Lunne et al., 1986, Robertson, 1990). The cone penetrometer does not provide a measurement of static resistance, but records the resistance at a certain penetration rate (now standardized to 20 mm/s). Therefore, pore water pressures are induced in the soil at the location of the cone point and sleeve that can differ significantly from the "neutral" pore water pressure. In dense fine sands, due to dilation, the induced pore pressures can be negative. In pervious soils, such as sands, they are small, while in less pervious soils, such as silts and clays, they can be quite large. Measurements with the piezocone showed that the cone resistance must be corrected for the pore pressure acting on the cone shoulder (Baligh et al., 1981; Campanella et al., 1982).

The cone penetrometer test is economical, supplies continuous records with depth, and allows a variety of sensors to be incorporated with the penetrometer. The direct numerical values produced by the cone test have been used as input to geotechnical formulae, usually of empirical nature, to determine capacity and settlement, and for soil profiling.

Early on, information about the soil type was approximate and the cone penetrometer was limited to determining the location of soil type boundaries and no details were provided. The soil type had to be confirmed from the results of conventional borings, with the exception of empirical interpretations limited to the geological area where they had been developed. Begemann (1965) is credited with having presented the first rational soil profiling method. With the advent of the piezocone, the CPTu, the cone penetrometer was established as an accurate site investigation tool.

BRIEF SURVEY OF SOIL PROFILING METHODS

Begemann (1965) pioneered soil profiling from the CPT, showing that, while coarse-grained soils generally demonstrate larger values of cone resistance, q_c, and sleeve friction, f_s, than do fine-grained soils, the soil type is not a strict function of either cone resistance or sleeve friction, but of the combination of the these values. Fig. 1 presents the Begemann soil profiling chart, showing (linear scales) q_c as a function of f_s. Begemann showed that the soil type is a function of the ratio between the sleeve friction and the cone resistance (the friction ratio, R_f). The friction ratio is indicated by the slope of the fanned-out lines.

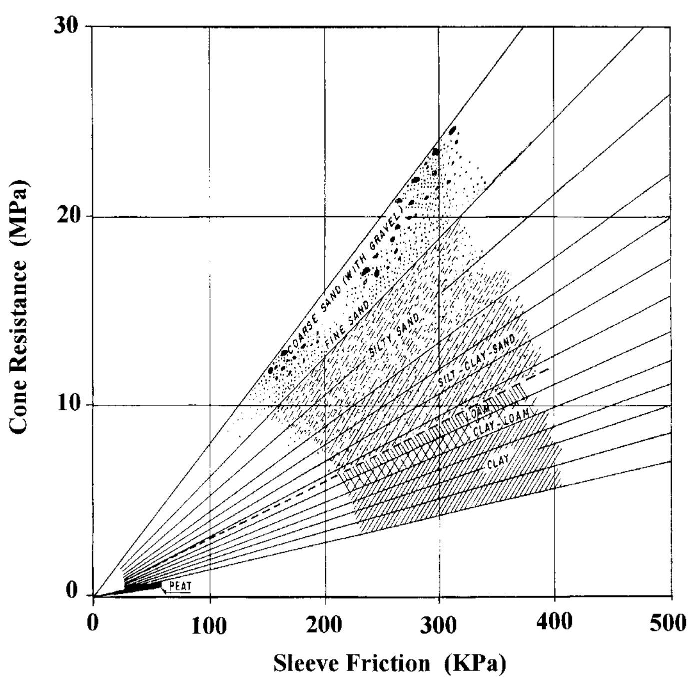

Fig. 1 The Begemann original profiling chart (Begemann, 1965)

The friction ratios identify the soil types as follows.

Soil Type as a Function of Friction Ratio (Begemann, 1965)

Coarse sand with gravel through fine sand	1.2 %	-	1.6 %
Silty sand	1.6 %	-	2.2 %
Silty sandy clayey soils	2.2 %	-	3.2 %
Clay and loam, and loam soils	3.2 %	-	4.1 %
Clay	4.1 %	-	7.0 %
Peat			>7 %

The Begemann chart was derived from tests in Dutch soils with the mechanical cone. The chart is site-specific, i. e., directly applicable only to the specific geologic locality where it was developed. For example, cone tests in sand usually shows a friction ratio smaller than 1 %. However, the chart has important general qualitative value.

Sanglerat et al., (1974) proposed the chart shown in Fig. 2, presenting data from an 80 mm diameter research penetrometer. The chart plots the cone resistance (logarithmic scale) versus the friction ratio (linear scale). This manner of plotting has the apparent advantage of showing the cone resistance as a direct function of the friction ratio and, therefore, of the soil type. However, plotting the cone resistance versus the friction ratio implies, falsely, that the values are independent of each other; the friction ratio would be the independent variable and the cone resistance the dependent variable. In reality, the friction ratio is the inverse of the ordinate and the values are patently not independent. That is, the cone resistance is plotted against its own inverse self, multiplied by a variable that ranges, normally, from a low of about 0.01 through a high of about 0.07. The plotting of data against own inverse values will predispose the plot to a hyperbolically shaped zone ranging from large ordinate values at small abscissa values through small ordinate values at large abscissa values. The resolution of data representing fine-grained soils is very much exaggerated as opposed to the resolution of the data representing coarse-grained soils. Simply, while both cone resistance and sleeve friction are important soil profiling parameters, plotting one as a function of the other distorts the information.

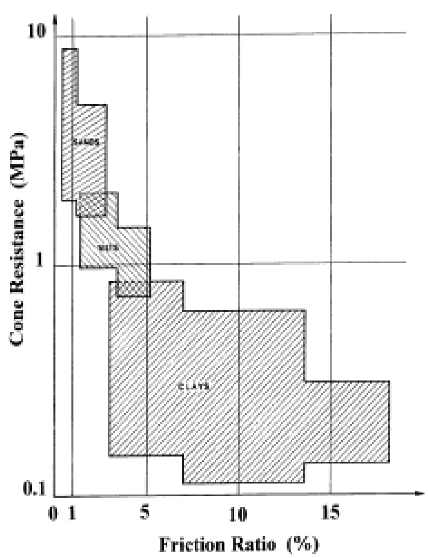

Fig. 2 Plot of data from research penetrometer (Sanglerat et al., 1974)

Notice, that Fig. 2 defines the soil type also by its upper and lower limit of cone resistance and not just by the friction ratio.

Schmertmann (1978) proposed the soil profiling chart shown in Fig. 3. The chart is based on results from mechanical cone data in "North Central Florida" and incorporates Begemann's CPT data and indicates zones of common soil type. It also presents boundaries for loose and dense sand and consistency (undrained shear strength) of clays and silts, which are imposed by definition and not related to the soil profile interpreted from the CPT results.

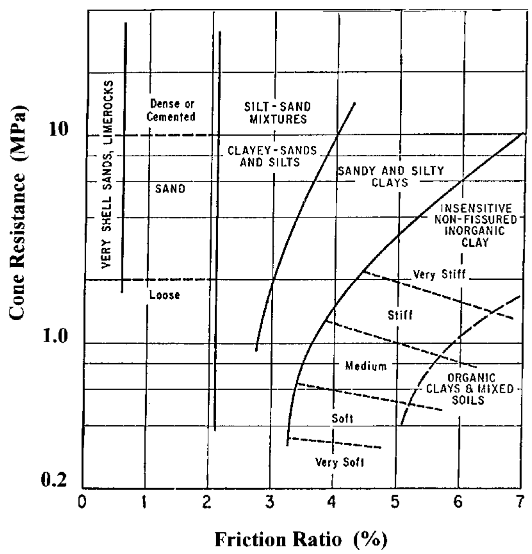

Fig. 3 The Schmertmann profiling chart (Schmertmann, 1978)

Also the Schmertmann (1978) chart presents the cone resistance as a plot against the friction ratio, that is, the data are plotted against their inverse self. Fig. 4 shows the Schmertmann chart converted to a Begemann type graph (logarithmic scales), re-plotting the Fig. 3 envelopes and boundaries as well as text information. When the plotting of the data against own inverse values is removed, a visual effect comes forth that is quite different from that of Fig. 3. Note also that the consistency boundaries do not appear very logical when seen in this undistorted manner of presentation.

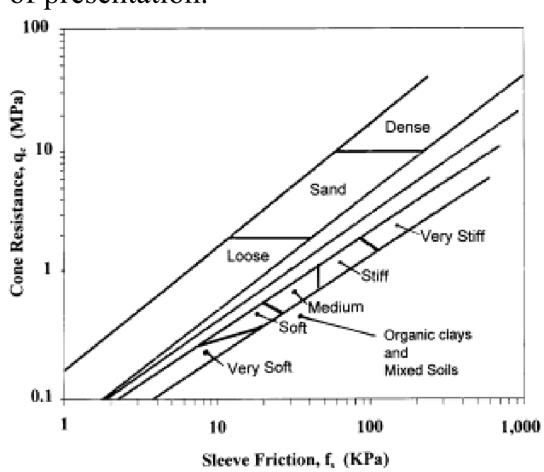

Fig. 4 The Schmertmann profiling chart converted to a Begemann type profiling chart

Schmertmann (1978) states that the correlations shown in Fig. 3 may be significantly different in areas of dissimilar geology. The chart is intended for typical reference and includes two warnings: "Local correlations are preferred" and "Friction ratio values decrease in accuracy with low values of q_c". Schmertmann also mentions that soil sensitivity, friction sleeve surface roughness, soil ductility, and pore pressure effects can influence the chart correlation. Notwithstanding the caveat, the Schmertmann chart is still commonly applied "as is" in North American practice.

Douglas and Olsen (1981) were the first to propose a soil profiling chart based on tests with the electrical cone penetrometer. They published the chart shown in Fig. 5 which appends classification per the unified soil classification system to the soil type zones. The chart also indicates trends for liquidity index and earth pressure coefficient, as well as sensitive soils and "metastable sands". The Douglas and Olsen chart envelops several zones using three upward curving lines representing increasing content of coarse-grained soil and four lines with equal sleeve friction. This way, the chart distinguishes an area (lower left corner of the chart) where soils are sensitive or "metastable". Comparing the Fig. 5 chart with the Fig. 3 chart, a difference emerges in implied soil type response**:** while in the Schmertmann chart the soil type envelopes curve downward, in the Douglas and Olsen chart they curve upward. Zones for sand and for clay are approximately the same in the two charts, however.

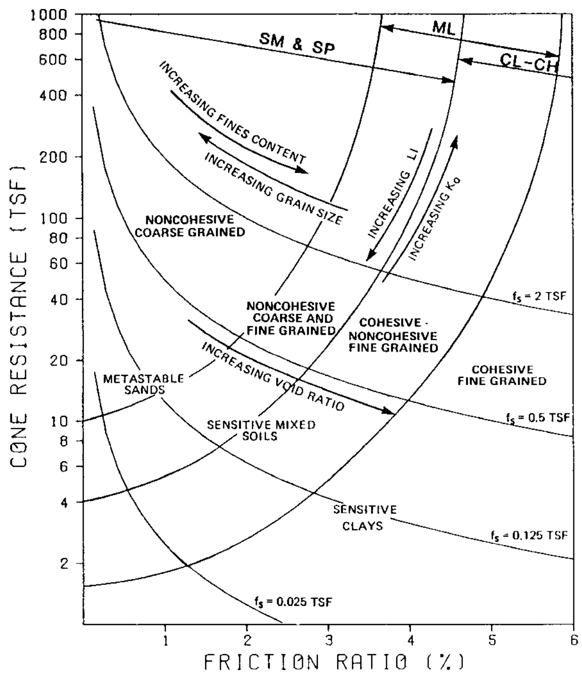

Fig. 5 Profiling chart per Douglas and Olsen (1981)

The authors consider a comparison between the Schmertmann and the Douglas and Olsen charts (Figs. 3 and 5) to be more relevant if the charts are prepared per the Begemann type of presentation. Fig. 6 shows the Douglas and Olsen chart converted to a Begemann type graph. The figure includes the three curved envelopes and the four lines with equal sleeve friction and a heavy dashed line which identifies an approximate envelop of the zones indicated to represent "metastable" and "sensitive" soils. Comparing the Begemann type presentations of the Douglas and Olsen chart (Fig. 6) and Schmertmann (Fig. 4) chart, the former offers a smaller band width for dense sands and sandy soils (q_c larger than 10 MPa) and a larger band width in the low range of cone resistance (q_c smaller than 1 MPa).

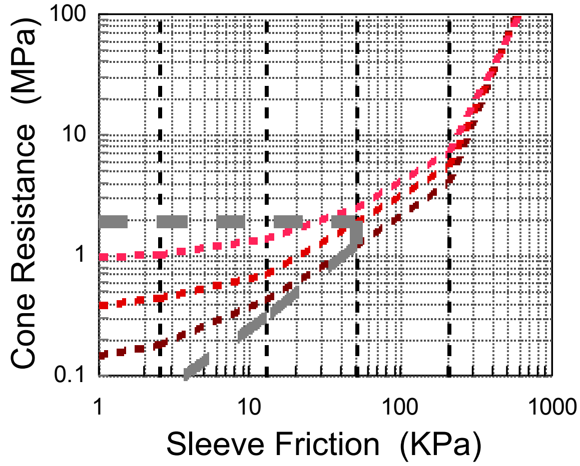

Fig. 6 The Douglas and Olsen profiling chart converted to a Begemann type chart

Vos (1982) suggested using the electrical cone penetrometer for Dutch soils to identify soil types from the friction ratio, as shown below. The percentage values are similar but not identical to those recommended by Begemann (1965).

Soil Type as a Function of Friction Ratio (Vos, 1982)

Coarse sand and gravel	<0.5%		
Fine sand	1.0 %	-	1.5 %
Silt	1.5 %	-	3.0 %
Clay	3.0%	-	5.0%
Clay	4.1 %	-	7.0 %
Peat			>5 %

Jones and Rust (1982) developed the soil profiling chart shown in Fig. 7, which is based on the piezocone using the measured total cone resistance and the measured ***excess*** pore water pressure mobilized during cone advancement. The chart presents the excess pore water pressure plotted against net cone resistance (total overburden stress subtracted from total cone resistance). The chart is interesting because it identifies also the density (compactness condition) of coarse-grained soils and the consistency of fine-grained soils. However, the suggestion that high negative pore water pressures (indicating dilatancy) could be measured in very soft clays is surely a result of an overzealous desire for symmetry in the chart. Vermeulen and Rust (1995) present a large number of data plotted using the chart (with slight modification of the plotting axes).

Robertson et al., (1986) and Campanella and Robertson (1988) were the first to present a chart based on the piezocone with the cone resistance corrected for pore pressure at the shoulder according to Eq. 1.

$$q_t = q_c + u_2(1-a) \quad (1)$$

where q_t = cone resistance corrected for pore water pressure on shoulder
q_c = measured cone resistance
u_2 = pore pressure measured at cone shoulder
a = ratio between shoulder area (cone base) unaffected by the pore water pressure to total shoulder area

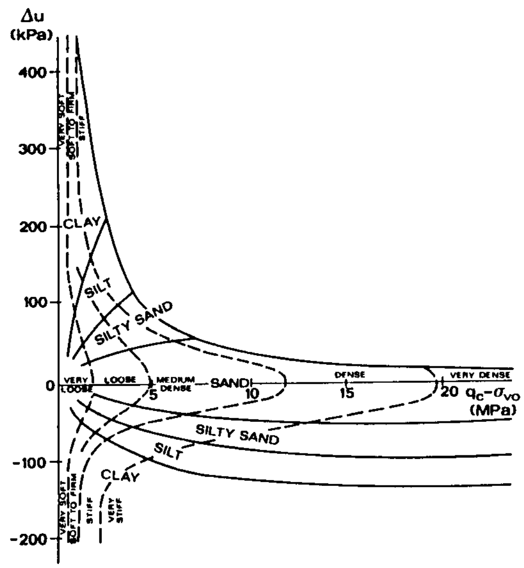

Fig. 7 Profiling chart per Jones and Rust (1982)

The Robertson et al. (1986) profiling chart is presented in Fig. 8. The chart identifies numbered areas that separate the soil types in twelve zones, as follows.

1. Sensitive fine-grained soil
2. Organic soil
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. Sand to gravelly sand
11. Very stiff fine-grained soil
12. Overconsolidated or cemented sand to clayey sand

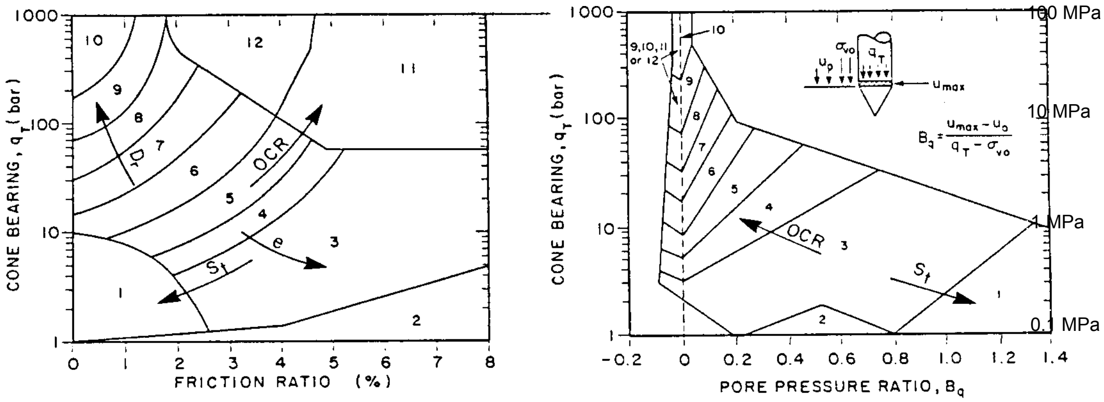

Fig. 8 Profiling chart per Robertson et al. (1986)

A novel feature in the profiling chart is the delineation of Zones 1, 11, and 12, representing somewhat extreme soil responses thus enabling the CPTu to uncover more than just soil grain size. The rather detailed separation of the in-between zones, Zones 3 through 10, indicate a gradual transition from fine-grained to coarse-grained soil.

The Robertson et al. (1986) profiling chart introduced a pore pressure ratio, B_q, defined by Eq. 2, as follows.

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_v} \tag{2}$$

where B_q = pore pressure ratio
u_2 = pore pressure measured at cone shoulder
u_0 = in-situ pore pressure
q_t = cone resistance corrected for pore water pressure on shoulder
σ_v = total overburden stress

Directly, the B_q-chart shows zones where the u_2 pore pressures become smaller than the initial pore pressures (u_0) in the soil during the advancement of the penetrometer, resulting in negative B_q-values. Otherwise, the B_q-chart appears to be an alternative rather than an auxiliary chart; one can use one or the other depending on preference. However, near the upper envelopes, a CPTu datum plotting in a particular soil-type zone in the friction ratio chart will not always appear in the same soil-type zone in the B_q-chart. Robertson et al. (1986) points out that "occasionally soils will fall within different zones on each chart" and recommends that the user study the pore pressure rate of dissipation (if measured) to decide which zone applies to questioned data.

The pore pressure ratio, B_q, is an inverse function of the cone resistance, q_t. Therefore, also the B_q-plot represents the data as a function of their own self values, in conflict with general principles of data representation.

Senneset et al., (1989) produced a soil classification chart based on plotting corrected cone resistance, q_t , against pore pressure ratio, B_q, as shown in Fig. 9. The chart is limited to the area where q_t is smaller than 16 MPa, i. e., the zone Robertson et al. (1986) denoted sensitive soil. It identifies limits of density and consistency (dense, stiff, soft, etc.) that appear to be somewhat lower than those normally applied in North American practice, as, for example, indicated in Fig. 3.

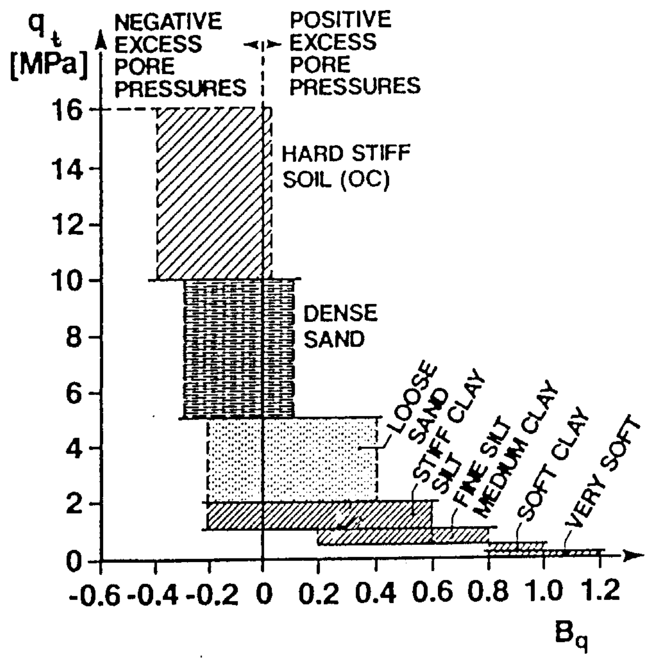

Fig. 9 Profiling chart per Senneset et al. (1989)

In comparing the chart to the Sanglerat chart shown in Fig. 2, it appears that the introduction of q_t and plotting against B_q, as opposed to R_f, avoids exaggerating the resolution in the clay region.

Eslami and Fellenius (1996) proposed a pore pressure ratio, B_E, defined, as follows.

$$B_E = \frac{(u_2 - u_0)}{u_0} \tag{3}$$

where B_E = "Effective" pore pressure ratio

A diagram showing q_t versus B_E provides a more perceptible picture of the pore pressure induced by the cone and it does not violate the principles of plotting. The authors believe that research may show that the pore pressure ratio B_E will be useful for assessing liquefaction potential, degree of overconsolidation, and compressibility of sand and silt soils. It is also hypothesized that the B_E-ratio may show to be useful in predicting the magnitude of increase (set-up) of capacity of driven piles between initial driving and after the soils have reconsolidated.

Robertson (1990) proposed a refinement of the Robertson et al. (1986) profiling chart, shown in Fig. 10, plotting a "normalized cone resistance", q_{cnrm}, against a "normalized friction ratio", R_{fnrm} in a cone resistance chart. The accompanying pore pressure ratio chart plots the "normalized cone resistance" against the pore pressure ratio, B_q, defined by Eq. 2 applying the same B_q-limits as the previous chart (Zone 2 is not included in Fig. 10).

The normalized cone resistance is defined by Eq. 4, as follows.

$$q_{cnrm} \quad = \quad \frac{(q_t - \sigma_v)}{\sigma'_v} \tag{4}$$

where q_t = cone resistance corrected for pore water pressure on shoulder
σ_v = total overburden stress
σ'_v = effective overburden stress
$(q_t - \sigma_v)$ = net cone resistance

The normalized friction factor is defined as the sleeve friction over the net cone resistance, as follows.

$$R_{cnrm} \quad = \quad \frac{f_s}{q_t - \sigma'_v} \tag{5}$$

where f_s = sleeve friction

The numbered areas in the profiling chart separate the soil types in nine zones, as follows.

1. Sensitive, fine-grained soils
2. Organic soils and peat
3. Clays [clay to silty clay]
4. Silt mixtures [silty clay to clayey silt]
5. Sand mixtures [sandy silt to silty sand]
6. Sand [silty sand to clean sand]
7. Sand to gravelly sand
8. Sand – clayey sand to "very stiff" sand
9. Very stiff, fine-grained, overconsolidated or cemented soil

The two first and two last soil types are the same as those used by Robertson et al. (1986) and Types 3 through 7 correspond to former Types 3 through 10. The Robertson (1990) normalized profiling chart has seen extensive use in engineering practice (as has the Robertson et al., 1986 chart).

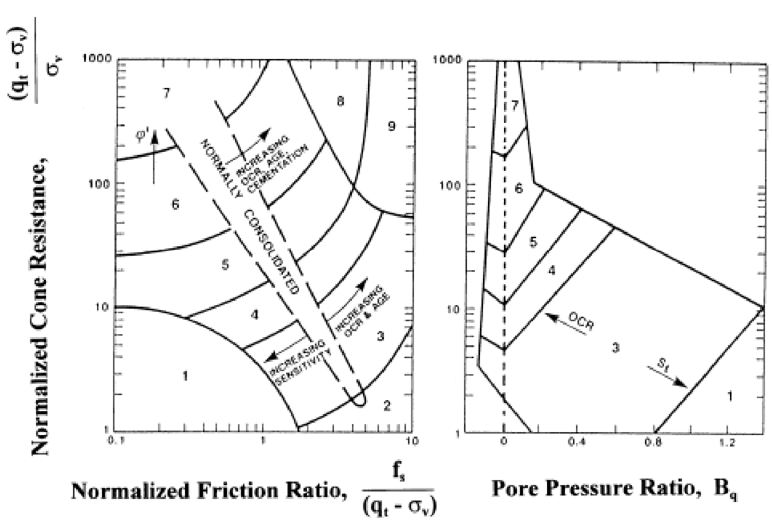

Fig. 10 Profiling chart per Robertson (1990)

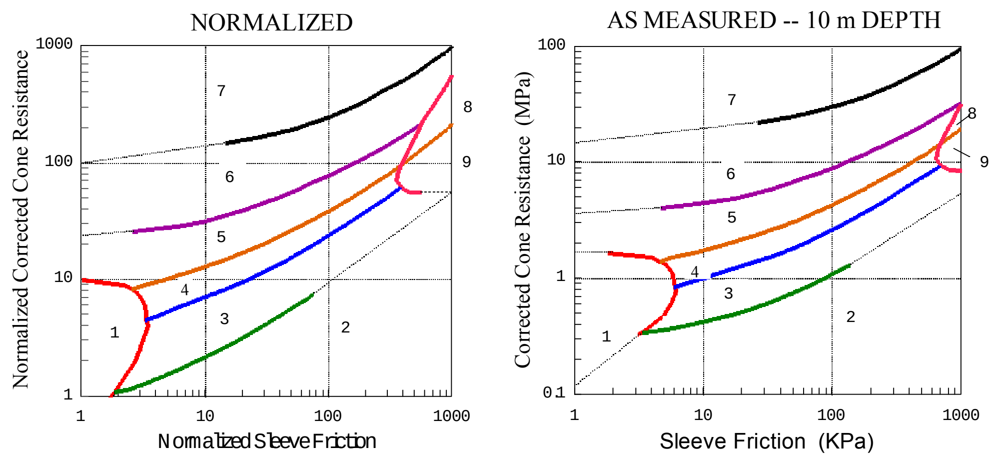

Fig. 11 The Robertson (1990) profiling chart converted to Begemann type charts
Left: Normalized corrected cone resistance vs. normalized sleeve friction
Right: Corrected cone resistance vs. sleeve friction

The normalization was proposed to compensate for the cone resistance dependency on the overburden stress and, therefore, when analyzing deep CPTu soundings (i. e., deeper than about 30 m) a profiling chart developed for more shallow soundings does not apply well to the deeper sites. At very shallow depths, however, the proposed normalization will tend to lift the data in the chart and imply a coarser soil than is necessarily the case. Moreover, the effective stress at depth is a function of the weight of the soil and, to a greater degree, of the pore pressure distribution with depth. Where soil types alternate between light soils and dense soils (soil densities can range from 1,400 kg/m^3 through 2,100 kg/m^3) and/or where upward or downward gradients exist, the normalization is unwieldy. For these reasons, it would appear that the normalization merely exchanges one difficulty for another.

For reference to the Begemann type chart, Fig. 11 (above) shows the envelopes of the Robertson (1990) converted to a Begemann type chart. The ordinate is the same and the abscissa is the multiplier of the normalized cone resistance and the normalized friction factor of the original chart (the normalized sleeve friction is the sleeve friction divided by the effective overburden stress). Where needed, the envelopes have been extended with a thin line to the frame of the diagram. As reference to Figs. 4 and 6, Fig. 11 also presents the usual Begemann type profiling chart converted from Fig. 10 under the assumption that the data apply to a depth of about 10 m at a site where the groundwater table lies about 2 m below the ground surface. This chart is approximately representative for a depth range of about 5 to 30 m. Comparing the "normalized" chart with the "as measured" chart does not indicate that normalization would be advantageous.

Eslami and Fellenius (1997) developed a soil profiling method when investigating the use of cone penetrometer data in pile design. They compiled a database consisting of CPT and CPTu data associated with results of boring, sampling, laboratory testing, and routine soil characteristics of cases from 18 sources reporting data from 20 sites in 5 countries. About half of the cases were from piezocone tests, CPTu, and include pore pressure measurements (u_2). Non-CPTu tests were from sand soils and were used with the assumption that each u_2-value is approximately equal to the neutral pore pressure (u_0). The database values are separated on five main soil type categories listed below.

1. Sensitive and Collapsible Clay and/or Silt
2. Clay and/or Silt
3. Silty Clay and/or Clayey Silt
4. Sandy Silt and/or Silty Sand
5. Sand and/or Sandy Gravel

The data points were plotted in a Begemann type profiling chart and envelopes were drawn enclosing each of the five soil types. The envelopes are shown in Fig. 12. The database does not include cases with cemented soils or very stiff clays, and, for this reason, no envelopes for such soil types are included in the chart.

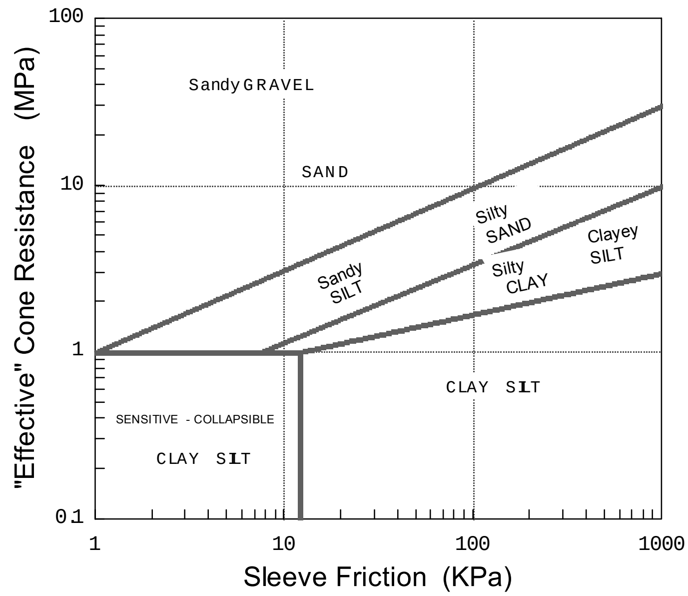

Fig. 12 The Eslami-Fellenius profiling chart

Plotting an "effective" cone resistance defined by Eq. 6 was found to provide a more consistent delineation of envelopes than a plot of only the cone resistance.

$$q_E = (q_t - u_2) \qquad (6)$$

where q_E = "effective" cone resistance
q_t = cone resistance corrected for pore water pressure on shoulder (Eq. 1)
u_2 = pore pressure measured at cone shoulder

The q_E-value was shown to be a consistent value for use in relation to soil responses such as pile shaft and pile toe resistances (Eslami 1996, Eslami and Fellenius, 1995; 1996; 1997). Notice that, as mentioned by Robertson (1990), the measured pore water pressure is a function of where the pore pressure gage is located. Therefore, the q_E-value is by no means a measurement of effective stress in conventional sense. Because the sleeve friction is a rather approximate measurement, no similar benefit was found in producing an "effective" sleeve friction. In dense, coarse-grained soils, the q_E-value differs only marginally from the q_t-value. In contrast, cone tests in fine-grained soils could generate substantial values of excess pore water pressure causing the q_E-value to be much smaller than the q_t-value.

The Eslami-Fellenius chart is simple to use and requires no adjustment to estimated effective stress and total stress. The chart is primarily intended for soil type (profiling) analysis of CPTu data. With regard to the boundaries between the main soil fractions (clay, silt, sand, and gravel), international and North American practices agree, but differences exist with regard to how soil-type names are modified according to the contents of other than the main soil fraction. The chart assumes the lower and upper boundaries for adjectives, such as clayey, silty, sandy to be 20 % and 35 %, "some" to mean 10 % through 20 %, and "trace" to mean smaller than 10 % by weight as indicated in the Canadian Foundation Engineering Manual (1985).

A soil profiling chart based on a Begemann type plot, such as the Eslami-Fellenius (1996) method can easily be expanded by adding delineation of strength and consistency of fine-grained soils and relative density and friction angle of coarse-grained soils per the user preferred definitions or per applicable standards. No doubt, CPTu sounding information from a specific area or site can be used to further detail a soil profiling chart and result in delineation of additional zones of interest. However, there is a danger in producing a very detailed chart inasmuch the resulting site dependency easily gets lost, leading an inexperienced user to apply the detailed distinctions beyond their geologic validity.

Other early profiling charts were proposed by Searle (1979), Olsen and Farr (1986), Olsen and Malone (1988), Erwig (1988). CPTu charts similar to similar to that of Robertson (1990) were proposed , Larsson and Mulabdic (1991), Jefferies and Davies (1991, 1993), and Olsen and Mitchell (1995).

COMPARING THE ROBERTSON (1990) METHOD TO THE ESLAMI AND FELLENIUS (1997) METHOD

To provide a comparison between the Robertson (1990) profiling chart and the Eslami-Fellenius (1997) soil profiling methods, three short series of CPTu data were compiled from sites with very different geologic origin, where the soil profiles had been established independently of the CPTu. The borehole information provides soil description and water content of recovered samples. For one of the cases, the grain size distribution is also available. The soil and CPTu information is compiled in Table 1. The three sites are:

1. Northwestern University, **Evanston**, Illinois (Finno, 1989). The soil profile consists of 7 m of sand deposited on normally consolidated silty clay. The CPTu data were obtained with a piezometer attached to the cone face (u_1) and not behind the shoulder (u_2). The method of converting the pore pressure measurement to the u_2-value presented by Finno (1989) has been accepted here, although the conversion is disputed. For comments, see Mayne et al. (1990).

2. Along the shore of Fraser River, **Vancouver**, British Columbia (personal communication, V. Sowa, 1998). The soil profile consists of an 18 m thick deltaic deposits of clay, silt, and sand. The first four data points are essentially variations of silty clay or clayey silt. The fifth is a silty sand.

3. University of Massachusetts, **Amherst**, Massachusetts (personal communication, P. Mayne, 1998). The soil profile (Lutenegger and Miller, 1995)

consists of a 5 m thick homogeneous overconsolidated clayey silt. This case includes also information on grain size distribution. The borehole records show the soil samples for data points Nos. 3 through 7 to be essentially identical. Notice that the u_2-measurements indicate substantial negative values, that is, the overconsolidated clay dilates as the cone is advanced.

For each case, the soil information in Table 1 is from depths where the CPTu data were consistent over a 0.5 m length. Then, the CPTu data from 150 mm above and below the middle of this depth range were averaged using geometric averaging, preferred over the arithmetic average as it is less subject to influence of unrepresentative spikes and troughs in the data (which is here a redundant effort, however, as the records contain no such spikes and troughs).

The results of the soil profiling of the CPTu data are shown in Fig. 13.

Evanston data: The first three samples are from a sand soil and both methods identify the CPTu data accordingly. The remaining data points (Nos. 4 through 7) given as Silty Clay in the borehole records are identified as Clay/Silt by the Eslami-Fellenius and as Clay to Silty Clay by the Robertson method; that is, both methods agree with the independent soil classification.

Vancouver data: Both methods properly identify the first four data points to range from Clayey Silt to Silty Clay in agreement with the independent soil classification. The fifth sample (Silty Sand) is identified correctly by the Eslami-Fellenius method as a Sand close to the boundary to Silty Sand and Sandy Silt. The Robertson method identifies the soil as a Sandy Silt to Clayey Silt, which is essentially correct, also.

Amherst data: Both methods identify the soils to be silt or clay or silt and clay mixtures. Moreover, both methods place Points 3 through 7 on the same soil type boundary line, that is, confirming the similarity between the soil samples. However, the spread of plotted points appear to be larger for the Robertson method; possibly because its profiling does not consider the pore pressures developed by the advancing penetrometer (but for correction for the pore pressure on the shoulder, of course), while the Eslami-Fellenius method does account (per Eq. 6) for the negative pore pressures that developed.

TABLE 1 Site Information

No.	Depth (m)	Description	Water Content (%)	Soil Fractions: Clay (%)	Silt (%)	Sand (%)	CPTu Data: q_t (MPa)	f_s (KPa)	u_2 (KPa)
Evanston, IL (Groundwater table at 4.5 m)									
1	1.5	SAND, Fine to medium, trace gravel	29				25.08	191.5	49.8
2	3.4	SAND, Medium, trace gravel	16				3.48	47.9	-16.0
3	6.7	SAND, Fine, trace silt, organics	26				32.03	162.8	111.7
4	8.5	Silty CLAY, trace sand	28				0.51	21.1	306.4
5	9.5	Silty CLAY, little gravel	22				0.99	57.5	39.6
6	12.8	Silty CLAY, little gravel	23				0.69	19.2	383.0
7	16.5	Silty CLAY, little gravel	24				0.77	17.2	427.1
Vancouver, BC (Groundwater table at 3.5 m)									
1	3.7	CLAY to Clayey SILT	52				0.27	16.1	82.5
2	5.8	Clayey SILT to SILT	34				1.74	20.0	177.1
3	10.2	Silty CLAY	47				1.03	13.4	183.5
4	14.3	Silty CLAY	40				4.53	60.2	54.3
5	17.5	Silty SAND	25				10.22	77.8	118.5
Amherst, MA (Groundwater table at 2.0 m)									
1	0.6	SAND and SILT, trace clay	20	10	30	60	2.04	47.5	-9.4
2	1.5	Clayey SILT, trace sand	28	23	67	10	2.29	103.3	-47.3
3	2.0	Clayey SILT, trace sand	36	21	75	4	1.87	117.0	-69.5
4	2.5	Clayey SILT, trace sand	29	33	65	2	1.86	117.0	-70.3
5	3.0	Clayey SILT, trace sand	40	36	62	2	1.37	46.8	-66.3
6	3.5	Clayey SILT, trace sand	53	40	58	2	1.38	48.9	-50.7
7	4.0	Clayey SILT, trace sand	60	40	58	2	0.91	17.9	-46.9
8	4.5	Clayey SILT	30	42	57	1	0.55	12.9	-29.3

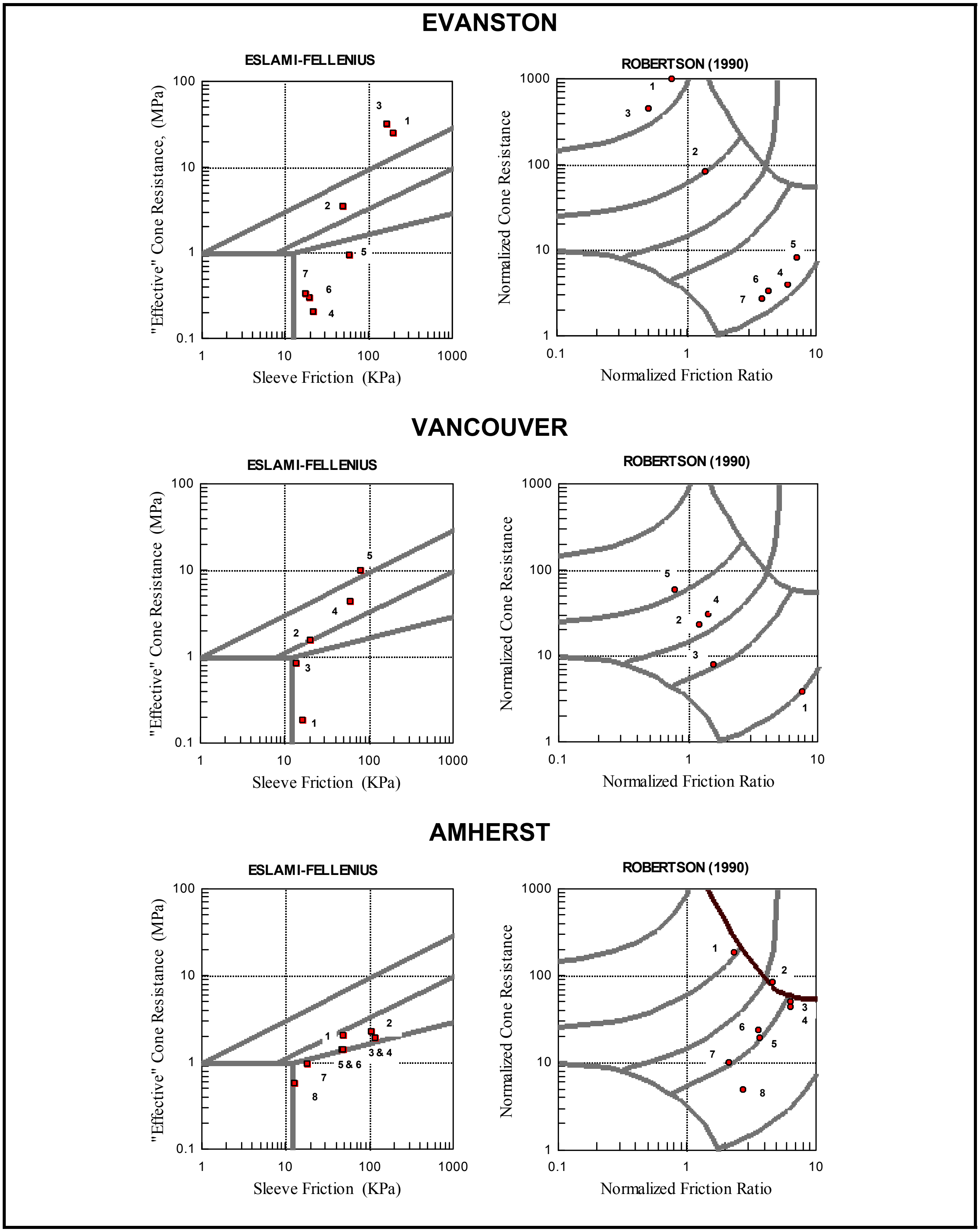

Fig. 13 Comparison between the Table 1 data plotted in Eslami-Fellenius and Robertson profiling charts

CONCLUSIONS

1. The CPT methods (mechanical cones) do not correct for the pore pressure on the cone shoulder and the profiling developed based on CPT data may not be relevant outside the local area where they were developed. The error due to omitting the pore water pressure correction is large in fine-grained soils and smaller in coarse-grained soils.
2. Except for the profiling chart by Begemann (1965) and Eslami-Fellenius (1997), all of the referenced soil profiling methods plot the cone resistance versus its own inverse value in one form of another. This generates data distortion and violates the rule that dependent and independent variables must be rigorously separated. The Eslami-Fellenius (1997) method avoids the solecism of plotting data against their own inverted values and associated distortion of the data.
3. Some profiling methods, e. g., Robertson (1990), include normalizations which require unwieldy manipulation of the CPT data. For example, in a layered soil, should a guesstimated "typical" total density value be used in determining the overburden stress or a value that accurately reflects density? Moreover, whether the soil is layered or not, determining the effective overburden stress (needed for normalization) requires knowledge of the pore pressure distribution. The latter is far from always hydrostatic but can have an upward or downward gradient; this information is rarely available.
4. The normalization by division with the effective overburden stress does not seem relevant. For example, the normalized values of fine-grained soils obtained at shallow depth (where the overburden stress is small) will often plot in zones for coarse-grained soil.
5. The Robertson (1990) and the Eslami-Fellenius (1997) CPTu methods of soil profiling were applied to data from three geographically separate sites having known soils of different types and geologic origins. Both methods identified the soil types accurately.

The CPTu is an excellent tool for the geotechnical engineer in developing a site profile. Naturally, it cannot serve as the exclusive site investigation tool and soil sampling is still required. However, when the CPTu is used to govern the depths from where to recover soil samples for detailed laboratory study, fewer sample levels are needed, reducing the costs of a site investigation while simultaneously increasing the quality of the information because important layer information and layer boundaries are not overlooked.

REFERENCES

Baligh, M. , Vivatrat, V., Wissa, A., Martin R., and Morrison, M., 1981. The piezocone penetrometer. Proceedings of Symposium on Cone Penetration Testing and Experience, American Society of Civil Engineers, ASCE, National Convention, St. Louis, October 26 - 30, pp. 247 - 263.

Begemann, H. K. S., 1953. Improved method of determining resistance to adhesion by sounding through a loose sleeve placed behind the cone. Proceedings of the 3rd International

Conference on Soil Mechanics and Foundation Engineering, ICSMFE, August 16 - 27, Zurich, Vol. 1, pp. 213 - 217.

Begemann, H. K. S., 1963. The use of the static penetrometer in Holland. New Zealand Engineering, Vol. 18, No. 2, p. 41.

Begemann, H. K. S., 1965. The friction jacket cone as an aid in determining the soil profile. Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, ICSMFE, Montreal, September 8 - 15, Vol. 2, pp. 17 - 20.

Campanella, R G., Gillespie, D., and Robertson, P. K., 1982. Pore pressures during cone penetration testing, Proceedings of the 2nd European Symposium on Penetration Testing, ESOPT-2, Amsterdam, May 24 - 27, Vol. 2, pp. 507 - 512.

Campanella, R. G., and Robertson, P. K., 1988. Current status of the piezocone test. Proceedings of First International Symposium on Penetration Testing, ISOPT-1, Orlando, March 22 - 24, Vol. 1, pp. 93 - 116.

Canadian Foundation Engineering Manual, 1985. Second Edition, Part 1 Fundamentals. Canadian Geotechnical Society, BiTech Publishers, Vancouver, BC, 456 p.

Douglas, B. J., and Olsen, R. S., 1981. Soil classification using electric cone penetrometer. American Society of Civil Engineers, ASCE, Proceedings of Conference on Cone Penetration Testing and Experience, St. Louis, October 26 - 30, pp. 209 - 227.

Erwig, H., 1988. The Fugro guide for estimating soil type from CPT data. Proceedings of Seminar on Penetration Testing in the UK, Thomas Telford, London, pp. 261 - 263.

Eslami, A., and Fellenius, B. H., 1995. Toe bearing capacity of piles from cone penetration test (CPT) data. Proceedings of the International Symposium on Cone Penetration Testing, CPT 95, Linköping, Sweden, October 4 - 5, Swedish Geotechnical Institute, SGI, Report 3:95, Vol. 2, pp. 453 - 460.

Eslami, A., and Fellenius, B. H., 1996. Pile shaft capacity determined by piezocone (CPTu) data. Proceedings of 49th Canadian Geotechnical Conference, September 21 - 25, St. John's, Newfoundland, Vol. 2, pp. 859 - 867.

Eslami, A., 1996. Bearing capacity of piles from cone penetrometer test data. Ph. D. Thesis, University of Ottawa, Department of Civil Engineering, 516 p.

Eslami, A., and Fellenius, B. H., 1997. Pile capacity by direct CPT and CPTu methods applied to 102 case histories. Canadian Geotechnical Journal, Vol. 34, No. 6, pp. 880 - 898.

Finno, R. J., 1989. Subsurface conditions and pile installation data. American Society of Civil Engineers, Proceedings of Symposium on Predicted and Observed Behavior of Piles, Evanston, June 30, ASCE Geotechnical Special Publication, GSP23, pp. 1 - 74.

Jefferies, M. G., and Davies, M. P., 1991. Soil classification using the cone penetration test. Discussion. Canadian Geotechnical Journal, Vol. 28, No. 1, pp. 173 - 176.

Jefferies, M. G., and Davies, M. P., 1993. Use of CPTu to estimate equivalent SPT N60. American Society for Testing and Materials, ASTM, Geotechnical Testing Journal, Vol. 16, No. 4, December, pp. 458 - 468.

Jones G. A., and Rust, E., 1982. Piezometer penetration testing, CUPT. Proceedings of the 2nd European Symposium on Penetration Testing, ESOPT-2, Amsterdam, May 24 - 27, Vol. 2, pp. 607-614.

Larsson, R., and Mulabdic, M., 1991. Piezocone tests in clay. Swedish Geotechnical Institute, SGI, Report No. 42, 240 p.

Lunne, T., Eidsmoen, D., and Howland, J. D., 1986. Laboratory and field evaluation of cone penetrometer. American Society of Civil Engineers, Proceedings of In-Situ 86, ASCE SPT 6, Blacksburg, June 23 - 25, pp. 714 - 729.

Mayne, P. W., Kulhawy F, and Kay, J. N., 1990. Observations on the development of pore water pressure during piezocone penetration in clays. Canadian Geotechnical Journal, Vol. 27, No. 4, pp. 418 - 428.

Olsen, R. S., and Farr, V., 1986. Site characterization using the cone penetration test. American Society of Civil Engineers, Proceedings of In-Situ 86, ASCE SPT 6, Blacksburg, June 23 - 25, pp. 854 - 868.

Olsen, R. S., and Malone, P. G., 1988. Soil classification and site characterization using the cone penetrometer test. Proceedings of First International Symposium on Cone Penetration Testing, ISOPT-1, Orlando, March 22 - 24, Vol. 2, pp. 887 - 893.

Olsen, R. S., and Mitchell, J. K., 1995. CPT stress normalization and prediction of soil classification. Proceedings of International Symposium on Cone Penetration Testing, CPT95, Linköping, Sweden, SGI Report 3:95, Vol. 2, pp. 257 - 262.

Robertson, P. K. and Campanella, R. G., 1983. Interpretation of cone penetrometer tests, Part I sand. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 718 - 733.

Robertson, P. K., Campanella, R. G., Gillespie, D., and Grieg, J., 1986. Use of piezometer cone data. Proceedings of American Society of Civil Engineers, ASCE, In-Situ 86 Specialty Conference, Edited by S. Clemence, Blacksburg, June 23 - 25, Geotechnical Special Publication GSP No. 6, pp. 1263 - 1280.

Robertson, P. K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151 - 158.

Sanglerat, G., Nhim, T. V., Sejourne, M., and Andina, R., 1974. Direct soil classification by static penetrometer with special friction sleeve. Proceedings of the First European Symposium on Penetration Testing, ESOPT-1, June 5 - 7, Stockholm, Vol. 2.2, pp. 337 - 344.

Schmertmann, J. H., 1978. Guidelines for cone test, performance, and design. Federal Highway Administration, Report FHWA-TS-78209, Washington, 145 p.

Searle, I. W., 1979. The interpretation of Begemann friction jacket cone results to give soil types and design parameters. Proceedings of 7th European Conference on Soil Mechanics and Foundation Engineering, ECSMFE, Brighton, Vol. 2, pp. 265 - 270.

Senneset, K., Sandven, R., and Janbu, N., 1989. Evaluation of soil parameters from piezocone test. In-situ Testing of Soil Properties for Transportation, Transportation Research Record, No. 1235, Washington, D. C., pp. 24 - 37.

Swedish State Railways Geotechnical Commission, 1922. Statens Järnvägars Geotekniska Kommission – Slutbetänkande. Swedish State Railways, Bulletin 2 (in Swedish with English summary), 228 p.

Vermeulen, N. and Rust, E., 1995. CPTu profiling – A numerical method. Proceedings of the International Symposium on Cone Penetration Testing, CPT 95, Linköping, Sweden, October 4 - 5, Swedish Geotechnical Institute, SGI, Report 3:95, Vol. 2, pp. 343 - 350.

Vos, J. D., 1982. The practical use of CPT in soil profiling, Proceedings of the Second European Symposium on Penetration Testing, ESOPT-2, Amsterdam, May 24 - 27, Vol. 2, pp. 933 - 939.