Estimating coefficient of consolidation from piezocone tests

P. K. Robertson
Civil Engineering Department, University of Alberta, Edmonton, Alta., Canada T6G 2G7
J. P. Sully
INTEVEP S.A., Caracas, Venezuela
D. J. Woeller
Conetec Investigations Ltd., Unit 3, 9113 Shaughnessy Street, Vancouver, B.C., Canada
T. Lunne
Norwegian Geotechnical Institute, P.O. Box 40 Täsen, N-0801, Oslo, Norway
J. J. M. Powell
Building Research Establishment, Garston, Watford WD2 7JR, United Kingdom
AND
D. G. Gillespie
British Columbia Ministry of Transportation and Highways, Geotechnical and Materials Branch, 3149 Production Way, Burnaby, B.C., Canada V5A 3H1

Received September 30, 1991
Accepted January 13, 1992

Data have been reviewed from sites in Europe and North and South America as well as published data from South Africa. The review has concentrated on dissipation data from piezocone tests (CPTU) to compare predicted coefficient of consolidation and permeability values using published interpretation techniques with available reference values. The results of this review have shown that the theoretical solutions provide reasonable estimates of the *in situ* coefficient of consolidation. Results were evaluated for pore-pressure data from different locations on the piezocone, and the least scatter in results was obtained with the pore-pressure element location immediately above the cone tip. A new correlation has been proposed to estimate *in situ* horizontal coefficient of permeability (k_h) from piezocone dissipation data.

Key words: *in situ*, coefficient consolidation, cone penetration test, permeability.

L'on a passé en revue des données provenant de sites en Europe et en Amérique du Nord et du Sud, de même que des données publiées en Afrique du Sud. L'on s'est arrêté particulièrement aux données de dissipation des essais de piézocône (CPTU) pour comparer le coefficient de consolidation prédit et les valeurs de perméabilité au moyen de techniques d'interprétation publiées avec les valeurs de référence disponibles. Les résultats de cette revue ont montré que les solutions théoriques fournissent des estimations raisonnables du coefficient de consolidation *in situ*. Des résultats ont été évalués en partant de données piézométriques mesurées à différentes positions sur le piézocône, et la dispersion la plus faible dans les résultats a été obtenue avec l'élément poreux situé immédiatement au-dessus de la pointe conique. Une nouvelle corrélation est suggérée pour évaluer le coefficient de perméabilité horizontal (k_h) *in situ* en partant des données de dissipation du piézocône.

Mots clés : *in situ*, coefficient de consolidation, l'essai de piézocône, perméabilité.

[Traduit par la rédaction]

Can. Geotech. J. **29**, 539–550 (1992)

Introduction

A project was initiated to review existing *in situ* test data and associated laboratory and performance data from well-documented geotechnical research sites. The objective of the review was to evaluate current interpretation methods of *in situ* tests for the determination of consolidation and permeability parameters of soils. Data were reviewed from over 30 sites in Europe and North and South America as well as published data from South Africa.

For the sites evaluated in this study, the largest data base of *in situ* test results available was from the cone penetration test with pore-pressure measurements (CPTU). Hence, to perform a reasonable evaluation of predictive techniques only CPTU data have been used for this study. The restriction of this study to the CPTU does not imply any bias towards this test but reflects common practice and availability of *in situ* dissipation test data.

The main advantages of the cone penetration test (CPT) are its simplicity, repeatability, and speed. The cone penetration test with pore-pressure measurements, commonly referred to as the piezocone test (CPTU), provides added advantages of (*i*) ability to distinguish drainage conditions during cone penetration; (*ii*) ability to correct measured cone penetration resistance (q_c) and to some extent sleeve friction (f_s), to account for unbalanced water forces due to unequal end area in cone designs; (*iii*) ability to assess equilibrium groundwater conditions; (*iv*) improved soil profiling and identification; (*v*) improved evaluation of geotechnical parameters; and (*vi*) ability to evaluate flow and consolidation characteristics.

The main objective of this paper is to present a summary of the comparison between the measured available reference values of coefficient of consolidation and permeability with the theoretically derived parameters from CPTU dissipation

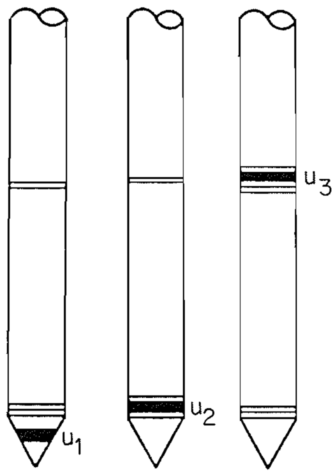

FIG. 1. Pore-pressure element locations (u_1–u_3) selected for this study.

tests. Full details of the study are given in a report by Robertson *et al.* (1990).

Data review

It is possible to measure pore pressures at various locations on a cone penetrometer. To assist in the definition of the pore-pressure data recorded at different locations, the following system has been used: (*i*) u_1 = pore pressure measured midway along the face of the cone tip; (*ii*) u_2 = pore pressure measured just above the cone tip; and (*iii*) u_3 = pore pressure measured above the friction sleeve on the shaft of the cone.

This system is the same as that used by Sully *et al.* (1988). Figure 1 shows these pore-pressure measurement locations.

The penetration pore pressures measured at any location on a cone can be divided into two components: (*i*) *in situ* equilibrium value u_0, which is controlled by local groundwater regime; and (*ii*) excess pore pressure generated by penetration of the cone, Δu, which is a function of both the soil behaviour and the cone geometry. Thus

$$[1] \quad u_{1,2,3} = u_0 + \Delta u_{1,2,3}$$

where the subscripts refer to the piezoelement location defined previously.

The decay with time of the excess pore-water pressure provides information concerning the coefficient of consolidation of the soil. To evaluate the dissipation of the generated excess pore pressure, cone penetration is stopped and the decay of pore pressure with time is recorded. The change in pore pressure is usually plotted against log time. Either the excess pore pressure or the normalized excess pore-pressure decay can be plotted:

$$[2] \quad \Delta u = u - u_0$$

or

$$[3] \quad U_t = \frac{u_t - u_0}{u_i - u_0} = \frac{\Delta u_t}{\Delta u_i}$$

where U_t = the normalized excess pore pressure at time t, u_t = the excess pore pressure at time t, and u_i = the initial excess pore pressure when penetration is stopped (t = 0). U_t usually varies between 1 (t = 0) and 0 when the excess pore pressure has completely dissipated and $u_t = u_0$.

TABLE 1. Suggested anisotropic permeability of clays (after Baligh and Levadoux 1980)

Nature of clay	k_h/k_v
No evidence of layering	1.2 ± 0.2
Slight layering, e.g., sedimentary clays with occasional silt dustings to random lenses	2–5
Varved clays in northeastern United States	10 ± 5

During cone penetration excess pore pressures are generated. These excess pore pressures are generally referred to as penetration pore pressures. In clean, medium- to coarse-grained sands the excess pore pressures dissipate as fast as they are generated, and measured penetration pore pressures are often equal to or very close to the static equilibrium pore pressure; hence the penetration process can be classed as drained. In fine-grained soils, such as silts and clays, the CPT penetration process is undrained, and large penetration pore pressures can be generated. Theories to predict consolidation parameters are based on an undrained penetration process, and hence the treatment of the available data is restricted primarily to fine-grained soils, where consolidation parameters are of interest.

The primary objective of this study was to compare theoretically derived values of consolidation parameters from CPTU dissipation tests with measured available reference values for different soil deposits. One of the major complications in this task was the selection of representative reference values of the consolidation parameters. The main consolidation parameters considered are the coefficient of consolidation (c) and the coefficient of permeability (k). Because of soil anisotropy and stratigraphy these values are often different in the horizontal and vertical directions (i.e., $c_v \neq c_h$ and $k_v \neq k_h$). The two principal methods available for deriving these reference values of consolidation parameters are (*i*) back analyses of field performance, and (*ii*) interpretation of laboratory tests on small samples.

The back analysis of field performance usually represents the ideal way to evalutate overall field consolidation parameters for a site. However, there are limited numbers of fully instrumented and well-documented field performance sites where CPTU or other *in situ* tests have been performed. The back analysis of field performance data is often complex, because variations in soil stratigraphy often make it difficult to evaluate the correct drainage path during consolidation. Minor interbedding of sand or silt layers can make the selection of the drainage path lengths difficult. Further problems related to back analysis of field performance data can also result from uncertainty in loading conditions and problems with field instrumentation. Hence, although field performance records, conceptually represent an ideal basis for evaluating reference values of consolidation parameters, there remains some uncertainty in the derived values. Back analysis of some of the field performance data provides an estimate of the vertical consolidation parameter (c_v), whereas the CPTU interpretation primarily provides an estimate of c_h (Baligh and Levadoux 1980). In the case of soils where vertical sand or wick drains have been installed, a more reliable estimate of c_h is obtained from back analysis of field performance due to the predominance of horizontal drainage and a clearer definition of drainage path.

TABLE 2. List of sites reviewed with associated references

Site (location)	Soil type[a]	Ref.
Amherst (U.S.A.)	Connecticut Valley varved clay; moderately to lightly overconsolidated	Baligh and Levadoux (1980)
Attakapa Landing (U.S.A.)	Soft clay and clayey silt; normally consolidated	M.T. Tumay (personal communication, 1990)
Saugus (U.S.A.)	Boston Blue Clay; moderately to lightly overconsolidated	Baligh and Levadoux (1980)
Brage, North Sea (Norway)	Silty marine clay and sandy silt; normally consolidated	Norwegian Geotechnical Institute (1988*a*)
Saint-Alban (Canada)	Champlain Sea Clay; very sensitive, lightly overconsolidated (aged)	Roy *et al.* (1982); Tavenas *et al.* (1974)
McDonald Farm (Canada)	Normally consolidated clayey silt	Gillespie and Campanella (1981); Gillespie (1981); Hers (1989)
Burnaby (Canada)	Normally consolidated clay silt	Gillespie and Campanella (1981)
Troll, North Sea (Norway)	Sandy marine clay; lightly overconsolidated	Norwegian Geotechnical Institute (1984)
Onsoy (Norway)	Marine clay silt; normally consolidated at depth	Mokkelbost (1988)
Snorre, North Sea (Norway)	Silty sandy stiff marine clay; lightly overconsolidated	Norwegian Geotechnical Institute (1988*b*)
Stjordal-Halsen (Norway)	Marine clay sandy silt; normally consolidated	Senneset *et al.* (1982)
Drammen (Norway)	Normally consolidated clay silt; sensitive	Mokkelbost (1988)
Strong Pit, B.C. (Canada)	Glaciomarine clayey silt; lightly to moderately overconsolidated	Campanella *et al.* (1988)
Colebrook Overpass, B.C. (Canada)	Sensitive silty clay; normally consolidated (aged)	Crawford and Campanella (1991)
Haga (Norway)	Lean marine clay; moderately overconsolidated	Mokkelbost (1988)
Fucino (Italy)	Clayey silt lake deposits; normally consolidated	Associazione Geotechnica Italiana (1979); Marchetti and Totani (1989)
Porto Tolle (Italy)	Soft clayey silt; normally consolidated	Battaglio *et al.* (1981); Jamiolkowski *et al.* (1979)
Guanabara Bay (Rio de Janeiro, Brazil)	Soft silty clay; normally consolidated	Sills *et al.* (1988)
Gainesville, Florida (U.S.A.)	Lake Alice Clay; clayey silty sand to clay	Gupta and Davidson (1986)
Norco, Louisiana (U.S.A.)	Silty sand clay; normally consolidated	Tumay and Acar (1984)
Langley, Lower 232 Street (Canada)	Sensitive clay silt; lightly overconsolidated to normally consolidated	Zavoral (1988)
Lulu Island, B.C. (Canada)	Organic clayey silt; normally consolidated	Hers (1989); Gillespie (1990)
Laing Bridge South, B.C. (Canada)	Soft deltaic clayey silt; normally consolidated	Le Clair (1988)
Trieste (Italy)	Homogeneous soft organic clay; normally consolidated	Battaglio *et al.* (1981); Jamiolkowski *et al.* (1983)
Site I/II (France)	Clayey silt	Parez and Bachelier (1981)
Glava (Norway)	Medium stiff marine clay; overconsolidated	Senneset *et al.* (1989)
SLS (U.S.A.)	Soft to medium silty clay; lightly overconsolidated	Kabir and Lutenegger (1990)
Storz 264, Nebraska (U.S.A.)	Florence Lake clays; normally consolidated	Lutenegger *et al.* (1988)

TABLE 2 (*concluded*)

Site (location)	Soil type[a]	Ref.
South Africa	Various normally consolidated materials (tailings, aluminum)	Jones and Van Zyl (1981)
Brent Cross (U.K.)	Stiff, heavily consolidated London Clay, fissured	Powell *et al.* (1988)
Bothkennar (U.K.) (Grangemouth)	Soft, slightly fissured organic silty clay	Powell *et al.* (1988)
Cowden (U.K.)	Stiff, glacial till	Powell and Quarterman (1988)
Madingley, (U.K.)	Gault Clay; stiff, fissured silty clay	Lunne *et al.* (1986)

[a]Lightly overconsolidated, $1 < \mathrm{OCR} < 4$; moderately overconsolidated, $4 < \mathrm{OCR} < 10$; heavily overconsolidated, $10 < \mathrm{OCR}$.

It is possible to correct the derived c_v values to c_h using the following equation:

$$[4] \quad c_h = c_v \frac{k_h}{k_v}$$

Based on North American experience, Baligh and Levadoux (1980) suggested values of k_h/k_v for various soil types (Table 1). A similar classification for k_h/k_v has been suggested by Jamiolkowski *et al.* (1985). Where field or laboratory measurements of directional properties are not available, Table 1 has been used to evaluate c_h from c_v.

The majority of sites reviewed for this project have consolidation and permeability parameters derived from laboratory testing on small, undisturbed samples. The primary laboratory test performed was the one-dimensional (oedometer) consolidation test, of which both incremental load and constant rate of strain results are available. One problem associated with the determination of consolidation and permeability parameters from laboratory testing relates to the small size of the sample and the unknown influence of sample disturbance. For soils that are interbedded or have some fabric, such as fissures or layering, laboratory testing on small intact samples can be misleading. Almost all recovered samples have some level of disturbance, hence laboratory-derived parameters may not be entirely representative of *in situ* conditions. Techniques have been developed to correct oedometer-derived consolidation parameters to account for disturbance (Sandbaekken *et al.* 1985). Two advantages with laboratory testing to determine consolidation parameters are that samples can be tested in both the vertical and horizontal direction to derive both c_h and c_v and that these parameters can be determined at different stress levels. Tavenas *et al.* (1983) discuss other limitations with the determination of consolidation parameters from oedometer tests.

Detailed *in situ* and laboratory data were obtained from over 30 sites. In almost all cases only interpreted laboratory and (or) field data were available and not the basic settlement or pore-pressure data which would have permitted independent derivation of parameters. A list of the sites from which data have been obtained is given in Table 2.

The laboratory reference coefficient of consolidation values have been selected at approximately the same overburden stress that existed at that depth in the field, i.e., at the same overconsolidation ratio (OCR) as exists *in situ*. This was considered to be a logical choice to predict the *in situ* response of the soil to imposed loads. If the soil is overconsolidated the reference value of c_v represents the overconsolidated range.

To ensure standardization, only time values for 50% dissipation of excess pore pressures ($U_t = 0.5$) have been taken from the *in situ* CPTU dissipation curves. Baligh and Levadoux (1980) suggest that at this degree of dissipation or greater the calculated c_h from piezocone dissipation should correspond to the *in situ* consolidation state of the soil.

Obviously, with any study of this kind, some degree of interpretation and (or) extrapolation has been necessary to maximize the data. In determining the average values of t_{50} from CPTU dissipations and c_v from laboratory consolidation tests, extreme values have been discarded where these have been considered to be nonrepresentative of the complementary data set on the particular soil. No distinction has been made between c_v values from different types of oedometer test or for that matter triaxial tests. Furthermore, no depth dependence of c_v from laboratory data has been considered, since the complete data set to permit this was not always available. However, for most sites this was not necessary, as global ranges adequately reflected the ranges of $c_{v,h}$ obtained at any one site. Nothwithstanding this, where stratigraphic changes warrant, separate $t_{50} - c_v$ averages have been used for separate layers.

Theoretical background of CPTU dissipation test

In the past 15 years, several techniques have been developed to evaluate consolidation characteristics from dissipation tests using the CPTU (Torstensson 1975, 1977; Baligh and Levadoux 1980; Gupta and Davidson 1986; Teh 1988).

A CPTU dissipation test consists of stopping cone penetration and monitoring the decay of excess pore pressures (Δu) with time. Excess pore pressure is defined as the difference between the penetration pore pressure (u) and the static equilibrium pore pressure (u_0). From the dissipation data an approximate value of the coefficient of consolidation in the horizontal direction (c_h) can be derived.

A comprehensive study and review of this topic was published by Baligh and Levadoux (1986), and the relevant conclusions from this study are as follows. (1) The simple uncoupled solutions provide reasonably accurate predictions

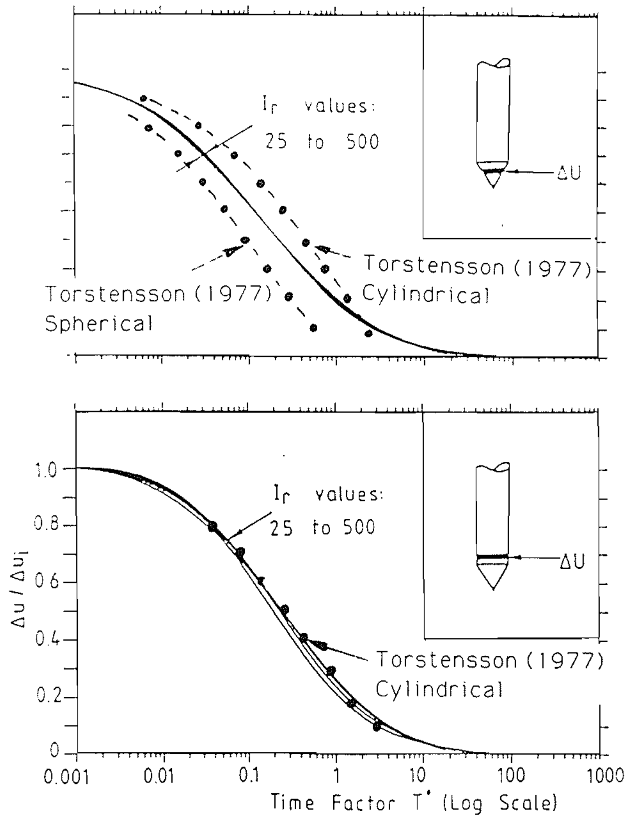

FIG. 2. Normalized dissipation curves in log time based on theoretical solution by Teh and Houlsby (1991).

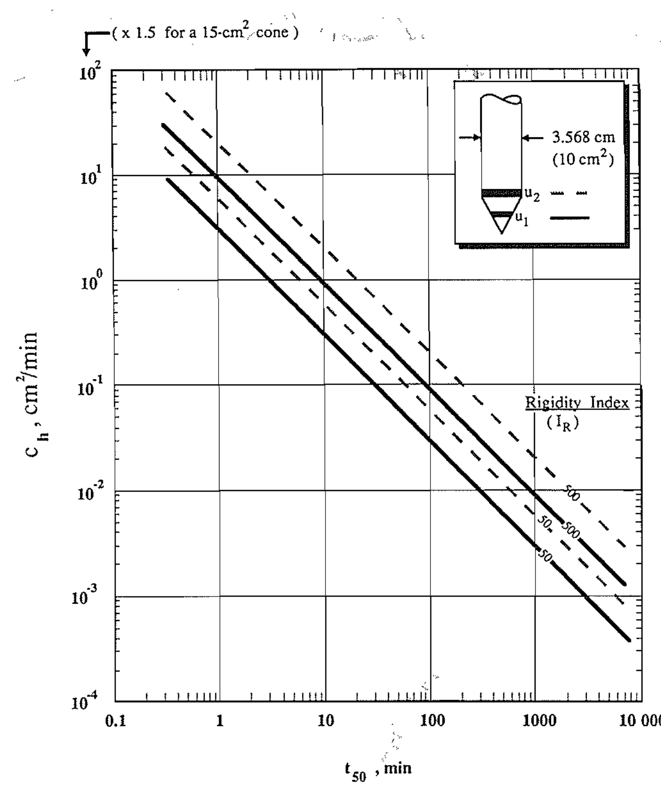

FIG. 3. Chart for evaluation of c_h from t_{50} for 10 and 15 cm² piezocone (based on theoretical solution by Teh and Houlsby 1991).

of the dissipation process. (2) Consolidation is taking place predominantly in the recompression mode for dissipation less than 50%. (3) Initial distribution of excess pore pressures around the probe has a significant influence on the dissipation process.

Research by Torstensson (1977) and Teh and Houlsby (1991) has also illustrated the importance of soil stiffness on the pore-pressure dissipation. The soil stiffness is usually represented by the rigidity index I_r ($= G/s_u$, where G is the shear modulus and s_u is the undrained shear strength).

At present, the existing theoretical solutions provide reasonable approximations of the initial distribution of excess pore pressures around a probe in soft, normally to lightly overconsolidated soils. However, in overconsolidated soils (OCR $\gg$ 4) the existing solutions provide a rather poor estimate of the initial distribution of pore pressure and hence have not been used extensively to evaluate consolidation characteristics in such soils.

In stiff, heavily overconsolidated soils, the pore-pressure gradient around a 60° cone penetrometer can be extremely large. This gradient of pore pressure often results in dissipations recorded above the cone tip which initially increase before decreasing to the final equilibrium value. Campanella and Robertson (1988) suggested that this type of response is due to the redistribution of excess pore pressures around the cone before the primarily radial drainage occurs, although poor saturation of the pore-pressure element can also cause a similar response.

Variations in the initial distribution of excess pore pressures around the probe represent one of the major difficulties for the theoretical solutions. If the initial distribution is significantly different than the theoretical distribution, the shape of the measured pore-pressure dissipation curve will be different than the theoretical curves. Errors in the measurement of the initial pore pressure and estimates of the equilibrium pore pressure have the least effect on c_h for $U_t = 0.5$, (i.e., t_{50} compared with other values of U_t (Baligh and Levadoux 1986).

Teh and Houlsby (1991) observed that the theoretical dissipation curves could be normalized using the following modified time factor, T^*:

$$[5] \quad T^* = \frac{c_h t}{R_2 I_r^{1/2}}$$

where T^* = modified time factor for a given probe geometry and porous element location, c_h = coefficient of consolidation in the horizontal direction, t = measured time, R = radius of the probe, and I_r = rigidity index = G/s_u Figures 2 shows the modified time factors derived by Teh and Houlsby (1991) and compares them with those derived by Torstensson (1977) for the pore-pressure element locations immediately above the tip (u_2) and on the face of the tip (u_1). It is interesting to note that the simplified solutions by Torstensson (1977) provide essentially the same values as the most recent solutions by Teh and Houlsby (1991). However, since the solutions by Torstensson (1977) are based on cavity expansion (cylindrical and spherical), they are unable to clearly define different response curves for different pore-pressure element locations.

The solutions given by Teh and Houlsby (1991) represent the most recent and comprehensive theoretical study of the CPTU dissipation test. Therefore, this method has been selected to form the framework of comparison for this study. A chart has been developed (Fig. 3) that plots the

TABLE 3. Summary of CPTU field dissipation results for u_1 position in terms of t_{50} and laboratory oedometer results

Site (location)	Pore-pressure location	t_{50}, min		c_v, cm^2/min		k_h/k_v
		Range	Average	Range	Average	
Guanabara Bay (Rio de Janeiro, Brazil)	u_1	3.8–9.2	5.6	0.012–0.06	0.036	1
Saugus, I-5 (U.S.A.)	u_1	4.9–8.8	6.7	0.06–0.12	0.09	2(>18 m)
	u_1	0.2–2.4	1.1	0.12–0.6	0.36	2(<18 m)
Colebrook overpass, B.C. (Canada)	u_1	1–7.2	2.9	0.018–0.17	0.093	2.5
McDonald Farm (Canada)	u_1	2.8–6.0	4.4	1.1–3.3	2.2	0.7–1.1
Strong Pit, B.C. (Canada)	u_1	6.3–10.5	8.4	0.038–0.17	0.057	2
Fucino (Italy)	u_1	2.6–21	8.5	0.003–0.014	0.008	2
Porto Tolle (Italy)	u_1	2.8–8.9	5.2	0.13–0.21	0.17	1.4
Brage (Norway)	u_1	0.13–2.0	1.05	0.46–17.5	5.4	1
Drammen (Norway)	u_1	1.6–4.5	3.5	0.004–0.008	0.006	1
Haga (Norway)	u_1	14.6	14.6	0.17–0.35	0.26	2
Onsoy (Norway)	u_1	2–12	5.9	0.06–0.13	0.1	2.5
Snorre (Norway)	u_1	2–7	4.3	0.05–0.17	0.11	1
Stjordal-Halsen (Norway)	u_1	0.15–0.53	0.27	2.9–14.7	7.9	1
Troll (Norway)	u_1	3–24	8.3	0.04–0.16	0.12	1
Amherst (U.S.A.)	u_1	0.05–2.5	0.48	0.06–0.12	0.09	8
Attakapa Landing (U.S.A.)	u_1	8–50	24	0.014–0.024	0.019	1
Lake Alice Clay (U.S.A.)	u_1	1.2–13.3	7.31	0.06–0.57	0.43	1
Saint-Alban (Canada)	u_1	4.1–6.5	5.0	0.003–0.006	0.0045	1
Norco, Louisiana (U.S.A.)	u_1	94.7–117.7	106.2	0.006–0.018	0.12	1(10–15 m)
	u_1	49.4–69.9	59.7	0.012–0.036	0.024	1(20–35 m)
	u_1	25.7–33.4	29.6	0.012–0.09	0.051	1(35–40 m)
Cowden (U.K.)	u_1	17–70	33	0.03–0.114	0.072	1
Madingley (U.K.)	u_1	1.5–12.0	4.5	0.019–0.048	0.04	2
Brent Cross (U.K.)	u_1	60–280	128	0.005–0.019	0.0119	2

derived value of c_h based on the solutions by Teh and Houlsby (1991), against the time for 50% dissipation for two of the selected pore-pressure element locations (u_1 and u_2). The chart shown in Fig. 3 will therefore form the framework for the comparison between predicted and measured coefficients of consolidation (c_h). For cones with a 15-cm^2 cross-sectional area, the scale of the derived c_h needs to be increased by a factor of 1.5, as indicated in Fig. 3.

Campanella and Robertson (1988) suggested that the applicability and meaning of the theoretical solutions is complicated by several phenomena, such as importance of vertical as well as horizontal dissipation; effects of soil disturbance; uncertainty over distribution, level, and changes in total stresses; soil anisotropy, nonlinearity, and creep; soil layering or nearness to a layer boundary; influence of macrofabric, such as fissuring; and influence of clogging and smearing of the porous filter element. Despite these above limitations, Campanella and Robertson (1988) suggested that the CPTU dissipation test provides an economic and useful means of evaluating approximate consolidation properties, soil macrofabric, and related drainage paths of natural, fine-grained soil deposits.

Evaluation of CPTU dissipation data

Tables 3–5 present a summary of the compiled data in terms of measured CPTU dissipation times (t_{50}) and reference coefficient of consolidation values. Figures 4 and 5 present all the available CPTU data in terms of t_{50} for each of the pore-pressure locations u_1 and u_2 and reference c_h values obtained from laboratory tests on undisturbed samples. For the purpose of clear presentation, only average values of t_{50} and c_h have been shown. The available data for pore-pressure location u_3 was very limited (only 7 sites) and has not been presented in this paper. Full details of all the data are given in the report by Robertson *et al.* (1990).

The practise at the Norwegian Geotechnical Institute (NGI) is to correct the oedometer c_v values for sample disturbance. Details of the correction procedure are given by Sandbaekken *et al* (1985). Although this correction appears to be quite large for some sites, the relative change in c_h values is generally quite small in terms of the overall range of values encountered.

The results show the following main points. (1) The trend and magnitude of the measured data (t_{50}, c_h) is generally consistent with the proposed theoretical framework. (2) Of the pore-pressure locations considered, the u_2 data appear

TABLE 4. Summary of CPTU field dissipation results for u_2 position in terms of t_{50} and laboratory oedometer results

Site (location)	Pore-pressure location	t_{50}, min		c_v, cm^2/min		k_h/k_v
		Range	Average	Range	Average	
Guanabara Bay (Rio de Janeiro, Brazil)	u_2	6.1–16.5	11.3	0.012–0.06	0.036	1
Colebrook Overpass, B.C. (Canada)	u_2	8.8–98	61	0.018–0.17	0.093	2.5
Laing Bridge South (Canada)	u_2	0.75–11.5	4.4	0.102–0.27	0.19	1
Langley, Lower 232 Street (Canada)	u_2	25–83	42	0.19–0.018	0.074	3
Lulu Island, B.C. (Canada)	u_2	2.5–10	5.3	1.9–4.2	3.1	1
McDonald Farm (Canada)	u_2	2.1–7.8	5.0	1.1–3.3	2.2	0.7–1.1
Strong Pit, B.C. (Canada)	u_2	37–66	46	0.038–0.17	0.057	2
Brage (Norway)	u_2	0.22–2.5	1.17	0.46–17.5	5.4	1
Drammen (Norway)	u_2	15–26	21	0.004–0.008	0.006	1
Haga (Norway)	u_2	12–44	25.4	0.17–0.35	0.26	2
Onsoy (Norway)	u_2	7–40	21	0.06–0.13	0.1	2.5
Snorre (Norway)	u_2	33	33	0.05–0.17	0.11	1
Stjordal-Halsen (Norway)	u_2	1–4.4	2.85	2.9–14.7	7.9	1
Troll (Norway)	u_2	90–180	137	0.04–0.16	0.12	1
SLS (U.S.A.)	u_2	29.6	29.6	0.04–0.6	0.12	
Storz 264, Nebraska (U.S.A.)	u_2	60–190	105	0.006–0.06	0.024	1
Saint-Alban (Canada)	u_2	9.6–13	11.3	0.003–0.006	0.0045	1
Glava (Norway)	u_2	—	18	0.14–0.21	0.16	1.5
Site I/II (France)	u_2	7.5–15	11.5	0.9	0.9	1
Trieste (Italy)	u_2	20–48	32	0.006–0.012	0.012	1
Porto Tolle (Italy)	u_2	15–25	20	0.13–0.21	0.14	1.4
Cowden (U.K.)	u_2	—	44	0.03–0.114	0.072	1
Bothkennar (U.K.)	u_2	12–90	39	0.079–0.048	0.03	1

TABLE 5. Summary of CPTU field dissipation results for u_3 position in terms of t_{50} and laboratory oedometer results

Site (location)	Pore-pressure location	t_{50}, min		c_v, cm^2/min		k_h/k_v
		Range	Average	Range	Average	
Guanabara Bay (Rio de Janeiro, Brazil)	u_3	27–44	35.5	0.012–0.06	0.036	1
Saint-Alban (Canada)	u_3	15.6–18.5	17.1	0.003–0.006	0.0045	1
Laing Bridge South (Canada)	u_3	6–22	12.5	0.1042–0.27	0.19	1
Attakapa Landing (U.S.A.)	u_3	23–295	103	0.014–0.024	0.019	1
Strong Pit (Canada)	u_3	94–150	122	0.038–0.17	0.057	2
Brage (Norway)	u_3	0.67–4.3	2.2	0.46–17.5	5.4	1
McDonald Farm (Canada)	u_3	8.3–16.7	12.5	1.1–3.3	2.2	0.7–1.1

to show the least scatter and compare well with the proposed theoretical framework. (3) For the u_1 and u_3 locations, the data show a scatter larger than the range suggested by the theoretical framework. (4) The reference laboratory c_h values tend to be less than the predicted values for pore-pressure locations u_1 and u_3.

For two sites, Lake Alice and Norco, the data and soil conditions are such that sufficient data are available to derive c_h and t_{50} values for different stratigraphic sequences of the profile. Figure 6 presents the values of c_h and t_{50} for these sites and shows that the trend of c_h with t_{50} is essentially parallel to the theoretical framework.

Two sites (Saint-Alban and Drammen) appear to consistently plot below the theoretical framework and data from other sites. Both these sites comprise either sensitive or structured clays. Hence, the remolding, due to cone penetration and possibly sampling, may have some influence on the t_{50} and laboratory c_h values.

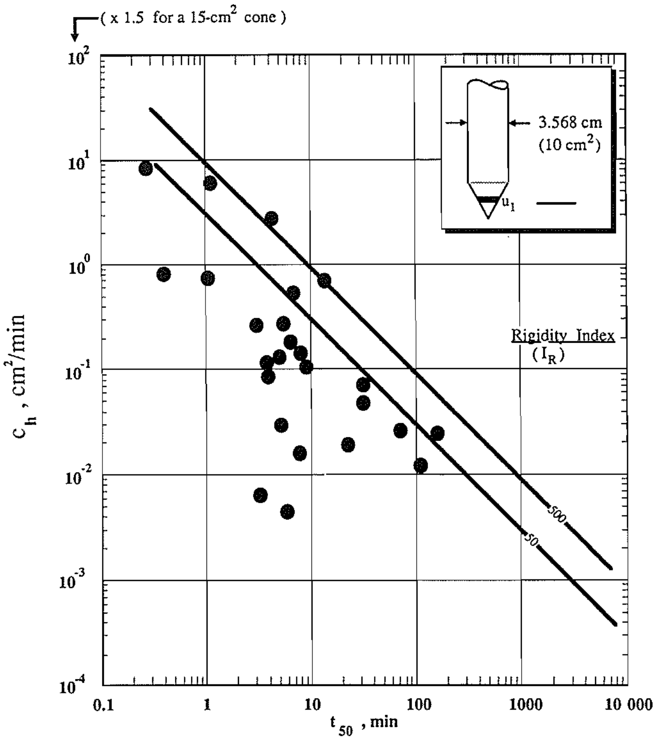

FIG. 4. Average laboratory oedometer c_h results and CPTU results in terms of t_{50} for u_1 pore-pressure location.

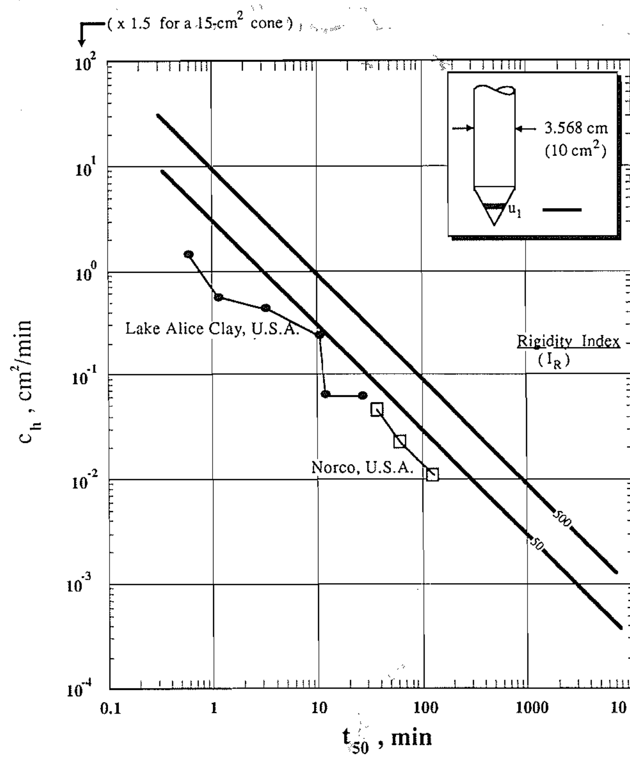

FIG. 6. Variation of c_h and t_{50} with depth for two sites.

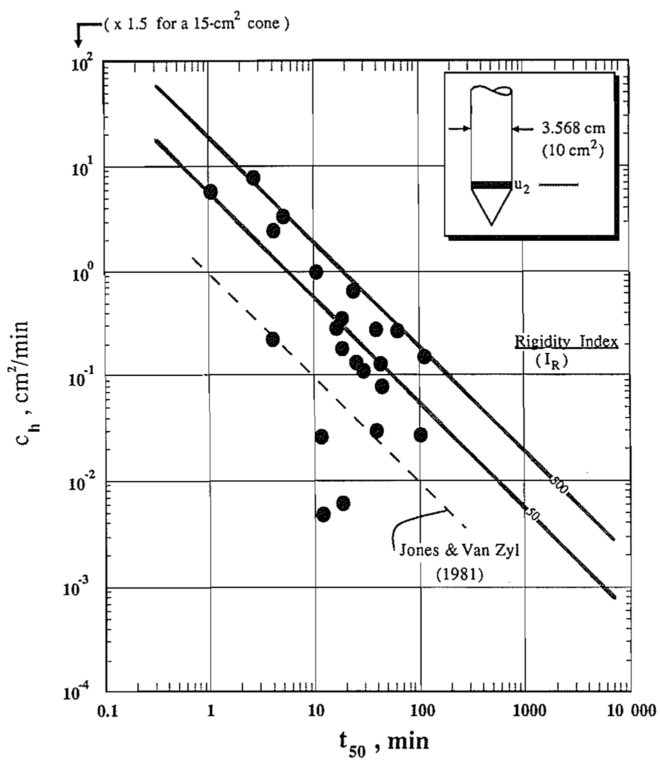

FIG. 5. Average laboratory oedometer c_h results and CPTU results in terms of t_{50} for u_2 pore-pressure location.

For the u_2 location, the majority of t_{50} values are between 10 and 100 min. It may not be economically convenient to stop the penetration process long enough to obtain the t_{50} time. Hence, it may be desirable to perform and interpret shorter dissipations. The theoretical solutions allow interpretation at any degree of dissipation. However, the measured dissipation curves are often different than the theoretical curves, resulting in different values of c_h for different degrees of consolidation. To illustrate this effect, data from one site (Onsoy) have been analyzed to compare the c_h values from different degrees of consolidation with the c_h values derived from t_{50} for different pore-pressure element locations. This comparison is shown in Fig. 7 and suggests the following. (1) For the pore-pressure element location u_1, the c_h derived from degrees of dissipation shorter than 50% can be significantly larger than c_h at 50% dissipation. (2) For the pore-pressure element location u_2, the c_h derived from degrees of dissipation shorter than 50% is only slightly larger than c_h at 50% dissipation. The large variation in c_h for short dissipation times may be a function of (*i*) test procedure in terms of unloading on the face of the cone due to the stress relaxation in the push rods during the pause in penetration; (*ii*) limitation of the theory to adequately predict the initial distribution of pore pressure; (*iii*) changes in the influence of the ratio c_h/c_v due to pore-pressure element location, especially for the u_1 location; (*iv*) variation in stress history effects at degrees of dissipation less than 50%; and (*v*) errors in measurement of initial penetration pore pressure and (or) equilibrium pore pressure (u_0).

The data presented in Figs. 4–7 represent laboratory-derived values of c_h. For a limited number of sites reference values of c_h could also be derived from published back analysis of field performance. The comparison of the field (c_h, field) and laboratory (c_h, lab) derived values are shown in Figs. 8 and 9 for average values of t_{50} and c_h for the u_1 and u_2 locations, respectively. A summary of the available data relating field and laboratory c_h values is shown in Table 6. Figures 8 and 9 show that the values of c_h from field performance are generally larger than c_h derived from

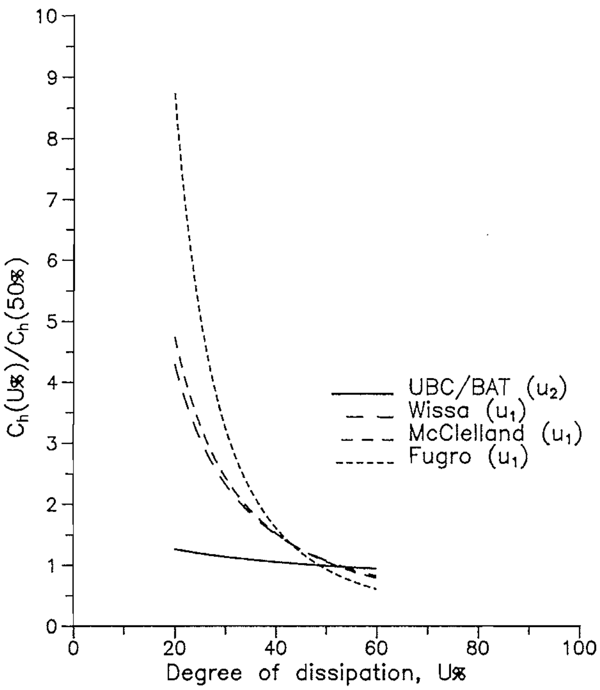

FIG. 7. Variation in c_h determined at different degrees of dissipation compared with c_h determined at 50% dissipation. Onsoy clay dissipation data from Soares *et al.* (1987), and c_h values from Torstensson cavity expansion theory. Curves are best fit lines to individual data sets for the cones UBC/BAT, Wissa, McClelland, and Fugro.

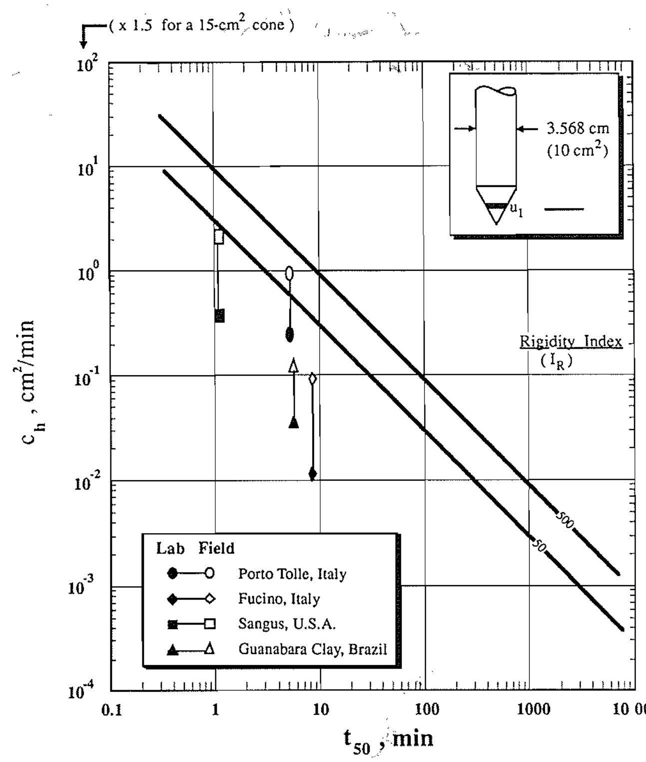

FIG. 8. Comparison between field back-analyzed and laboratory c_h values for CPTU t_{50} values for u_1 pore-pressure location.

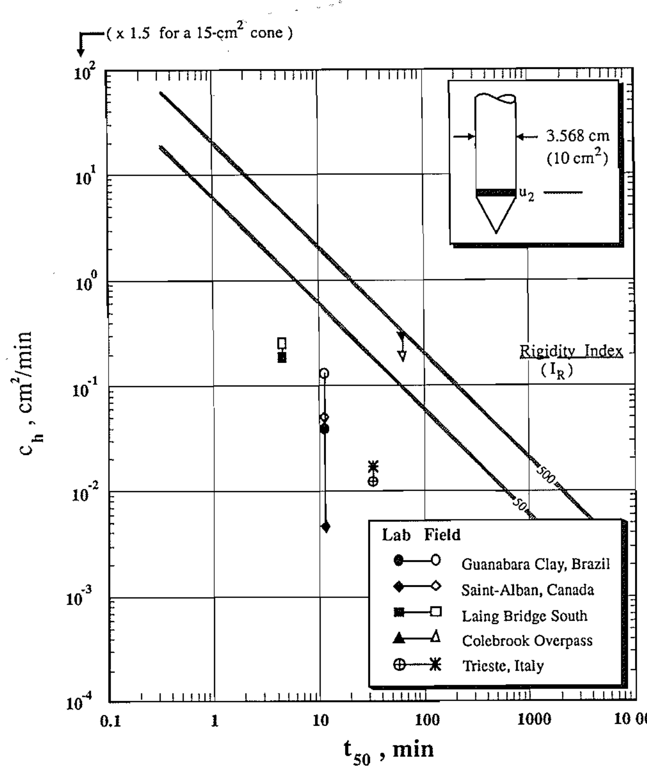

FIG. 9. Comparison between field back-analyzed and laboratory c_h values for CPTU t_{50} values for u_2 pore-pressure location.

laboratory testing and that the largest difference appears to occur for the more sensitive and structured clays. This is consistent with the results presented by Tavenas *et al.* (1986) Figures 8 and 9 would then suggest that, if the diference between c_h(lab) and c_h(field) is similar for most sites, then the data shown in Figs. 4 and 5 would generally plot closer to the theoretical framework. However, the difference between c_h(field) based on performance and c_h(lab) also appears to be a function of soil structure.

So far, data have only been presented in terms of c_h. This is because the rate of pore-pressure dissipation during a pause in the CPT is theoretically related to c_h. However, data are available from some of the sites to evaluate the direct empirical correlation between t_{50} and the laboratory-derived values of the horizontal coefficient of permeability k_h. A summary of the available data is shown in Table 7. Figure 10 presents the available data of k_h and t_{50} and compares these with a preliminary relationship proposed by Schmertmann (1978). For dissipation times (t_{50}) less than 0.5 min, the penetration process appears to be partially drained, and no correlation or data exists. Some of the reasons for the observed large scatter are probably due to variations in soil compressibility as well as the uncertainty in the reference values.

Conclusions

Data have been reviewed from sites in Europe and North and South America, as well as published data from South Africa. The review has been limited to data from cone penetration tests with pore-pressure measurements (CPTU). The main objective of the review has been to apply existing published interpretation techniques to interpret CPTU dissipation test data and compare predicted consolidation and permeability parameters to available reference values. The results of this review can be summarized as follows:

(1) The theoretical solutions proposed by Teh and Houlsby (1991) and Torstensson (1977) provide a reasonable

TABLE 6. Summary of estimated c_h ratios from laboratory and back-analysed field values

Site (location)	Pore-pressure location	t_{50} avg., min	c_h, cm^2/min Lab	Field	c_h(field)/ c_h(lab)
Guanabara Bay (Rio de Janeiro, Brazil)	u_1	5.6			
	u_2	11.3	0.036	0.122	3.4
	u_3	35.5			
Colebrook Overpass, B.C. (Canada)	u_2	61	0.23	0.162	0.7
Laing Bridge South (Canada)	u_2	4.4			
			0.19	0.24	1.3
	u_3	12.5			
Saint-Alban (Canada)	u_2	11.3	0.0045	0.048	10.7
	u_3	17.1			
Fucino (Italy)	u_1	8.5	0.016	0.09	5.6
Porto Tolle (Italy)	u_1	5.2	0.24	0.9	3.8
Trieste (Italy)	u_2	32	0.012	0.016	1.3
Saugus (U.S.A.)	u_1	1.1	0.36	2.1	5.8

TABLE 7. Summary of CPTU dissipation results and laboratory permeability results

Site (location)	Pore-pressure location	t_{50} avg., min	k_h range, cm/s	k_h avg., cm/s
Colebrook Overpass, B.C. (Canada)	u_1	2.9		
	u_2	61	2×10^{-8}	2×10^{-8}
McDonald Farm (Canada)	u_1	4.4		
	u_2	5.0	4×10^{-7}	4×10^{-7}
	u_3	12.5		
Saint-Alban (Canada)	u_1	5.0		
	u_2	11.3	2×10^{-7} to 4×10^{-7}	3×10^{-7}
	u_3	17.1		
Langley, Lower 232 Street (Canada)	u_2	42	8×10^{-8}	8×10^{-8}
Porto Tolle (Italy)	u_1	5.2	—	6.9×10^{-8}
Trieste (Italy)	u_2	32	—	8×10^{-9}
Brage (Norway)	u_1	1.05	3.1×10^{-8} to 1.24×10^{-6}	4.4×10^{-7}
Drammen (Norway)	u_1	3.5	1×10^{-8}	1×10^{-8}
	u_2	21		
Haga (Norway)	u_1	14.6	2×10^{-8}	7×10^{-8}
	u_2	25.4	-2.1×10^{-7}	
Onsoy (Norway)	u_1	5.9	3×10^{-8}	1.33×10^{-7}
	u_2	21	5×10^{-7}	
Snorre (Norway)	u_1	4.3	4.12×10^{-9}	1.18×10^{-8}
	u_2	33	-2.8×10^{-8}	
Stjordal-Halsen (Norway)	u_1	0.27	1.2×10^{-6}	2.37×10^{-6}
	u_2	2.85	-3.8×10^{-6}	
Troll (Norway)	u_1	8.3	4.7×10^{-9}	4.04×10^{-8}
	u_2	137	-2.2×10^{-7}	
Amherst (U.S.A.)	u_1	0.48	3×10^{-7} -1×10^{-6}	6×10^{-7}
Attakapa Landing (U.S.A.)	u_1	24	2.2×10^{-8}	2.56×10^{-8}
	u_3	103	-3×10^{-8}	
Saugus (U.S.A.)	u_1	6.7	4.5×10^{-8}	8.5×10^{-8}
	u_2	13.4	3×10^{-7}	
Bothkennar (U.K.)	u_2	39	4.76×10^{-8} to 3.33×10^{-7}	1.78×10^{-7}
Madingley (U.K.)	u_1	4.5	7.61×10^{-10} to 1.74×10^{-9}	1.36×10^{-9}
Cowden (U.K.)	u_1	33	1.93×10^{-8} to	2.85×10^{-8}
	u_2	44	4.44×10^{-8}	

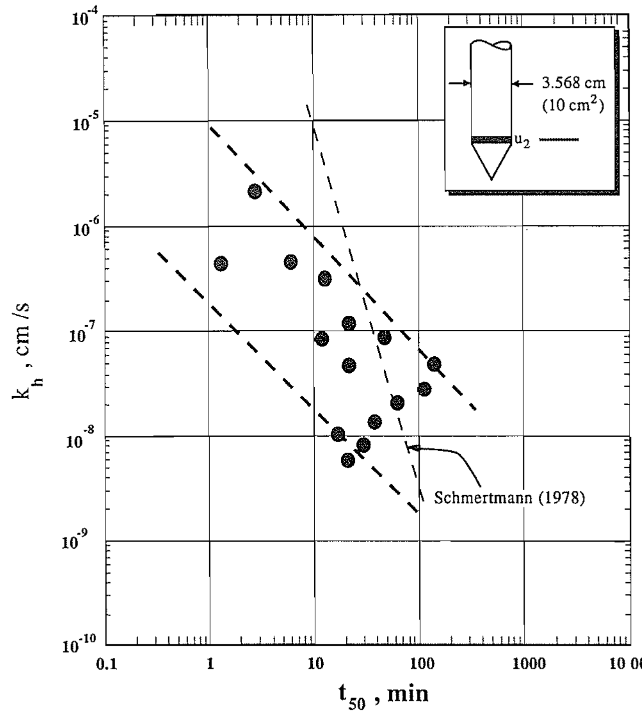

FIG. 10. Average values for laboratory-derived horizontal coefficient of permeability (k_h) and CPTU t_{50} for u_2 pore-pressure location.

estimate of the *in situ* horizontal coefficient of consolidation (c_h).

(2) The most consistent results showing the least scatter were obtained with the pore-pressure element location immediately above the cone tip (u_2).

(3) The expected reliability for estimating c_h for the u_2 position is about plus or minus one-half an order of magnitude, whereas for the u_1 position this increases to a full one order of magnitude. These magnitudes of expected reliability are consistent with previous published experience for the determination of c_h or c_v.

A tentative correlation between the rate of dissipation (t_{50}) and horizontal coefficient of permeability (k_h) has also been proposed, although considerable scatter exists in the data.

The application of these correlations requires that the CPTU must be performed with fully saturated pore-pressure elements. Although data from many sites have been compiled and reviewed, there is a need for continued research. This review has illustrated the difficulty in determining the coefficient of consolidation of soils from traditional site investigation and laboratory results. The interpretation techniques available for the CPTU appear to produce estimates of the coefficient of consolidation with similar accuracy as existing traditional sampling and laboratory testing. The CPTU values of c_h are slightly higher than the laboratory values but agree better with c_h values back calculated from full-scale performance. However, more data should be collected to improve the correlations suggested in this paper. There is a need for more data from sites where field performance has been monitored so that the reference values of the coefficient of consolidation can be better determined. More data are needed to clarify the influence of OCR and soil structure on the proposed correlations.

Acknowledgements

This work was performed under a contract (23420-9-M644-01-OSC) for Energy, Mines and Resources Canada. The authors would like to acknowledge the assistance of Prof. R.G. Campanella, C.B. Crawford, Dr. J-M. Konrad, Dr. A. Whittle, Prof. M. Tumay, Prof. A. Lutenegger, and Prof. M. Jamiolkowski for providing valuable data to assist with this study.

Associazione Geotechnica Italiana. 1979. Experiences on the time-settlement behaviour of some Italian soft clays. Proceedings, 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton, vol. 1, pp. 1–11.

Baligh, M.M., and Levadoux, J.N. 1980. Pore pressure dissipation after cone penetration. Massachusetts Institute of Technology Department of Civil Engineering, Research Report R80–11.

Baligh, M.M., and Levadoux, J.N. 1986. Consolidation after undrained piezocone penetration. II: interpretation. ASCE Journal of Geotechnical Engineering, **112**: 727–745.

Battaglio, M., Jamiolkowski, M., Lancellotta, R., and Maniscalco, R. 1981. Piezometer probe test in cohesive deposits. Proceedings, ASCE Conference on Cone Penetration Testing and Experience, St. Louis, pp. 264–300.

Campanella, R.G., and Robertson, P.K. 1988. Current status of the piezocone test. Proceedings, 1st International Symposium on Penetration Testing, vol. 1, Orlando, Fla., pp. 93–116.

Campanella, R.G., Sully, J.P., and Robertson, P.K. 1988. Interpretation of piezocone soundings in clay — a case history. Proceedings, Penetration Testing in the UK, Birmingham, England, pp. 203–208.

Crawford, C.B., and Campanella, R.G. 1991. Comparison of field consolidation with laboratory and *in situ* tests. Canadian Geotechnical Journal, **28**: 103–112.

Gillespie, D.G. 1981. The piezometer cone penetration test. M.A.Sc. thesis, Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Gillespie, D.G. 1990. Evaluating velocity and pore pressure data from the cone penetration test. Ph.D. thesis, Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Gillespie, D., and Campanella, R.G. 1991. Consolidation characteristics from pore pressure dissipation after piezometer cone penetration. Soil Mechanics Series 47, Department of Civil Engineering, The University of British Columbia. Vancouver, B.C.

Gupta, R.C., and Davidson, J.L. 1986. Piezoprobe determined coefficient of consolidation. Soils and Foundations, **26**(3): 12–22.

Hers, I. 1989. The analysis and interpretation of the cone pressuremeter in cohesive soils. M.A.Sc. thesis, Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Jamiolkowski, M., Lancellotta, R., Marchetti, S., *et al.* 1979. Design parameters for soft clays. Proceedings, 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton, pp. 27–57.

Jamiolkowski, M., Lancellotta, R., and Wolski, W. 1983. Precompression and speeding up consolidation. Proceedings, 8th European Conference on Soil Mechanics and Foundation Engineering, Helsinki, vol. 3, pp. 1201–1226.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T., and Lancellotta, R. 1985. New developments in field and laboratory testing of soils. Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, pp. 57–153.

Jones, G.A., and Van Zyl, D.J.A. 1981. The piezometer probe—A useful tool. Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, vol. 2, pp. 489–496.

Kabir, M.G., and Lutenegger, A.J. 1990. In situ estimation of the coefficient of consolidation of clays. Canadian Geotechnical Journal, **27**: 58–67.

LeClair, D.G. 1988. Prediction of embankment performance using in-situ tests. M.A.Sc. thesis, Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Lunne, T., Powell, J.J.M., Quarterman, R.S.T., and Eidsmoen, T.E. 1986. Piezocone testing in overconsolidated clays. Proceedings, 39th Canadian Geotechnical Conference, Ottawa, pp. 209–218.

Lutenegger, A.J., Kabir, M.G., and Saye, S.R. 1988. Use of penetration tests to predict wick drain performance in a soft clay. Proceedings, 1st International Symposium on Penetration Testing, Orlando, Fla., vol. 2, pp. 843–848.

Marchetti, S., and Totani, G. 1989. C_h evaluations from DMTA dissipation curves. Proceedings, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, vol. 1, pp. 281–286.

Mokkelbost, K.H. 1988. Application of dilatometer for pile design. Dilatometer tests at four Norwegian sites. Joint Research Project NGI/BRE, Norwegian Geotechnical Institute, Oslo. Internal Report 521610-1.

Norwegian Geotechnical Institute. 1984. Oppdragsrapport til Norsk Hydro A/S., Troll Ost 1984, grunnundersokelser, laboratorierapport. Bind II — geologi. Report 84215-2,

Norwegian Geotechnical Institute. 1988*a*. Contract report to Norsk Hydro A/S. Brage Soil Survey 1987, Contract No. KO 1636/TU-5171, Geotechnical Data, vols. 1 and 2. Report 87261-03.

Norwegian Geotechnical Institute. 1988*b*. Contract report to Saga Petroleum A/S. Snorre soil investigation 1987 Block 34/7. Soil parameter report for design. Report 87260-3.

Parez, L., and Bachelier, M. 1981. C_{vr}, K_h, δ_{au} déterminés par pénétration statique. Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, vol. 2, pp. 553–556.

Powell, J.J.M., and Quarterman, R.S.T. 1988. The interpretation of cone penetration tests in clays, with particular reference to rate effects. Proceedings, 1st International Symposium on Penetration Testing, Orlando, Fla., pp. 903–910.

Powell, J.J.M., Quarterman, R.S.T., and Lunne, T. 1988. Interpretation and use of the piezocone test in UK clays. Proceedings, Penetration Testing in the UK, Birmingham, England, pp. 151–156.

Robertson, P.K., Sully, J.P., Woeller, D.J., *et al.* 1990. Guidelines for interpretation of CPTU test data for determination of consolidation and permeability parameters of soils. Report prepared by ConeTec Investigations Ltd. for Energy, Mines and Resources Canada, Contract No. 23420-9-M644/01-OSC.

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.

Sandbaekken, B., Berre, T., and Lacasse, S. 1985. Oedometer testing at the Norwegian Geotechnical Institute. Proceedings, Symposium on Consolidation behaviour for Soils, Ft. Lauderdale, Fla. American Society for Testing and Materials, Special Technical Publication 892, pp. 329–353.

Schmertmann, J.H. 1978. Guidelines for cone penetration tests performance and design. Federal Highways Administration, Washington, D.C., Report FHWA-TS-78-209.

Senneset, K., Janbu, N., and Svano, G. 1982. Strength and deformation parameters from cone penetration tests. Proceedings, 2nd European Symposium on Penetration Testing, Amsterdam, pp. 863–870.

Senneset, K., Sandven, R., and Janbu, N. 1989. Evaluation of soil parameters from piezocone tests. *In* In situ testing of soil properties for transportation. Transportation Research Board, National Research Council, Washington, D.C.,Transportation Research Record 1235, pp. 24–37.

Sills, G.C., Almeida, M.S.S., and Danziger, F.A.B. 1988. Coefficient of consolidation from piezocone dissipation tests in a very soft clay. Proceedings, 1st International Symposium on Penetration Testing, Orlando, Fla., vol. 2, pp. 967–974.

Soares, M.M., Lunne, T., and Lacasse, S. 1987. In-situ site investigation techniques and interpretation of offshore practise. Interpretation of dissipation tests in Onsoy clay. Norwegian Geotechnical Institute, Oslo, Norway, Internal Report 40019-5.

Sully, J.P., Campanella, R.G., and Robertson, P.K. 1988. Overconsolidation ratio of clays from penetration pore pressures, ASCE Journal of Geotechnical Engineering, **114**(2): 209–216.

Tavenas, F.A., Chapeau, C., La Rochelle, P., and Roy, M. 1974. Immediate settlements of three test embankments on Champlain Clay. Canadian Geotechnical Journal, **11**: 109–141.

Tavenas, F., Leblond, P., Jean, P., and Leroueil, S. 1983. The permeability of natural soft clays, part I: methods of laboratory measurement. Canadian Geotechnical Journal, **20**: 629–644.

Tavenas, F., Tremblay, M., Larouche, G., and Leroueil, S. 1986. In-situ measurement of permeability in soft clays. Proceedings, ASCE Conference on Use of In-situ Tests in Geotechnical Engineering, Blacksburg, Va. ASCE, New York, pp. 1034–1048.

Teh, C.I., and Houlsby, G.T. 1991. An analytical study of the cone penetration test in clay. Géotechnique, **41**: 17–34.

Torstensson, B.A. 1975. Pore pressure sounding instrument. Proceedings, ASCE Specialty Conference on In Situ Measurement of Soil Properties, Raleigh, N.C., vol. 2, pp. 48–54.

Torstensson, B.A. 1977. The pore pressure probe. Nordiske Mote, bergemekanikk, Oslo, Norway, Paper 34, pp. 34.1–34.15.

Tumay, M.T., and Acar, Y.B. 1984. Piezocone penetration testing in soft cohesive soils. *In* Strength testing of marine sediments: laboratory and in situ measurements. American Society for Testing and Materials, Special Technical Publication 883, pp. 72–82.

Zavoral, D.Z. 1988. Laboratory testing evaluation of cone penetration test co-relations for geotechnical soil parameters. B.A. Sc. thesis, Department of Geological Engineering, University of British Columbia, Vancouver, B.C.