

## Estimating coefficient of consolidation from piezocone tests

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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.

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### 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

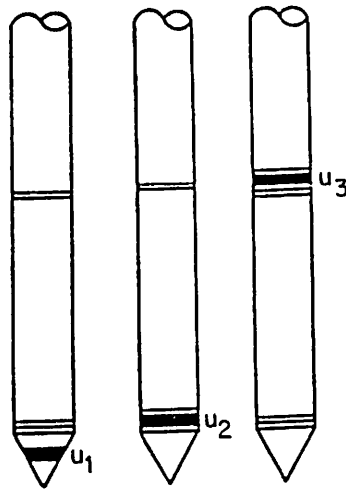


FIG. 1. Pore-pressure element locations ( $u_1$ - $u_3$ ) selected for this study.

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,  $\Delta u$ , 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	$1.2 \pm 0.2$
Slight layering, e.g., sedimentary clays with occasional silt dustings to random lenses	2-5
Varved clays in northeastern United States	$10 \pm 5$

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 evaluate 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 <sup>a</sup>	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 (1988a)
Saint-Alban (Canada)	Champlain Sea Clay; very sensitive, lightly overconsolidated (aged)	Roy <i>et al.</i> (1982); Tavenas <i>et al.</i> (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 (1988b)
Stjordal-Halsen (Norway)	Marine clay sandy silt; normally consolidated	Senneset <i>et al.</i> (1982)
Drammen (Norway)	Normally consolidated clay silt; sensitive	Mokkelbost (1988)
Strong Pit, B.C. (Canada)	Glaciomarine clayey silt; lightly to moderately overconsolidated	Campanella <i>et al.</i> (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 <i>et al.</i> (1981); Jamiolkowski <i>et al.</i> (1979)
Guanabara Bay (Rio de Janeiro, Brazil)	Soft silty clay; normally consolidated	Sills <i>et al.</i> (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 <i>et al.</i> (1981); Jamiolkowski <i>et al.</i> (1983)
Site I/II (France)	Clayey silt	Parez and Bachelier (1981)
Glava (Norway)	Medium stiff marine clay; overconsolidated	Senneset <i>et al.</i> (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 <i>et al.</i> (1988)

TABLE 2 (concluded)

Site (location)	Soil type <sup>a</sup>	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 <i>et al.</i> (1988)
Bothkennar (U.K.) (Grangemouth)	Soft, slightly fissured organic silty clay	Powell <i>et al.</i> (1988)
Cowden (U.K.)	Stiff, glacial till	Powell and Quarterman (1988)
Madingley, (U.K.)	Gault Clay; stiff, fissured silty clay	Lunne <i>et al.</i> (1986)

<sup>a</sup>Lightly overconsolidated,  $1 < OCR < 4$ ; moderately overconsolidated,  $4 < OCR < 10$ ; heavily overconsolidated,  $10 < OCR$ .

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. Notwithstanding 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 ( $\Delta u$ ) 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

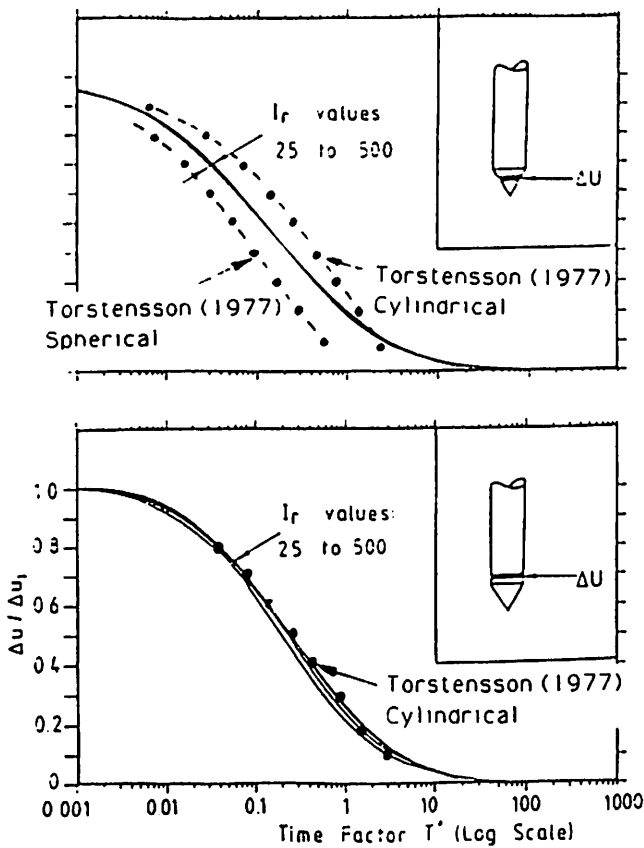


FIG. 2. Normalized dissipation curves in log time based on theoretical solution by Teh and Houslby (1991).

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 Houslby (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  $60^\circ$  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 distribu-

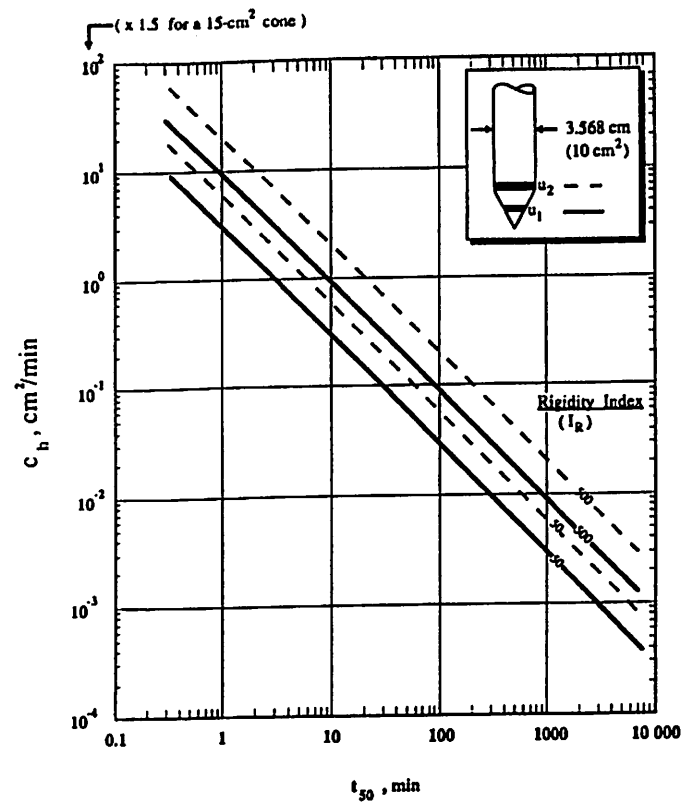


FIG. 3. Chart for evaluation of  $c_h$  from  $t_{50}$  for 10 and 15  $\text{cm}^2$  piezocone (based on theoretical solution by Teh and Houslby 1991).

tion 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 Houslby (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 Houslby (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 Houslby (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 Houslby (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 <sup>2</sup> /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<sup>2</sup> 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 <sup>2</sup> /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	1
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 <sup>2</sup> /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.

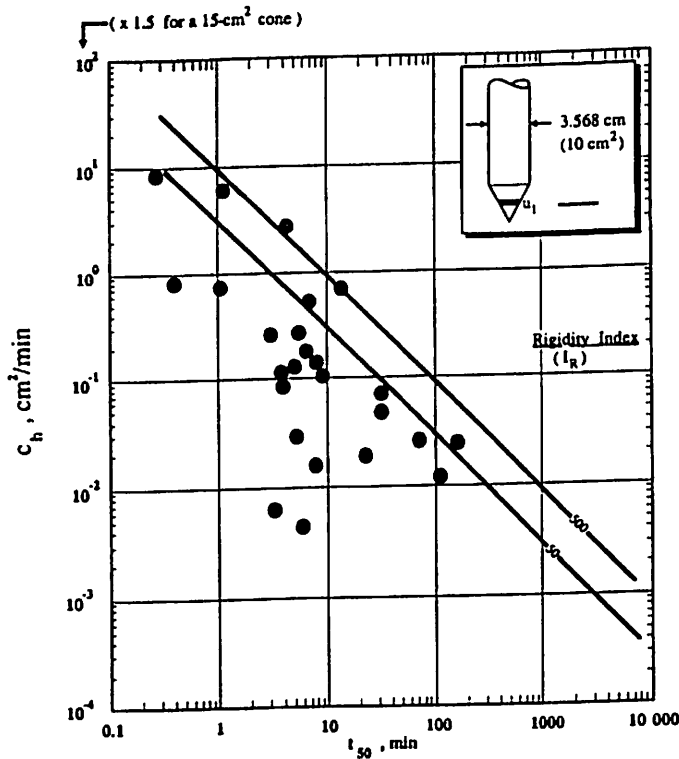


FIG. 4. Average laboratory oedometer  $c_h$  results and CPTU results in terms of  $t_{50}$  for  $u_1$  pore-pressure location.

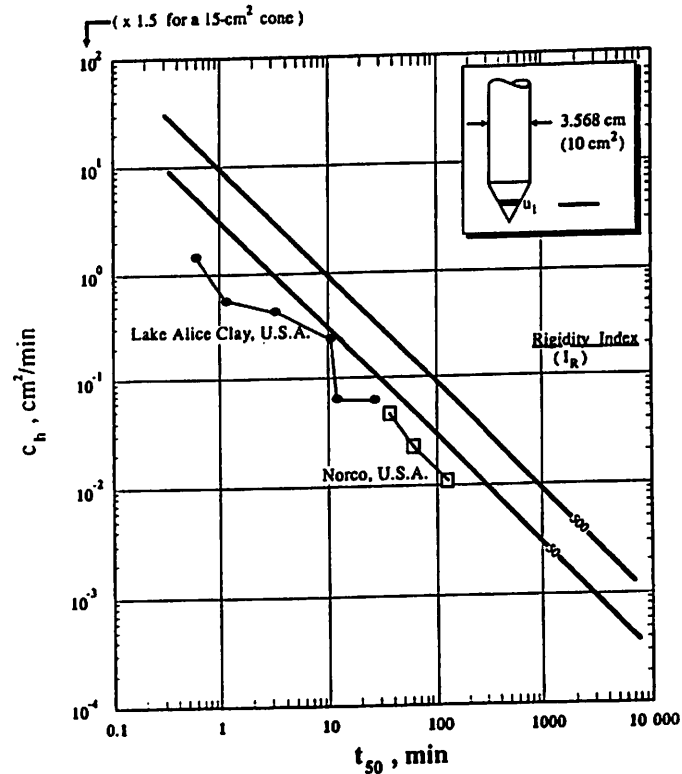


FIG. 6. Variation of  $c_h$  and  $t_{50}$  with depth for two sites.

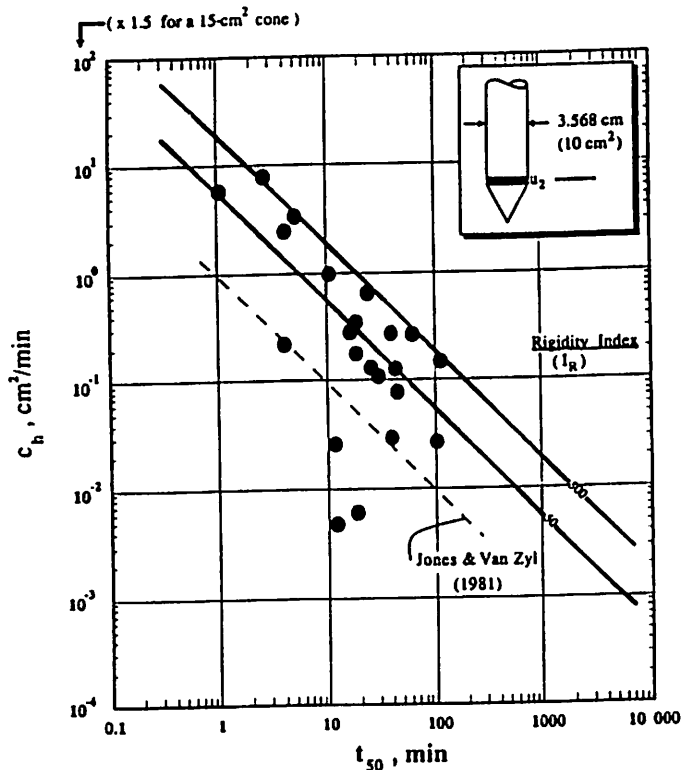


FIG. 5. Average laboratory oedometer  $c_h$  results and CPTU results in terms of  $t_{50}$  for  $u_2$  pore-pressure location.

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 inter-

pretation 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



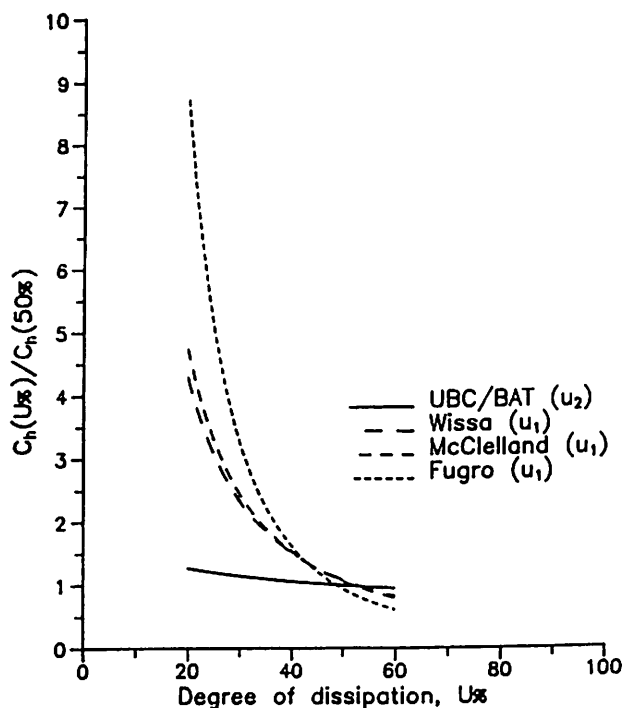


FIG. 7. Variation in  $c_h$  determined at different degrees of dissipation compared with  $c_h$  determined at 50% dissipation. Onsoy clay dissipation data from Soares *et al.* (1987), and  $c_h$  values from Torstensson cavity expansion theory. Curves are best fit lines to individual data sets for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

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 difference between  $c_h(\text{lab})$  and  $c_h(\text{field})$  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  $c_h(\text{field})$  based on performance and  $c_h(\text{lab})$  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

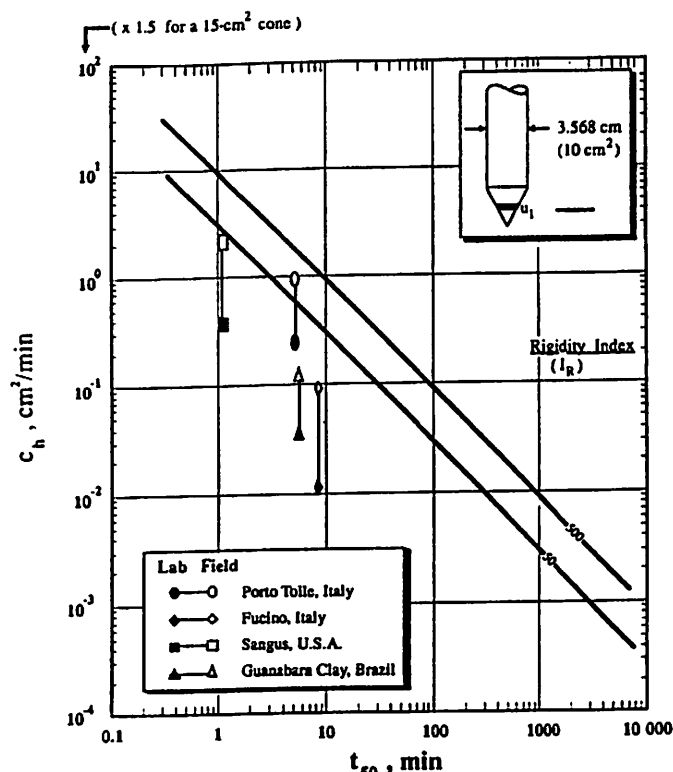


FIG. 8. Comparison between field back-analyzed and laboratory  $c_h$  values for CPTU  $t_{50}$  values for  $u_1$  pore-pressure location.

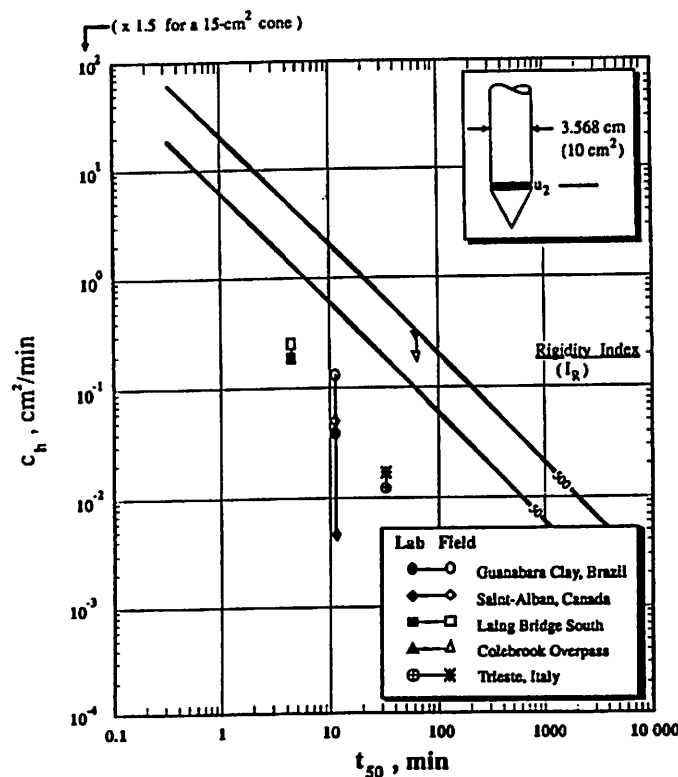


FIG. 9. Comparison between field back-analyzed and laboratory  $c_h$  values for CPTU  $t_{50}$  values for  $u_2$  pore-pressure location.

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 <sup>2</sup> /min		$c_h(\text{field})/$ $c_h(\text{lab})$
			Lab	Field	
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
Saint-Alban (Canada)		$u_3$	12.5		
	$u_2$	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	$2 \times 10^{-8}$	$2 \times 10^{-8}$
McDonald Farm (Canada)	$u_1$	4.4		
	$u_2$	5.0	$4 \times 10^{-7}$	$4 \times 10^{-7}$
	$u_3$	12.5		
Saint-Alban (Canada)	$u_1$	5.0		
	$u_2$	11.3	$2 \times 10^{-7}$ to $4 \times 10^{-7}$	$3 \times 10^{-7}$
Langley, Lower 232 Street (Canada)	$u_3$	17.1		
	$u_2$	42	$8 \times 10^{-8}$	$8 \times 10^{-8}$
Porto Tolle (Italy)	$u_1$	5.2	—	$6.9 \times 10^{-8}$
Trieste (Italy)	$u_2$	32	—	$8 \times 10^{-9}$
Brage (Norway)	$u_1$	1.05	$3.1 \times 10^{-8}$ to $1.24 \times 10^{-6}$	$4.4 \times 10^{-7}$
			$1 \times 10^{-8}$	$1 \times 10^{-8}$
Drammen (Norway)	$u_1$	3.5		
	$u_2$	21		
Haga (Norway)	$u_1$	14.6	$2 \times 10^{-8}$	$7 \times 10^{-8}$
	$u_2$	25.4	$-2.1 \times 10^{-7}$	
Onsoy (Norway)	$u_1$	5.9	$3 \times 10^{-8}$	$1.33 \times 10^{-7}$
	$u_2$	21	$5 \times 10^{-7}$	
Snorre (Norway)	$u_1$	4.3	$4.12 \times 10^{-9}$	$1.18 \times 10^{-8}$
	$u_2$	33	$-2.8 \times 10^{-8}$	
Stjordal-Halsen (Norway)	$u_1$	0.27	$1.2 \times 10^{-6}$	$2.37 \times 10^{-6}$
	$u_2$	2.85	$-3.8 \times 10^{-6}$	
Troll (Norway)	$u_1$	8.3	$4.7 \times 10^{-9}$	$4.04 \times 10^{-8}$
	$u_2$	137	$-2.2 \times 10^{-7}$	
Amherst (U.S.A.)	$u_1$	0.48	$3 \times 10^{-7}$	$6 \times 10^{-7}$
			$-1 \times 10^{-6}$	
Attakapa Landing (U.S.A.)	$u_1$	24	$2.2 \times 10^{-8}$	$2.56 \times 10^{-8}$
	$u_3$	103	$-3 \times 10^{-8}$	
Saugus (U.S.A.)	$u_1$	6.7	$4.5 \times 10^{-8}$	$8.5 \times 10^{-8}$
	$u_2$	13.4	$3 \times 10^{-7}$	
Bothkennar (U.K.)	$u_2$	39	$4.76 \times 10^{-8}$ to $3.33 \times 10^{-7}$	$1.78 \times 10^{-7}$
			$7.61 \times 10^{-10}$ to $1.74 \times 10^{-9}$	$1.36 \times 10^{-9}$
Madingley (U.K.)	$u_1$	4.5		
			$1.93 \times 10^{-8}$ to $4.44 \times 10^{-8}$	$2.85 \times 10^{-8}$
Cowden (U.K.)	$u_1$	33		
	$u_2$	44		

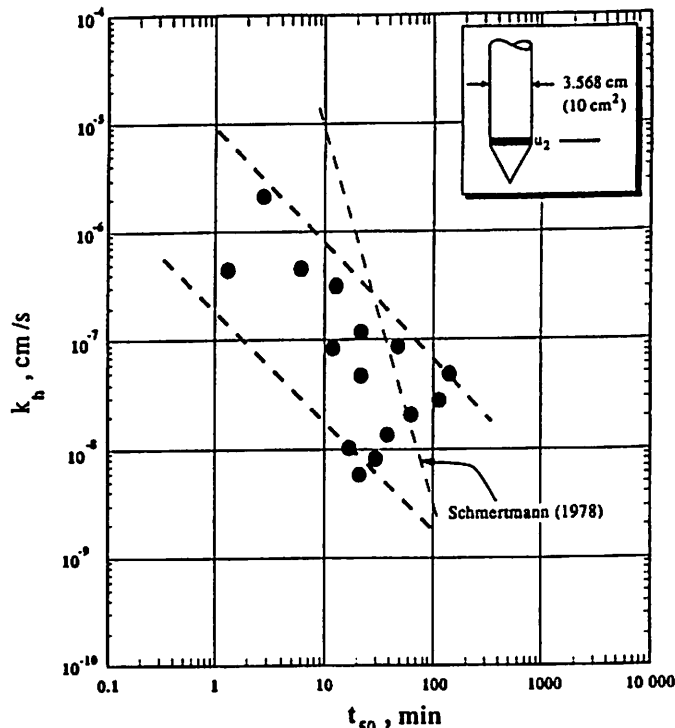


FIG. 10. Average values for laboratory-derived horizontal coefficient of permeability ( $k_h$ ) and CPTU  $t_{50}$  for  $u_2$  pore-pressure location.

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.

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