GUIDELINES FOR INTERPRETATION OF CPTU TEST DATA FOR DETERMINATION OF CONSOLIDATION AND PERMEABILITY PARAMETERS OF SOILS

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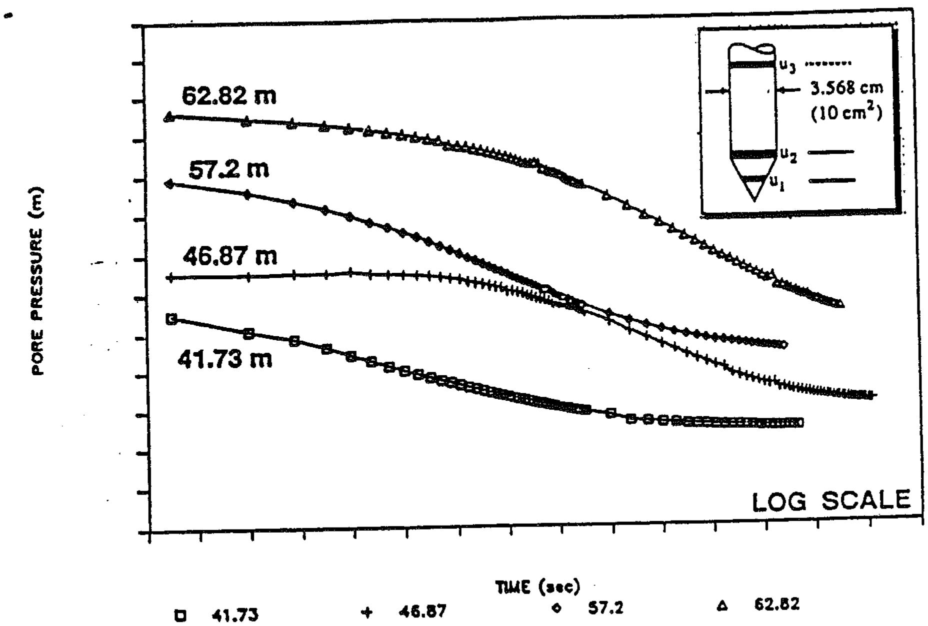

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PREPARED BY:

CONETEC INVESTIGATIONS LTD.
2443 BETA AVENUE
BURNABY, B.C.
V5C 5NI

PREPARED FOR:

ENERGY MINES & RESOURCES
ATLANTIC GEOSCIENCE CENTRE
P.O. BOX 1006
DARTMOUTH, NOVA SCOTIA
B2Y 4A2

Guidelines for Interpretation of *CPTU* data for the Determination of Consolidation and Permeability Parameters of Soils

The Department of Supply and Services acting on behalf of Energy, Mines and Resources

Contract number 23420-9-M644-01-0SC

Prepared by

ConeTec Investigation Ltd., British Columbia, Canada; Norwegian Geotechnical Institute (NGI), Norway; and, Building Research Establishment (BRE), United Kingdom

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Preface

This report has been prepared by ConeTec Investigations Ltd., under the direction of Dr. P. K. Robertson with the assistance of J. Sully, D. J. Woeller, T. Lunne and J. J. M. Powell.

Executive Summary

Data has been reviewed from over 30 sites in Europe and North and South America as well as published data from South Africa. The review has concentrated on 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:

- The theoretical solutions proposed by Teh (1988) and Torstensson (1977) provide a reasonable estimate of the in-situ horizontal coefficient of consolidation (c_h).

- The least scatter in results was obtained with the pore pressure element location immediately behind the cone tip (u_2).

- The recommended procedure to estimate the in-situ field value of c_h is as follows:

  c_h (field) = A/t_{50}

  where: c_h is in cm^2/min
  t_{50} is in minutes, and
  $A = 6$ for u_1 (face location)
  $A = 10$ for u_2

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

- Based on the available data it would appear that the recommended values do not apply for either highly sensitive or structured clays.

- For short dissipation times the CPTU can be interpreted using the slope of the square-root time plot using the following:

  c_h (field) = m^2/B

  where: c_h is in cm^2/min
  m^2 is in min^{-1}
  $B = 5.56 \times 10^{-2}$ for u_1
  $B = 3.34 \times 10^{-2}$ for u_2

- The m^2 method appears to be slightly better for estimating c_h from short dissipations in soft low OCR soils.

- Penetration pore pressures and dissipation times (t_{50}) can be useful additional data to enhance the interpretation of the CPTU for soil classification.

- The in-situ horizontal permeability (k_h) can be estimated from CPTU dissipation data using:

$$k_h = \frac{C}{t_{50}}$$

where: k_h is in cm/sec
t_{50} is in minutes, and
C varies from 3×10^{-7} to 1×10^{-5}

- The application of these correlations requires that the CPTU must be performed with fully saturated pore pressure elements.

Although data from over 30 sites have been compiled and reviewed further research is still required to clarify the following points:

- Influence of soil structure, sensitivity and fabric.
- Interpretation procedures for heavily overconsolidated soils.

Reliable data should continue to be collected and reviewed to improve the correlations and to increase data related to

- short dissipation times
- soil type interpreation, especially fines content for low plastic soils, and
- field performance.

1. Introduction

The Department of Supply and Services (DSS) acting on behalf of Energy, Mines and Resources (EMR) have requested the development of guidelines for interpretation of *in-situ* test data for the determination of consolidation and permeability parameters of soils. As a logical approach to developing these guidelines, the DSS has suggested a review of existing *in-situ* and associated laboratory test data from well documented geotechnical research sites in North America and Europe.

Based on the above requirements, the following aims and objectives were defined by the project participants:

a) Compile available *in-situ* test data and associated reference field and laboratory data from sites in North America and Europe.

b) Apply existing published interpretation techniques to interpret the *in-situ* test data and compare predicted consolidation and permeability parameters to the available reference values.

c) Based on (b) determine the optimum interpretation technique for *in-situ* test method and comment on the expected reliability, also document the conditions required for the reliability indicated.

d) Determine the feasibility of using penetration pore pressures and/or dissipation data to enhance the interpretation of the cone penetration test (CPTU), with special emphasis on grain size distribution information.

e) Document areas where available predictive techniques are least reliable and where further research is required.

The main participants in this project are:

ConeTec Investigations Ltd., British Columbia, Canada;
Norwegian Geotechnical Institute (NGI), Norway; and,
Building Research Establishment (BRE), United Kingdom.

Authorization to proceed with this project was given in accordance with Contract number 23420-9-M644-01-0SC.

2. Data Review

In-situ and laboratory data related to this project were obtained from many sources. The primary sources were as follows:

- ConeTec Investigations Ltd. (ConeTec)
- Norwegian Geotechnical Institute (NGI)
- Building Research Establishment (BRE)

Additional data was kindly provided by the following:

- University of British Columbia (UBC)
  (Prof. R.G. Campanella and C.B. Crawford)
- Bedford Inst. of Oceanography
  (Dr. D.G. Gillespie)
- University of Waterloo
  (Dr. J.-M. Konrad)
- M. I. T. , USA, (Dr. A. Whittle, and Prof. M. Baligh)
- Louisiana State University, USA, (Prof. M. Tumay)
- Torino University, Italy, (Prof. M. Jamilokowski)
- University of Massachusetts, USA, (Prof. A. Lutenegger)

Data was also taken from the geotechnical literature.

Several *in-situ* test techniques have been suggested to estimate the coefficient of consolidation, some of which are listed in Table 1.

TABLE 1. Reported use of in-situ tests to estimate coefficient of consolidation (Adapted from Kabir and Lutenegger,1990)

In-situ Test	*Reference*
Dilatometer (DMT)	Marchetti *et al.*. (1986) Robertson *et al.* (1988b) Marchetti and Totani (1989)
Piezocone (CPTU)	Hansbo *et al.* (1981) Battaglio *et al.* (1981) Robertson *et al.* (1988a) Lutenegger *et al.* (1988)
Piezoblade	Lutenegger *et al.* (1985) Kabir and Lutenegger (1990)
Pressuremeter holding tests	Clarke *et al.* (1979) Fahey and Carter (1986) Benoit and Clough (1986) Fahey and Foley (1987
Push-in piezometer	Bennett *et al.* (1985) Foott *et al.* (1987)
Self-Boring Permeameter	Baguelin *et al.* (1974) Tavenas *et al.* (1983)
BAT tests	Torstensson (1984) Rad et al. (1988) Lunne et al. (1989)

For the sites evaluated in this study, the largest data base of *in-situ* dissipation data available was from the CPTU. Hence, to perform a statistically valid evaluation of predictive techniques only CPTU data have been used for this study. However, reference and comments are made concerning other *insitu* test data, where appropriate. 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, repeatibility and speed. The cone penetration test with pore pressure measurements (CPTU) provides added advantages of:

- ability to distinguish drainage conditions during cone penetration;
- ability to correct measured cone penetration resistance (q_c), and to some extent f_s, to account for unbalanced water forces due to unequal end area in cone designs;
- ability to assess equilibrium groundwater conditions;
- improved soil profiling and identification;
- improved evaluation of geotechnical parameters; and,
- ability to evaluate flow and consolidation characteristics.

These advantages are the primary reasons that the CPTU is one of the most commonly used *in-situ* tests for evaluation of consolidation characteristics.

It is possible to measure pore pressures at various locations on a cone penetrometer and Figure 1 illustrates the major locations currently used. To assist in the definition of the pore pressure data recorded at different locations, the following system shall be used:

u_1 = pore pressure measured on the face of the cone tip.

u_2 = pore pressure measured just behind the cone tip.

u_3 = pore pressure measured behind the friction sleeve on the shaft of the cone.

No distinction is made for the various locations possible on the cone tip itself (u_1, i.e. midheight tip, etc.) since published data suggests that the measured pressures at all locations on the face are very similar (Figure 2).

The penetration pore pressure measured at any location on a cone can be divided into two components:

- *in-situ* equilibrium value, u_o, which is controlled by local groundwater regime,

- exess pore pressure generated by the penetration process of the cone, Δu, which is a function of both the soil and cone geometry.

Thus;

$$u_{1,2,3,} = u_o + \Delta u_{1,2,3} \qquad (1)$$

Where the subscripts refer to the piezo-element location defined previously.

It is the decay with time of the excess pore water pressure that provides information concerning the flow characteristics of the soil. In order to evaluate the dissipation of the generated excess pore pressure, cone penetration is stopped and the decay of pore pressure with time is recorded. A typical dissipation curve in a normally consolidated fine grained soil is shown in Figure 3. The change in pore pressure can be plotted against log time or square-root time depending on the type of analyses to be performed (see Section 3). Either the excess pore pressure or the normalized excess pore pressure decay can be plotted:

$$\Delta u = u - u_o \qquad (2)$$

or

$$U(t) = \frac{u_t - u_o}{u_i - u_o} = \frac{\Delta u(t)}{\Delta u_i} \qquad (3)$$

where: $U(t)$ = the normalized excess pore pressure at time t
u_t = the excess pore pressure at time t
u_i = the initial excess pore pressure when penetration is stopped (t = 0)

$U_{(t)}$, therefore varies between 1 (t = 0) and 0 when the excess pore pressure has completely dissipated and $u(t) = u_o$.

As mentioned earlier, during cone penetration excess pore pressures are generated. These excess pore pressures are generally referred to as penetration or dynamic 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 require 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 is to compare theoretically derived values of consolidation parameters from CPTU dissipation tests with measured reference values for different soil deposits. One of the major complications in this task is 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:

1. Back analyses of field performance, and,

2. Interpretation of laboratory tests on small samples.

In-situ permeability tests using the self-boring permeameter have been sugested by Tavenas et al. (1983) as the most promising method for evaluating the "real" consolidation parameters, providing smear on the porous elements can be avoided. However, the independent determination of soil compressibility (m_v) from laboratory testing is required which greatly restricts this approach, also the self-boring permeameter is a slow and expensive *in-situ* test.

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* dissipation 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 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 analyses of field performance generally provides an estimate of the vertical consolidation parameter (c_v), whereas the CPTU interpretation primarily provides an estimate of c_h. In the case of soils where 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 predominace of horizontal drainage and a clearer definition of drainage path.

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

$$c_h = c_v \,.\, k_h / k_v \tag{4}$$

This is a very approximate correction since it assumes that the soil compressibility is isotropic (i.e., $m_v = m_h$). However, considering all other uncertainties in the evaluation of c_h this appears to be a reasonable assumption. Based on North American experience Ladd (1976) suggested values of k_h/k_v for various soil types (Table 2). A similar classification for k_h/k_v has been sugested by Jamiolkowski et al. (1985).

TABLE 2: Suggested anisotopic permeability of clays
(After Ladd, 1976)

	Nature of clay	k_h/k_v
1.	No evidence of layering	1.2 ± 0.2
2.	Slight layering, e.g. sedimentary clays with occasional silt dustings to random lenses	2 to 5
3.	Varved clays in North - Eastern U.S.	10 ± 5

Table 2 has been used to evaluate c_h from c_v unless data are available to avoid this assumption.

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, both incremental load and constant rate of strain. The major problems 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 samples have some level of disturbance, hence laboratory derived parameters can be influenced by sample disturbance. 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; samples can be tested in both the vertical and horizontal direction to derive both c_h and c_v and parameters can be determined at different stress levels.

As outlined earlier, detailed *in-situ* and laboratory data were obtained from three main sources. This information is summarized in a tabulated form in Appendix A. The complete data in the form of dissipation curves, oedometer plots and additional site/soil type information are contained in the subsequent appendices in Volume 2. Supplementary data provided by other research centres are also packaged as the primary source information. However, in almost all cases only interpreted laboratory and/or field data are available and not the data/plots which would permit independent derivation of parameters. Data has also been obtained from the literature to broaden the data base. This has been necessary due to the variation in piezocone design which permits pore pressure measurement at any one of several locations; as a consequence, more information is required to obtain a sufficient sample size for each of the pore pressure measurement locations. A complete summary of the sites from which data has been obtained is given in Table 3.

TABLE 3

List of sites for which both laboratory and CPTU dissipation studies have been performed

Site/Location	Soil Type	Reference
Amherst (USA)	Connecticut Valley Varved Clay-Moderately to lightly overconsolidated	Baligh and Levadoux (1980)
Attakapa Landing (USA)	Soft clay and clayey silt Normally consolidated	Tumay (1990)
Saugus (USA) (Boston Blue Clay)	Boston Blue Clay Moderately to lightly overconsolidated	Baligh and Levadoux (1980)
Brage, North Sea,(Norway)	Silty marine clay and sandy silt Normally consolidated	NGI, 1988
St. Albans (Canada) (Champlain Sea Clay)	Champlain Sea Clay Very sensistive, lightly OC (aged)	Roy et al. (1982) Tavenas et al. (1974)
McDonald Farm (Canada)	Normally consolidated clayey silt	Gillespie and Campanella (1988) Gillespie (1981) Hers (1989)
Burnaby (Canada)	Normally consolidated clay silt	Gillespie and Campanella (1981)
Troll, North Sea (Norway)	Sandy marine clay Lightly overconsolidated	NGI (1984)
Onsoy (Norway)	Marine clay silt Normally consolidated at depth	Mokkelbost,(1988)
Snorre, North Sea (Norway)	Silty sandy stiff marine clay Lightly oveconsolidated	NGI (1988)

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 (1990)
Haga (Norway)	Lean marine clay Moderately overconsolidated	Mokkelbost,(1988)
Fucino (Italy)	Clayey silt lake deposits Normally consolidated	Assoc. Geot. Italiana (1979) Marchetti and Totani (1989)
Porto Tolle (Italy)	Soft clayey silt Normally consolidated	Battaglio et al 91981) Jamiolkowski et al. (1979)
Guanabara Bay (Rio de Janeiro, Brazil)	Soft silty clay Normally consolidated	Sills et al, 1988
Gainesville, Florida (USA) (Lake Alice Clay)	Lake Alice Clay Clayey silty sand to clay	Gupta and Davidson (1986)
Norco, Louisiana (USA)	Silty sand clay Normally consolidated	Tumay and Acar (1984)
Langley, Lr. 232 St. (Canada)	Sensitive clay silt Lightly OC 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 OC	Senneset et al. (1989)
SLS (USA)	Soft to medium silty clay Lightly OC	Kabir and Lutenegger (1990)

Storz 264 Nebraska (USA)	Florence Lake Clays Normally consolidated	Lutenegger et al. (1988)
South Africa	Various Normally consolidated materials (tailings, alluminum)	Jones Van Zyl (1987)
Brent Cross, (UK)	Stiff, heavily overconsolidated London Clay, fissured	Powell et al, (1988)
Bothkennar, (UK) (Grangemouth)	Soft, slightly fissured organic silty clay	Powell et al, (1988)
Cowden, (UK)	Stiff, Glacial Till	Powell and Quarterman (1988)
Madingley, (UK) (Gault Clay)	Stiff, fissured silty clay	Lunne et al, (1986)

Note: Lightly OC $1 < OCR < 4$
Moderately OC $4 < OCR < 10$
Heavily OC $10 < OCR$

The laboratory reference coefficienet 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 is considered to be a logical choice to predict the *in-situ* response to imposed loads. If the soil is overconsolidated the refernce value of c_v represents the overconsolidated range. However, if the soil is loaded beyond the preconsolidation pressure the value of c_v will decrease significantly. This change in response due to loading cannot be directly accounted for in this project.

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 the calculated c_h from piezocone dissipation should correspond to the normally consolidated condition 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 a result of soil fabric and sample size. No distinction has been made between c 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 to permit this was not always available. However, for most sites this was not necessary as global ranges generally adequately reflected the ranges of $c_{v,h}$ obtained at any one depth. Notwithstanding this, where stratigraphic changes warrant, separate t_{50} - c_v averages have been used for separate layers.

3. 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. A summary of some of the main techniques is given in Table 4.

Table 4
Summary of Theoretical Solutions commonly employed for Analysis of CPTU Dissipation Tests (After Kabir and Lutenegger, 1990)

Technique	Model
Torstensson (1975,1977)	One-dimensional radial solutions corresponding to cylindrical and spherical cavities. Isotropic, elastic-perfectly plastic materials. Linear uncoupled one-dimensional finite difference consolidation analysis
Baligh and Levadoux (1986)	Estimation of strain from velocity field and strain path of different elements from streamlines. Linear isotropic material. Two-dimensional uncoupled consolidation (Terzaghi-Rendulic) theory.
Gupta and Davidson (1986)	Cavity expansion theory after making necessary corrections for the *in-situ* probe measurements. Isotropic and anisotropic conditions. Linear transient uncoupled axisymmetric consolidation problem.
Houlsby and Teh (1988)	Solution from the strain path method is taken as the initial stress state. Finite element analysis including the effect of continuous penetration is considered. Uncoupled Terzaghi-rendulic consolidation theory, solved using an alternating-direction implicit finite difference scheme.

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_o). From the dissipation data an approximate value of the coefficient of consolidation in the horizontal direction (c_h) can be derived. An ideal pore pressure dissipation curve is illustrated in Figure 3.

A comprehensive study and review of this topic was published by Baligh and Levadoux (1986), and the relevant conclusions from this study were:

1. The simple uncoupled solutions provide reasonably accurate predictions 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 have a significant influence on the dissipation process.

Research by Torstensson (1977) and Houlsby and Teh (1988) has also illustrated the importance of the soil stiffness, usually represented by the rigidity index I_r (= G/s_u).

At present, the existing theoretical solutions provide reasonable estimates of the initial distribution of excess pore pressures around a probe in soft, normally to lightly overconsolidated soils. However, in overconsolidated soils (OCR >> 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 degree cone penetrometer can be extremely large, as illustrated in Figure 2. This gradient of pore pressure often results in dissipations recorded behind the cone tip that 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 this response.

Variations in the initial distribution of excess pore pressures around the probe represents one of the major difficulties for the theoretical solutions. If the initial distribution is significantly different than the theoretical distribution, the shape of the measure pore pressure dissipation curve will be different than the theoretical curves. Soares et al. (1987) suggest that the initial pore pressures should be corrected for non-uniform effects and found that this gave better agreement with cavity expansion solutions. Hence, different values of c_h will be derived for different degrees of consolidation. It has been common practise to measure the time for 50% of the excess pore pressure to dissipate, i.e., t_{50}. Errors in the measurement of the initial pore pressure and estimate of the equilibrium pore pressure have the least effect on c_h for $U_{(t)} = 0.5$, compared with other values of $U_{(t)}$ (Baligh and Levadoux, 1986).

It is difficult to directly compare the different theoretical solutions because of variations in pore pressure element location and soil stiffness (I_r). However, Houlsby and Teh (1988) observed that the theoretical dissipation curves could be normalized using the following Modified Time Factor, T*:

$$T^* = \frac{c_h t}{R^2 \sqrt{I_r}} \tag{5}$$

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.

I_r = rigidity index = G/s_u

Figure 4 shows the Modified Time Factors derived by Houlsby and Teh (1988) and compares them with those derived by Torstensson (1977) for the pore pressure element locations immediately behind 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) provides essentially the same values as the most recent and comprehensive solutions by Houlsby and Teh (1988).

Because the solution by Torstensson (1977) is based on cavity expansion it is unable to clearly define different response curves for different pore pressure element locations.

The solution by Houlsby and Teh (1988) represents 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 (Figure 5) that plots the derived value of c_h, based on the solution by Houlsby and Teh (1988) and Teh (1988), against the time for 50% dissipation for the three main pore pressure element locations (u_1, u_2 and u_3). The chart shown in Figure 5 will therefore form the framework for the comparison between predicted and measured coefficients of consolidation (c_h).

Occasionally, it may not be economically convenient to stop the penetration process long enough to obtain the t_{50} time. To provide a framework for interpretation for short duration dissipation tests Teh (1988) proposed the analyses of the data in the form of a square-root of time plot. Teh (1988) observed that the initial dissipation of the theoretical curves in square-root time could be approximated by a straight line. If the gradient of this linear section is m, then c_h can be evaluated from the equation:

$$c_h = (m/M)^2 \sqrt{I_r}\, R^2 \tag{6}$$

where M = gradient corresponding to the theoretical curve for a given probe geometry and porous element location

m = measured gradient of the initial linear dissipation

The procedure based on the slope of the linear section of a square-root of time dissipation plot has also been included on the chart in Figure 5.

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 shown in Figure 5.

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, non-linearity and creep;
- non-linearity due to soil layering or nearness to a layer boundary;
- influence of macrofabric, such as fissuring;

- influence of clogging and smearing of the porous filter element.

In spite of the above limitations, Campanella and Robertson (1988) have 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. In this study the theoretical solution proposed by Teh (1988) will only be used as a framework for the comparison between dissipation times (t_{50}) and reference values of c_h.

4. CPTU Procedures

There are generally few differences in field test procedures between standard CPT and CPTU soundings, except those related to saturation, penetration and pauses in penetration.

Saturation

It is generally recognized that complete saturation of the piezocone is essential (Campanella et al., 1981; Battaglio et al., 1981; Lacasse and Lunne, 1982). Pore pressure response can be inaccurate and sluggish for poorly saturated piezocone systems. Both the maximum pore pressures and dissipation times can be seriously affected by air entrapment. Response to dynamic pore pressures can be significantly affected by entrapped air within the sensing element, especially for soft, low permeability soils (Acar, 1981; Gillespie,1990). Saturation procedures generally consist of the following operations:

- deairing filter elements
- deairing cone
- assembly of cone and filter element(s)
- protection of system during handling and penetration in unsaturated soils, if required.

In the early days of piezocone testing, it was common to deair the filter elements and cone by boiling the complete system, but this proved to seriously affect the life-time of the cones, and is generally no longer done.

General practice today is to carefully saturate the filter elements in the laboratory by placing them in a high vacuum with the de-aired saturating fluid for times ranging from 3 to 24 hours. The filter elements are then placed in a small fluid filled container for transportation into the field.

Campanella and Robertson (1988) suggested that the voids in the cone itself should be deaired by flushing with a suitable fluid, usually the same fluid used for saturation of the elements. Techniques for saturating piezocones will depend on individual cone design. The cone and filter elements should be assembled while submerged in the saturating fluid.

The next step after cone preparation and assembly is the lowering of the string of cone rods and penetration. A thin protective rubber sleeve or container is sometimes placed over the cone. To avoid premature rupture of the protective sleeve, a small hole is sometimes pushed using a "dummy" cone of a larger diameter than the piezocone. Sometimes a hand dug or predrilled hole is made depending on circumstances and soil stratigraphy. Predrilling to the water level is not always necessary if the filter element and saturating fluid develop a high air entry value to prevent loss of saturation. However, soil suctions can be very large in unsaturated clays where predrilling may be necessary. Campanella and Robertson (1988) recommend that the entire saturation procedure be repeated after each sounding, including a change of filter element.

One of the major difficulties with piezocone testing can be the evaluation of saturation. Frequently it has been suggested to check saturation before penetration. Unfortunately, it is very difficult to check saturation and it is questionable if such a check would be reliable since even a small amount of entrapped air produced during the handling or early penetration can drastically increase the system compressibility. Generally, saturation is evaluated by careful review of the pore pressure data during the sounding. Several examples have been published to illustrate the pore pressure response from poorly saturated piezocones (Battaglio et al., 1981; Campanella and Robertson, 1981; Lacasse and Lunne, 1982). In general, the pore pressure response for poorly saturated piezocones is sluggish and detailed macro-structure and soil stratigraphy is subdued. In the offshore situation the backpressure from the water depth improves the saturation so that procedures for saturation of the piezocone may be less critical than for shallow onshore soundings.

The requirement for complete saturation is very important at shallow depth where equilibrium pore pressures (u_o) are very low. Once significant penetration below the water table has been achieved (>5 m) the resulting equilibrium water pressure is often sufficient to ensure saturation. Penetration at shallow depth in saturated sands can produce negative pore pressures behind the tip which may cause temporary cavitation if u drops below about -100 kPa.

Rate of Penetration

The standard rate of penetration for CPTU is 2 cm/sec. For penetration in medium grained clean sands and coarser materials, these pore pressures dissipate almost as fast as they are generated and penetration takes place under drained conditions. For penetration in fine grained soils, such as clays and clayey silts, significant excess pore pressures can be generated because of their relatively low permeability, and penetration takes place under predominantly undrained conditions. Penetration into fine sands and silty sands can generate excess pore pressures, but penetration may be taking place under partially drained conditions.

Correct interpretation of the pore pressure data requires some knowledge that the penetration is predominantly undrained. Radial consolidation theory can be used to obtain a plausible estimate of the upper limit to soil permeability for which the piezo element will observe undrained pore pressures. The estimate of the upper limit to permeability for undrained penetration depends on the following: soil compressibility and stiffness, size of the cone and size and location of porous element. For standard 10 cm^2 base area cones and a 5 mm thick porous element located on or just behind the tip, Campanella and Robertson (1988) suggested a plausible upper limit to soil permeability for undrained penetration (at 2 cm/sec) is in the order of 1×10^{-7} m/s. A partially drained CPTU response may be observed for soils with a permeability in the range of 1×10^{-4} m/s to 1×10^{-7} m/s, that is, soils such as fine sands and silts. For permeabilities greater than about 1×10^{-4} m/s penetration is most likely fully drained. These values are approximate but have been generally confirmed by field observations. It has often been suggested that if penetration is partially drained at 2 cm/sec. the rate of penetration could be increased or decreased to produce an undrained or drained penetration, respectively. However, since the permeability of soils varies by orders of magnitude, the change in penetration rate required to significantly change the drainage process would generally also vary by orders of magnitude (Campanella et al., 1983). Penetration rates of 20 cm/sec. or faster and 0.2 cm/sec. or slower become impractical and also introduce further strain rate effects.

Pore Pressure Dissipation

Penetration is generally performed in 1 m strokes since most push-rods are generally 1 m in length. This produces pauses in the penetration process which last from about 15 to 90 seconds, depending on the individual pushing assembly. During these pauses in penetration any excess pore pressures start to dissipate.

If saturation is not maintained the measured pore pressures are smaller than the maximum. If a dissipation test is then performed the measured pore pressure dissipation rate will be influenced by the lack of initial saturation.

Sometimes the push rods are clamped to the pushing rig during the dissipation test. Although this stops the movement of the top of the push rods, the cone tip will continue to move very slightly as the elastic strain energy in the rods releases and the tip load reduces. The longer the push rods, and the greater the tendency for the soil to creep, the more significant this movement may be. This movement alters the total stresses in the soil around the conical tip and may influence the measured decay of pore pressures with time. It is generally agreed that this is only significant with the pore pressure element located on the face of the cone (u_1).

For penetration in stiff heavily overconsolidated clays the penetration pore pressures measured behind the tip (u_2) are smaller than the pore pressures measured on the face (u_1). Hence, when penetration is stopped for a dissipation test the u_2 pore pressures often initially rise before falling towards the equilibrium pore pressure (u_0) Therefore, the selection of the initial pore pressure can be difficult to calculate t_{50}. In this study, the initial pore pressure was selected at the point where the pore pressure starts to decrease.

5. Evaluation of CPTU Dissipation Data

Data from over 30 sites in Europe and North and South America have been compiled and reviewed. Table 5 presents a summary of the compiled data in terms of measured CPTU dissipation times (t_{50}) and reference coefficient of consolidation values. Reference values of vertical coefficient of consolidation (c_v) have been corrected to c_h using the measured or assumed values of k_h/k_v shown in Table 5.

Figures 6, 7 and 8 present all the available data in terms of CPTU t_{50} for each of the pore pressure locations u_1, u_2 and u_3 and reference c_h values obtained from oedometer tests on undisturbed samples.

For the laboratory derived values of c_v, it was decided to select the value determined 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*. The practise at NGI is to correct the oedometer c_v values for sample disturbance. Details of the correction procedure are given by Sandbaekken et al (1986) and are summarized in Appendix C. 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:

- The trend of the measured data (t_{50}, c_h) is consistent with the proposed theoretical framework.

- Of the three pore pressure locations considered, the u_2 data shows the least scatter and compares well with the lower bound of the proposed theoretical framework.
- For the u_1 and u_3 locations, the data shows a scatter larger than suggested by the theoretical framework.
- The reference values of laboratory c_h tend to be less than the predicted values for pore pressure locations u_1 and u_3.

The majority of reference c_h values from laboratory tests required a correction from c_v based on the ratio k_h/k_v, as shown in Table 5. Although this correction appears to be quite large for some sites, the relative change in position of the data points is generally quite small in relation to the four (4) orders of magnitude variation in c_h values encountered.

The data presented in Table 5 and Figures 6 to 8 for a given site represent a range of values for the soil profile. For the purpose of presentation an average value has been shown. In the Appendix to this volume the data are presented in terms of the average values along with the range of measured values for both c_h and t_{50}. The range represents a combination of the following:

- Variation in c_h and t_{50}, due to soil variability.
- Variation in measured t_{50} due to procedure and interpretation, such as selection of initial and final pore pressure.
- Variation in c_h due to uncertainty in interpretation of available data.

The large size of the ranges illustrate the degree of uncertainty with the derived values, especially with the reference c_h values.

It appears that for most of the data obtained the rods were not clamped during the dissipation stage. As stated earlier, for the u_1 location unloading may occur during the initial dissipation stages and that this unloading appears to be more severe in stiff, heavily overconsolidated clays.

For two sites, the data and soil conditions are such that sufficient data is available to derive c_h and t_{50} values for different stratigraphic sequences of the profile. Figure 9 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 (St. Albans and Drammen) appear to consistently plot below the theoretical framework and the other data. Both these sites comprise either sensitive or structured clays. Hence, the remolding, due to cone penetration and possibly sampling, may have influence on the t_{50} and laboratory c_h values.

For the u_2 location, the majority of t_{50} values are between 10 minutes and 60 minutes. 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 Figure 10 and suggests the following:

- The c_h derived from degrees of dissipation shorter than 50% can be significantly larger than c_h (50%).
- Depending on pore pressure element location, the c_h (20%) can be 1.5 to 10 times larger than c_h (50%).
- The largest variation appears to be for pore pressure element location u_1.

The large variation for short dissipation times may be a function of:

- Test procedure in terms of unloading on the face of the cone.
- Limitation of the theory to adequately predict the initial distribution of pore pressure.
- Variations in the ratio of c_h/c_v, especially for the u_1 location.
- Variation in stress history effects at degrees of dissipation less than 50%.

For some sites, it was possible to interpret the CPTU dissipation in terms of the square-root time method. Figures 11 and 12 presents the available data in terms of the slope of the square-root time plot and reference c_h values from laboratory oedometer tests. A summary of the available data is shown in Table 6. Although the data are rather limited, Figures 11 and 12 indicate that the proposed theoretical framework provides good values of c_h. Again, the sensitive clay data fits well below the other data.

The data presented in Figures 6 to 12 represent laboratory derived values of c_h. For a limited number of sites reference values of c_h could also be derived from back-analysis of field performance. The comparison of the field (c_h, field) and laboratory (c_h, lab) derived values are shown in Figures 13 and 14 for average values of t_{50} and c_h for u_1 and u_2 locations respectively. A summary of the available data relating field and laboratory c_h values is shown in Table 7. Figures 13 and 14 show that the field performance values of c_h are all larger than c_h derived from laboratory testing and that the largest difference appears to occur for the more sensitive and structured clays. This is consistent with the reasults presented by Tavenas et al (1986) for tests performed in homogeneous and stratified Champlain Sea clays. Figures 13 and 14 would then suggest that, if the difference between c_h (lab) and c_h (field) is similar for most sites, then the data shown in Figures 6, 7 and 8 would plot closer to the theoretical framework. However, the difference between c_h (field) based on performance and c_h (lab) appears to be a function of soil structure.

So far data has only been presented in terms of c_h. Data are available from some of the sites to evaluate the correlation between t_{50} and the laboratory derived values of the coefficient of permeability k. A summary of the available data is shown in Table 8. Figure 15 presents the available data of k and t_{50} and compares these with a preliminary relationship proposed by Schmertmann (1974).

Although the scatter is large, there does appear to be a trend. For dissipation times (t_{50}) less than 0.5 minutes, 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 the variation in soil compressibility and stress history as well as the uncertainty in the reference values.

6. Improved Interpretation of Soil Type with CPTU

One of the primary applications of the Cone Penetration Test (CPT) is for stratigraphic profiling. Considerable experience exists concerning the identification and classification of soil types from CPT data. Several soil classification charts exist for CPT and CPTU.

Some of the most comprehensive recent work on soil classification using electric cone penetrometer data was presented by Douglas and Olsen (1981). One important distinction made by Douglas and Olsen (1981) was that CPT classification charts cannot be expected to provide accurate predictions of soil type based on grain size distribution but provide a guide to soil behaviour type. The CPT data provide a repeatable index of the aggregate behaviour of the *in-situ* soil in the immediate area of the probe.

In recent years, soil classification charts have been adapted and improved based on an expanded data base (Robertson et al., 1986, Olsen and Farr, 1986). Also, research has illustrated the importance of cone design and the effect that water pressures have on the measured penetration resistance and sleeve friction due to unequal end areas (Campanella et al., 1982; Baligh et al., 1981). Thus, cones of slightly different designs, but conforming to the International Standard (ISSMFE - 1977) and Reference Test Procedure (ISOPT - 1988), will give slightly different values of q_c and f_s, especially in soft clays and silts.

Recent studies have shown that even with careful procedures and corrections for pore pressure effects, the measurement of sleeve friction (f_s) is often less accurate and reliable than the tip resistance (Lunne et al., 1986; Gillespie, 1990). Cones of different designs will often produce variable friction sleeve measurements. This can be caused by small variations in mechanical and electrical design features, as well as small variations in tolerances.

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

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{vo}} \tag{7}$$

where

u_2 = pore pressure measured between the cone tip and the friction sleeve

u_o = equilibrium pore pressure

σ_{vo} = total overburden stress

q_t = cone penetration resistance corrected for unequal end area effects. (Campanella and Robertson, 1982).

The original chart by Senneset and Janbu (1984) used q_c. However, it is generally agreed that the chart and B_q should use the corrected total cone resistance, q_t.

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

A problem that has been recognized for some time with soil classification charts that use q_t and R_f is that soils can change in their apparent classification as cone penetration resistance increases with increasing depth. This is due to the fact that q_t, f_s and u all tend to increase with increasing overburden stress. For example, in a thick deposit of normally consolidated clay the cone resistance (q_c) will increase linearly with depth resulting in an apparent change in CPT classification for large changes in depth. Existing classification charts are based predominantly on data obtained from CPT profiles extending to a depth of less than 30 m. Therefore, for CPT data obtained at significantly greater depths some error can be expected using existing CPT classification charts that are based on q_t (or q_c) and R_f.

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

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

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

Normalized cone resistance, $$Q_t = \frac{q_t - \sigma_{vo}}{\sigma_{vo}'} \quad (8)$$

Normalized friction ratio, $$F_r = \frac{f_s}{q_t - \sigma_{vo}} \quad (9)$$

Pore pressure ratio, $$Bq = \frac{\Delta u}{q_t - \sigma_{vo}} \quad (10)$$

Based on these normalized parameters and using the extensive CPTU data base now available in published and unpublished souceces, modifed soil behaviour type classification charts have been proposed by Robertson (1990) and shown in Figure 16.

The two charts shown in Figure 16 represent a three-dimensional classification system that incorporates all three pieces of CPTU data. For basic CPT data where only q_c and f_s are available the lefthand chart (Figure 16) can be used. The error in using uncorrected q_c data will generally only influence the data in the lower part of the chart where normalized cone resistance is less than about 10. This part of the chart is for soft, fine grained soils where q_c can be small and penetration pore pressures (u_2) can be large.

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

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

Robertson (1990) suggested that the charts in Figure 16 are still global in nature and should be used as a guide to define soil behaviour type based on CPT and CPTU data. Factors such as changes in, stress history, *in-situ* stresses, sensitivity, stiffness, macrofabric and void ratio will also influence the classification.

Occasionally, soils will fall within different zones on each chart; in these cases judgement is required to correctly classify the soil behaviour type. Often the rate and manner in which the excess pore pressures dissipate during a pause in the cone penetration will significantly aid in the classification. For example, a soil may have the following CPTU parameters: $q_t = 0.9$ MPa, $f_s = 40$ kPa and $\Delta u = 72$ kPa at a depth where $\sigma_{vo} = 180$ kPa and $\sigma_{vo}' = 90$ kPa. Hence, the normalized CPTU parameters are:

$$Q_t = 8$$

$$F_r = 5.6\%$$

$$Bq = 0.1$$

Using these normalized parameters the soil would classify as a slightly overconsolidated clay (clay to silty clay) on the normalized friction ratio chart and as a silt mixture (clayey silt to silty clay) on the normalized pore pressure ratio chart. However, if the rate of pore pressure dissipation during a pause in penetration were very slow this would add confidence to the classification as a clay. If the dissipation were more rapid, say 50% dissipation in 2 to 4 minutes (2 mins < t_{50} < 4 mins), the soil is more likely to be a clayey silt.

In an effort to improve the interpretation, dissipation rates (t_{50}) have been compiled and correlated with fines content (% passing 0.74 mm sieve size, #200 sieve) and soil type. Figure 17 presents a summary of the data compiled relating t_{50} with fines content. For t_{50} < 50 sec. (0.5 min), the penetration process is partially drained and there appears to be no correlation exists between fines content and t_{50}. However, for t_{50} < 50 sec. the penetration process appears to become undrained and a linear relationship appears to exist between t_{50} and fines content. This correlation, however, is unlikely to be unique for all soils, since grain size distribution, plasticity, structure and soil sensitivity will also influence the t_{50} results.

Figure 18 presents a chart that correlates the normalized cone penetration resistance (Q_c) with t_{50} for different soil behaviour types. Figure 18 represents an extra dimension to the soil behaviour charts proposed by Robertson (1990) using normalized penetration Q_c, F_r and B_q (Figure 16).

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

7. Summary and Recommendations

Data from over 30 sites in Europe and North and South America have been compiled and reviewed. CPTU dissipation times have been compared with laboratory and field derived values of the horizontal coefficient of consolidation (c_h).

The least scatter in the results was obtained with the pore pressure element location immediately behind the cone tip (u_2). Based on the data compiled, the recommended correlations to derive the field value of c_h from CPTU dissipation rates is the following equation:

$$c_h \text{ (field)} = A / t_{50} \tag{11}$$

where c_h is in cm^2/min and t_{50} is in minutes and where the corresponding values of A for each pore pressure location are given in Table 9. The data from location u_3 is insufficient to allow any recommendation to be made.

Table 9. Recommended and Measured Range of A values for Equation 11

Pore Pressure Element Location	Measured Range	Recommended Value
u_1	0.6 to 60	6
u_2	1.8 to 45	10

Basically, the expected reliability for the u_2 position should be about one-half an order of magnitude of the in-situ value, whereas for the u_1 position this increases to a full one order of magnitude. These magnitudes are consistent with previous published experience in the determination of the coefficient of consolidation. The recommended correlation is shown in Figure 19, which is very similar to the values predicted by Teh (1988) and Torstensson (1977)

Since there is less scatter with the u_2 position, this is the recommended position for estimating c_h in normally to moderately overconsolidated soils (OCR < 10).

Based on the available data reviewed, it would appear that the recommended values in Table 9 do not apply for either highly sensitive or structured clays.

For short dissipation times the CPTU can be interpreted using the slope of the square-root time plot in the following way:

$$c_h = m^2/B \tag{12}$$

where c_h is in cm^2/min and m^2 is in min^{-1}.

The corresponding values of B for each pore pressure location are given in Table 10.

Table 10. Recommended and Measured Range of B Values for Equation 12

Pore Pressure Element Location	Measured Range ($x10^{-2}$)	Recommended Values ($x10^{-2}$)
u_1	10 to 3.34	5.56
u_2	6.67 to 1.67	3.34

However, these values are tentative, due to the limited amount of data available in this format. Also, the dissipation data did not always show well defined initial linear portions, which may result from high pore pressure gradient around the cone tip, especially in stiffer soils.

Degrees of dissipation less than 50% can also be applied to estimate c_h. Available data suggests that c_h can be significantly overestimated for short degrees of dissipation (see Figure 10). Site specific corrections can be developed to allow the application of equation 11. However, correction can be significant for the u_1 pore pressure location.

The application of these correlations requires that the CPTU must be obtained for fully saturated pore pressure elements.

Penetration pore pressure and dissipation times (t_{50}) can be used to enhance the interpretation of CPTU data for soil behaviour type using Figures 16 to 18. Although the pore pressure data improves the interpretation of soil type, the classification is still influenced by grain size distribution, plasticity, structure, soil sensitivity and stress history (OCR).

A tentative correlation has been proposed to estimate soil permeability from CPTU dissipation rates (Figure 15).

8. Recommended Research

Although data from over 30 sites have been compiled and reviewed, there is a need for continued research. More data should be collected to improve the correlations suggested in this report. There is a need for more data from sites where field performance has been monitored. More data is needed to clarify the influence of OCR, soil structure sensitivity and fabric. There is also a need for increased data for soil type interpretation, esepcially for fines content in low plastic soils.

TABLES

Table 5. Summary of CPTU field dissipation results in terms of t_{50} and laboratory oedometer results

Site (Location)	Pore Pressure Location	t_{50} Range (min)	t_{50} Ave (min)	c_v Range (cm^2/min)	c_v Ave (cm^2/min)	k_h/k_v
Guanabara Bay (Rio de Janeiro, Brazil)	u_1	3.8 - 9.2	5.6	0.012 - 0.06	0.036	1
Saugus, I-5 (Boston Blue Clay) (USA)	u_1	4.9 - 8.8	6.7	0.06 - 0.12	0.09	2 (> 18m)
	u_1	0.2 - 2.4	1.1	0.12 - 0.6	0.36	2 (< 18m)
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

Table 5. Summary of CPTU field dissipation results in terms of t_{50} and laboratory oedometer results (Continued)

Site (Location)	Pore Pressure Location	t_{50} Range (min)	t_{50} Ave (min)	c_v Range (cm^2/min)	c_v Ave (cm^2/min)	k_h/k_v
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 , (USA)	u_1	0.05 - 2.5	0.48	0.06 - 0.12	0.09	8
Attakapa Landing (USA)	u_1	8 - 50	24	0.014 - 0.024	0.019	1
Lake Alice Clay (USA)	u_1	1.2 - 13.3	7.31	0.06 - 0.57	0.43	1
St. Albans (Champlain Sea Clay - Canada)	u_1	4.1 - 6.5	5.0	0.003 - 0.006	0.0045	1
Norco, Louisiana (USA)	u_1	94.7 - 117.7	106.2	0.006 - 0.018	0.12	1 (10-15m)
	u_1	49.4 - 69.9	59.7	0.012 - 0.036	0.024	1 (20-35m)
	u_1	25.7 - 33.4	29.6	0.012 - 0.09	0.051	1 (35-40m)
Cowden (UK)	u_1	17 - 70	33	0.03 - 0.114	0.072	1
Madingley (UK)	u_1	1.5 - 12.0	4.5	0.019 - 0.048	0.04	2
Brent Cross (UK)	u_1	60 - 280	128	0.005 - 0.019	0.0119	2

Table 5. Summary of CPTU Field Dissipation Results in terms of t_{50} and Laboratory Oedometer Results (continued)

Site (Location)	Pore Pressure Location	t_{50} Range (min)	t_{50} Ave (min)	c_v Range (cm^2/min)	c_v Ave (cm^2/min)	k_h/k_v
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
Lower 232St (Canada)	u_2	25 - 83	42	0.19 - 0.018	0.074	3
Lulu Island (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

Table 5. Summary of CPTU Field Dissipation Results in terms of t_{50} and Laboratory Oedometer Results (continued)

Site (Location)	Pore Pressure Location	t_{50} Range (min)	t_{50} Ave (min)	c_v Range (cm^2/min)	c_v Ave (cm^2/min)	k_h/k_v
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 (USA)	u_2	29.6	29.6	0.04 - 0.6	0.12	
Storz 264, (USA)	u_2	60 - 190	105	0.006 - 0.06	0.024	1
St. Albans (Champlain Sea Clay -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	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 (UK)	u_2	-	44	0.03 - 0.114	0.072	1
Bothkennar (UK)	u_2	12-90	39	0.079 - 0.048	0.03	1

Table 5. Summary of CPTU Field Dissipation Results in terms of t_{50} and Laboratory Oedometer Results (continued)

Site (Location)	Pore Pressure Location	t_{50} Range (min)	t_{50} Ave (min)	c_v Range (cm^2/min)	c_v Ave (cm^2/min)	k_h/k_v
Guanabara Bay (Rio de Janeiro, Brazil)	u_3	27 - 44	35.5	0.012 - 0.06	0.036	1
St. Albans (Champlain Sea Clay -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 (USA)	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

Table 6. Summary of Field CPTU Dissipation results in terms of m^2 and Laboratory Oedometer Results

Site (Location)	Pore Pressure Location	t_{50} Ave (min)	m^2 Ave	c_h (cm^2/min)
Brage (Norway) (15 cm^2 cone)	u_1	1.05	1.15	5.4
	u_2	1.17	0.9	
Drammen (Norway)	u_1	3.5	0.94	0.006
	u_2	21	0.04	
Onsoy (Norway)	u_1	5.9	0.12	0.25
	u_2	21	0.017	
Stjordal -Halsen (Norway)	u_1	0.27	0.15	7.9
	u_2	2.85	0.123	
Troll (Norway)	u_1	8.3	0.075	0.12

Table 7. Summary of estimated c_h ratios from laboratory and back-analysed field values

Site (Location)	Pore Pressure Location	t_{50} Ave (min)	c_h (lab) (cm^2/min)	c_h (field) (cm^2/min)	$\frac{c_h\text{ (field)}}{c_h\text{ (lab)}}$
Guanabara Bay (Rio de Janeiro, Brazil)	u_1 u_2 u_3	5.6 11.3 35.5	0.036	0.122	3.4
Colebrook Overpass, B.C. (Canada)	u_2	61	0.23	0.162	0.7
Laing Bridge South (Canada)	u_2 u_3	4.4 12.5	0.19	0.24	1.3
St. Albans (Canada)	u_2 u_3	11.3 17.1	0.0045	0.048	10.7
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, (USA)	u_1	1.1	0.36	2.1	5.8

Table 8. Summary of CPTU dissipation results and laboratory permeability results

Site (Location)	Pore Pressure Location	t_{50} Ave (min)	k Range (cm/s)	k Ave (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		
St. Albans. (Canada)	u_1	5.0		
	u_2	11.3	$2 - 4 \times 10^{-7}$	3×10^{-7}
	u_3	17.1		
Langley,(Lr 232) (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}-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}	

Table 8. Summary of CPTU dissipation results and laboratory permeability results (continued)

Site (Location)	Pore Pressure Location	t_{50} Ave (min)	k Range (cm/s)	k Ave (cm/s)
Snorre (Norway)	u_1 u_2	4.3 33	4.12×10^{-9} -2.8×10^{-8}	1.18×10^{-8}
Stjordal -Halsen (Norway)	u_1 u_2	0.27 2.85	1.2×10^{-6} -3.8×10^{-6}	2.37×10^{-6}
Troll (Norway)	u_1 u_2	8.3 137	4.7×10^{-9} -2.2×10^{-7}	4.04×10^{-8}
Amherst , (USA)	u_1	0.48	3×10^{-7} -1×10^{-6}	6×10^{-7}
Attakapa Landing (USA)	u_1 u_3	24 103	2.2×10^{-8} -3×10^{-8}	2.56×10^{-8}
Saugus (USA)	u_1 u_2	6.7 13.4	4.5×10^{-8} 3×10^{-7}	8.5×10^{-8}
Bothkennar (UK)	u_2	39	4.76×10^{-8} to 3.33×10^{-7}	1.78×10^{-7}
Madingley (UK)	u_1	4.5	7.61×10^{-10} to 1.74×10^{-9}	1.36×10^{-9}
Cowden (UK)	u_1 u_2	33 44	1.93×10^{-8} to 4.44×10^{-8}	2.85×10^{-8}

k = permeability at strain corresponding to P_o'

References

Associazione Geotechnica Italiana. 1979. Experiences on the Time-Settlement Behaviour of some Italian Soft Clays. Proc. 7th European Conference SMFE, Brighton, 1, 1-11.

Baligh, M.M and Levadoux, J.N., 1980. Pore Pressure Dissipation after Cone Penetration. Research Report R80-11, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, M.A.

Baligh, M.M., Vivatrat, V. and Ladd, C.C. 1980. Cone Penetration in Soil Profiling, ASCE Journal of Geotechnical Engineering Division, Vol. 106, GT4, April, pp. 447-461.

Baligh, M.M., Azzouz, A.S., Wissa, A.Z.E., Martin, R.T. and Morrison, M.H. 1981. The Piezocone Penetrometer, ASCE Geotechnical Division, Symposium on Cone Penetration Testing and Experience, St. Louis, pp. 247-263.

Baligh, M.M. 1975. Theory of Deep Site Static Cone Penetration Resistance. Research Report R75-56, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA.

Battoglio, M., Jamiolkowski, M., Lancellotta, R. and Maniscalco, R. 1981. Piezometer Probe Test in Cohesive Deposits. Proc. ASCE, Cone Penetration Testing and Experience, St. Louis, 264-300.

Bennett, R.H., Li, H., Valent, P.J., Lipkin, J. and Esrig, M.I. 1985. In-Situ Undrained Shear Strengths and Permeabilities Derived from Piezometer Measurements. American Society for Testing and Materials, Special Technical Publication 883, 83-100.

Campanella, R.G., Robertson, P.K. and Gillespie, D. 1983. Cone Penetration Testing in Deltaic Soils. Canadian Geotechnical Journal, 20:23-35.

Campanella, R.G., Sully, J.P. and Robertson, P.K. 1990. Comparison of Field Consolidation with Laboratory and In-situ Tests. Accepted for publication in Canadian Geotechnical Journal.

Campanella, R.G., Robertson, P.K. and Gillespie, D., 1981, In-Situ Testing in Saturated Silt, (Drained or Undrained?) Proceedings, 34th Int. Geotech. Conf. Fredericton, Canada.

Campanella, R.G. and Robertson, P.K., 1988, Current Status of the Piezocone Test, Invited Lecture, First International Conf. on Penetration Testing, ISOPT-1, Disney World, March, A.A. Balkema, pp.93-116.

Campanella, R.G., Gillespie, D. and Robertson, P.K. 1982. Pore Pressures During Cone Penetration Testing., Proceeding 2nd European Symposium on Penetration Testing, ESOPT-11, pp. 507-512.

Chan, A.A. 1982. An Analysis of Dissipation of Pore Pressures After Cone Penetration. M.S. thesis, Department of Civil Engineering, Louisianna State University, Baton Rouge, LA.

Clarke, B.G., Carter, J.P., and Wroth, C.P. 1979. In-Situ Determination of the Consolidation Characteristics of Saturated Clays. Proceedings, 7th European Conference on Soil Mechanics and foundation Engineering, Brighton, England, Vol. 2, pp. 207-213.

Douglas, B.J. and Olsen, R.S. 1981. Soil Classification Using Electric Cone Penetrometer. Symposium on Cone Penetration testing and Experience, Geotechnical Engineering Division, ASCE, St. Louis, pp. 209-227.

Douglas, B.J., Strutynsky, A.I., Mahar, L.J. and Weaver, J. 1985. Soil Strength Determinations from the Cone Penetration Test., Proceedings of Civil Engineering in the arctic Offshore, San Francisco.

Fahey, M., and Carter, J.P. 1986. Some Effects of Rate of Loading and Drainage on Pressuremeter Tests in Clays. Specialty Geomechanics Sym.: Interpretation of Field Testing for Design parameters, Adelaide, Australia, pp. 50-55.

Foott, R., Koutsoftas, D.C., and Handfelt, L.D. 1987. Test Fill at Chek Lap Kok, Hong Kong. ASCE Journal of Geotechnical Engineering, 113:106-126.

Gillespie, D.G., 1981, The Piezometer Cone Penetration Test, MA Sc Thesis, Dept. of Civil Eng., University of British Columbia, December 1981.

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. 1981. 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.

Gillespie, D.G. 1981. The Piezometer Cone Penetration Test. MA.Sc. Thesis, Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Gillespie, D.G. and Campanella, R.G. 1981. Consolidation Characteristics from Pore Pressure Dissipation after Piezometer Cone Penetration. Soil Mechanics Series No. 47, Department of Civil Engineering, 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.

Gupta, R.C. and Davidson, J.L. 1986. Piezoprobe Determined Coefficient of Consolidation. Soils and Foundations, 26(3);12-22.

Hegg, U., Jamiolkowski, M., Lancellotta, R. and Parvis, E. 1983. Problems of Evaluating Consolidation Coefficients from Observed Dissipation Rates. Discussion to Session 6, Proc. 8th European Conf. SMFE., Helsinki, Vol. 3, 1240-1242.

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.

Houlsby, G.T., and Teh, T.I. 1988. Analysis of the Piezocone in Clay. Proceedings, 1st International Symposium on Penetration Testing, Florida, Vol. 2, pp. 777-783.

Houlsby, G. 1988. Discussion Session Contribution, Penetration Testing in the U.K., Birmingham.

ISOPT, 1988. International Sym. on Penetration Testing. Report of the ISSMFE Technical Committee on Penetration Testing, CPT Working Party. Vol. 1, pp. 27-51.

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

Jamiolkowski, M., Lancellotta, R. and Wolski, W. 1983. Precompression and Speeding Up Consolidation. General report, 8th Euro. conf. soil Mech. Found. Eng., Helsinki, May, Vol. 3, 1207-1226.

Jamiolkowski, M. and Robertson, P.K. 1988. Future trends for Penetration Testing, Closing Address, Proceedings Penetration Testing in the U.K., Birmingham, England.

Jamiolkowski, M., Lancellotta, R., Marchetti, S., Nova, R. and Pasqualini, E. 1979. Design Parameters for Soft Clays. SOA Report, 7th European Conf. SMFE, Brighton.

Jones, G.A. and Rust, E.A. 1982. Piezometer Penetration Testing CUPT. Proceedings of the 2nd European Sym. on Penetration Testing, ESOPT II, Amsterdam, Vol. 2, pp. 607-613.

Jones, G.A. and Van Zyl, D.J.A. 1981. The Piezometer Probe-A Useful Tool. Proc. 10th Int. Conf. Soil Mech. Found. Eng., Stockholm, Vol. 2, 489-496.

Kabir, M.G. and Lutenegger, A.J. 1990. In Situ Estimation of the Coefficient of Consolidation of Clays. Can. Geot. Journal, Vol. 27, 1, 58-67.

Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G. 1977, Stress-Deformation and strength Characteristics, proceedings, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. 11, pp. 421-494.

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.

Levadoux, J.N. and Baligh, M.M. 1986. Consolidation After Undrained Piezocone Penetration. 1: prediction. ASCE Journal of geotechnical Engineering, 112:707-726.

Lunne, T., Eidsmoen, T., Gillespie, D. and Howland, J.D. 1986. Laboratory and Field Evaluation of Cone Penetrometers, proceedings of In-Situ '86, Specialty Conf., ASCE, Blacksburg, Virginia.

Lunne, T., Powell, J.J.M., Quarterman, R.S.T. and Eidsmoen, T.E., 1986, Piezocone Testing in Overconsolidated Clays, Proc. 39th Canadian Geot. Conf., Ottawa, pp 209-218

Lutenegger, A.J., Kabir, M.G. and Saye, S.R. 1988. Use of Penetration Tests to Predict Wide Drain Performance in a Soft Clay. Proc. 1st Int. Sym. Penetration Testing, Orlando, March, Vol. 2, 843-848.

Lutenegger, A.J., Saye, S.R., and Kabir, M.G. 1988. Use of Penetration Tests to Predict Wick Drain Performance in a Soft Clay. Proceedings, 1st International Symposium on Penetration Testing, Florida, 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, Brazil.

Marchetti, S. and Totani, G. 1989. C_h Evaluations from DMTA Dissipation Curves. Proc. 12th Int. Conf. Soil Mech. Found. Eng., Rio de Janeiro. Vol. 1, 281-286.

Mokkelbost, K.H.,1988, Application of Dilatometer for pile design. Dilatometer tests at four Norwegian Sites. Joint Res. Project NGI/BRE NGI Internal Report No. 521610-1 October 20th., 1988

NGI 1988, Contract Report to Norsk Hydro A/S, Brage Soil Survey 1987, Contract No. KO 1636/TU-5171, Geotectnical Data Vol. 1,2. Rep. No. 87261-03, 1 March 1988

NGI 1984, Oppdragsrapport til Norsk Hydro A/S., Troll Ost 1984, grunnundersokelser, laboratorierapport. Bind II - geologi. Ropport 84215-2, oktober 1984

NGI 1988, Contract report to Saga Petroleum A/S. Snorre soil investigation 1987 Block 34/7. Soil parameter report for design. Report 87260-3, January 1988

Olsen, R.S. 1984. Liquefaction Analysis Using the Cone Penetration Test. Proceedings of the 8th world Conf. on Earthquake Eng., San Francisco.

Olsen, R.S. and Farr, J.V. 1986. Site Characterization Using the Cone Penetration Test, Proceedings of In-situ '86, ASCE Specialty Conf., Blacksburg, Virginia.

Parez, L. and Bachelier, M. 1981. C_{vr}, K_h, δ_{au} Determines Par Penetration Statique. Proc. 10th Int. Conf. Soil Mech. and Found. Eng., Stockholm, Vol. 2, 553-556.

Powell, J.J.M. and Quaterman, R.S.T., 1988, The Interpretation of Cone Penetration Tests in clays - with particular reference to rate effects., Proc. ISOPT-1, Florida, pp903-910

Powell, J.J.M., Quarterman, R.S.T. and Lunne, T., 1988, Interpretation and use of the piezocone test in UK Clays, Proc. Penetration Testing in the UK, pp 151-156.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Grieg, J. 1986. Use of Piezometer Cone Data, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K. 1990. Soil Classification Using the Cone Penetration Test. Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151-158.

Robertson, P.K., Campanella, R.G., Brown, p.T., and Robinson, K.E. 1988a. Prediction of Wick Drain Performance Using Piezometer Cone Data. Canadian Geotechnical Journal, 25:56-61.

Robertson, P.K., Campanella, R.G., Gillespie, D., and By, T. 1988b. Excess Pore Pressures and the Flat Dilatometer Test. Proceedings, 1st Int. Sym. on Penetration Testing, Florida, Vol. 2, pp. 567-576.

Roy, M., Tremblay, M., Tavenas, F. and LaRochelle, P. 1982. Development of Pore Pressures in Quasi-Static Penetration Tests in Sensitive Clay. Canadian Geotechnical Journal, 19(2), 124-138.

Schmertmann, J.H., 1978a, Guidelines for Cone Penetration Test, Performance and Design, Federal Highway Administration, Report FHWA-TS-78-209, Washington, July 1978, 145 pgs.

Senneset, K., Sandven, R. and Janbu, N. 1989. The Evaluation of Soil Parameters from Piezocone Tests. Sym. on In Situ Testing of Soil Properties for Transportation Facilities, National Research Council, Washington, D.C., January.

Senneset, K., Sandven, R., Lunne, T., By, T., and Amundsen, T. 1988. Piezocone Tests in Silty Soils. Proceedings, 1st Int. Sym. on Penetration Testing, Florida, Vol. 2, pp. 955-966.

Senneset, K. and Janbu, N. 1984. Shear Strength Parameters Obtained from Static Cone Penetration Tests, ASTM STP 883, Symp., San Diego.

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. Proc. 1st Int. Sym. in Penetration Testing, Florida, Vol. 2, 967-974.

Tavenas, F.A., Chapeau, C., LaRochelle, P. and Roy, M. 1974. Immediate Settlements of three Test Embankments on Champlain Clay. Canadian Geotechnical Journal, 11(1), 109-

Tavenas, F., Leroueil, S., and Roy, M. 1982. The Piezocone Test in Clays: Use and Limitations. Proceedings, 2nd European Symposium on Penetration Testing, Amsterdam, pp.889-894.

Torstensson, B.A. 1975. Pore Pressure Sounding Instrument. Proceedings, ASCE Specialty Conference on In Situ Measurement of Soil Properties, Raleigh, NC, Vol. 2, pp. 48-54.

Torstensson, B.A. 1977. The Pore Pressure Probe. Nordiske Mote, bergemekanikk, Oslo, Norway, Paper no. 34, pp. 34.1-34.15.

Torstensson, B.-A. 1975, Pore Pressure Sounding Instrument, Proc., ASCE Spec. Conf. on In-Situ Measurement of Soil Properties, Vol. II, Raleigh, N.C., pp. 48-54.

Torstensson, B.-A. 1977. The Pore Pressure Probe, Nordiske Geotekniske Mote, Oslo, Paper No. 34.1-34.15.

Tumay, M.T., and Acar, Y. B. 1985. Piezocone Penetration Testing in Soft Cohesive Soils. Int Strength testing of marine sediments: laboratory and in situ measurements. American Society for Testing and Materials, Special Technical Publication 883, pp. 78-82.

Tumay, M.T. and Acar. 1984. Piezocone penetration testing in soft cohesive soils. In Strength Testing of Marine Sediments: Laboratory and in Situ Measurements. Amer. Soc. Testing and Materials, Special Technical Publication 883, 72-82.

Tumay, M.T. 1980. Personal written communication.

Wroth, C.P. 1984. Interpretation of In-Situ Soil Test, 24th Rankine Lecture, Geotechnique, 34, pp. 449-489.

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.

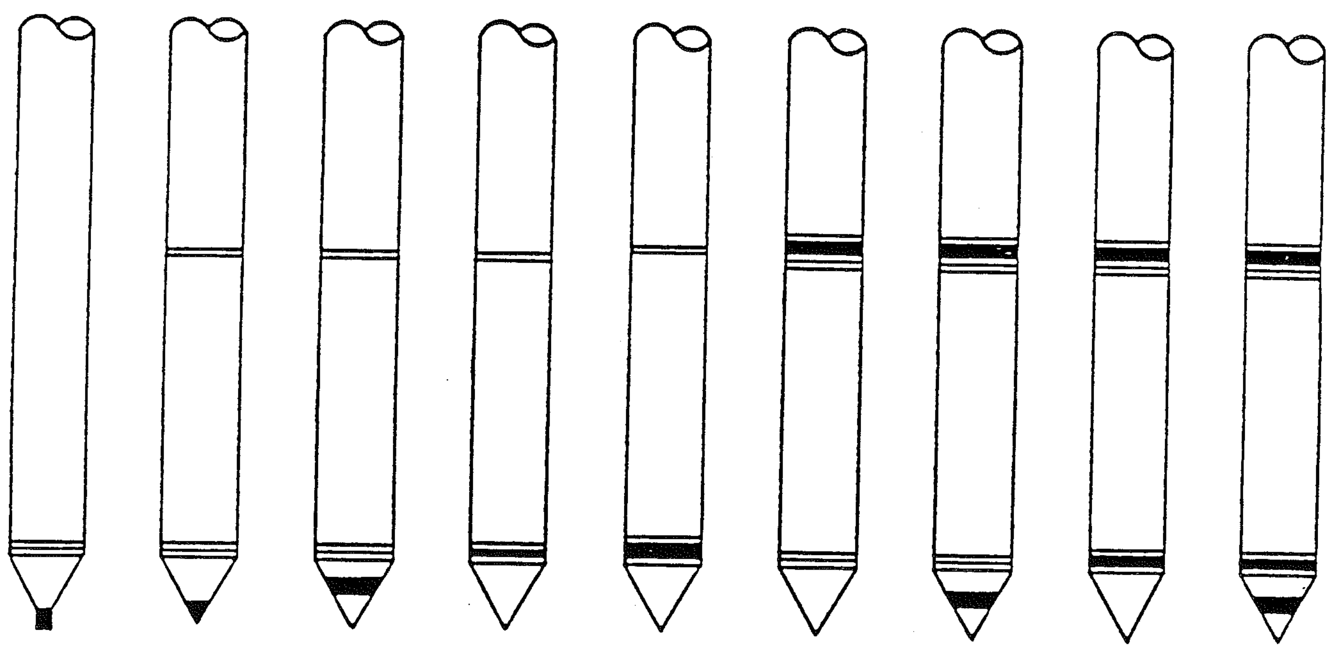


Figure 1 Pore pressure element locations

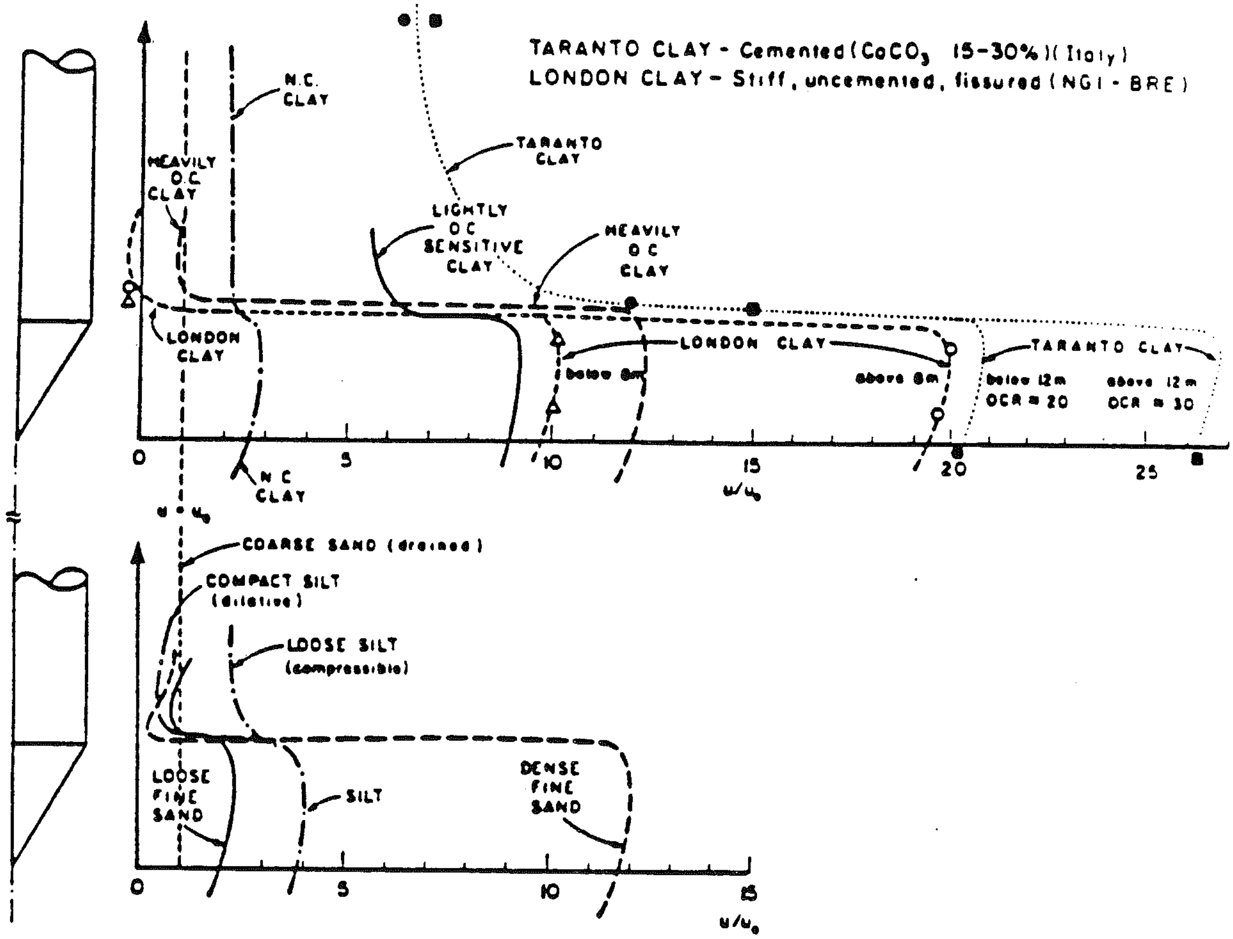


Figure 2 Distribution of pore pressures around a penetrating cone (After Robertson et al. 1986)

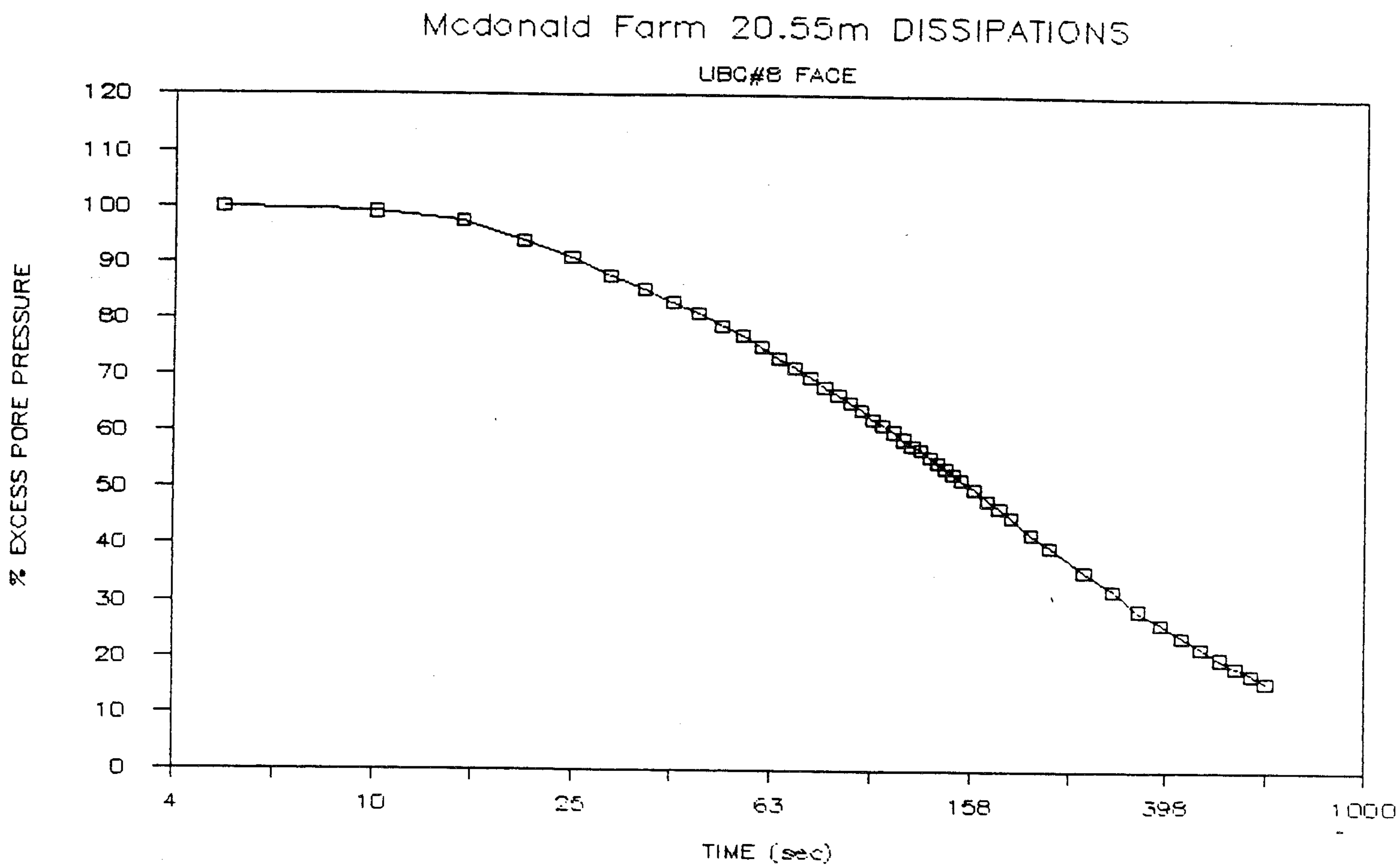


Figure 3 Typical pore pressure dissipation curve in log time

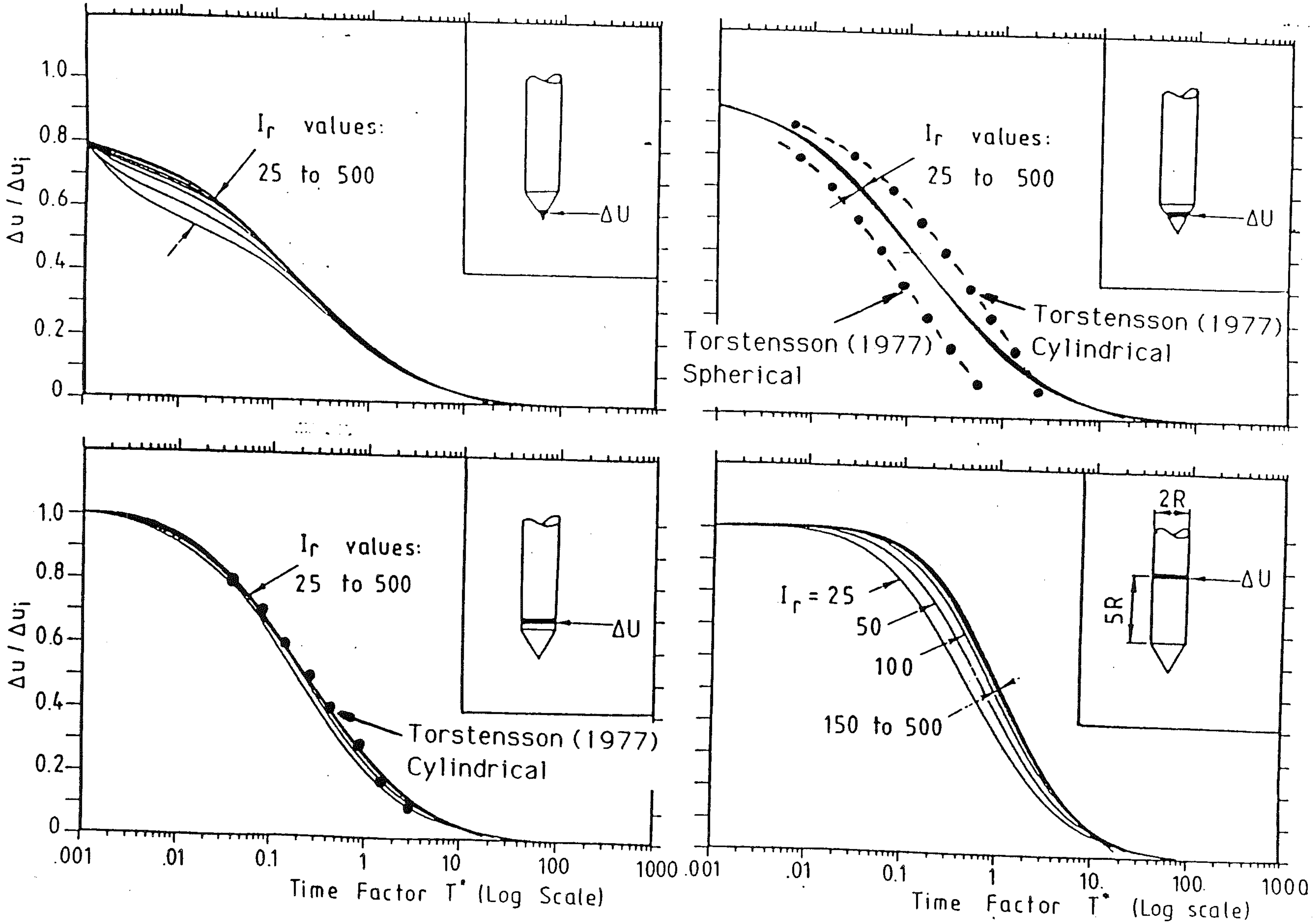


Figure 4 Normalized dissipation curves in log time based on theoretical solution by Teh (1988)

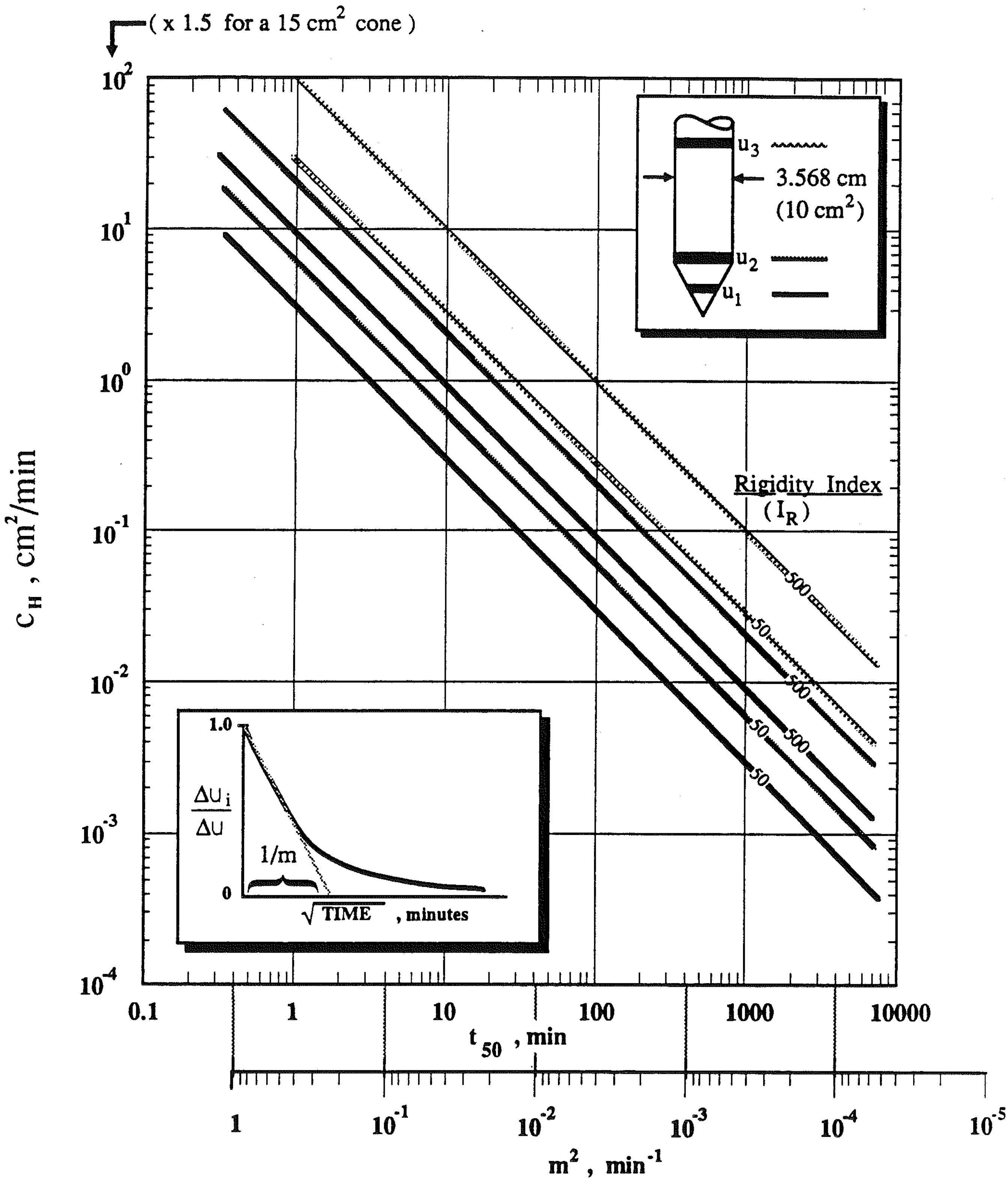


Chart for Evaluation of c_H from t_{50} or m^2 for 10 cm^2 and 15 cm^2 Piezocone
(Based on theoretical solution by Teh,1988)
Figure 5

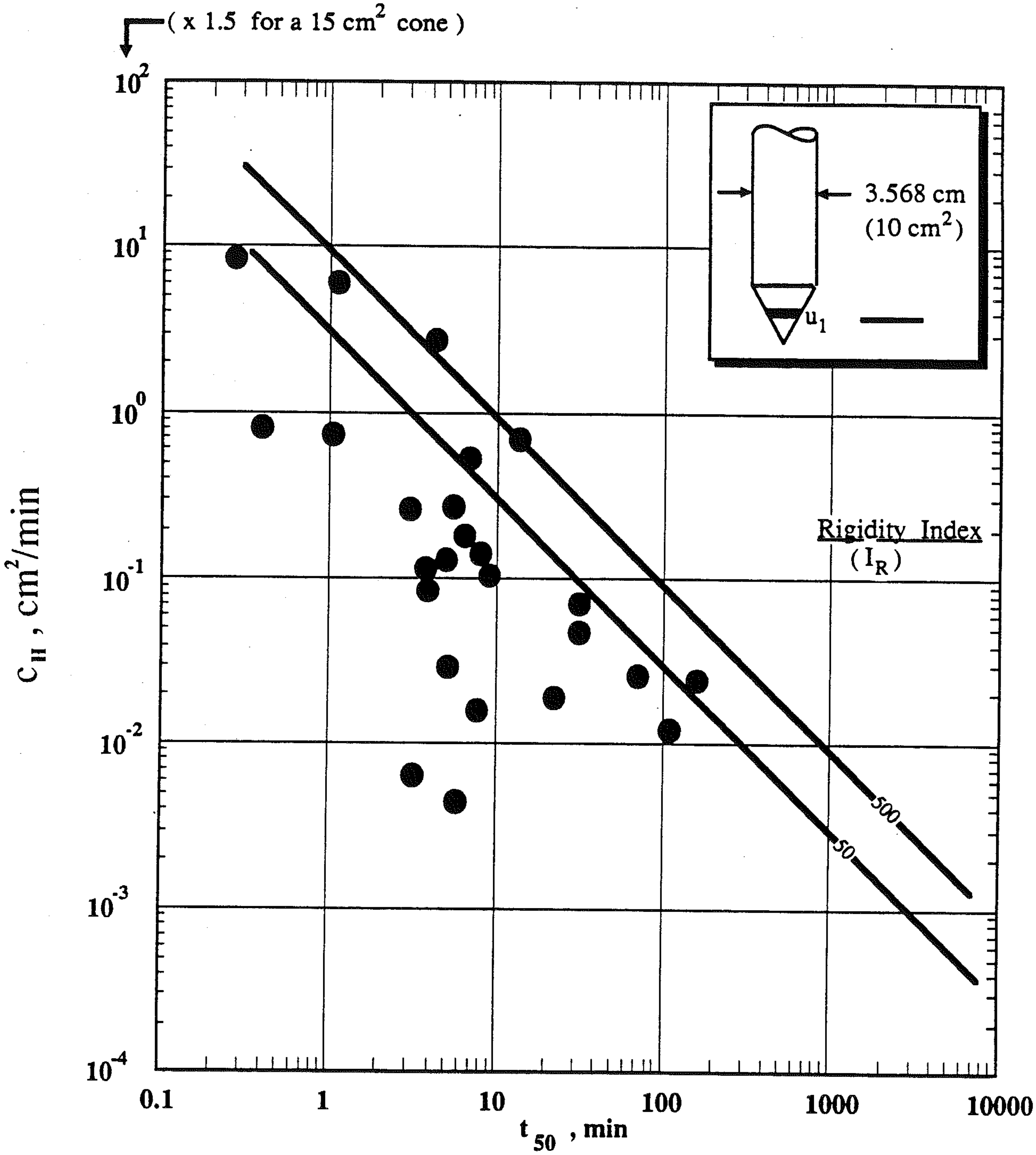


Figure 6 Average CPTU results in terms of t_{50} and laboratory oedometer results (u1)

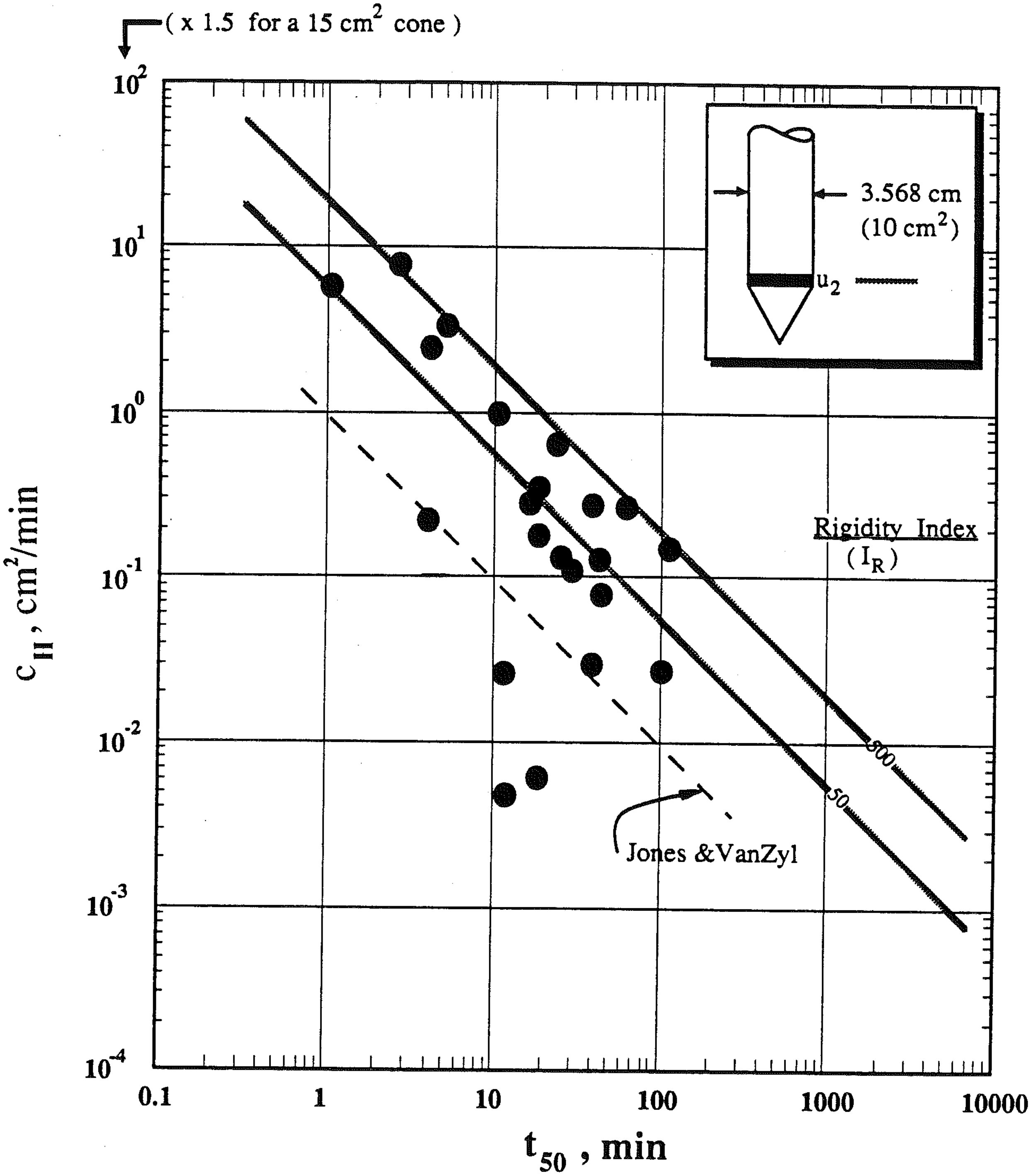


Figure 7 Average CPTU results in terms of t_{50} and laboratory oedometer results (u2)

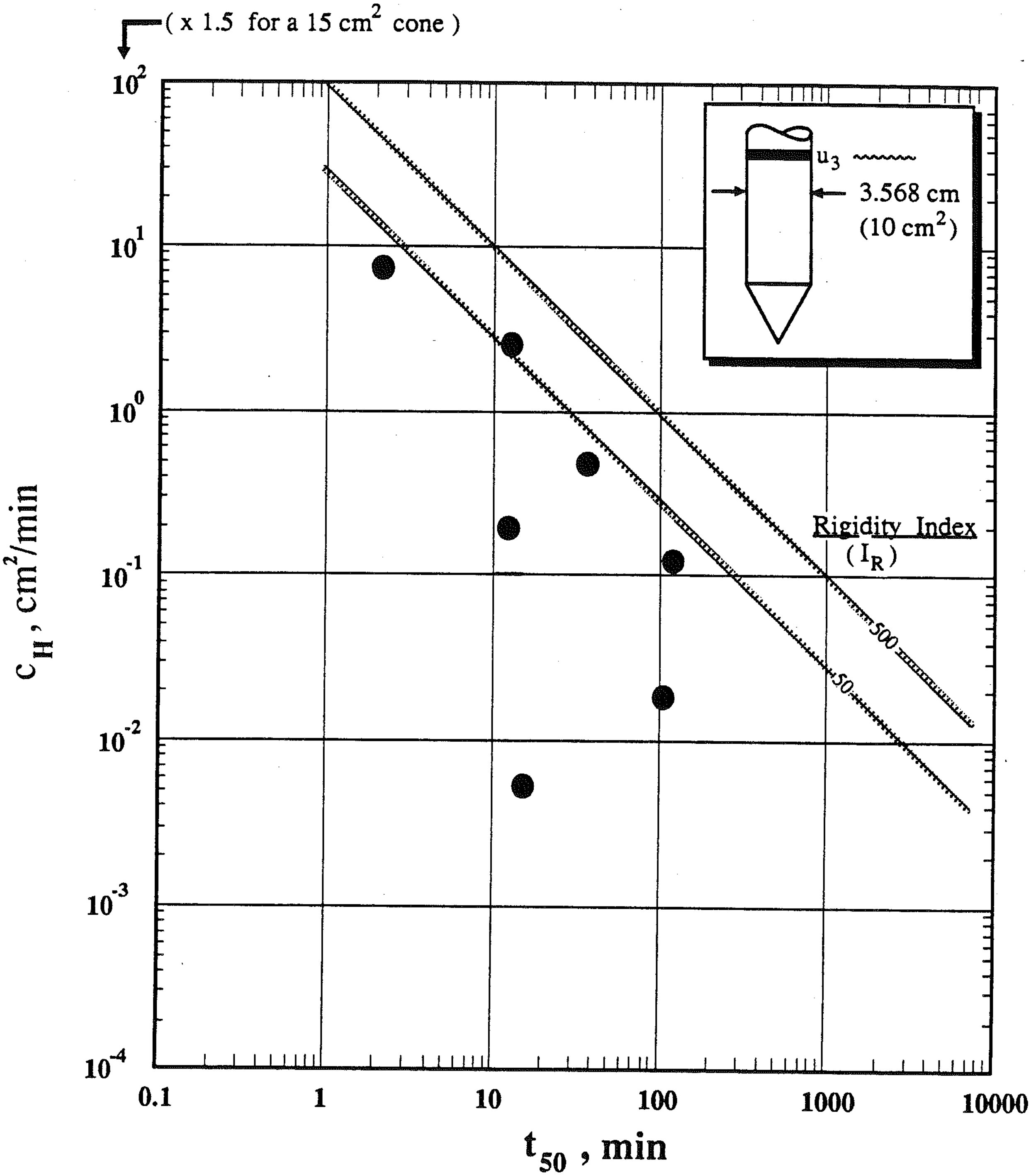
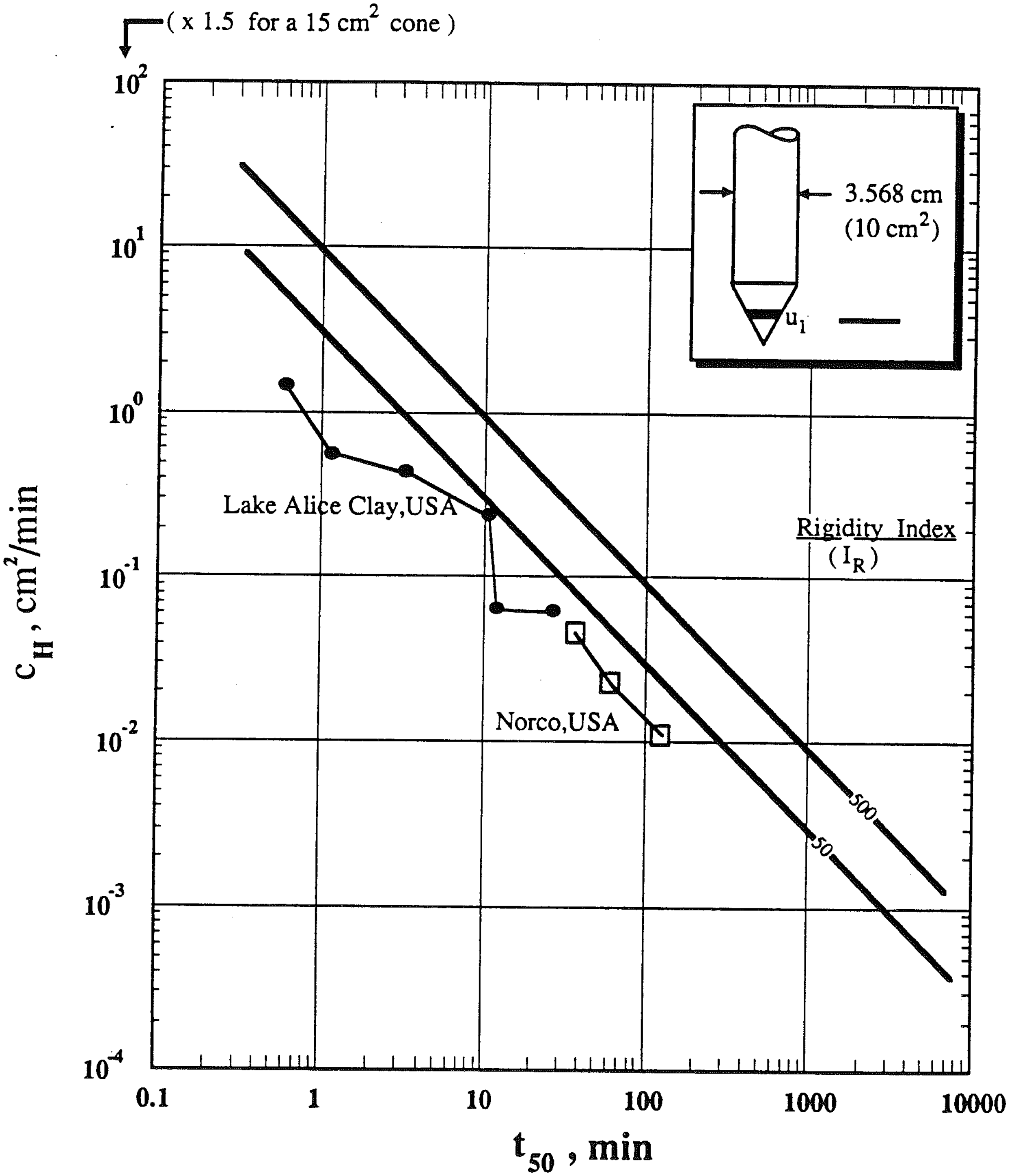


Figure 8 Average CPTU results in terms of t_{50} and laboratory oedometer results (u3)



Variation of c_H and t_{50} with depth for two sites

Figure 9

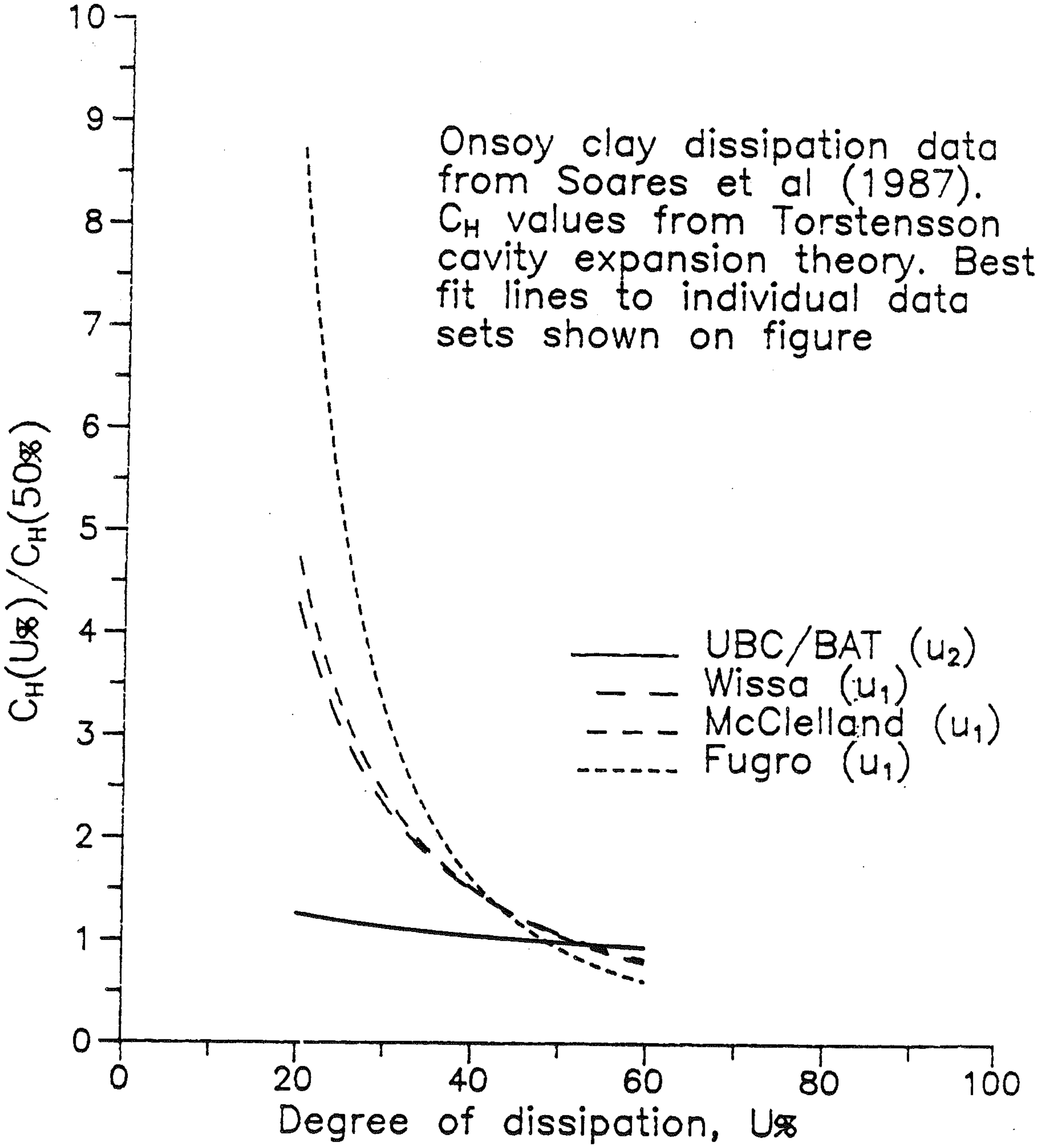


Figure 10 Variation in c_h determined at different degrees of dissipation compared to c_h determined at 50% dissipation

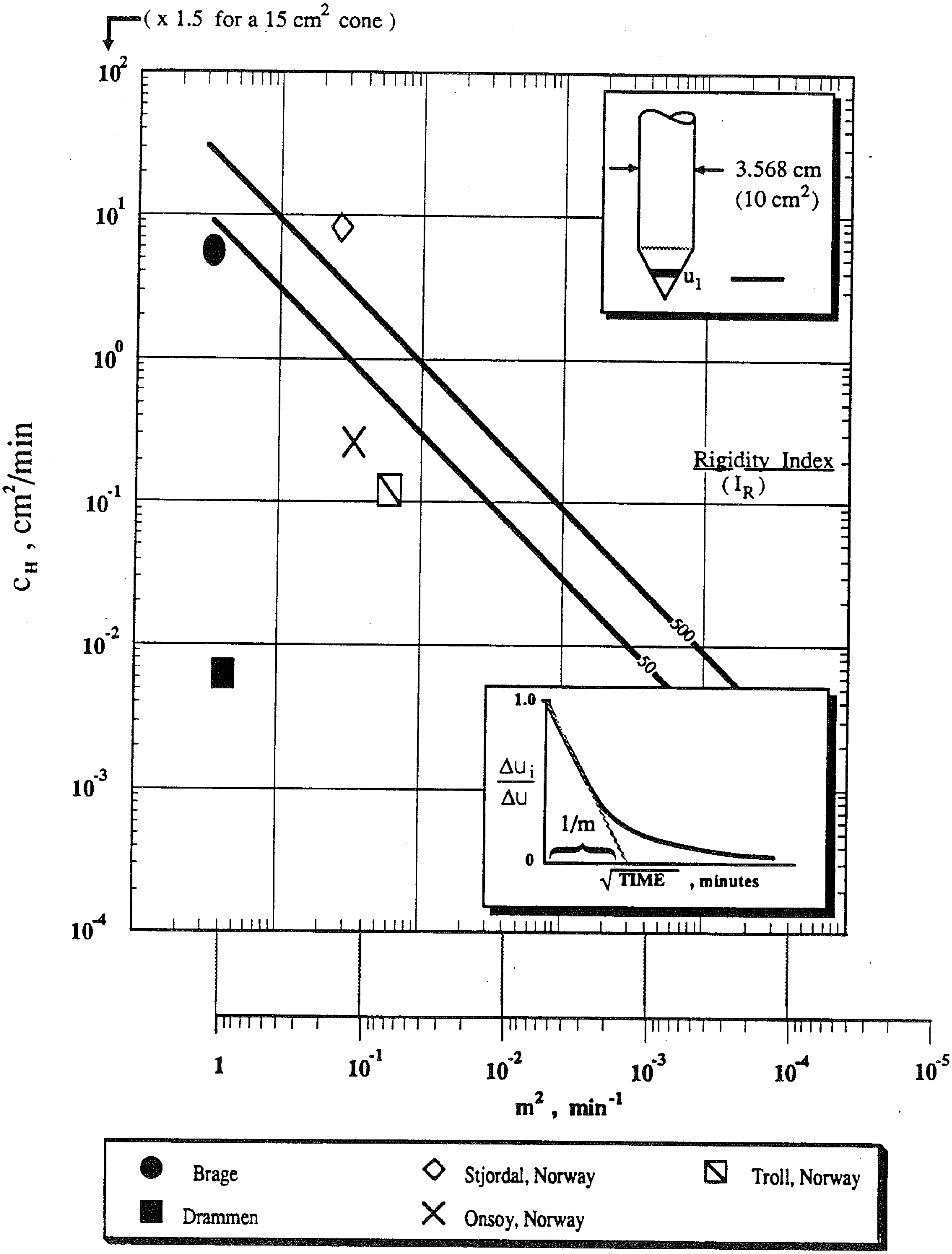


Figure 11 CPTU results in terms of m^2 and laboratory oedometer results (u1)

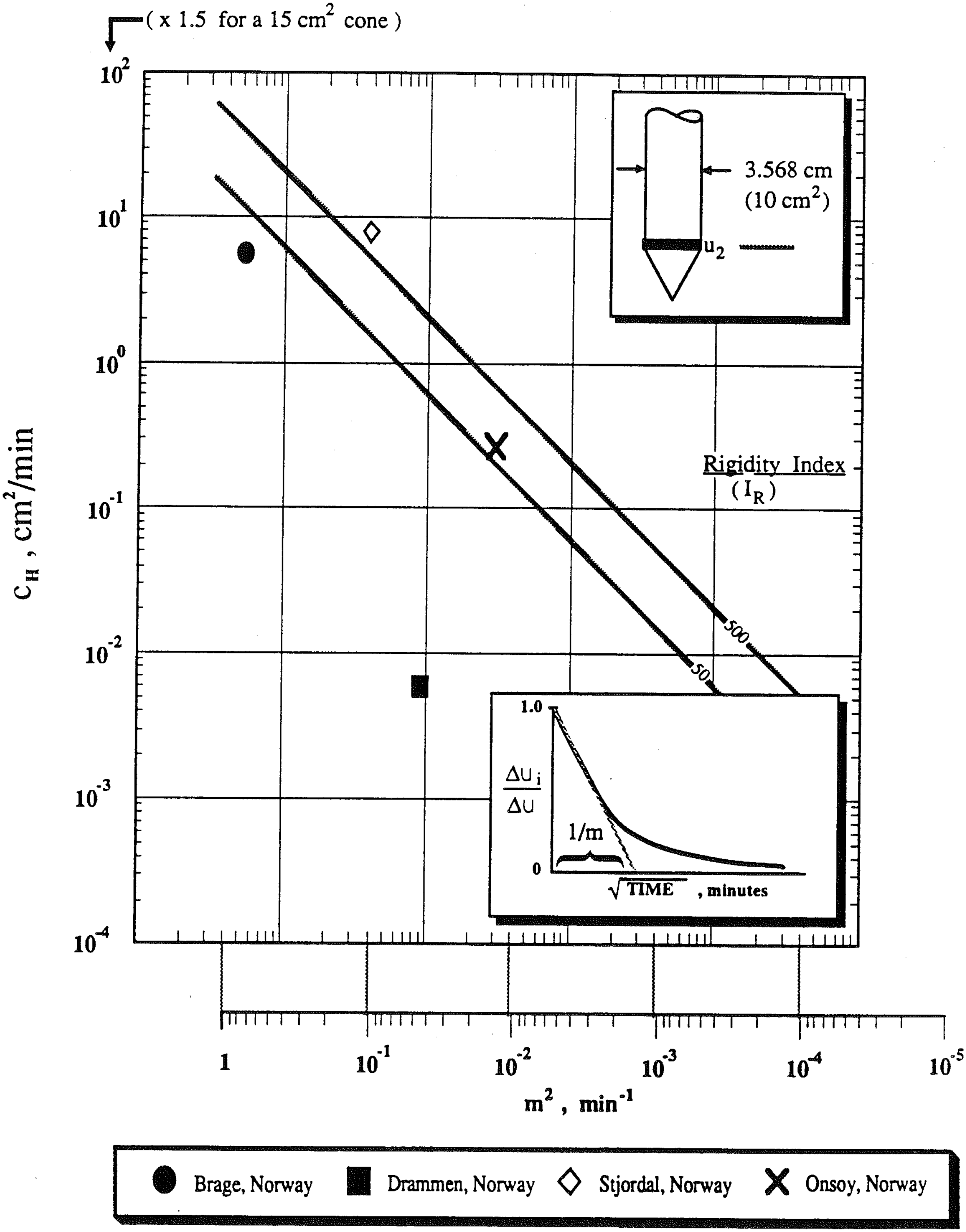
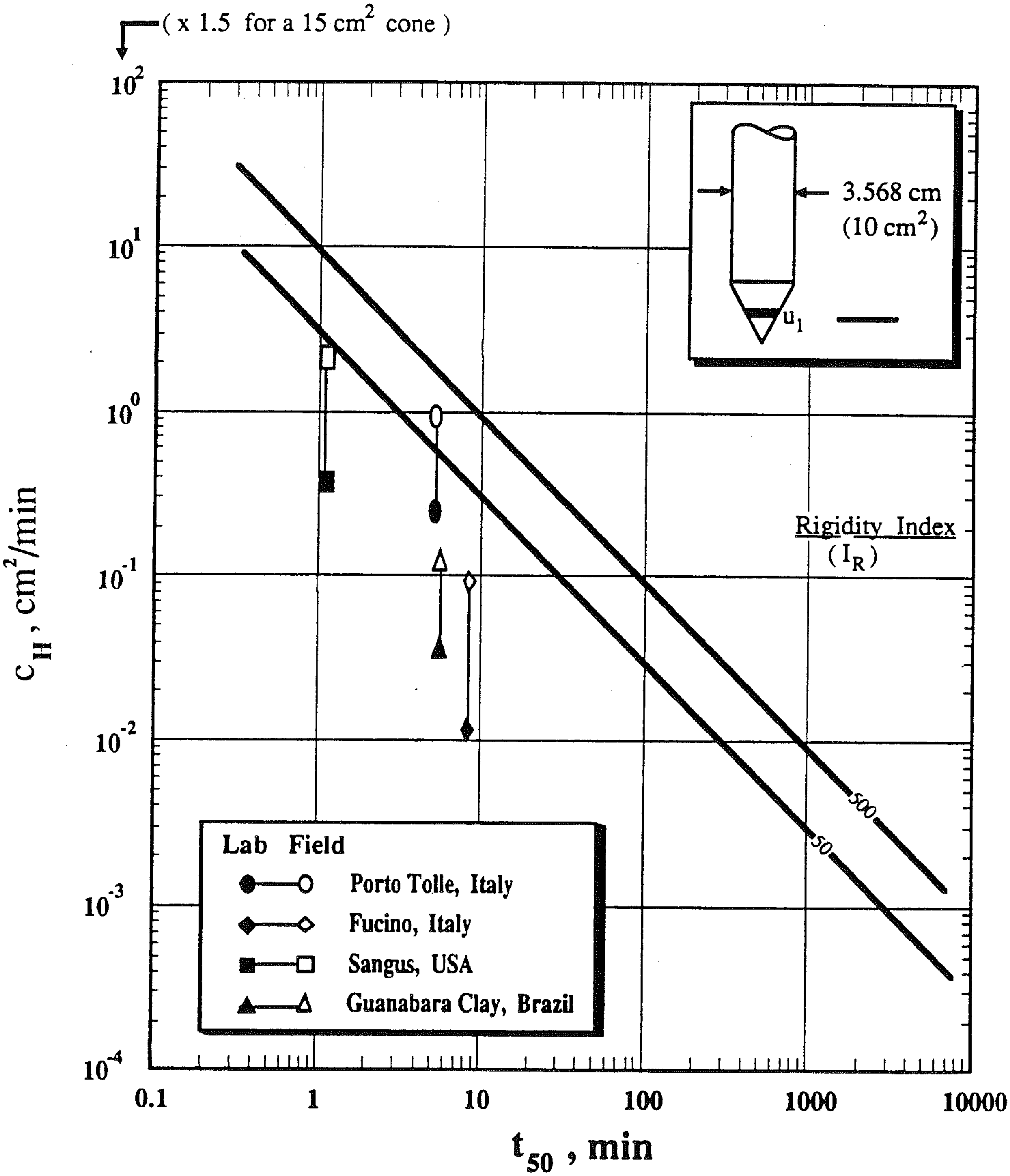
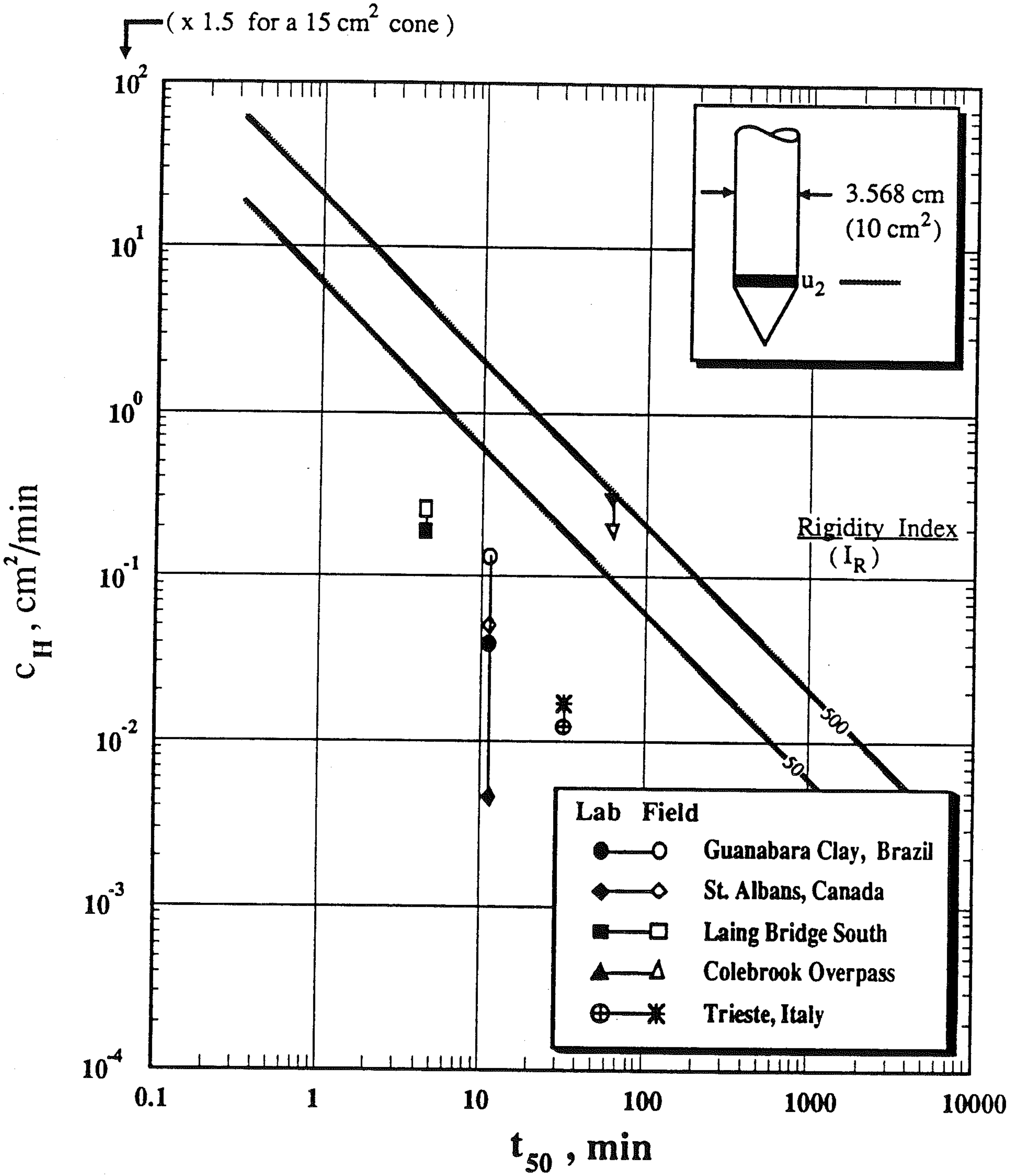


Figure 12 CPTU results in terms of m^2 and laboratory oedometer results (u2)



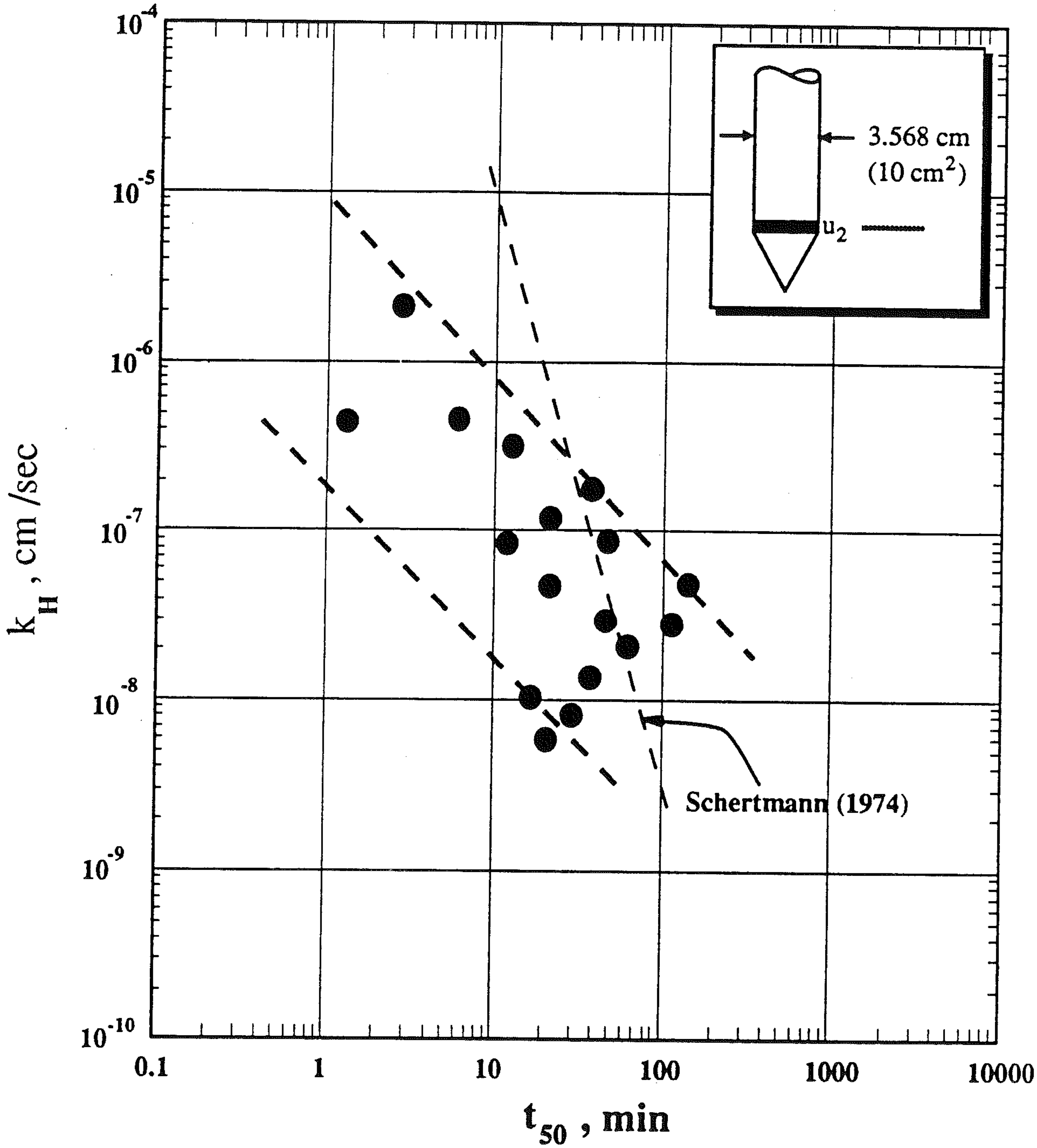
Comparison between field and lab C_h values - correlation with t_{50} from CPTU (u_1)

Figure 13



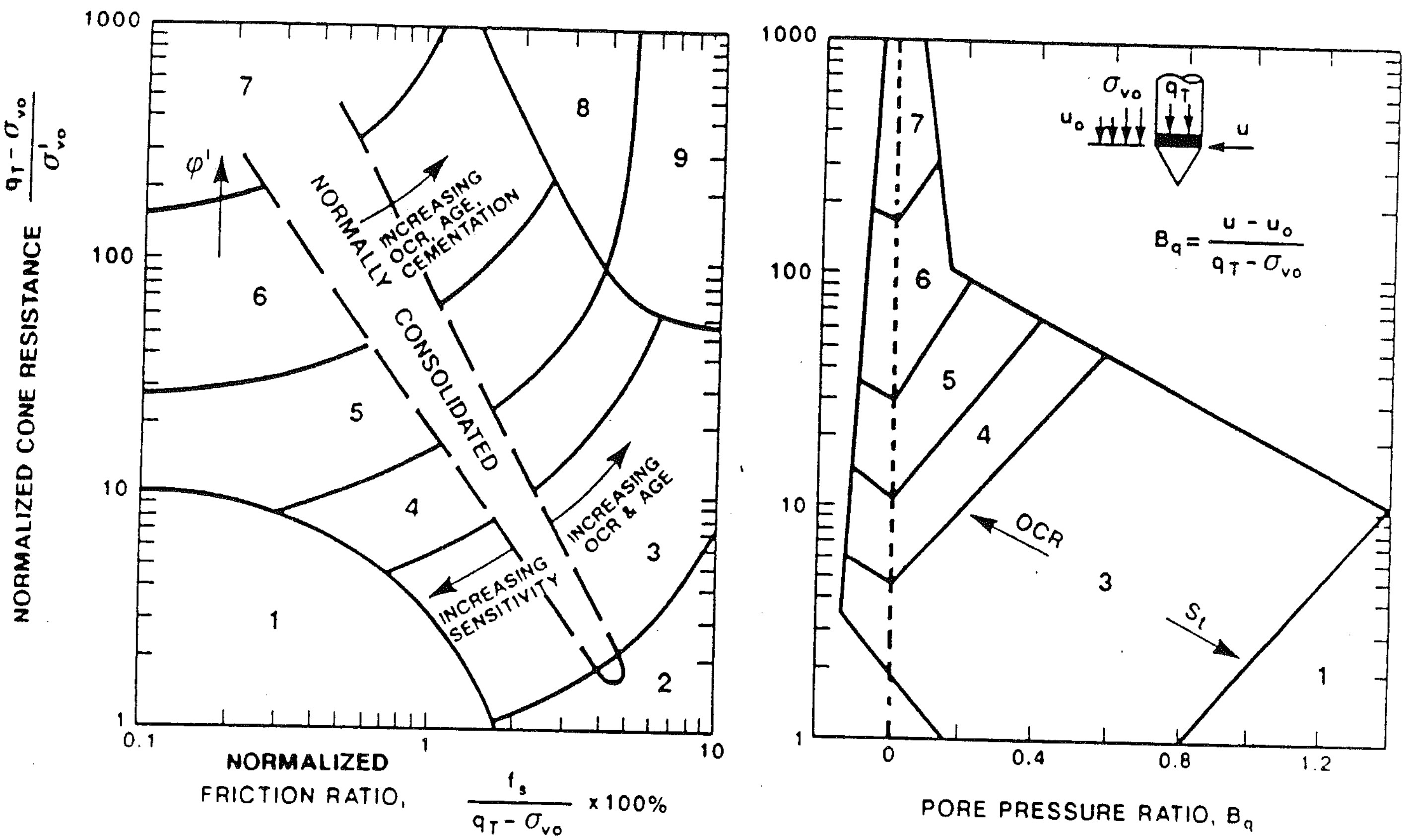
Comparison between field and lab C_h values - correlation with t_{50} from CPTU (u_2)

Figure 14



Proposed Chart for Evaluation of k_H from t_{50} for 10 cm^2 Piezocone (u2)

Figure 15



1. SENSITIVE FINE GRAINED
2. ORGANIC SOILS - PEATS
3. CLAYS - CLAY TO SILTY CLAY
4. SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
5. SAND MIXTURES - SILTY SAND TO SANDY SILT
6. SANDS - CLEAN SAND TO SILTY SAND
7. GRAVELLY SAND TO SAND
8. VERY STIFF SAND TO CLAYEY* SAND
9. VERY STIFF FINE GRAINED*

(*) HEAVILY OVERCONSOLIDATED OR CEMENTED

Figure 16 Soil behaviour type soil classification chart for CPTU (After Robertson 1990)

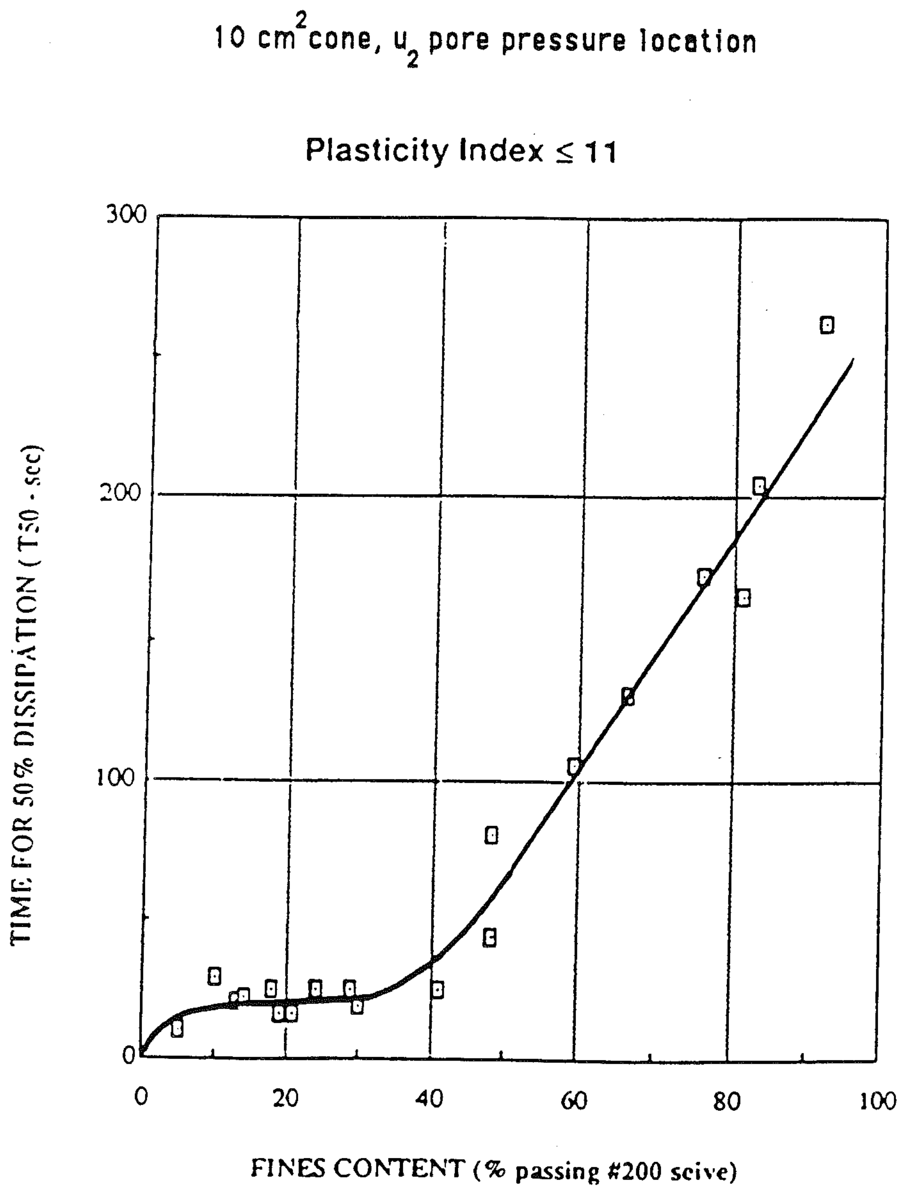
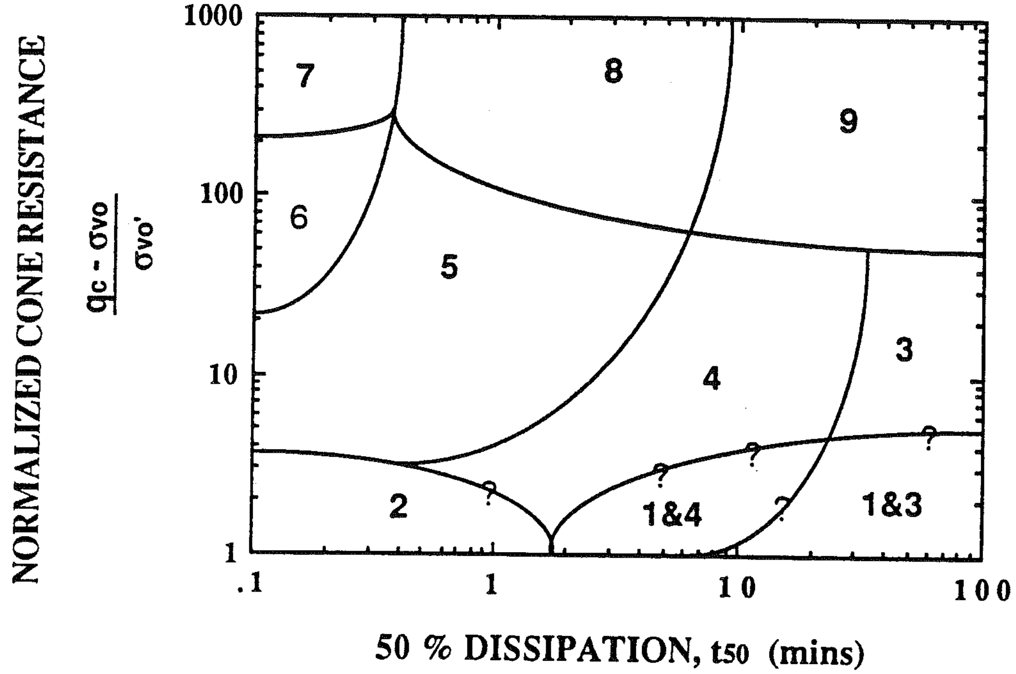


Figure 17 Variation of t_{50} with percent fines content

Proposed soil behaviour type chart

based on CPTU dissipation times

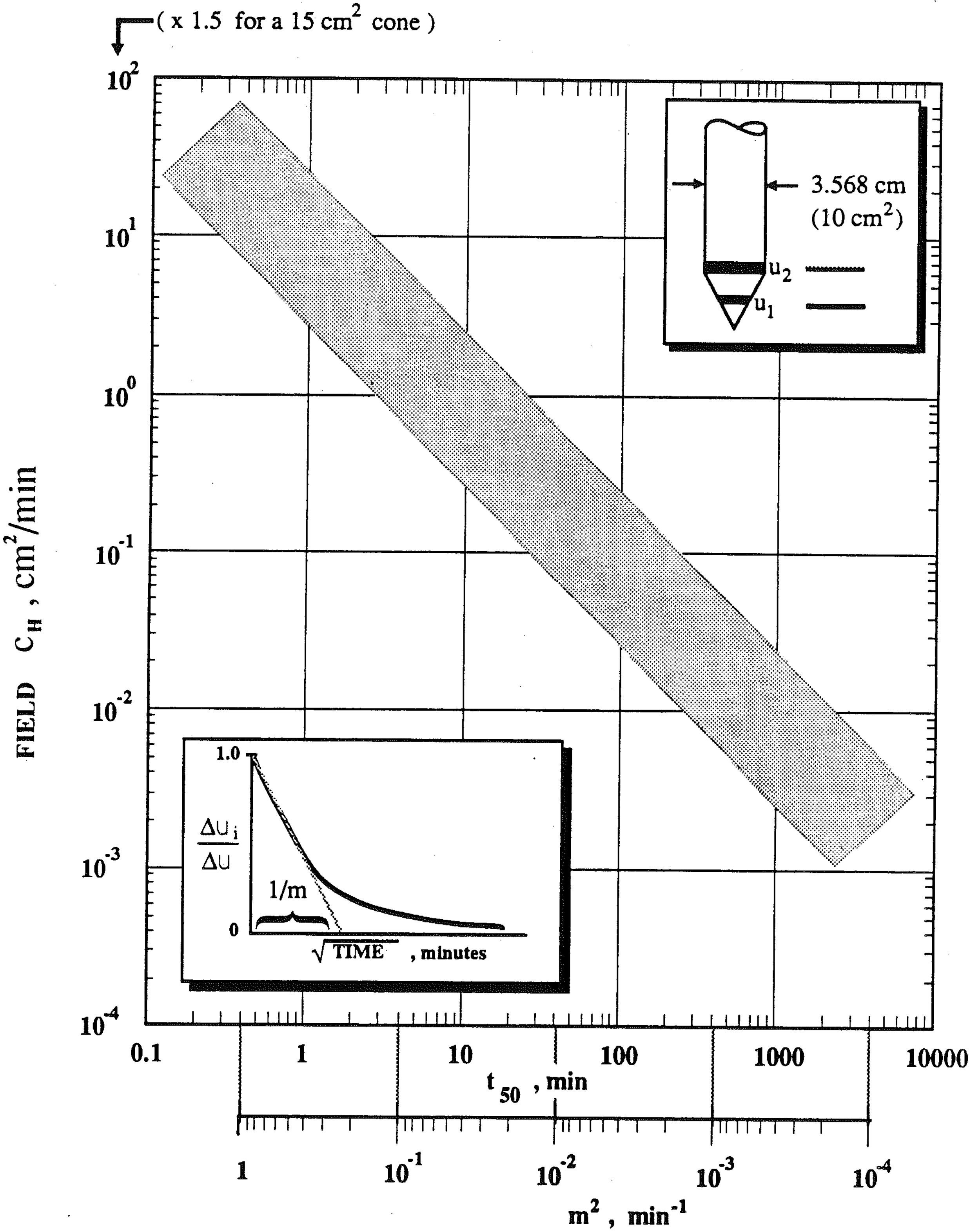


($10cm^2$ Cone, U_2 Pore Pressure location)

1. SENSITIVE FINE GRAINED
2. ORGANIC SOILS - PEATS
3. CLAYS - CLAY TO SILTY CLAY
4. SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
5. SAND MIXTURES - SILTY SAND TO SANDY SILT
6. SANDS - CLEAN SAND TO SILTY SAND
7. GRAVELLY SAND TO SAND
8. VERY STIFF SAND TO CLAYEY* SAND
9. VERY STIFF FINE GRAINED*

(*) HEAVILY OVERCONSOLIDATED OR CEMENTED

Figure 18 Proposed soil behaviour type classification based on CPTU dissipation data



Proposed Chart for Evaluation of field c_H from t_{50} or m^2 for 10 cm^2 and 15 cm^2 Piezocone

Figure 19

ADDITIONAL FIGURES

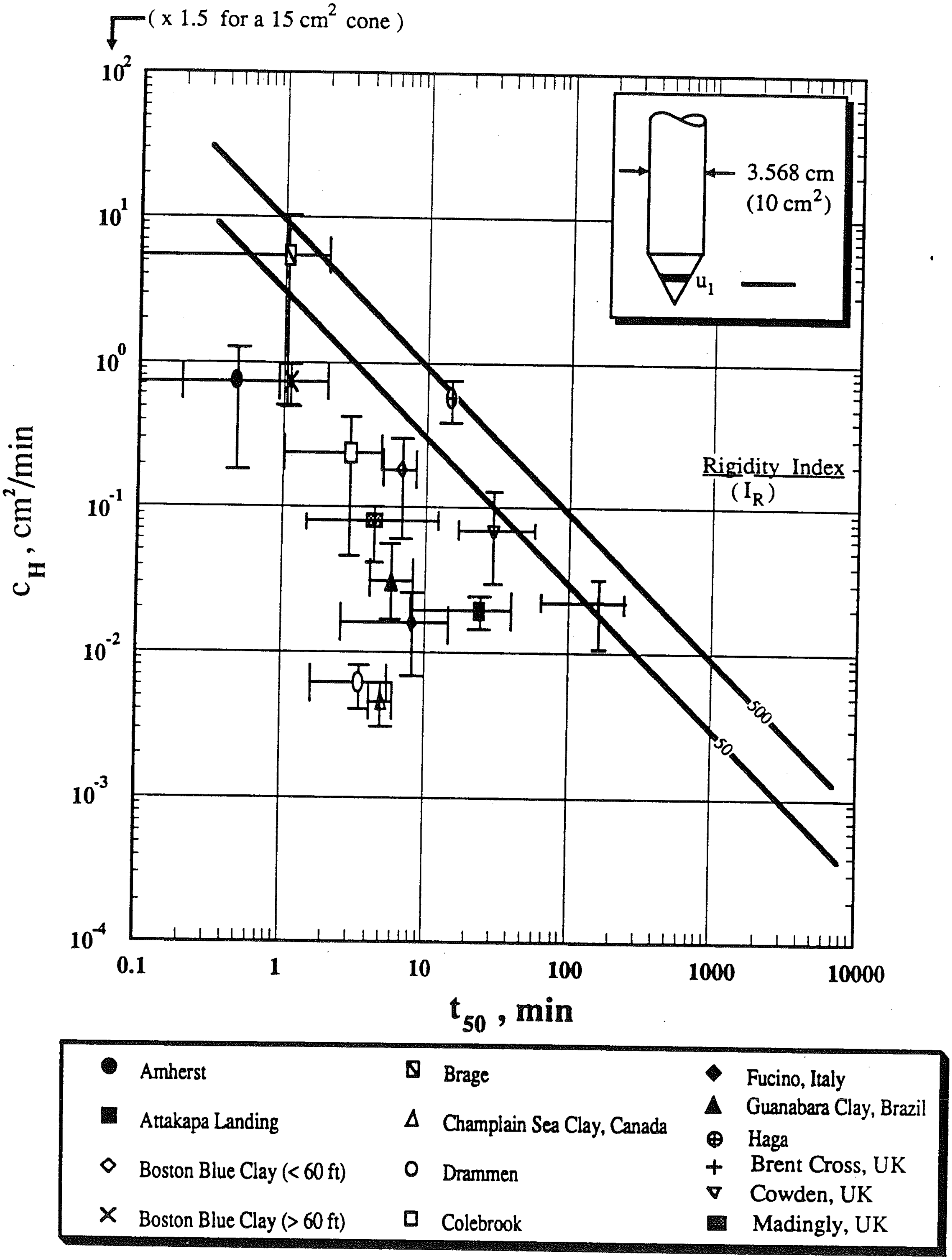


Figure 6a CPTU results in terms of t_{50} and laboratory oedometer results (u1)

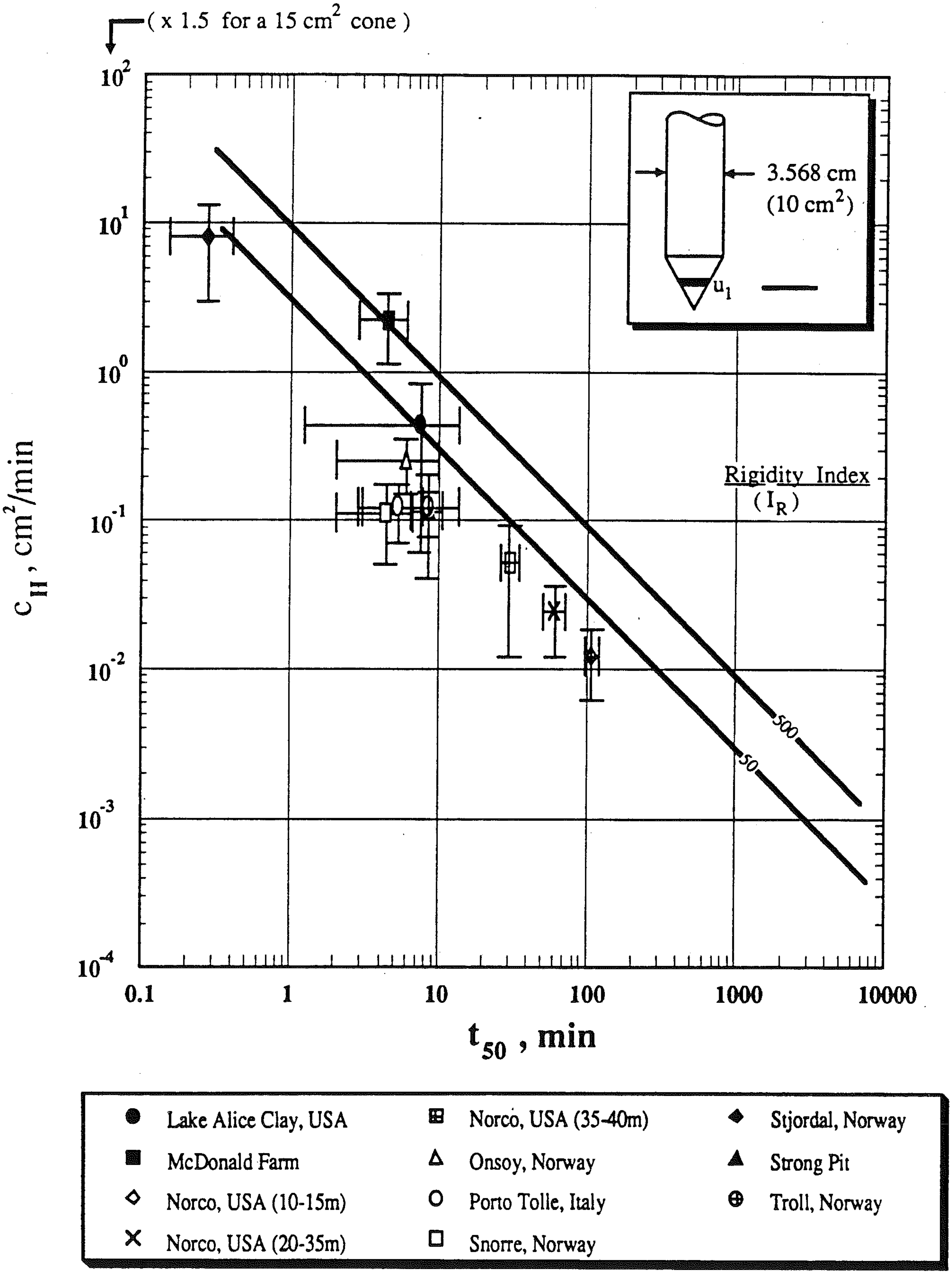


Figure 6b CPTU results in terms of t_{50} and laboratory oedometer results (u1)

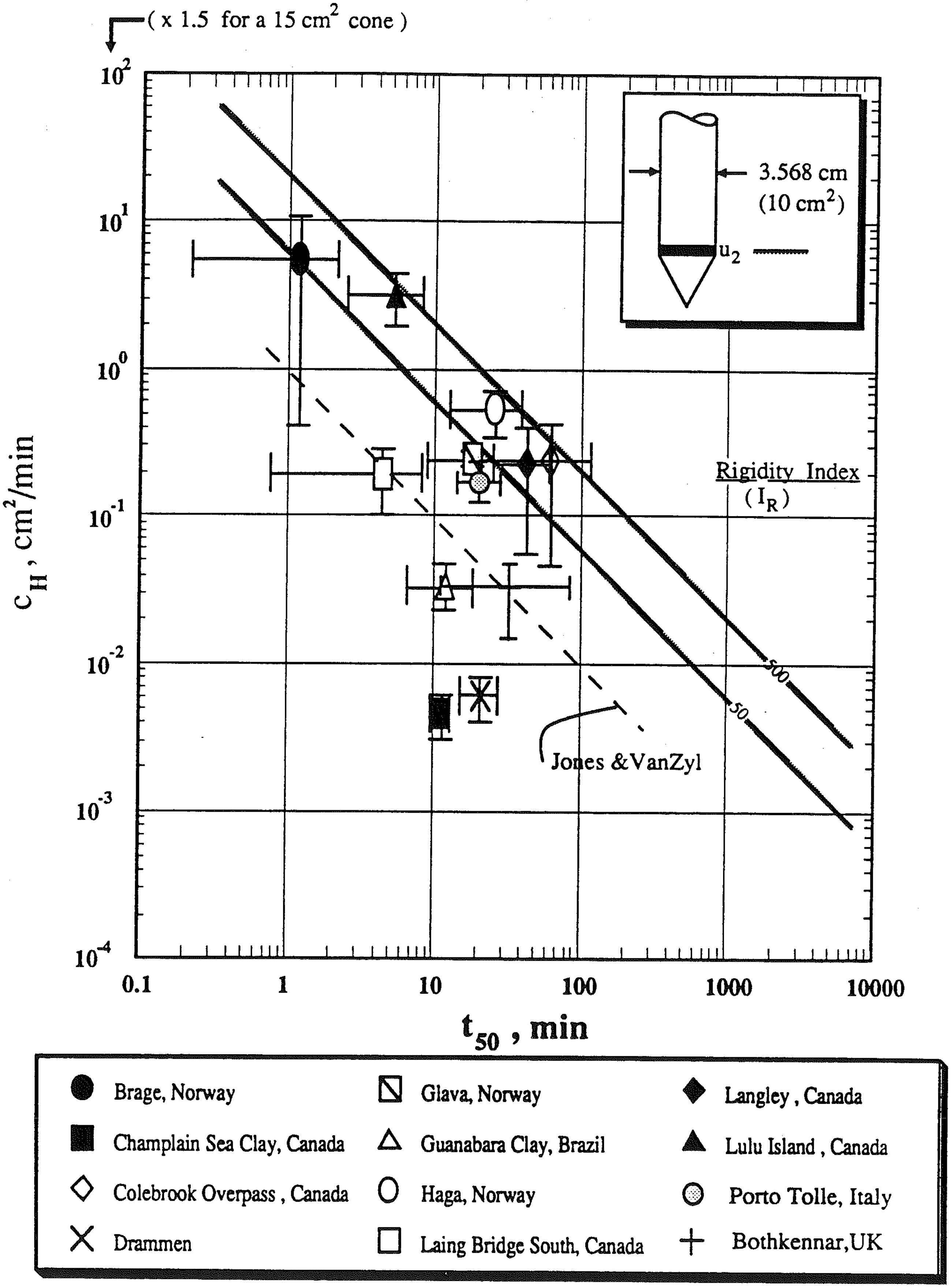


Figure 7a CPTU results in terms of t_{50} and laboratory oedometer results (u2)

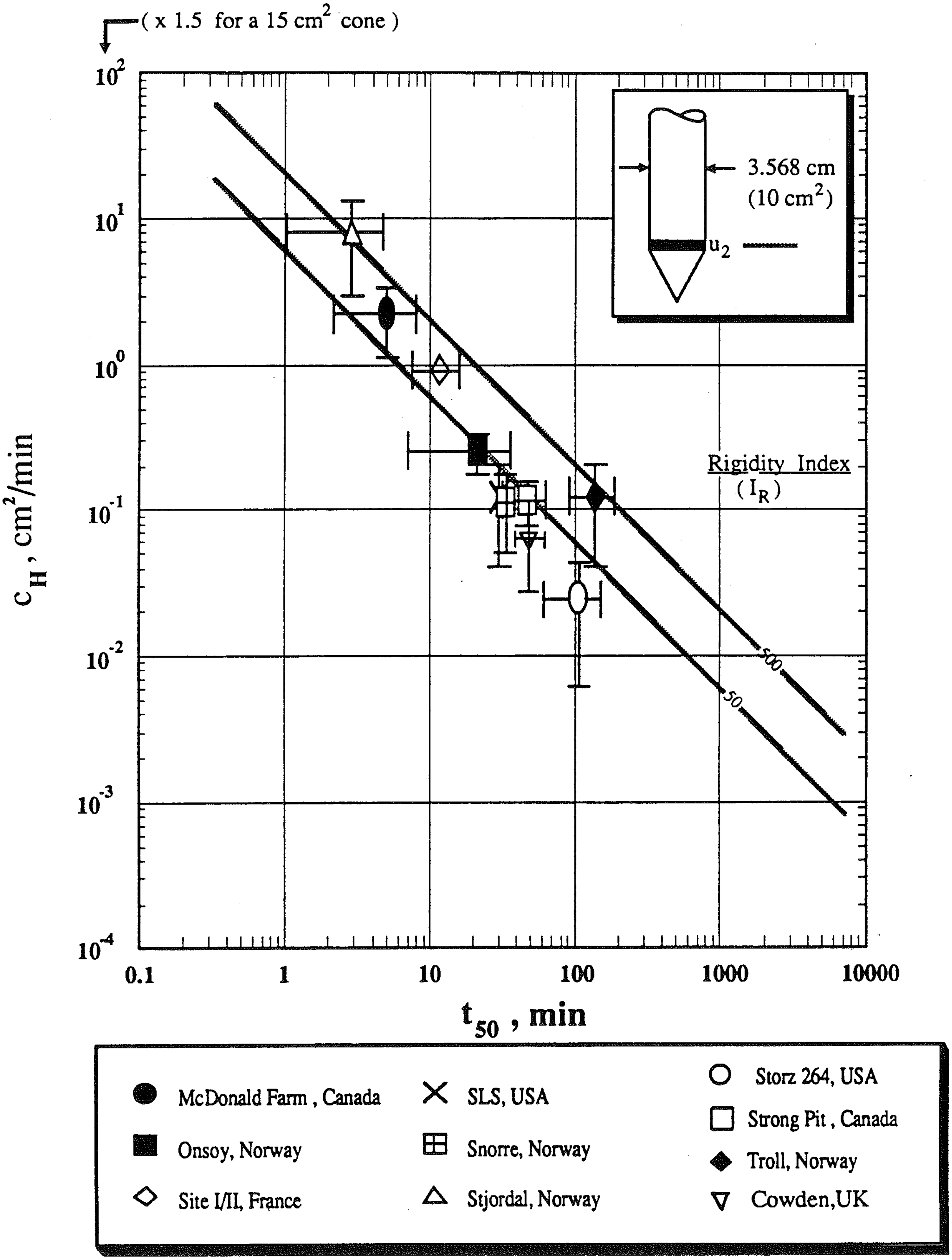
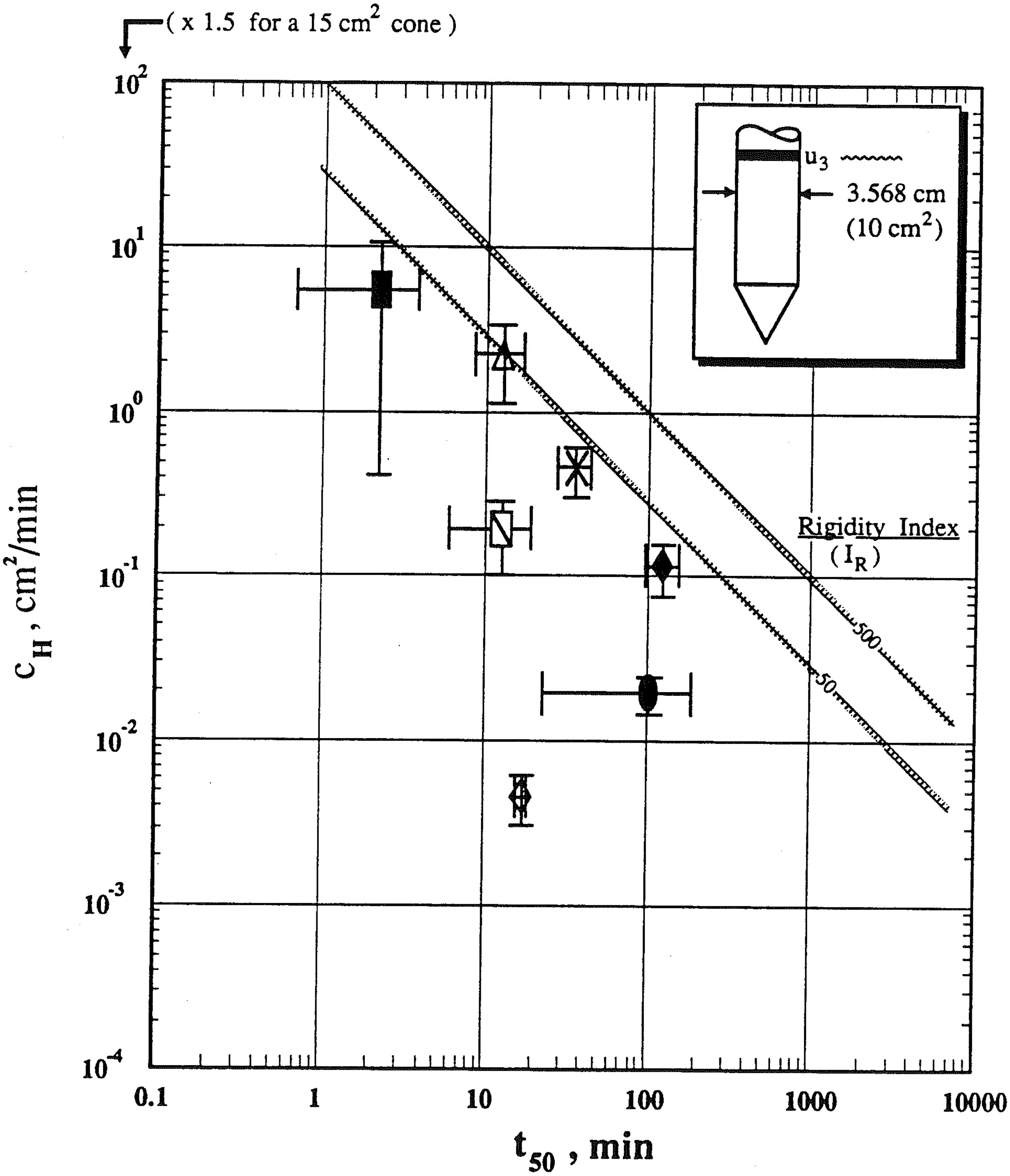


Figure 7b CPTU results in terms of t_{50} and laboratory oedometer results (u2)



● Attakapas Landing	✕ Guanabara Clay, Brazil	◆ Strong Pit, Canada
■ Brage, Norway	⧅ Laing Bridge South, Canada	
◇ Champlain Sea Clay, Canada	△ McDonald Farm, Canada	

Figure 8 CPTU results in terms of t_{50} and laboratory oedometer results (u3)