

THE UNIVERSITY OF BRITISH COLUMBIA



**CONSOLIDATION CHARACTERISTICS FROM
PORE PRESSURE DISSIPATION AFTER
PIEZOMETER CONE PENETRATION**

by

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TABLE OF CONTENTS

PAGE

1. Introduction	1
2. Dynamic Pore Pressures	1
3. Distribution of Excess Pore Pressure	2
4. Consolidation Process After Penetration	3
5. Influence of Piezometer Saturation	5
6. Solutions Available for Obtaining Consolidation Characteristics	6
7. Data Reduction	8
8. Comparison of Existing Solutions	9
8.1 Fit of Existing Solutions	9
8.2 Comparison between Predicted and Laboratory Measured Values	10
9. Conclusions	12
Appendix A: Recommended Procedures for Obtaining Consolidation Characteristics from Pore Pressure Dissipation	14
Appendix B: Values of r^2T for Various Degrees of Dissipation	16
References	17

1. Dynamic Pore Pressure and Bearing Logs	
a) Richmond Site	
b) Burnaby Site	1a
2. Piezometer, Friction Bearing Cone C6FPS-2UBC	2a
3. Excess Pore Pressures Around Installed Piles: Case Histories ..	2b
4. Excess Pore Pressure Distribution Around a Cylindrical Cavity with Time	4
5. Influence of Porous Stone Location on Pore Pressure Dissipation	5a
6. Influence of Saturation on Pore Pressure Response of Piezometer Cone	5b
7. Time Factors for Predicting the Coefficient of Consolidation ...	6b
8. Pore Pressure Dissipations at Richmond and Burnaby Sites	8a
9. Fit of Theoretical Solutions to Field Data: Richmond Site	9c
10. Fit of Theoretical Solutions to Field Data: Burnaby Site	9c
11. Anisotropic Permeability of Clays	15a

LIST OF TABLES

PAGE

-
1. Summary of Existing Solutions for Pore Pressure Dissipation 6a
 2. Comparison between Laboratory Data and Calculated Coefficients
of Consolidation: Richmond Site 9a
 3. Comparison between Laboratory Data and Calculated Coefficients
of Consolidation: Burnaby Site 9b

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1. INTRODUCTION

During cone penetration testing in cohesive soils, high excess positive pore pressures have been observed around the cone. The development of this positive pore pressure reduces the effective stress and has long been known to ease the installation of piles through clay soils. Dissipation of excess pore pressures has, in a few instances, been documented and compared to the increase in load carrying capacity of piles. The parameter governing the consolidation process around either a cone penetrometer or a driven pile is the coefficient of consolidation.

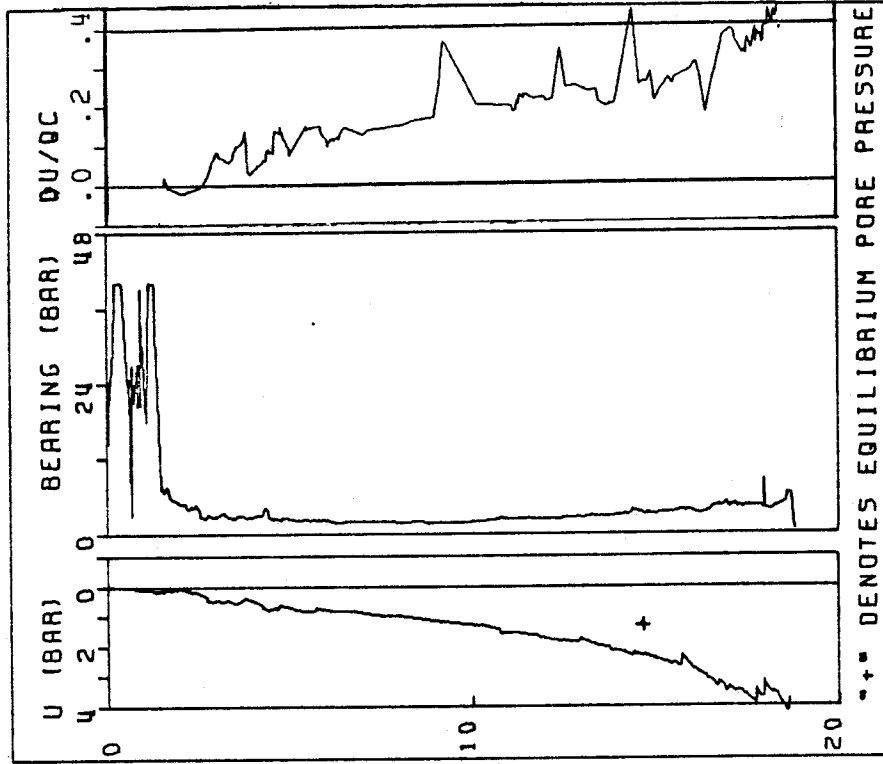
This paper illustrates the dependency of the rate of pore pressure dissipation on the coefficient of consolidation and attempts to make use of this fact by using pore pressure dissipation records from two different sites to obtain consolidation characteristics for the soils. Laboratory tests conducted on samples obtained from the sites will be used for comparison.

It is hoped that this paper will outline and test the validity of existing solutions for pore pressure decay after penetration sounding and will enhance the use of the electric piezometer cone as an instrument for site investigations.

2. DYNAMIC PORE PRESSURES

During cone penetration tests, high excess pore pressures have been observed. Figures 1a and 1b show the magnitude of these pore

Burnaby Site



Richmond Site

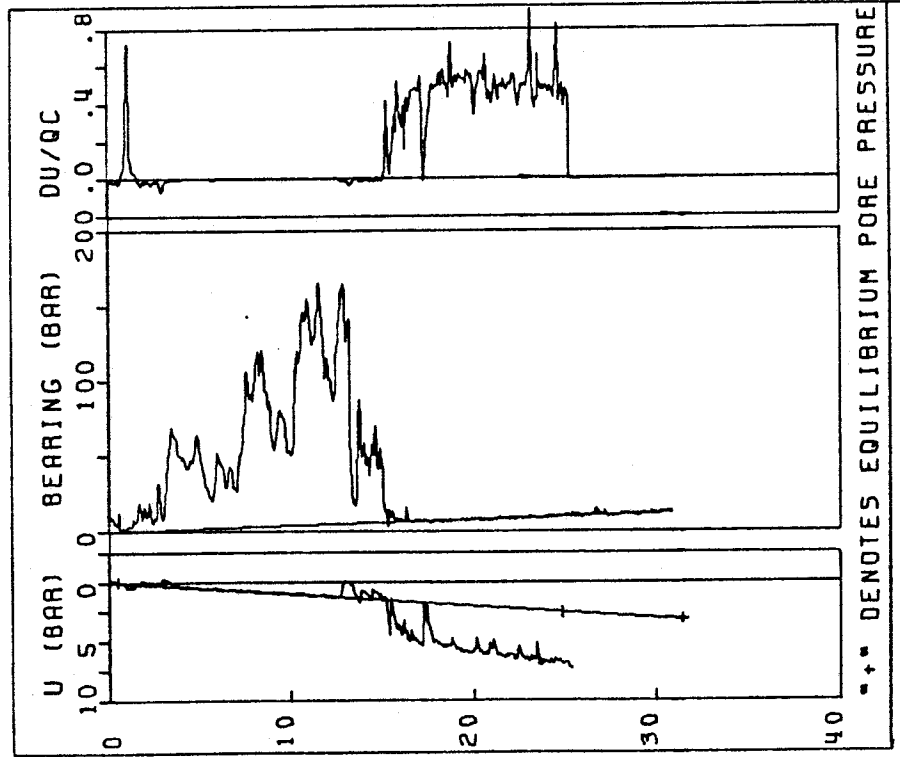


Figure 1. Dynamic Pore Pressure and Bearing Logs

pressures for two different sites. Throughout this study, excess pore pressure or dynamic pore pressure, Δu , will be used rather than the measured pore pressure, u , because of its more fundamental nature.

The electric piezometer friction bearing cone used in this investigation employs a porous plastic filter element located just behind the cone tip (see Figure 2). For the location of the porous element used, the dynamic pore pressure ratio, $\Delta u/q_c$ approaches 0.60 in the normally consolidated silts and clays tested. The excess pore pressures around the cone exceed the initial vertical effective stress by a factor of approximately 2.2. Research on the usefulness of the pore pressure ratio is ongoing and will not be discussed here.

3. DISTRIBUTION OF EXCESS PORE PRESSURE

During cone penetration sounding, pore pressures are measured at only one particular location on the cone. Considerable debate still exists over the optimum location for the porous filter (Roy *et al.*, 1979). For the purpose of consolidation analysis, however, some assumption must be made of the distribution of pore pressures around the cone, as well as at the cone surface.

Alignment problems preclude the possibility of actually measuring pore pressures around the cone. By using measurements taken during pile installation, some insight can be gained into the distribution of excess pore pressures.

A collection of case histories of excess pore pressure measurement has been compiled and shown in Figure 3. Although there is considerable

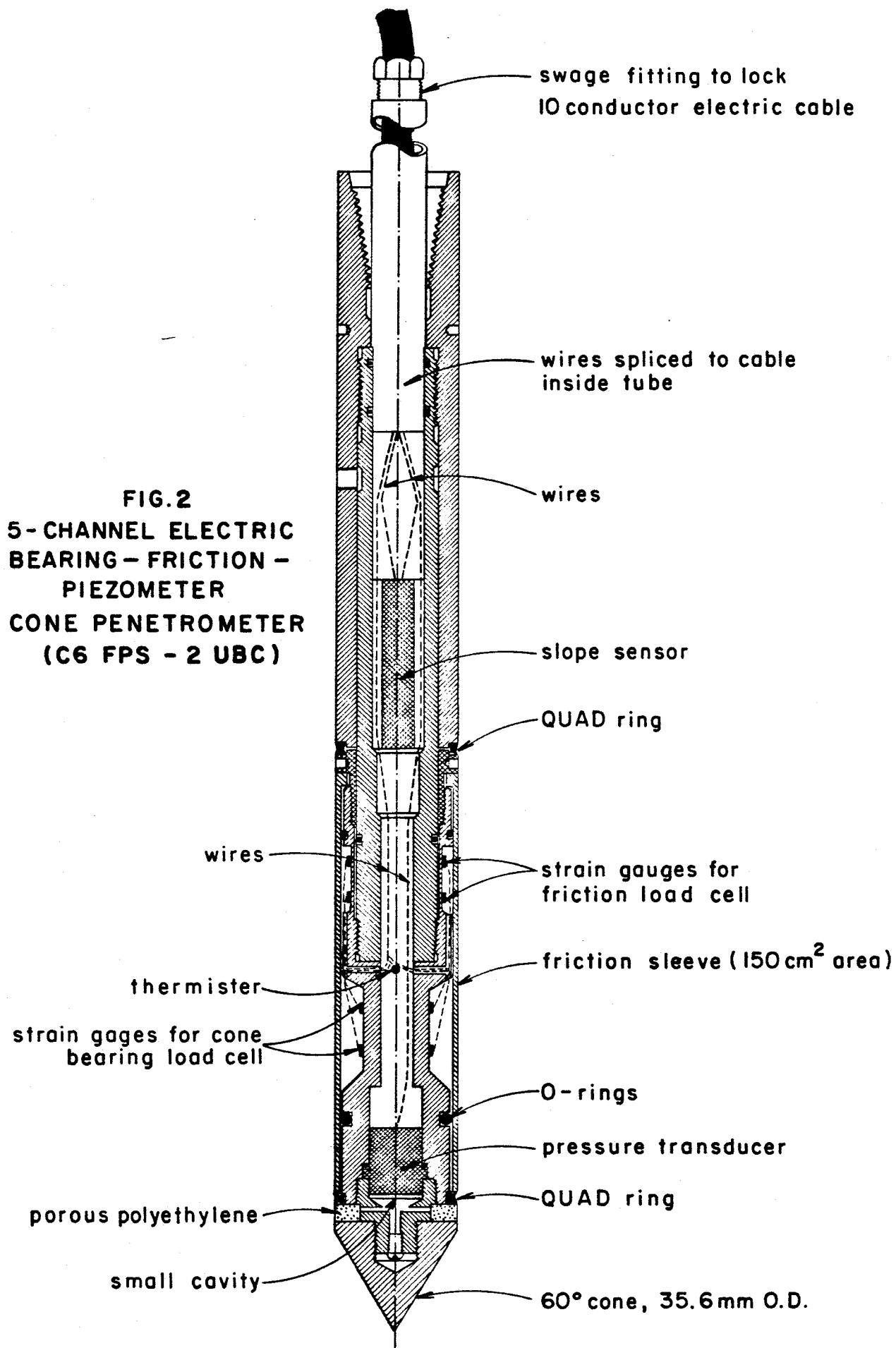


FIG. 2
5-CHANNEL ELECTRIC
BEARING - FRICTION -
PIEZOMETER
CONE PENETROMETER
(C6 FPS - 2 UBC)

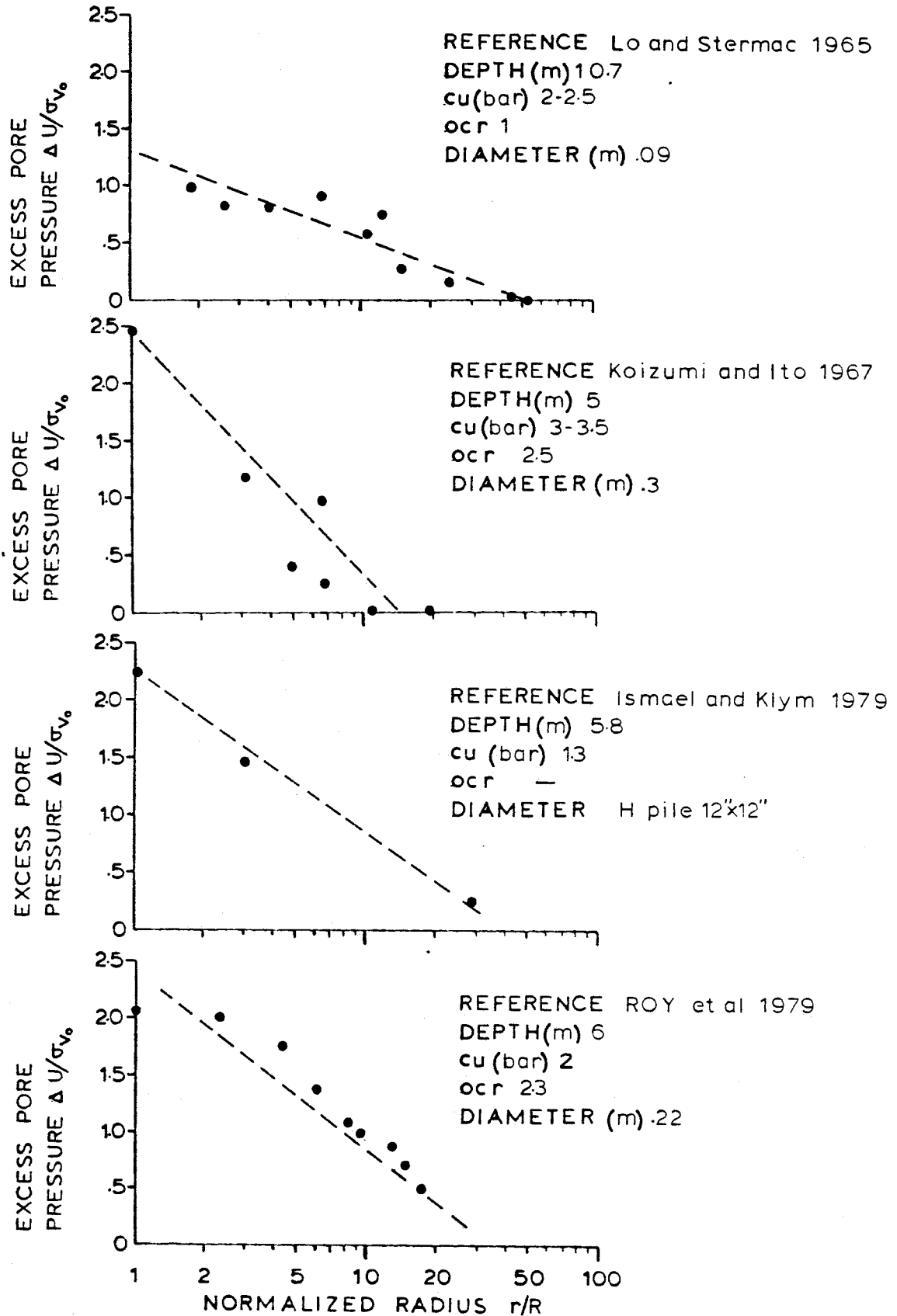


Figure 3. Excess Pore Pressure Distribution Around Installed Piles: Case Histories

scatter, the magnitude of excess pore pressure appears to decrease with the logarithm of the radius from the pile axis, as indicated by the dashed line in Figure 3.

4. CONSOLIDATION PROCESS AFTER PENETRATION

Before attempting to use the results of available theories of excess pore pressure dissipation, it is important to understand the process that takes place when penetration is stopped.

Penetration testing induces large strains on the soil in the immediate area of the cone, inducing some degree of remolding of the soil; the exact nature of the remolding depends on the soil type and location relative to the cone. Considerable research has been conducted in this area (for example, see Mitchell et al., 1978). The importance of the remolding process is that when penetration is stopped, the consolidation process takes place in a partially remolded soil.

As well as partially remolding the soil, the massive cavity expansion produced by the cone generates large pore pressures in cohesive soils. These high pore pressures reduce the effective stress. Upon arrest of penetration, these pore pressures dissipate, and the effective stress around the cone increases. This implies that the consolidation process occurs with the soil in an overconsolidated state, at least during early stages of dissipation. The changes in excess pore pressure that take place after driving are shown in Figure 4 in a nondimensionalized form using the analytical solution of Randolph and Wroth, 1979.

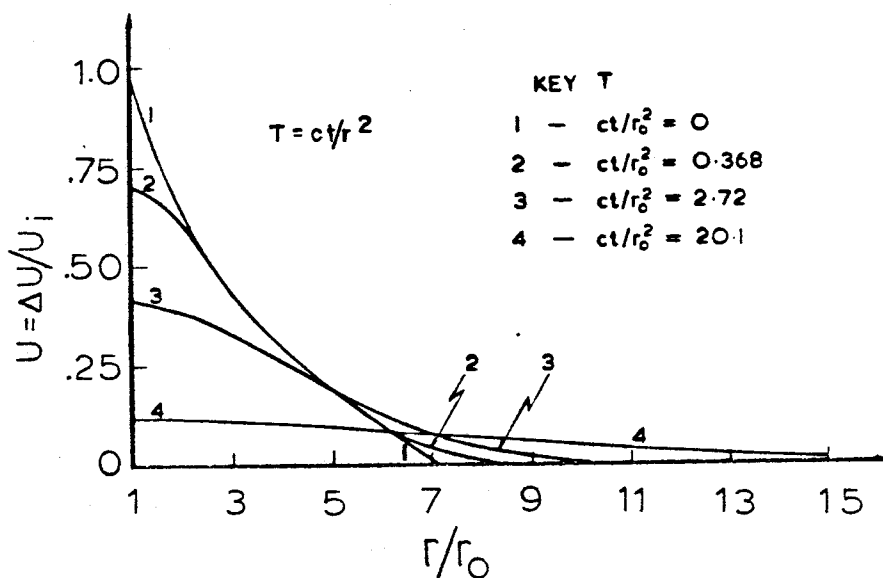


Figure 4. Excess Pore Pressure Distribution Around a Cylindrical Cavity with Time. Adapted from Randolph and Wroth, 1979.

Dissipation of the excess pore pressures must occur away from the cone, since the cone itself is a rigid, impermeable boundary. It is generally accepted that for the porous stone located some distance above the tip (say 10 X radius), only radial drainage can take place. These are the conditions that take place for the pressuremeter and for the shaft portion of piles. With the usual location of the porous element at or near the tip, some debate exists as to the influence of possible vertical drainage. Baligh, 1980, conducted sensitivity analysis using his two-dimensional analysis and concluded that dissipation is slightly faster around a spherical cavity than a cylindrical cavity. Torstensson, 1977, offers solutions to both cylindrical and spherical dissipation, but it is not clear which analysis is most useful.

In an attempt to gain some insight into the importance of the vertical component of consolidation, a series of field tests was

conducted. By conducting dissipations at the same depth in holes 1 - 2 metres apart, but with different cone tips that effectively moved the porous filter element up the shaft from the tip, the importance of the vertical component can be shown. Figure 5 shows the dissipation of excess pore pressures for two of the tips. The dissipation rate for the standard cone is only slightly greater than for the cone with a 25 centimetre tip extension. Cones with 12.5 and 38 centimetre tip extensions displayed a similar result to the cone with the 25 centimetre tip extension. It can be concluded that, at least for this soil, horizontal drainage dominates the consolidation process, and therefore, it is the coefficient of consolidation in the horizontal direction that the record reflects. Only in the unlikely case were the vertical permeability is greater than the horizontal permeability would the vertical component become very important.

5. INFLUENCE OF PIEZOMETER SATURATION

To accurately determine the initial excess pore pressure, it is essential that the pore pressure measuring system be saturated. If the porous element or the transducer chamber is not completely saturated, the measured pore pressure may not be correct. Figure 6 shows the contrast between results obtained with a saturated and unsaturated piezometer element. The unsaturated cone records an increase in the measured pore pressure after the arrest of penetration. The actual time delay to peak excess pore pressure is a function of the system

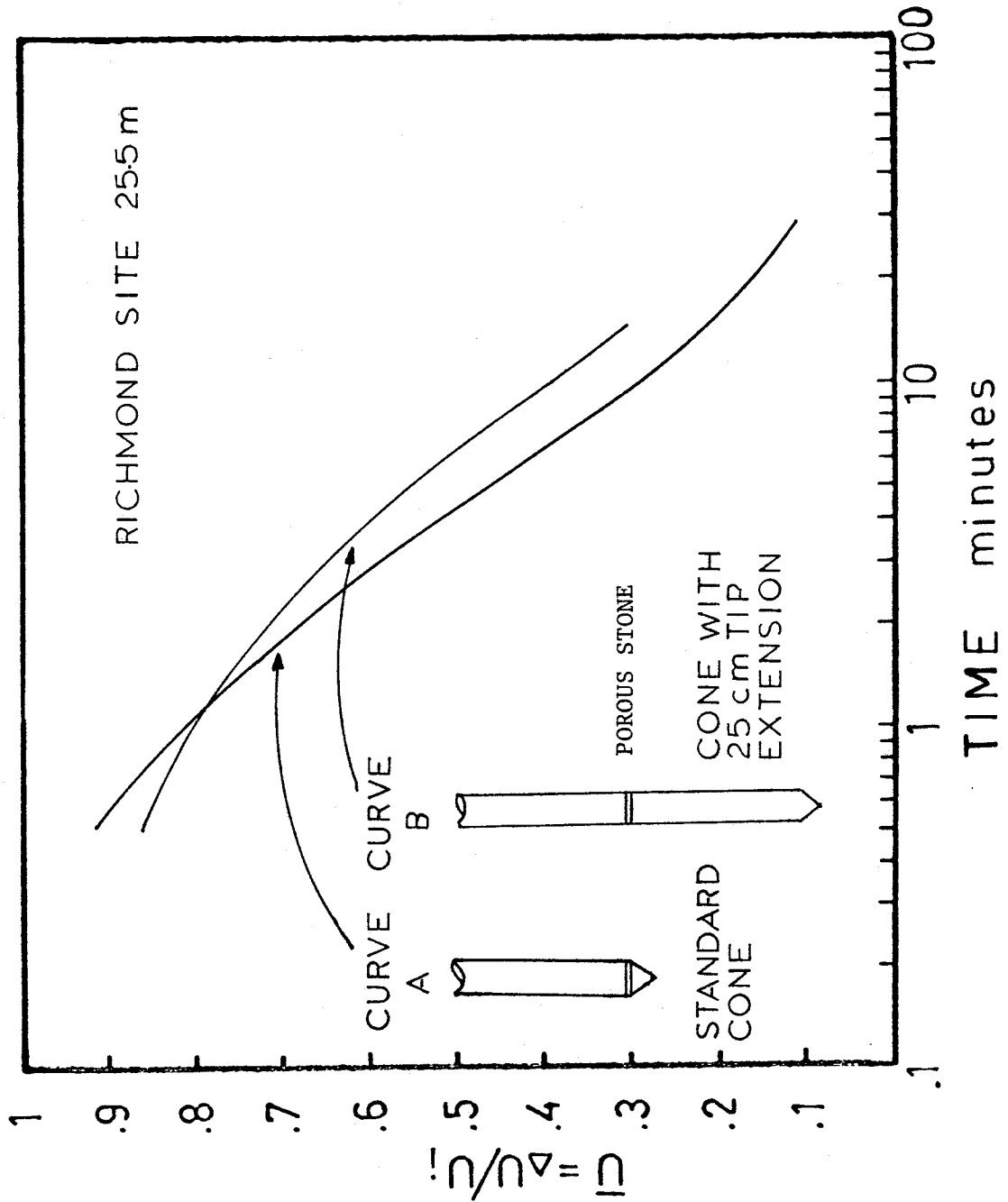


Figure 5. Influence of Porous Stone Location on Pore Pressure Dissipation

BURNABY SITE - VERY SOFT SILTY CLAY

$q_c = 4 \text{ kPa} , k = 10^{-7} \text{ cm/sec}$

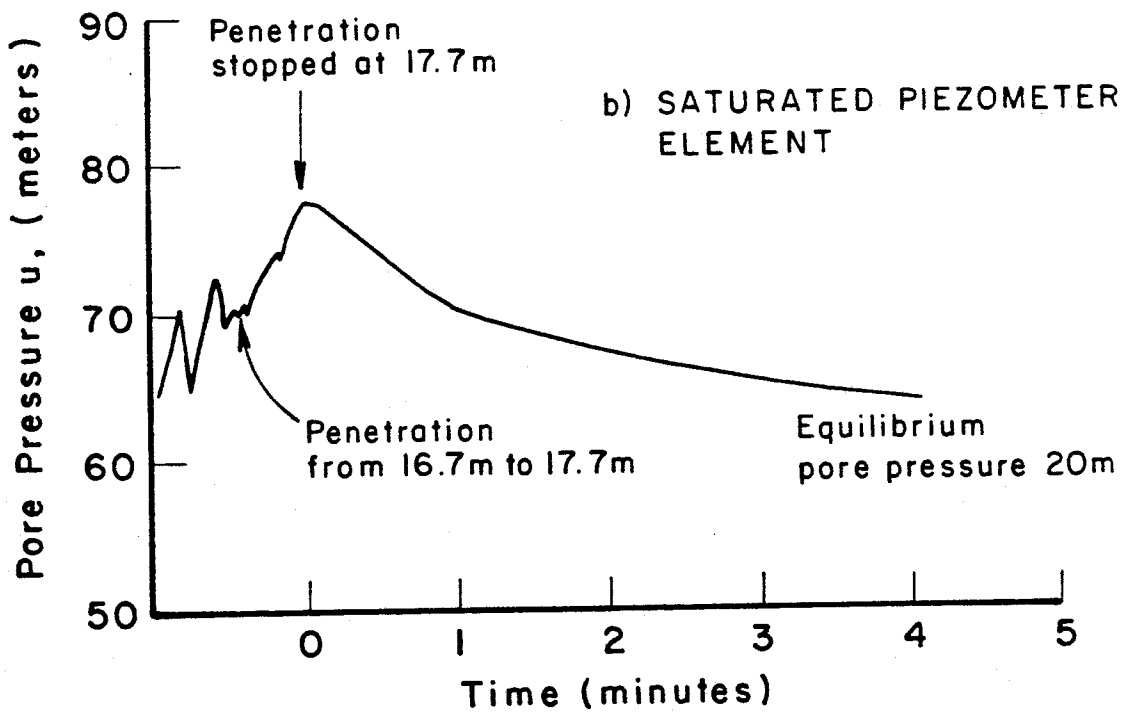
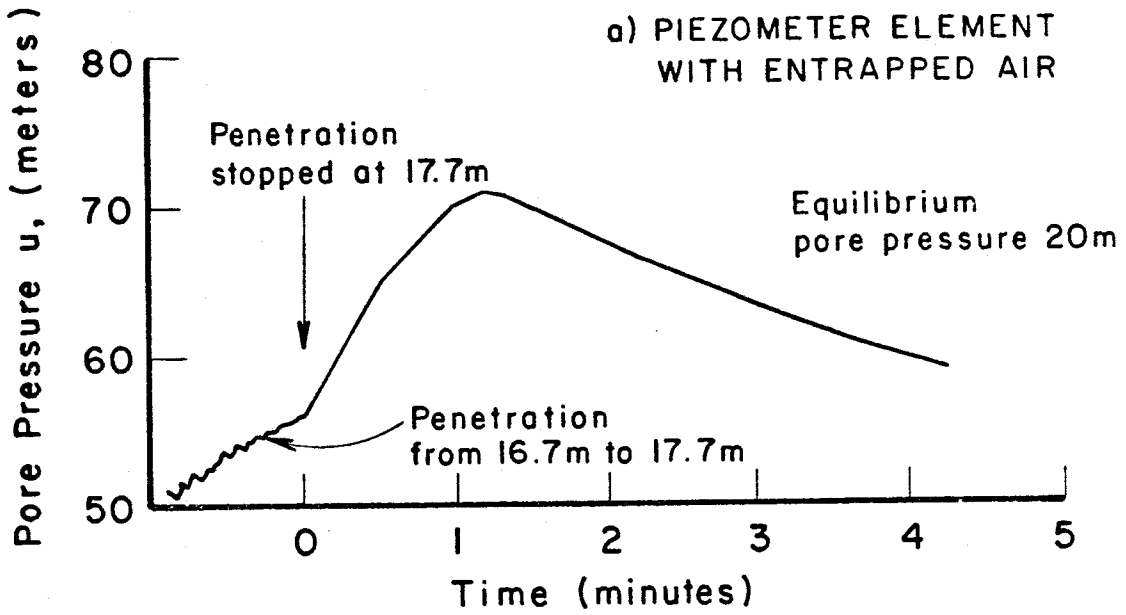


Figure 6. Influence of Saturation on Pore Pressure Response of Piezometer Cone

flexibility, as well as soil compressibility and permeability. Pore pressure dissipations of the type shown in Figure 6 for the unsaturated cone have often been reported; it was generally assumed that high shear strains adjacent to the cone resulted in positive dilatancy and a zone of low pore pressures adjacent to the cone. A redistribution of pore pressures before dissipation was thought to explain the increase in pore pressure after penetration.

During field testing, some indication of the degree of saturation can be obtained when penetration is restarted after recording a pore pressure decay. For the piezometer cone used in this study, excess pore pressures have recovered to their predissipation levels within 10 centimetres of resumed penetration.

6. SOLUTIONS AVAILABLE FOR OBTAINING CONSOLIDATION CHARACTERISTICS

Several solutions are available for analysis of dissipation records. These solutions are shown in Table 1, which highlights the major differences between the solutions. In order to compare results of the different solutions, they have all been nondimensionalized in the same manner and shown in Figure 7. Figure 7 shows the decay of excess pore pressure, \bar{u} , plotted against a nondimensional time factor, $T = ct/r^2$. Use of the time factor, T , allows a quick calculation for the coefficient of consolidation, c .

Examination of Figure 7 reveals several important points. The solutions by Baligh and Levadoux, 1980, and Randolph and Wroth, 1979, plus the cylindrical solution by Torstensson, 1977, yield essentially

Table 1. Summary of Existing Solutions for Pore Pressure Dissipations

Author	Cavity Type	Material Model	Initial Pore Pressure Distribution	Proposed Applications	Remarks
Baligh & Levadoux 1980	combined radial and spherical	non-linear Boston Blue clay	from F.E. studies using strain path method	consolidation characteristics	shows very small influence of spherical component of dissipation
Randolph & Wroth 1979	cylindrical	elastic-plastic	$\frac{\Delta u_i}{R} = \frac{2 c_u \ln\left(\frac{R}{r}\right)}{(G/c_u)^{1/2} r_0}$	consolidation around piles pressuremeter analysis	analytical solution
Soderberg 1962	cylindrical	elastic-plastic	$\frac{u_i}{u_i} = \frac{r_i}{r}$	consolidation around piles	
Torstensson 1977	cylindrical	elastic-plastic	$\frac{\Delta u_i}{R} = \frac{2 c_u \ln\left(\frac{R}{r}\right)}{(G/c_u)^{1/2} r_0}$	consolidation characteristics	proposes average of two results
Torstensson 1977	spherical	elastic-plastic	$\frac{\Delta u_i}{R} = \frac{4 c_u \ln\left(\frac{R}{r}\right)}{(G/c_u)^{1/3} r_0}$	consolidation characteristics vertical drains	

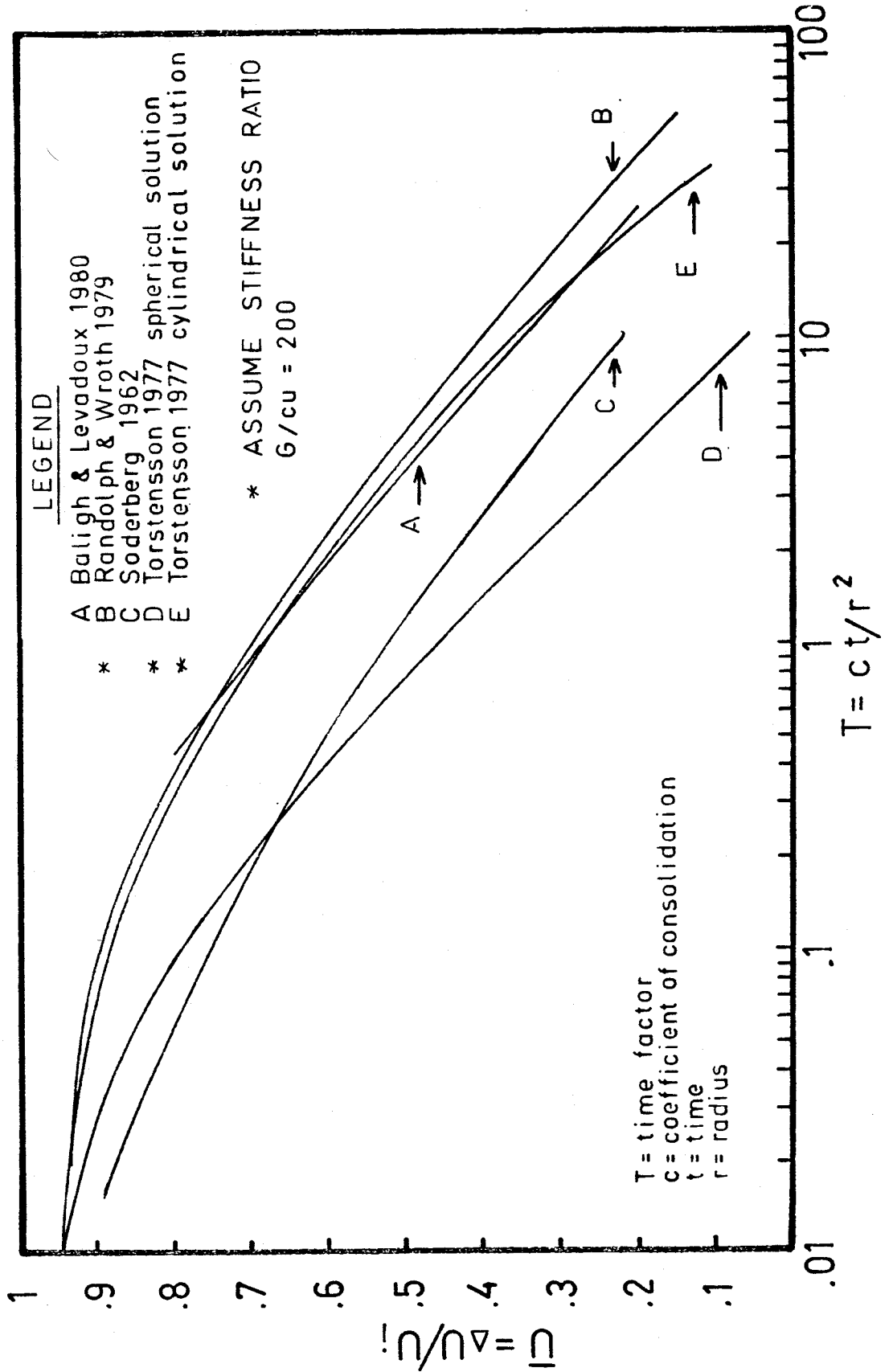


Figure 7. Time Factors for Predicting the Coefficient of Consolidation

the same result. The solution by Randolph and Wroth and the cylindrical solution by Torstensson were solved after making the same assumption regarding the initial pore pressure distribution. The consolidation analysis, however, was treated slightly differently. Randolph and Wroth used an analytical solution, whereas Torstensson used a finite difference approach.

The solution by Baligh and Levadoux was solved in quite a different manner. Although Baligh used a slight variation to the log normal distribution of excess pore pressure, he concluded that there was little difference in the predicted time factor, T . The major difference between Baligh and Levadoux's solution and the other solutions is that Baligh and Levadoux conducted a two-dimensional analysis and thus, in addition to radial drainage, included a vertical component. As previously stated, addition of this radial component does not seem to have changed the time factor, T , appreciably.

In contrast, Torstensson, 1977, shows remarkably different solutions for the spherical and cylindrical solutions. The time factor, T , at any different level of dissipation is about 5 times greater for the cylindrical solution. A partial explanation for the apparent discrepancy in the relative importance of including some component of vertical drainage arises because, as well as using a different geometric pattern for dissipation, Torstensson determines the initial excess pore pressure distribution in a different manner. For the spherical case, he uses a spherical cavity expansion, whereas for the cylindrical case, he uses a cylindrical expansion. The two cases result in a much

different distribution of initial excess pore pressures. The cylindrical solution results in a much larger zone of increased pore pressures (see equations in Table 1).

Soderberg, 1962, uses a distribution of excess pore pressure that decays with the inverse of the radius.

The solutions by Randolph and Wroth and by Torstensson require an estimate of the soil stiffness ratio. The reason for this is that a stiff soil will extend a zone of influence much larger than a soft soil.

7. DATA REDUCTION

Field dissipation records of pore pressure dissipation, recorded as a function of time, need to be corrected by subtracting the ambient static pore pressure from the measured excess values. The excess pore pressures are then nondimensionalized by dividing by the initial excess pore pressure, then plotted against time. Data for the two sites investigated in this report are shown in Figure 8. In this manner, the coefficient of consolidation can be calculated at any dissipation level by using the measured time and theoretical value of the time factor, T . Thus, for any of the five existing solutions:

$$c = \frac{Tr^2}{t}$$

where t = time

T = time factor

r = radius of probe

c = coefficient of consolidation

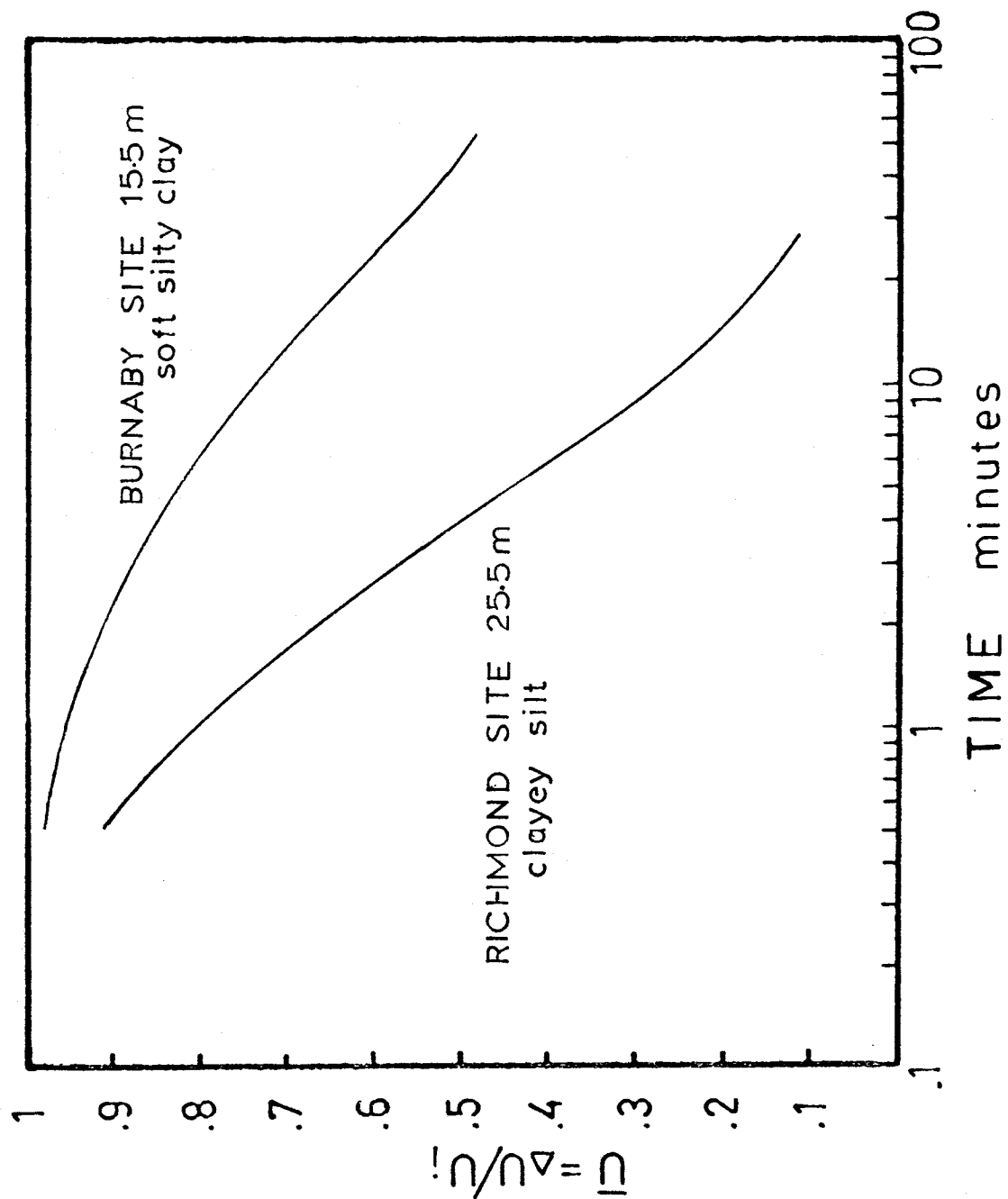


Figure 8. Pore Pressure Dissipations at Richmond and Burnaby Sites

8. COMPARISON OF EXISTING SOLUTIONS

For any theoretical solution of pore pressure dissipation to be regarded as useful, it must satisfy several criteria:

- 1) be repeatable;
- 2) predict the same coefficient of consolidation at all levels of consolidation; and
- 3) compare well to reference values.

Repeated tests done at the same depth in adjacent holes confirm the repeatability of the test.

8.1 Fit of Existing Solutions

For the dissipation curves shown in Figure 8, predicted values of the coefficient of consolidation have been compared in Tables 2 and 3. To compare the fit of the different solutions to the field data, Figures 9 and 10 compare Torstensson's spherical and cylindrical solutions to the field dissipation curves. Torstensson's cylindrical solution is shown, because of its similarity to the solutions by Baligh and Levadoux and by Randolph and Wroth. To plot the dissipation curves on the same scale as the theoretical solutions, time factors, T , have been calculated using the laboratory measured values for c . The laboratory values have been selected only as a means of comparing the shape of the dissipation curves.

Figure 9 compares the analytical solutions to the data from the Richmond site. The field curves show a more rapid rate of dissipation at high levels of dissipation than either the spherical or cylindrical solution. Use of either of the solutions, therefore, is somewhat

Table 2. Comparison between Laboratory Data and Calculated Coefficients of Consolidation : Richmond Site

Site A: Macdonald Site - Richmond, B.C., clayey silt 25.5 m

Laboratory Results - CRSC tests

Undisturbed Sample: C_v - normally consolidated = .75 cm²/min.

C_v - overconsolidated = 6.21 cm²/min.

Remolded Sample: C_v - normally consolidated = .10 cm²/min.

C_v - overconsolidated = 1.20 cm²/min.

Table of Predicted Values for Coefficient of Consolidation (cm²/min.)

Percent Dissipation

Method	20	40	50	60	80
Baligh & Levadoux	1.17	2.14	2.73	3.31	5.67
Randolph & Wroth (G/cu = 200)	1.06	2.61	3.79	5.13	8.27
Randolph & Wroth (G/cu = 50)	.45	1.27	1.82	2.23	2.53
Soderberg	.16	.57	.76	1.54	2.12
Torstensson spherical (G/cu = 200)	.27	.45	.61	.72	1.06
Torstensson spherical (G/cu = 75)	.16	.27	.37	.43	.54
Torstensson cylindrical (G/cu = 200)	.93	2.27	3.03	3.59	4.88
Torstensson cylindrical (G/cu = 75)	.47	1.14	1.52	1.80	2.50

Table 3. Comparison between Laboratory Data and Calculated Coefficient of Consolidation : Burnaby Site

Site B: Burnaby Lake Site, soft silty CLAY 15.5 m

Laboratory Results - CRSC tests

C_v - normally consolidated: .084 cm²/min
 C_v - overconsolidated : .60 cm²/min.

Table of Predicted Values for Coefficient of Consolidation (cm²/min)

Method	Percent Dissipation		
	20	40	50
Baligh & Levadoux	.23	.27	.27
Randolph & Wroth (G/cu = 200)	.21	.33	.37
Randolph & Wroth (G/cu = 50)	.07	.14	.15
Soderberg	.03	.07	.07
Torstensson spherical (G/cu = 200)	.050	.060	.060
Torstensson spherical (G/cu = 75)	.030	.040	.040
Torstensson cylindrical (G/cu = 200)	.18	.29	.30
Torstensson cylindrical (G/cu = 75)	.090	.15	.15

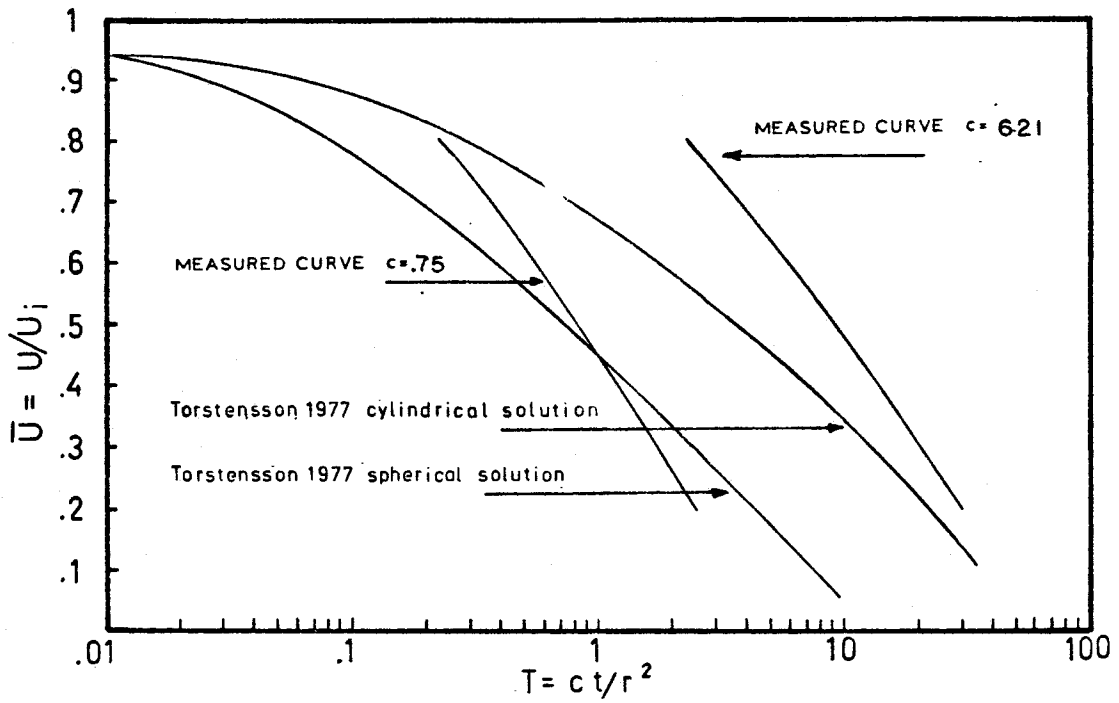


Figure 9. Fit of Theoretical Solutions to Field Data: Richmond Site

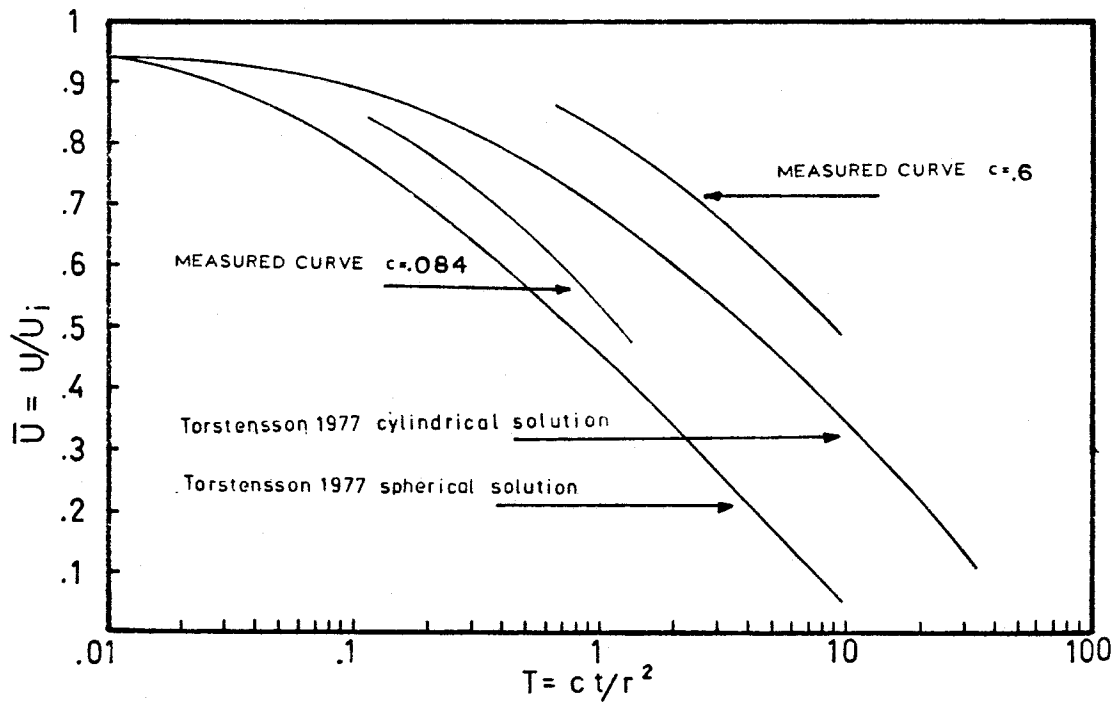


Figure 10. Fit of Theoretical Solutions to Field Data: Burnaby Site

complicated by the necessity of deciding which level of dissipation is best used to predict the actual coefficient of consolidation.

Data from the Burnaby site, a soft silty clay, plotted in Figure 10 compares well with either the spherical or cylindrical solutions. The predicted coefficient of consolidation, therefore, is nearly independent of the degree of dissipation.

Probable causes for departure of the field data from the theoretical solutions include:

- 1) an incorrect assumption of the initial excess pore pressure;
- 2) significant material nonhomogeneities near the cone; and
- 3) inappropriate consolidation analysis.

The most probable cause of the poor fit between the theoretical solutions and the field data from Richmond lies in the assumption of the initial pore pressure distribution. Considerably more confidence can be placed in the data obtained at the Burnaby site.

8.2 Comparison between Predicted and Laboratory Measured Values

Constant rate of strain consolidation tests were performed on samples obtained at the two sites. Although the undisturbed samples were used to measure the vertical coefficient of consolidation, an absence of visible layering in the samples tested indicates that the ratio of the vertical to horizontal coefficient of consolidation for the two sites should be close to .1.

A comparison between the laboratory values and the predicted coefficient of consolidation is best done at one degree of dissipation.

At the early stages of dissipation, consolidation definitely takes place along the reconsolidation curve in effective stress space. At higher levels of dissipation, some uncertainty about the total stress state confuses the state of effective stress and, therefore, which parameter c_h , normally consolidated, or c_h , overconsolidated, is most appropriate. Torstensson, 1977, recommends calculating the coefficient of consolidation at the 50% level of dissipation.

Using only the 50% level of dissipation, Tables 2 and 3 can be used to compare the different methods. For the Richmond site, Table 2, the predicted coefficient of consolidation for the cylindrical solution by Randolph and Wroth and by Torstensson, at a stiffness ratio G/c_u of 200, and the two dimensional solution by Baligh and Levadoux predict a coefficient of consolidation c_h only slightly higher than the c_v calculated in the overconsolidated range for the remolded sample.

This may reflect the partial remolding around the cone. The predicted value from Torstensson's spherical solution compares well with the c_v from the normally consolidated portion of the undisturbed sample. This result is not entirely expected, considering the overconsolidated partially remolded state of the soil around the cone.

Results from the Burnaby site show a similar trend to those of the Richmond site: the cylindrical solution at the lower stiffness ratio and the solution by Baligh and Levadoux predict a coefficient of consolidation about one half of the c_v measured in the overconsolidated state of the undisturbed sample. Again, considering the effects of partial remolding, this value is reasonable. Torstensson's spherical

solution underpredicts even the c_v for the normally consolidated sample. As in the case with the Richmond sample, the solution by Soderberg should probably be discounted as being a poor indicator.

9. CONCLUSIONS

The two cylindrical solutions for pore pressure dissipation around the cone by Randolph and Wroth and by Torstensson, and the two-dimensional solution by Baligh and Levadoux, provide a reasonable prediction of the horizontal coefficient of consolidation in the overconsolidated state.

Torstensson's spherical solution underpredicts the coefficient of consolidation; this is probably due to the assumption of the initial pore pressure distribution arrived at during spherical cavity expansion.

The solutions by Randolph and Wroth and by Torstensson that require a stiffness ratio are not sensitive to the stiffness ratio chosen. For a four-fold increase in stiffness ratio, the predicted coefficient of consolidation changes by a factor of about 2.

The solution by Soderberg should be discounted, as it showed the poorest fit to the field data.

Provided that equilibrium pore pressures are not required, it is necessary, for the purpose of obtaining consolidation characteristics, to wait past the 50% level of dissipation.

The interpretation of results requires some judgement in the event that different coefficients of consolidation are predicted at

different stages of dissipation. Considering the variability of laboratory results and soil nonhomogeneity, this difficulty may be of small consequence.

An outline of the solution technique, a table of time factors, and some recommendation of possible methods of obtaining soil permeability can be obtained from Appendices A and B.

APPENDIX A

Recommended procedures for obtaining consolidation characteristics from pore pressure dissipation:

- 1) Calculate the dissipation of excess pore pressures by subtracting the known or estimated static value from the measured values and divide by the initial excess pore pressure.
- 2) Plot the decay of excess pore pressures with time, and select the time to the 20, 40, and 50% levels of dissipation.
- 3) Using the tabulated values of r^2T given in Appendix B, calculate:

$$c_h \text{ probe} = \frac{r^2T}{t}$$

Appendix B lists the solutions by Baligh and Levadoux and by Torstensson only because of the similarity of the solution by Randolph and Wroth to the solution by Torstensson.

The spherical solution was found in this report to underpredict the expected value of c_h , but values of r^2T are given for the sake of completeness.

The appropriate stiffness ratio can be selected from:

G/cu = 50 - soft clay

G/cu = 400 - stiff clay

(after Randolph and Wroth, 1979).

- 4) Check the validity of the solution method by comparing the predicted coefficient of consolidation at the different levels of dissipation.

- 5) Since it can be safely assumed that the soil compressibility is isotropic, then:

$$c_{v_{oc}} = c_{h \text{ probe}} \times \frac{k_v}{k_h}$$

An estimate of the ratio k_v/k_h can be obtained from Figure 11, on the following page, after Baligh and Levadoux, 1980.

Soil remolding around the cone caused the predicted value of $c_{h \text{ probe}}$ to underpredict the expected c_h overconsolidated by a factor of about 2. This factor would depend on many factors such as soil sensitivity and structure.

- 6) An estimate of soil permeability can be obtained from:

$$k_h = c_h m v \gamma_w$$

$$k_v = c_v m v \gamma_w$$

where, in the absence of any other data, mv can be predicted from:

$$mv = \frac{1}{E} = \alpha q_c \quad \text{where } \alpha = 7 \text{ for clay}$$

(after Mitchell, 1978).

Anisotropic Permeability of Clays

<u>Nature of Clay</u>	<u>k_h/k_v</u>
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

Fig. II Anisotropic Permeability of Clays
after: Baligh and Levadoux 1980

APPENDIX B

Values of r^2T for various degrees of dissipation:

DEGREE OF DISSIPATION	20%	40%	50%	60%	80%
BALIGH SOLUTION	1.39	5.99	11.46	20.5	85.1
TORSTENSSON SPHERICAL DISTRIBUTION					
E/Su = 500	0.34	1.46	2.58	3.99	10.4
E/Su = 400	0.32	1.27	2.16	3.57	9.04
E/Su = 300	0.27	1.11	1.92	3.12	7.48
E/Su = 200	0.21	0.88	1.50	2.45	6.05
E/Su = 100	0.18	0.62	1.02	1.58	3.66
TORSTENSSON CYLINDRICAL DISTRIBUTION					
E/Su = 500	1.07	6.78	13.59	26.4	74.8
E/Su = 400	0.94	5.55	11.30	21.5	64.0
E/Su = 300	0.77	4.36	8.91	17.03	51.6
E/Su = 200	0.57	3.37	7.34	12.10	32.1
E/Su = 100	0.43	2.64	4.33	7.89	19.1

Note: T is the Time Factor defined as $T = Cht/R^2$
 $R = 1.78$ cm for the U.B.C. Piezometer Cone.

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EVALUATION OF FIELD CPTU DISSIPATION DATA IN OVERCONSOLIDATED FINE-GRAINED SOILS

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SYNOPSIS: In stiff overconsolidated clays, the interpretation of piezocone dissipation data is complicated by the large pore pressure gradients that exist. These large pore pressure gradients around the tip of the cone modify the response such that, at locations behind the tip, the measured values may initially increase rather than dissipate. The form of the dissipation curve is thus non-standard and cannot be interpreted according to existing theories, developed for soft normally consolidated soils. However, it is possible to correct the non-standard curves, in a theoretically acceptable manner, in order to produce dissipation curves whose shape is in agreement with the existing interpretation methods and assumptions. Four generic types of dissipation curve are identified, three of which are specifically for stiff overconsolidated soils. The correction procedures for interpretation of dissipation results in OC soils are presented and applied to data from three stiff clay sites. The results are compared with the coefficient of consolidation obtained from laboratory oedometer tests.

BACKGROUND

Cone penetration testing with pore pressure measurement (CPTU or piezocone testing) is becoming a regular investigation technique in geotechnical site investigation practise. The penetration of the piezocone can be halted at any instant and the variation with time of the measured parameters can be monitored. It is usually the time variation of the pore pressure that is of interest as the results can be interpreted to provide estimates of the *in situ* horizontal coefficient of consolidation, c_h (Torstensson, 1977). Rather than the total pore pressure, it is the change in the excess pore pressure (δu) with time that is required for the evaluation of c_h , where δu is defined as:

$$\delta u = u_i - u_0 \quad (1)$$

u_i is the measured pore pressure at the depth of interest and u_0 is the equilibrium *in situ* pore pressure at the depth of the porous element on the piezocone.

Interpretation of dissipation records is generally based on a normalized excess pore pressure ratio, U , defined as:

$$U = \delta u(t)/\delta u_i = (u(t) - u_0)/(u_i - u_0) \quad (2)$$

where $\delta u(t)$ is the excess pore pressure at any time t after penetration is stopped, δu_i is the initial excess pore pressure at $t=0$, i.e. immediately on stopping penetration and $u(t)$ is the total pore pressure at any time t .

Three specific filter locations will be discussed in this paper and the excess pore pressure will be subscripted according to where the measurements are obtained (Fig. 1a):

$$\delta u_{1,2,3} = u_{1,2,3} - u_0 \quad (3)$$

(Here it is assumed that since all the porous filter locations are very close, the same value of u_0 can be used to calculate δu .)

It is usual practise to continue taking dissipation measurements until at least half the initial excess pore pressure has dissipated ($U = 0.5$). Interpretation of the dissipation results can be achieved using either cavity expansion theory or the strain path approach. Comparisons of the available solutions and results from field studies suggest that the methods of Torstensson (1977), Levadoux (1980) and Teh (1987) all provide similar predictions of consolidation parameters from CPTU

dissipation data in normally consolidated soils. The relevance of any of the above solutions depends on many factors, the most important of which relates to how well the initial pore pressure distribution around the cone compares with the theoretical idealization employed by each of the models. In overconsolidated soils, large pore pressure gradients exist around the cone (Robertson et al., 1986; Sully et al., 1988) which may give rise to non-standard type dissipation curves. A typical set of excess pore pressure dissipations (Type I) in soft normally consolidated clay for the three above-mentioned filter locations are presented in Fig. 1(a). The corresponding normalized dissipation curves are shown in Fig. 1(b). All three pore pressure dissipation curves show a monotonic decrease in the excess pore pressure with time and agree with the theoretical dissipation curve. Under these conditions, the data can be interpreted according to any of the available theories to estimate the *in situ* consolidation parameter, c_h , which primarily governs the rate of dissipation for CPTU tests (Baligh and Levadoux 1980).

However, in overconsolidated soils with filters located at the u_2 and u_3 positions, the pore pressures on stopping penetration do not decrease immediately, rather they show an initial increase over a definite period of time before finally beginning to dissipate. At the u_1 filter location, the dissipation record may also be modified due to the unloading that occurs when penetration is stopped. The interpretation of these dissipation records to obtain predictions of c_h in overconsolidated soils are the subject of this paper.

DISSIPATION IN OVERCONSOLIDATED SOIL

A typical example of pore pressure dissipation at the three filter locations in a lightly overconsolidated fine grained soil ($OCR=4$) is illustrated in Fig. 2 using results obtained at the Strong Pit site. Similar types of dissipation record (Type II) in OC soils for locations behind the tip have been reported by Lutenecker and Kabir (1987), Coop (1987) and Lunne et al. (1986). Assuming that the filter system has been saturated correctly, the initial rise in pore pressures measured behind the tip can be attributed to the redistribution that occurs around the tip due to the large gradients that are generated in OC soils.

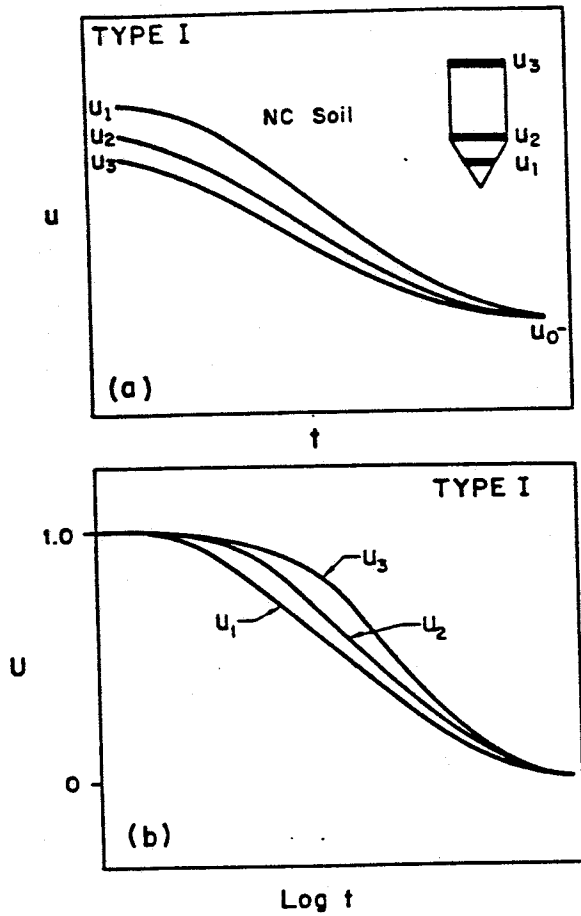


Fig. 1 Typical dissipation data for NC soil

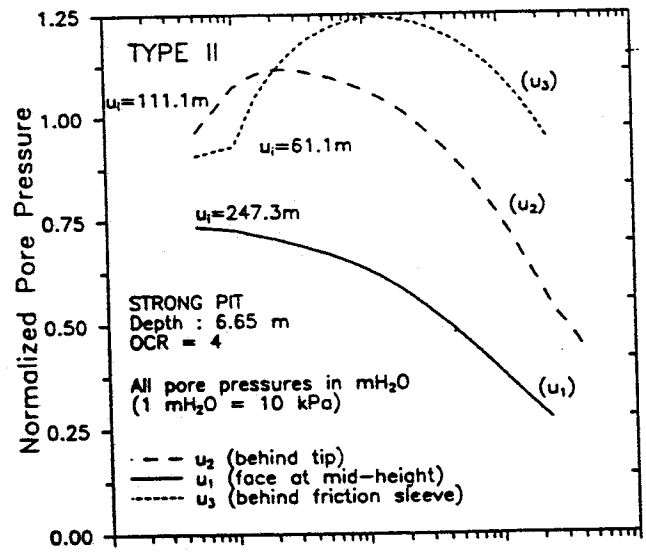


Fig. 2 Type II dissipation curves in OC soil

In moderately to heavily OC soils the pore pressures measured at locations behind the tip may be negative of hydrostatic or even below zero. In this case, on halting penetration the pore pressure increases to finally arrive at the *in situ* equilibrium value. Two types of dissipation curve may result depending on the soil characteristics:

- the measured pore pressure may increase over and above the

in situ equilibrium value if the rate of pore pressure redistribution is higher than the rate of dissipation. After reaching some peak value, the pore pressure then decreases until the equilibrium value is reached (Fig. 3, Type III).

-if the rate of dissipation is faster than the rate of redistribution, the pore pressure dissipation does not overshoot but directly arrives at the equilibrium value (Fig. 3, Type IV). Types II and III pore pressure response can be evaluated using the correction technique outlined below. Type IV can be considered as inverted dissipation and treated in the standard way (as dissipation of a negative excess).

Correction to Non-Standard Dissipation Curves for Initial Distribution Effects

Two alternative data manipulation approaches have been used to correct the above-type dissipation curves so that the available dissipation theories can be used to estimate values of the coefficient of consolidation. One approach is based on a log-time plot while the other is based on a square-root time representation.

Log-time plot correction

The data in Fig. 2, obtained in a lightly overconsolidated clay at Strong Pit, have to be corrected according to the location of the pore pressure measurement, i.e. either on the cone tip or behind the cone tip, since the redistribution that occurs affects the two sets of pore pressures in different ways.

On the tip, a sudden decrease in pore pressure occurs on halting penetration. In Fig. 2 the normalized pore pressure (as defined in Eq. (2)) 5 seconds after dissipation begins is already reduced by 25% due to the reduction in the bearing stress acting on the face of the cone. For this location, the initial maximum pore pressure used for normalizing the dissipation record is taken as the peak value once the initial unload has occurred, i.e. for this case the maximum value corresponds to the 5 sec. measurement and this time is taken as the zero time point (5 sec. are subtracted from the time register throughout the record).

For the behind tip locations, the maximum pore pressure is taken as the peak value that occurs during the post-penetration increase and the time at which this peak occurs is taken as the zero time of the dissipation record and all other times adjusted accordingly.

The data from Fig. 2, corrected in this way are replotted in Fig. 4 to show the new form of the normalized dissipation plot, adjusted to account for unloading and redistribution effects. Figure 4 does highlight the problems associated with the execution and interpretation of dissipation data in fine-grained soils for filter locations behind the tip. The time for 50% dissipation increases from 1100 seconds to approximately 10000 seconds as the filter moves from u_1 to u_3 .

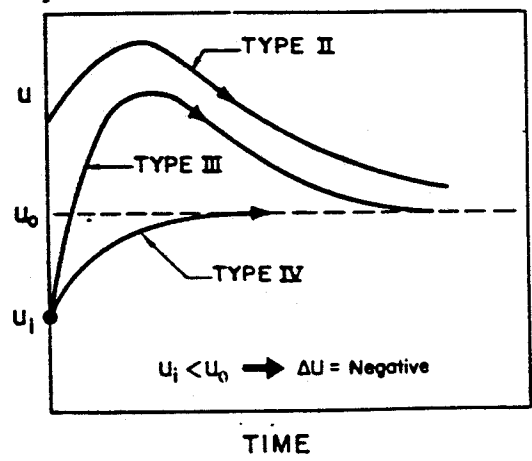


Fig. 3 Type III and IV dissipation curves in OC soil

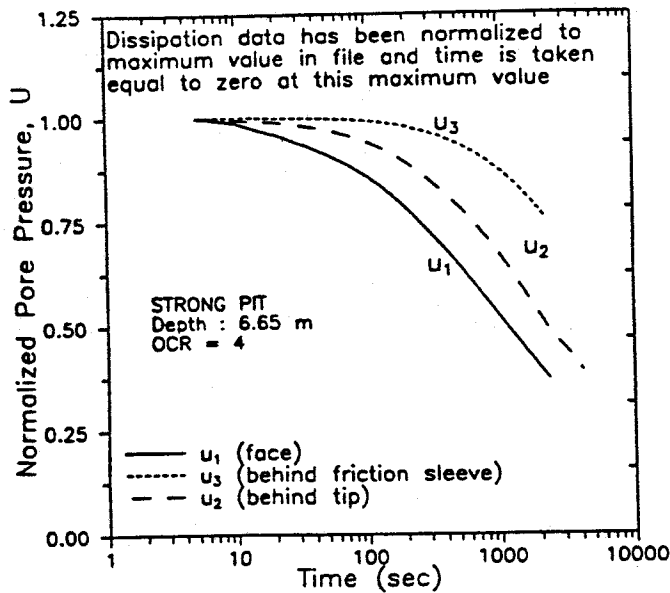


Fig. 4 Log-time corrected Type II data from Strong Pit

Root-time plot

In a similar fashion to the Taylor approach for interpreting oedometer data, it is also possible to adjust the dissipation data of Fig. 2 using a back-extrapolation technique on a square-root time plot. In the root-time plot, the dissipation that occurs after the initial peak caused by redistribution of pore pressure, depicts a straight line which can be back-extrapolated to $t=0$ in order to obtain a u_i for the corrected dissipation curve. This back-extrapolated value is then used to produce the normalized dissipation curve. The basis of the above correction technique is illustrated in Fig. 5 for measurement locations behind the tip. The principle is the same for measurement locations on the tip except that instead of an increasing pore pressure, the initial pore pressure suddenly drops as discussed previously.

The additional advantage of the root-time method is that the initial straight line portion can be extrapolated to 50% pore pressure reduction if short dissipation periods are used in the field and measured data to longer periods are not available (Fig. 6).

Alternately, the initial linear slope in the normalized pore pressure root time plot can be analysed to provide estimates of c_h using the theoretical approach suggested by Teh (1987). These extrapolations however require further verification prior to general use.

The two correction methods (log time and root time) described above will give rise to slightly different normalized dissipation curves since the initial corrected u_i values are, by definition, not the same. The resulting corrected dissipation data are compared in Fig. 7 (the root time plot has been reproduced in log time space for comparison purposes). While the dissipation curves for U less than 25% for both corrections may be different (as would be expected from the different u_i values) at $U=50\%$ the error is relatively small. In essence, the two correction techniques give the same values for t_{50} . The curves in Fig. 7 do, however, indicate the importance of the initial pore pressure value at $t=0$ on the normalized form of the dissipation curve.

EVALUATION OF PROCEDURE IN OC SOILS

Basis of Comparison

The log time correction procedure has been applied to CPTU dissipation data from three University of British Columbia research sites where overconsolidated soils are present in the profile. The results of comparisons between the log t correction and laboratory derived consolidation parameters are considered below. The root time method is not presented here since the difference between the

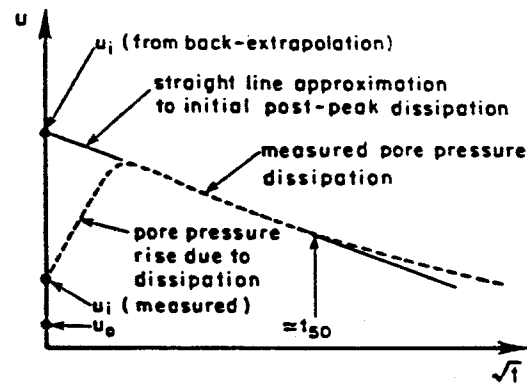


Fig. 5 Root-time correction procedure for Type II data

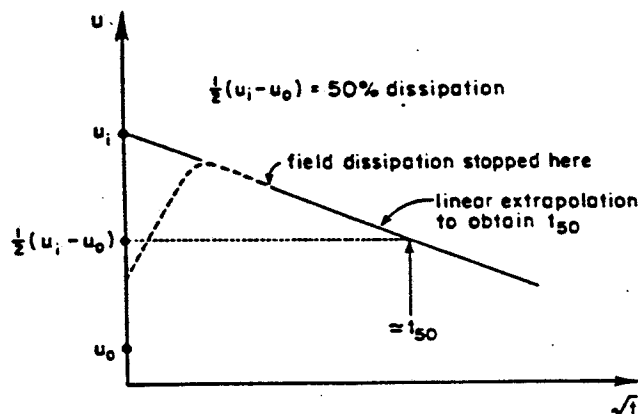


Fig. 6 Extrapolation of data in root-time plot to obtain t_{50}

resulting times from the two methods is only significant for short dissipation periods. All the results presented here are for 50% dissipation of the excess pore pressure.

The horizontal coefficient of consolidation can be evaluated from the corrected CPTU data using any of the available theories. For this study, the method proposed by Teh (1987) has been used. The advantage of this method is that it considers the effect of the rigidity index on the pore pressure dissipation and can be applied to any filter location. The c_h value is determined from:

$$c_h = (T^* R^2 I_R^{0.5}) / t_{50} \quad (4)$$

where T^* is the Teh and Hously (1988) modified time factor, R is the cone radius at the measurement location and I_R is the rigidity index of the soil. Results presented by Danziger (1990) suggest that the Hously and Teh (1988) approach provides more consistent c_h estimates than the other available methods.

For comparison with the CPTU interpretation, consolidation coefficients from incremental laboratory oedometer tests are presented. While these laboratory determined c_v values may not be wholly representative of in situ conditions, the results do provide a basis on which the relative CPTU magnitudes can be compared. Also, at the OC clay research sites no field data from full-scale monitoring are available and so laboratory reference values form the basis for comparison.

Laboratory c_v values are determined from root time plots at each incremental loading stage, so that both OC and NC data are available. Furthermore 1-D oedometer tests have also been performed on samples cut so as to obtain estimates of c_h . Hence the c_h/c_v ratio can be evaluated from the laboratory tests in order to correct the CPTU c_h values to c_v for direct comparison with the laboratory data.

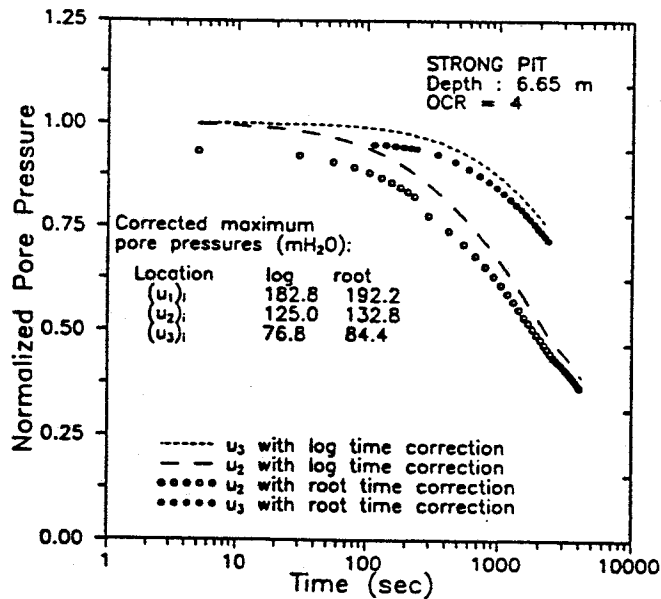


Fig. 7 Comparison of log and root-time corrected dissipation curves

Geotechnical Review of Sites Considered

The general geotechnical characteristics of the sites under consideration are presented in Table 1. The soils at Lr. 232 St. are soft sensitive clay silts. At the other two sites the clay silts are non-sensitive with undrained strengths up to 200 kPa.

Table 1 Geotechnical characteristics for sites considered.

Site	Depth range (m)	PI (%)	OCR	Range of σ'_v (kPa)	Range of σ'_{vm} (kPa)
Lr 232 St	1-5	21-30	3-10	16-40	90-205
Strong Pit	1-9	11-20	2-15	16-180	350-500
200th St	1-5	20	2-17	16-51	115-300

σ'_{vm} is the maximum past vertical pressure from oedometer tests

Results of Comparison

Only data from pore pressure locations u₁ and u₂ have been used for the purposes of comparison. The times required for 50% dissipation of the excess pore pressure measured at the u₃ location are prohibitively long and the use of piezocone dissipation tests at this location is not considered by the authors to be of practical interest unless graphical extrapolation techniques can be employed.

The coefficients of consolidation from CPTU (c_h) and laboratory oedometer (c_v) tests have been determined as described above. The obtained values are compared in Table 2.

Table 2 Comparison of field and laboratory coefficients of consolidation

Site	Measure. location	c_h from CPTU (cm ² /s)	c_v from oedom. (cm ² /s)	
			OC range	NC range
Lr 232	u ₁	0.002-0.005	0.006-0.1	0.0005-0.001
	u ₂	0.005-0.016		
Strong	u ₁	0.007-0.004	0.002-0.005	0.0006-0.001
	u ₂	0.01-0.006		
200 St	u ₁	0.014-0.047	0.05-0.18	0.001-0.03
	u ₂	0.045-0.054		

The c_h values from the two pore pressure measurement locations (u₁ and u₂) are in very good agreement with each other. Furthermore, the CPTU values would suggest that the t_{50} dissipation provides an estimate of the c_h corresponding to the OC condition, i.e. the in situ condition of the soil. Hence, it would appear that at degrees of dissipation of 50% or more, the theory provides estimates of the coefficient of consolidation relevant to the in situ stress history condition, (c_h)_{OC} at the sites studied. In addition to this, it is apparent from the types of dissipation curve considered, that stress history may be an important factor to be considered when interpreting CPTU dissipation data. The dissipation curve itself may be useful as a stress history indicator.

CONCLUSIONS

The c_h values from CPTU generally fall into the range suggested by the laboratory data. The correction method proposed here does give rise to consistent estimates of c_h based on a theoretically acceptable technique. Due to the limited data available, it is not possible to recommend a preferential pore pressure measurement location in terms of performing CPTU dissipation tests. The u₃ location would appear to involve prohibitively long dissipation times to be of practical use. On the other hand, the u₁ position would appear to be a good location due to the higher rate of dissipation involved. In fine-grained soils, the time required for dissipation of CPTU excess pore pressures may be very long and as a result impractical to perform in the field. In this respect, a graphical extrapolation technique would be useful whereby data from an initial short period of dissipation could be mathematically modelled to predict the complete dissipation curve. Alternatively, the root-time interpretation method can be used to extrapolate short duration dissipation tests.

The results presented here are preliminary and further evaluation is being performed.

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