An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils

John P. Sully, Peter K. Robertson, Richard G. Campanella, and David J. Woeller

Abstract: Dissipation of excess pore pressures during piezocone testing in firm to stiff overconsolidated fine-grained soils provides data curves that cannot be interpreted using published theoretical solutions. Available solutions are based on either cavity-expansion or strain-path methods, which have been developed for soft, normally consolidated soils and do not adequately model the response in overconsolidated soils. During penetration in overconsolidated soils, large pore-pressure gradients and unloading can exist as the soil moves past the singularity at the cone tip shoulder; these gradients modify the initial pore-pressure distribution around the tip and along the friction sleeve and give rise to nonstandard dissipation curves. The different types of response resulting from the modified pore-pressure distribution are discussed and classified and a correction technique is proposed. In this way, the application of available theoretical models can be used to evaluate in situ flow characteristics. Examples of the different types of pore-pressure response are presented from various sites worldwide.

Key words: cone penetration test, in situ, clay, overconsolidation ratio, pore pressure, dissipation, consolidation.

Résumé : La dissipation de l'excès des pressions interstitielles au cours de l'essai au piézocône dans les sols fins surconsolidés de consistance moyenne à ferme fournit des courbes de données qui ne peuvent pas être interprétées au moyen des solutions théoriques publiées. Les solutions disponibles sont basées sur les méthodes d'expansion de cavité ou de cheminement de déformation qui ont été développées pour les sols mous normalement consolidés et qui ne modélisent pas adéquatement la réponse dans les sols surconsolidés. Au cours de la pénétration dans les sols surconsolidés, de forts gradients de pression interstitielle et un déchargement peuvent se produire lorsque le sol se déplace le long de la singularité à l'épaulement de la pointe conique; ces gradients modifient la distribution initiale de la pression interstitielle autour de la pointe et le long du manchon de frottement et génèrent des courbes de dissipation non standard. Les différents types de réaction résultant de la distribution modifiée des pressions interstitielles sont discutés et classifiés, et une technique de correction est proposée. Ainsi, l'on peut appliquer les modèles théoriques disponibles pour évaluer les caractéristiques d'écoulement in situ. L'on présente des exemples des différents types de réaction de pression interstitielle provenant de divers sites à travers le monde.

Mots clés : essai de pénétration au cône, in situ, argile, rapport de surconsolidation, pression interstitielle, dissipation, consolidation.

[Traduit par la Rédaction]

Background

Cone penetration testing with pore pressure measurement (CPTU or piezocone testing) has become a popular investigation technique in geotechnical site investigation practice. The near-continuous data obtained during penetration (approx. every 5 cm) provide information related to the following soil response parameters: tip resistance (q_c), sleeve friction (f_s), and penetration pore pressure (u). In addition, inclination and temperature may also be recorded depending on the type of piezocone being used (Campanella and Robertson 1988). The penetration of the piezocone can be halted at any depth and the variation with time of the measured parameters can be monitored. Of the above three quantities (q_c, f_s, u), it is usually the 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

$$[1] \qquad \Delta u = u_i - u_o$$

and where

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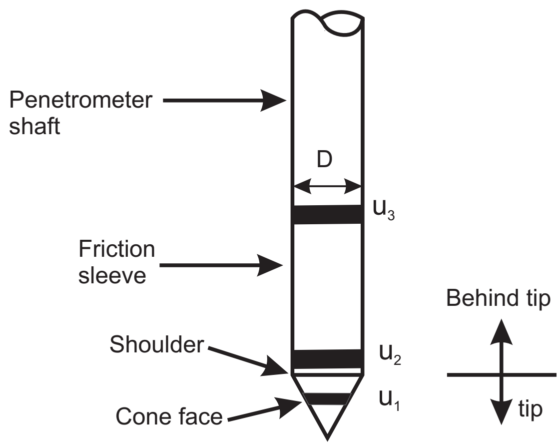
Fig. 1. Terminology used for indicating location of pore-pressure measurement.



u_i is the measured pore pressure at the depth of interest; and

u_o is the equilibrium in situ pore pressure at the depth of interest.

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

$$[2] \qquad U = \frac{\Delta u(t)}{\Delta u_i} = \frac{[u(t) - u_o]}{(u_i - u_o)}$$

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., on stopping penetration; and

$u(t)$ is the total pore pressure at any time t.

Hence, for standard dissipation records where the excess pore pressure shows a monotonic decrease with time, U varies between unity (at $t = 0$) and zero when 100% dissipation of the excess pore pressure has occurred.

With the development of piezocone equipment, it is now possible to measure penetration pore pressures at one or more locations on the cone. Three specific locations will be discussed in this paper and designated according to the scheme illustrated in Fig. 1. Hence, the excess pore pressure can also be subscripted according to where the measurements are obtained (Sully et al. 1988):

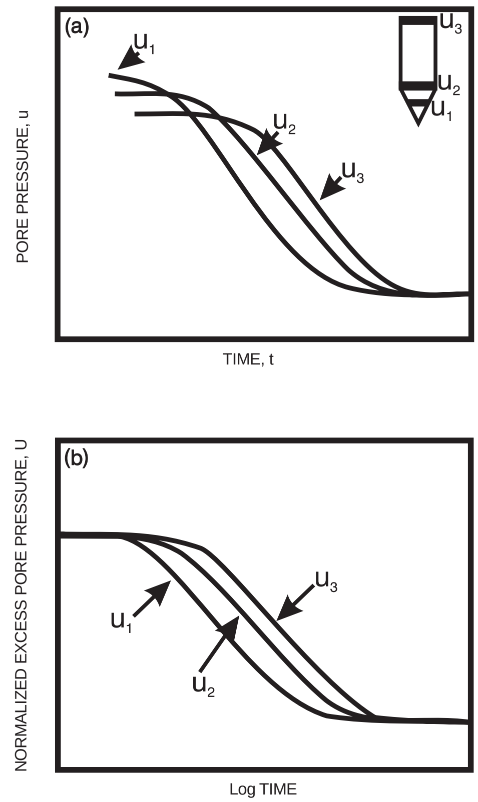
$$[3] \qquad \Delta u_{1,2,3} = u_{1,2,3} - u_o$$

Because of the small distance between each of the three measurement locations (0.1 m max. between u_1 and u_3), the magnitude of u_0 can be taken to be equal for each of the three positions.

Once the required dissipation data have been obtained during the penetration testing, the excess pore-pressure variation with time can be plotted if the equilibrium pore pressure is known (or measured). In low-permeability soils, it is usual practice 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 of the two main analytical approaches: cavity-expansion theory or the strain-path approach. Interpretation by dislocation methods has also been proposed by Elsworth (1993). Comparisons of the available solutions and results from field studies suggest that the cavity-expansion method of Torstensson (1977) and the strain-path approaches of Levadoux (1980) and Teh (1987) all provide similar predictions of consolidation parameters from CPTU dissipation data (Gillespie 1981; Kabir and Lutenegger 1990; Robertson et al. 1991). Robertson et al. (1991) have shown that these methods, although developed for normally consolidated soils, can be equally applied to overconsolidated soils. Furthermore, comparisons of field and laboratory data indicate that the trends in the measured (laboratory) and predicted (CPTU) data are consistent provided the microfabric and nature of the soils being tested are taken into consideration (Danziger 1990; Robertson et al. 1991). Limited published data are available to verify the more recent dislocation method.

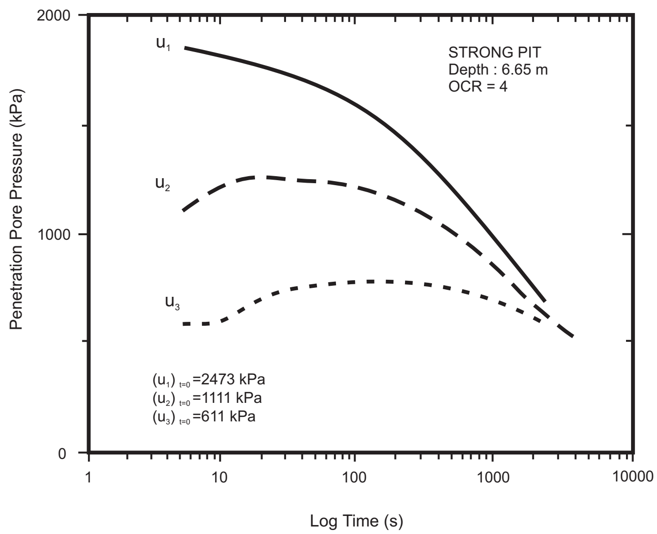
Fig. 2. Typical dissipation records for normally consolidated fine-grained soils.



However, 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. The initial distribution around the probe may be such that the applicability of these methods may be questioned or restricted only to normally consolidated soils.

A typical set of excess pore-pressure dissipations in soft, normally consolidated clay for the three above-mentioned filter locations is presented in Fig. 2*a*. The corresponding normalized dissipation curves are shown in Fig. 2*b*. All three pore-pressure dissipation curves show a monotonic decrease in the excess pore pressure with time and essentially

Fig. 3. Pore-pressure dissipation in stiff, moderately overconsolidated silty clay (Strong Pit).



agree with the theoretical models of the 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). The rate of dissipation is highest on the face of the cone and reduces with distance behind the tip; these effects are considered in the analysis by varying the time factor, T, according to location of the pore-pressure element and by using the radius of the probe at the location of the pore-pressure measuring sensor in the numerical calculations.

Under certain circumstances, the pore pressures measured behind the tip do not decrease immediately on stopping penetration; rather, they show an initial increase over a definite period of time before finally beginning to dissipate. Where the pore-pressure measurement system is completely saturated, dissipation records of this type are characteristic to filter locations located behind the cone tip (u_2 and u_3) for penetration in overconsolidated soils. The interpretation of these dissipation records to obtain predictions of c_h is the subject of this paper.

Pore-pressure dissipation in overconsolidated soil

A typical example of pore-pressure dissipation in a lightly overconsolidated fine grained soil (overconsolidation ratio OCR = 4) is illustrated in Fig. 3 using results obtained at Strong Pit in the Lower Mainland of British Columbia (Campanella et al. 1988). Similar types of dissipation record in overconsolidated soils for locations behind the tip have been reported by Tumay et al. (1981), Davidson (1985), BRE/NGI (1985), Kabir and Lutenegger (1987), Gillespie et al. (1988), Coop and Wroth (1989), Lunne et al. (1986), Gomez and Escalante (1987), and Bond and Jardine (1991).

The initial rise in pore pressures measured at locations behind the tip in overconsolidated soils may be explained by (*i*) poor saturation and (or) poor response of the measurement system such that a time lag in response to pore-pressure changes occurs, and (or) (*ii*) redistribution of pore pressure around the tip due to the large gradients that are generated in overconsolidated soils. (The Mandel–Cryer effect is not considered to be of major importance for locations behind the tip for the overconsolidated soils examined here.)

A further possibility for the increase in pore pressure has been suggested by Coop and Wroth (1989) as a result of the maximum penetration pore pressure being located at some point away from the shaft of the piezocone. These points will be considered briefly below.

Poor saturation and (or) poor response of the measurement system

If the rise in pore pressure on halting penetration were due solely to saturation and (or) measurement problems, then the effects should be equally frequent in normally consolidated and overconsolidated soils. This is not the case. Furthermore, data of this type have been reported by some of the main research centres around the world where overconsolidated soils have been studied and experimental techniques are well proven. It would thus appear that the anomalous pore-pressure rise is not due solely to poor field technique. However, it must also be borne in mind that in heavily overconsolidated soils pore pressures behind the cone tip may become negative and in some instances give rise to cavitation of the measuring system (Powell et al. 1988). If cavitation occurs, the measurement system may become desaturated and sluggish response will result, giving rise to curves somewhat similar to those shown in Fig. 3.

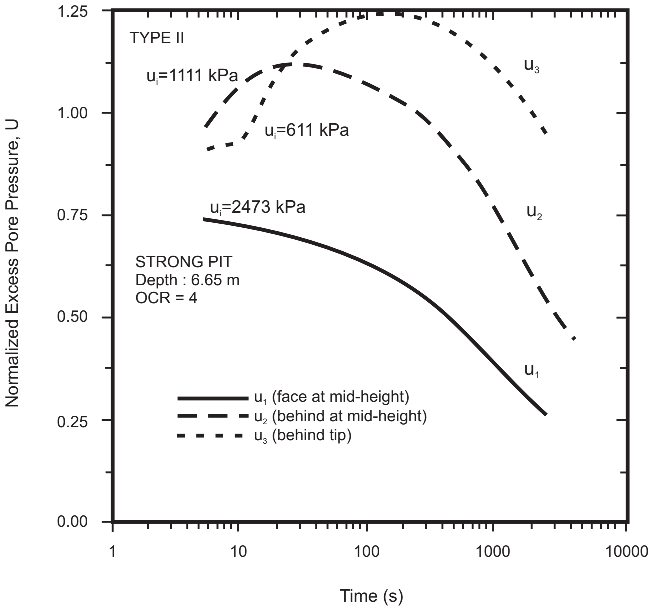
Redistribution of pore pressure

Due to the large gradient of pore pressures around the tip in overconsolidated and stiff, fine-grained soils (Robertson et al. 1986; Davidson 1985), drainage from the tip (high pore pressure) to the zone behind the tip (lower pore pressure) occurs, the rate of which is determined primarily by the soil permeability and the magnitude of the gradient. The effect of the flow around the tip on the pore pressures measured on the shaft also varies according to the soil stiffness and strength, parameters which also determine the pore-pressure gradient itself. The authors consider this to be the principal reason why pore pressures measured at locations behind the tip in overconsolidated and stiff, fine-grained soils show an initial increase when penetration is stopped followed by dissipation of excess pore pressures. It is also interesting to note that a comparison of the u_2 and u_3 pore pressures in Fig. 3 indicates that the u_2 value reaches a peak faster than the u_3 measurement. This is logical if the driving force for the initial increase is the pore-pressure gradient, since the gradient between the u_1 and u_2 locations is much higher than that between the u_2 and u_3 locations (Robertson et al. 1986; Sully et al. 1988; Whittle et al. 1991).

Maximum pore-pressure located away from the shaft

Coop and Wroth (1989) have suggested that the maximum penetration pore pressure in overconsolidated soils is located at some distance away from the shaft of the penetrometer. If this were the case, soil strength and stiffness would control not only the magnitude of the pore pressure behind the tip but also the distance of the maximum value from the shaft.

Fig. 4. Normalized pore-pressure dissipation for Strong Pit clay.



Hence, in normally consolidated soils the maximum pore pressure would be close to or on the shaft and pore-pressure decrease would occur on stopping penetration. As the soil becomes more overconsolidated, the location of the maximum pore pressure would move away from the shaft and progressively longer time delays would occur before the pore pressure reached its peak value after stopping penetration. This idea would imply, however, that at all locations on the shaft, irrespective of the distance behind the tip, the measured pore pressures would all attain peak values at the same time. This is not the case, as shown in Fig. 3, and in other overconsolidated dissipation data referenced above. General trends in published data also do not confirm the hypothesis of Coop and Wroth (1989).

Typical dissipation records in overconsolidated soils

Considering the data presented in Fig. 3 it is apparent that only the u_1 pore pressure shows initial dissipation on halting penetration. However, the unloading of the tip resistance causes a sudden decrease in the u_1 pore pressure. This sudden decrease modifies the dissipation record such that normalization with the initial maximum penetration pore pressure $(u_i)_1$ of 2473 kPa gives rise to a nonstandard dissipation record compared with that suggested by the available theories. For the locations behind the tip, the pore pressure initially increases when penetration is stopped, before finally decreasing and arriving at the in situ equilibrium value.

In conclusion, it appears that interpretation of pore-pressure dissipation records in overconsolidated soils is complicated by unloading effects and redistribution at all three filter locations considered here.

Commonly, the dissipation results are presented in terms of normalized curves, whereby the normalized pore pressure, U, at any time t, is given by eq. [2]. The normalized dissipation curves for the records in Fig. 3 are shown in Fig. 4. None of the curves follow the theoretical dissipation trends suggested by the available theories for normally consolidated soils and hence cannot be evaluated, as is, to provide information on the coefficient of consolidation of the soil. The departure of the field curves from the theoretical framework is considered to be a result of the mean normal stress reduction that occurs as the soil passes around the cone tip and along the shaft. None of the available theoretical approaches adequately considers the magnitude of the stress reduction in stiff overconsolidated soils and the important effect it has on the initial pore-pressure distribution around a penetrating cone.

Characteristic dissipation types

In normally consolidated soils, the pore-pressure dissipation curve for any filter location (Fig. 2) can be considered as a type I response. A type I response implies a monotonic decrease of the initial penetration excess pore pressure. In overconsolidated soils, several different responses may be obtained depending on soil characteristics and filter location. For the filter located on the cone tip (u_1), the unloading-type dissipation associated with overconsolidated soils can be classified as a type II response (Fig. 5*a*). A type II response is similar to a type I response once the pore-pressure reduction due to unloading has occurred.

A type III response is assigned to the locations behind the tip (u_2, u_3) where the excess pore pressure is greater than hydrostatic ($u(t) > u_o$), but increases on stopping penetration before dissipating (Fig. 5*b*). All of the above responses have been discussed earlier.

In moderately to heavily overconsolidated soils, the pore pressures measured at the location immediately behind the tip may be less than 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. (1) 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. 5*c*, type IV). The type IV curve is similar to the type III response, the difference being the degree of pore-pressure change or unloading that occurs. (2) 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. 5*c*, type V).

The standard approach for interpreting dissipation records in normally consolidated soils cannot be applied to the responses in overconsolidated soils (with the exception of an inverted type V), since dissipation does not follow the theoretical response, that is, a monotonic reduction with time. The theoretical framework for evaluating dissipation in soils showing responses similar to those defined by types II–IV is not available at present. However, the anomalous curves can be corrected to permit interpretation using the available theories. In fact, Elsworth (1993) suggests curve correction prior to applying the dislocation method for curves where pore pressure increases on stopping penetration.

Fig. 5. Idealized pore-pressure dissipation response in overconsolidated fine-grained soil for different measurement locations.

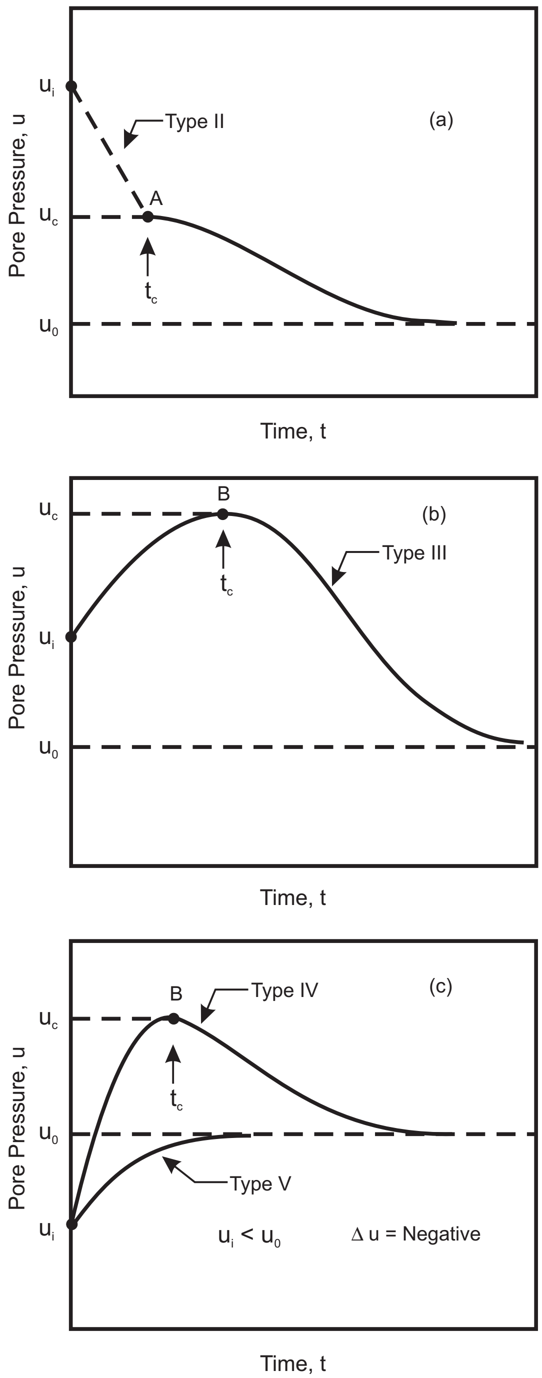
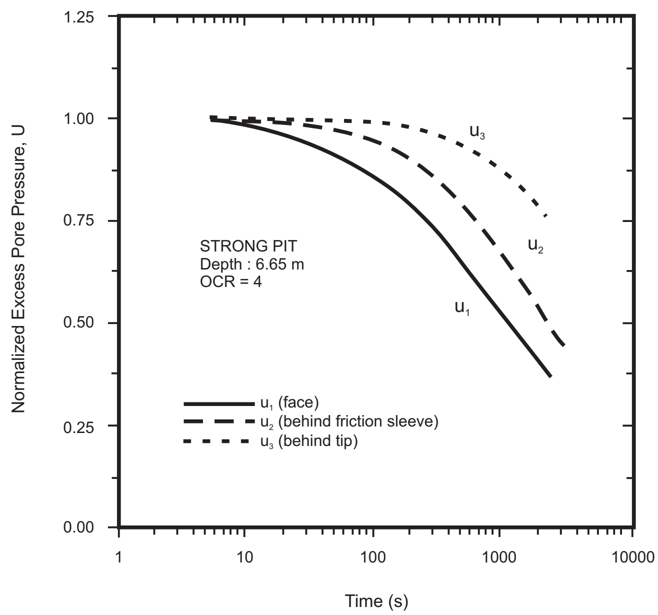


Fig. 6. Normalized dissipation – logarithm of time plot for data in Fig. 3 corrected for unloading (u_1) and redistribution (u_2 and u_3). The dissipation data have been normalized to the maximum value in the file, and time is taken equal to zero at this maximum value.



The type V response can be considered as an inverted dissipation and treated in the standard way (as dissipation of a negative excess). This type of response may occur in soils with an OCR of 4 or larger, although the type of response also depends on soil type and structure. For example, in heavily overconsolidated Taranto Clay (OCR = 20–40) the pore pressures behind the tip are positive, possibly due to the cementation present (Battaglio et al. 1986), whereas in fissured London Clay (OCR = 25–50) negative pore pressures are recorded (Powell and Uglow 1988).

Correction to dissipation curves for redistribution effects

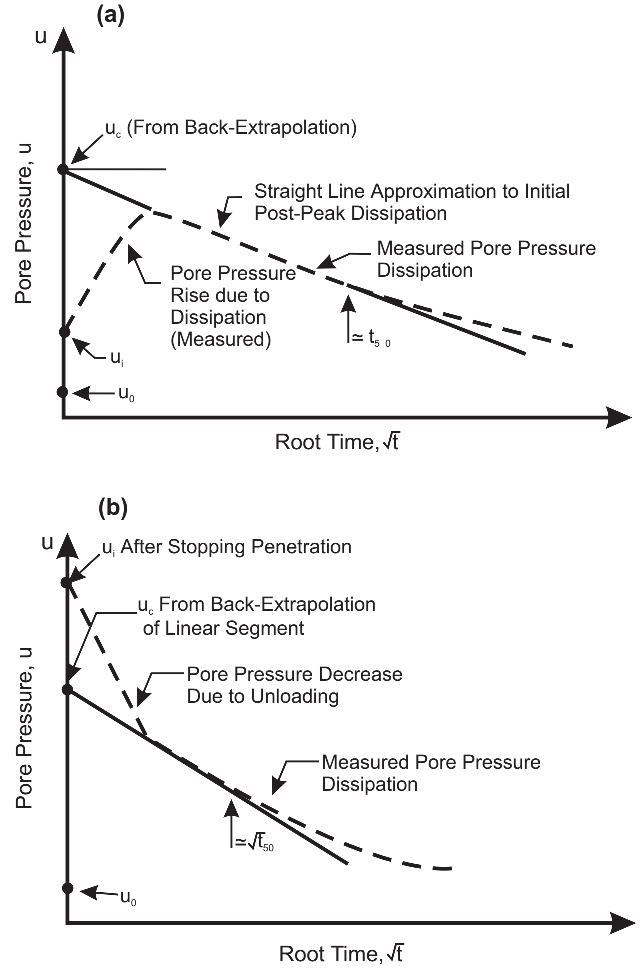
Two methods of data presentation can be utilized to correct the type II–IV dissipation curves so that the available dissipation theories can be used to estimate values of the coefficient of consolidation. One approach is based on a logarithm of time plot and the other is based on a square-root of time representation, both similar to the routine methods presently employed for laboratory consolidation data. Either approach can be used separately or combined to provide a check on the results obtained. The application of the methods is presented using the results in Fig. 4 as an example. The step-by-step correction procedure for both methods is outlined in the Appendix.

Logarithm of time plot correction

The data in Fig. 4 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 unloading and (or) redistribution that occurs affects the three sets of pore pressures in different ways.

On the tip, a sudden decrease in pore pressure occurs on halting penetration. In Fig. 4 the normalized pore pressure 5 s 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 (u_c), i.e., for this case the maximum value corresponds to the 5 s measurement and this time (t_c) is taken as the new zero time point (5 s are subtracted from the time register throughout the record). The maximum pore pressure for the dissipation

Fig. 7. Details of the square root of time method for evaluating pore-pressure dissipation data in overconsolidated soils according to response type: (*a*) types III to IV, and (*b*) type II.



record is taken as the peak value which occurs, for this particular record, at 5 s.

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 new zero time of the dissipation record, with all other times adjusted accordingly.

The data from Fig. 4, corrected in this way, are replotted in Fig. 6 to show the new form of the normalized dissipation plot, adjusted to account for unloading and redistribution effects. The method of correcting the data is considered to be theoretically acceptable, since it adjusts the dissipation file so that a monotonic reduction of a maximum pore pressure to the in situ equilibrium occurs thereby removing the anomalous effects discussed above.

Square root of time plot

It is also possible to adjust the dissipation data of Fig. 4 using a back-extrapolation technique on a square-root of time plot, similar to the Taylor method used for interpreting t_{50} values (50% reduction of excess pore pressure) from laboratory one-dimensional incremental consolidation testing. In the square root of time plot, the dissipation after the peak

Fig. 8. Measured dissipation data for stiff clay in square root of time plot.

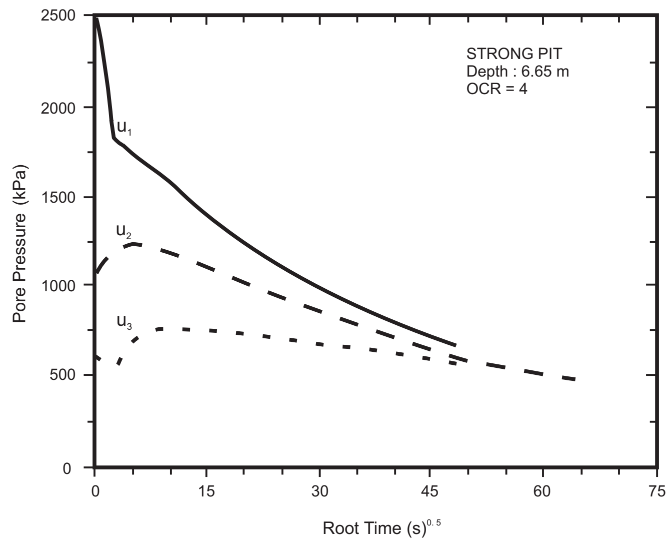
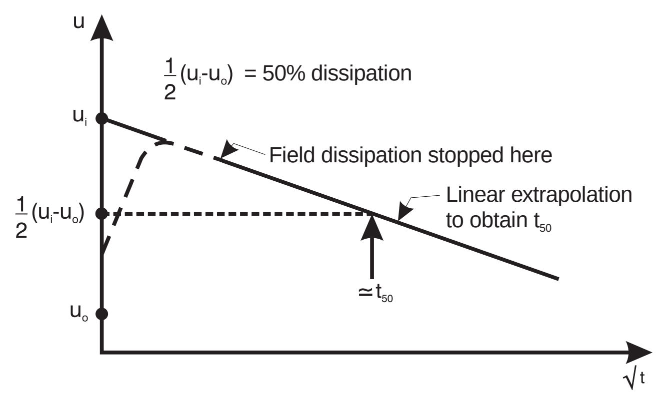


Fig. 9. Square root of time extrapolation for short-duration dissipation records.



caused by redistribution of pore pressure, initially depicts a straight line which can be back-extrapolated to $t = 0$ in order to obtain a modified u_c for the corrected dissipation curve. This value is then used to produce the normalized dissipation curve. The correction technique is illustrated in Fig. 7*a* 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 (Fig. 7*b*).

The data from Fig. 4 have been plotted in the square root of time base and are presented in Fig. 8. Direct estimates of t_{50} can be read directly from Fig. 8.

The additional advantage of the square root of 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. 9), or the initial linear slope in the normalized pore pressure – square root of time plot can be analyzed to provide estimates of c_h using the theoretical approach suggested by Teh (1987).

The two correction methods described above will give rise to slightly different normalized dissipation curves, since the

Table 1. Geotechnical characteristics for the University of British Columbia test sites considered.

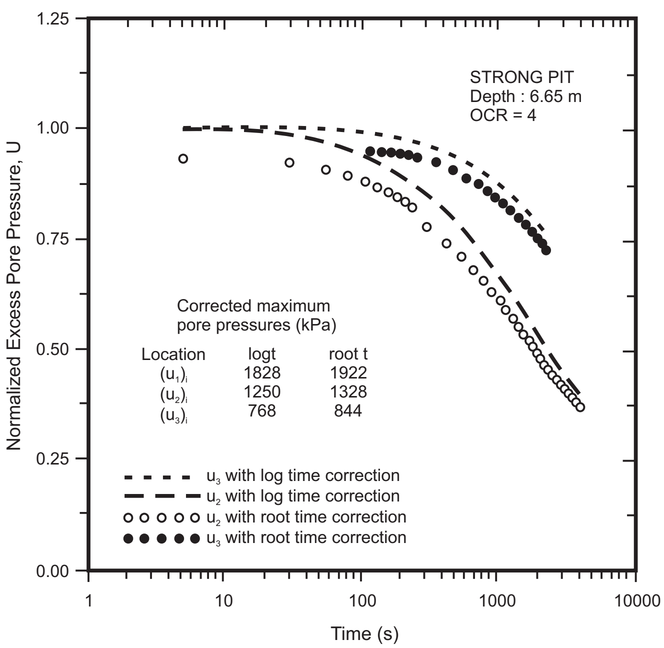
Site	Depth range (m)	PI (%)	OCR	Range of σ_{vo}' (kPa)	Range of σ_{vm}' (kPa)
Lower 232 Street	1–5	21–30	3–10	16–40	90–205
Strong Pit	1–9	11–20	2–15	16–180	350–500
200th Street	1–5	20	2–17	16–51	115–300

Note: PI, Plasticity Index; σ_{vo}', vertical effective stress presently acting in the ground on the soil; σ_{vm}', maximum past vertical effective pressure from incremental oedometer tests.

Table 2. Comparison of field and laboratory coefficients of consolidation.

Site	Filter location	$(c_h)_{OC}$ from CPTU (cm^2/s)	c_v from oedometer (cm^2/s)		c_h/c_v (oedometer)
			Overconsolidated	Normally consolidated	
Lower 232 Street	u_1	0.002–0.005	0.006–0.1	0.0005–0.001	2–3
	u_2	0.005–0.016			
Strong Pit	u_1	0.004–0.007	0.002–0.005	0.0006–0.001	1–2
	u_2	0.006–0.01			
200th Street	u_1	0.014–0.047	0.05–0.18	0.001–0.03	1.5–2
	u_2	0.045–0.054			

Fig. 10. Comparison of normalized dissipation curves after applying proposed logarithm of time (log time) and square root of time (root time) corrections.



initial corrected u_c values are, by definition, not the same. The resulting corrected dissipation data for the Strong Pit site are compared in Fig. 10 (the square root of time plot has been reproduced in logarithm of 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 between the predicted values is relatively small (5–10%). In essence, the two correction techniques give similar values for t_{50}. The curves in Fig. 10 do, however, indicate the importance of the initial pore-pressure value at t = 0 on the normalized form of the dissipation curve.

Evaluation of proposed procedures in overconsolidated soils

Basis of comparison

The two correction procedures have been applied to CPTU dissipation data from three University of British Columbia research sites where overconsolidated soils are present in the profile. These extrapolation techniques have also been verified using data presented by some of the major piezocone research centres worldwide (Robertson et al. 1991). The results of comparisons between the log t correction and laboratory-derived consolidation parameters are considered below. The square root of time method is not presented here because the difference between the 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 and, as suggested by Fig. 10, the difference between the two values of t_{50} is insignificant when taken in context of the overall magnitudes involved.

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. The c_h value is determined from

$$[4] \qquad c_h = \frac{T^* R^2 I_R^{0.5}}{t_{50}}$$

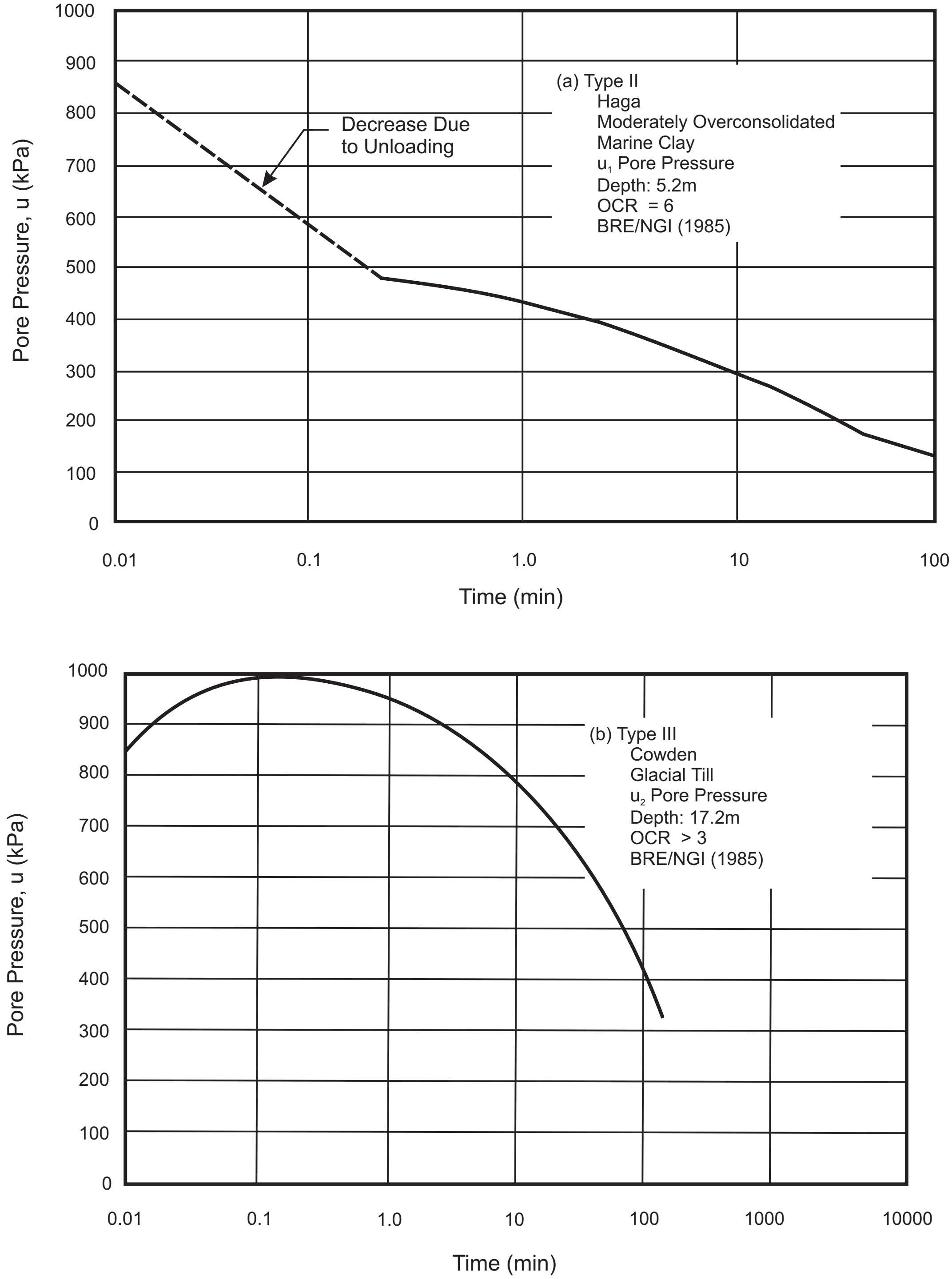
where

T^* is the Teh and Houlsby (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 Teh and Houlsby (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 values of

Fig. 11. Published dissipation data for overconsolidated fine-grained soils, illustrating the proposed classification scheme. (*a*) Type II response in Haga clay. (*b*) Type III response in glacial till at Cowden.



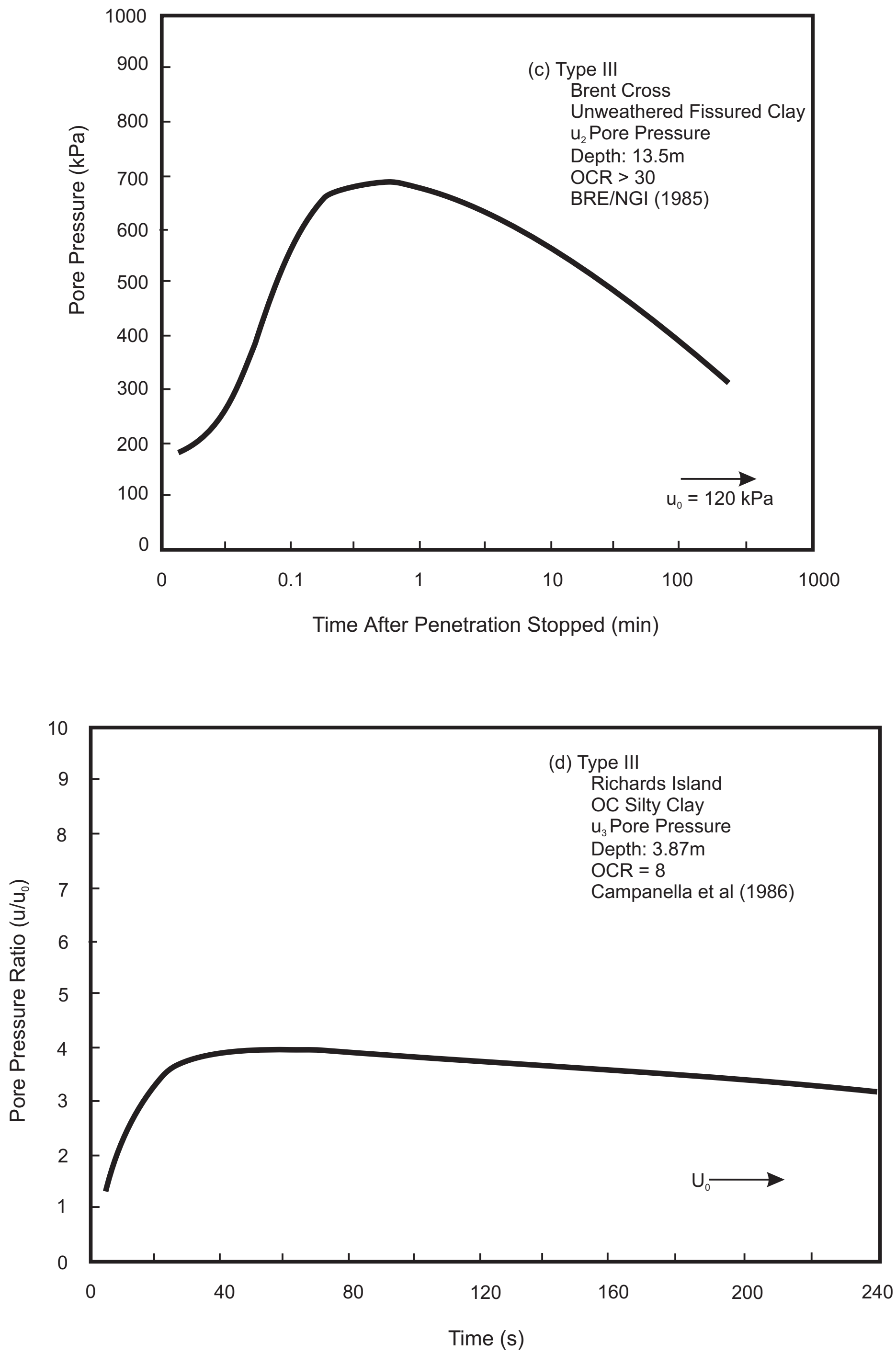
the coefficient of consolidation c_v may not be wholly representative of in situ conditions, the results do provide a basis on which the relative CPTU magnitudes can be compared. Field data are available from several international research sites where CPTU-derived values of c_h and the vertical coefficient of consolidation c_v can be compared with the results of large-scale field tests (Robertson et al. 1991).

Laboratory c_v values are determined from each loading stage so that both overconsolidated and normally consolidated data are available. Furthermore, one-dimensional oedometer tests have also been performed on directionally-cut samples to obtain estimates of c_h. Hence, the c_h/c_v ratio can be estimated from the laboratory tests to correct the CPTU c_h values to c_v for direct comparison with the laboratory data.

Geotechnical review of University of British Columbia sites considered

The general geotechnical characteristics of the University

Fig. 11 (*continued*). (*c*) Type III response in London Clay at Brent Cross. (*d*) Type III response in overconsolidated silty clay at Richards Island.

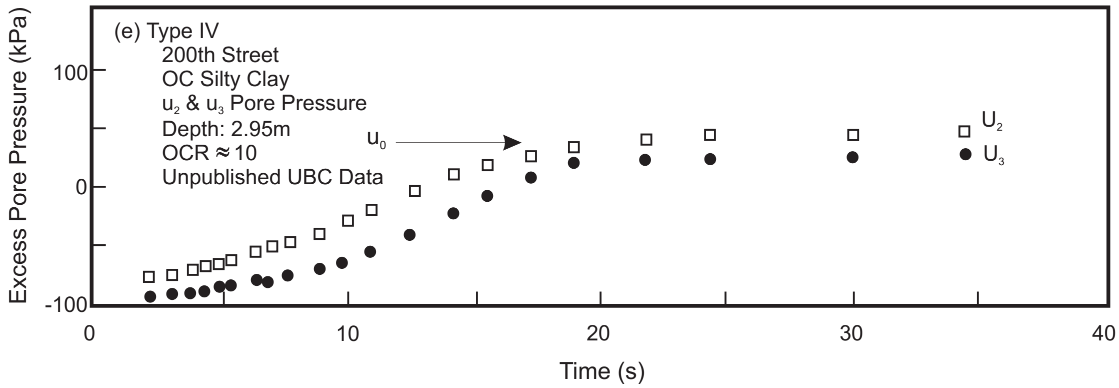


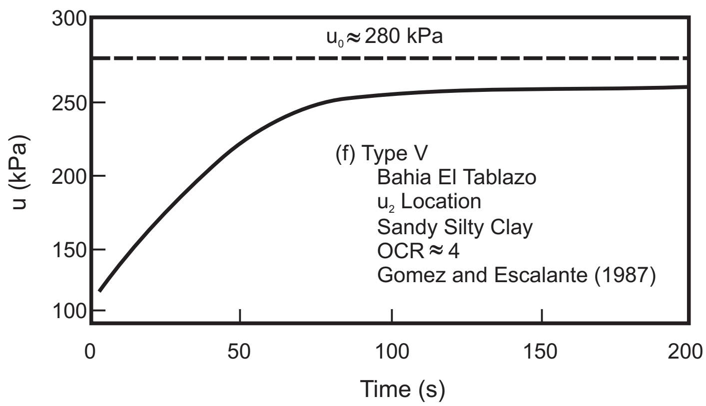
of British Columbia test sites under consideration are presented in Table 1. The soils at Lower 232 Street are soft, sensitive clay silts. At the other two sites the clay silts are nonsensitive, with undrained strengths up to 200 kPa.

Results of comparison for University of British Columbia sites

Only data from pore-pressure locations u_1 and u_2 have been used for the purposes of comparison. The times required for 50% dissipation of the excess pore pressure measured at the u_3 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. This is evident from the data in Fig. 6 which highlight the problems associated with the execution and interpretation of dissipation data at locations behind the tip, since the time periods required are very long in low-permeability soils. The time for 50% dissipation at the u_1 location is 1100 s (18 min), which increases to 2300 s for u_2 and approximately 10 000 s for

Fig. 11 (*concluded*). (*e*) Type IV response at 200th Street. (*f*) Type V response in stiff sandy silty clay.





u_3 (the u_3 record has been extrapolated for comparative purposes to obtain the 50% dissipation times).

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. For the Lower 232 Street data an average I_r value of 50 has been used to evaluate c_h, whereas values of 100 and 200 have been used for 200th Street and Strong Pit, respectively (the $I_r = G/S_u$ values have been obtained from in situ shear wave velocity measurements and field vane shear tests, where G is the maximum shear modulus, and S_u is the undrained shear strength).

The c_h values from the two pore-pressure measurement locations (u_1 and u_2) are in very good agreement. Furthermore, the CPTU values would suggest that the t_{50} dissipation provides an estimate of the c_h corresponding to the overconsolidated condition, i.e., the in situ condition of the soil. Considering the results presented by Baligh and Levadoux (1980) who show that, in normally consolidated soils, the t_{50} time corresponds to the $(c_h)_{NC}$, it would appear that at degrees of dissipation of 50% the theory provides estimates of the coefficient of consolidation relevant to the in situ stress history condition, $(c_h)_{OC}$. In addition, 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. As suggested by Kabir and Lutenegger (1987), the dissipation curve may be useful as a stress-history indicator.

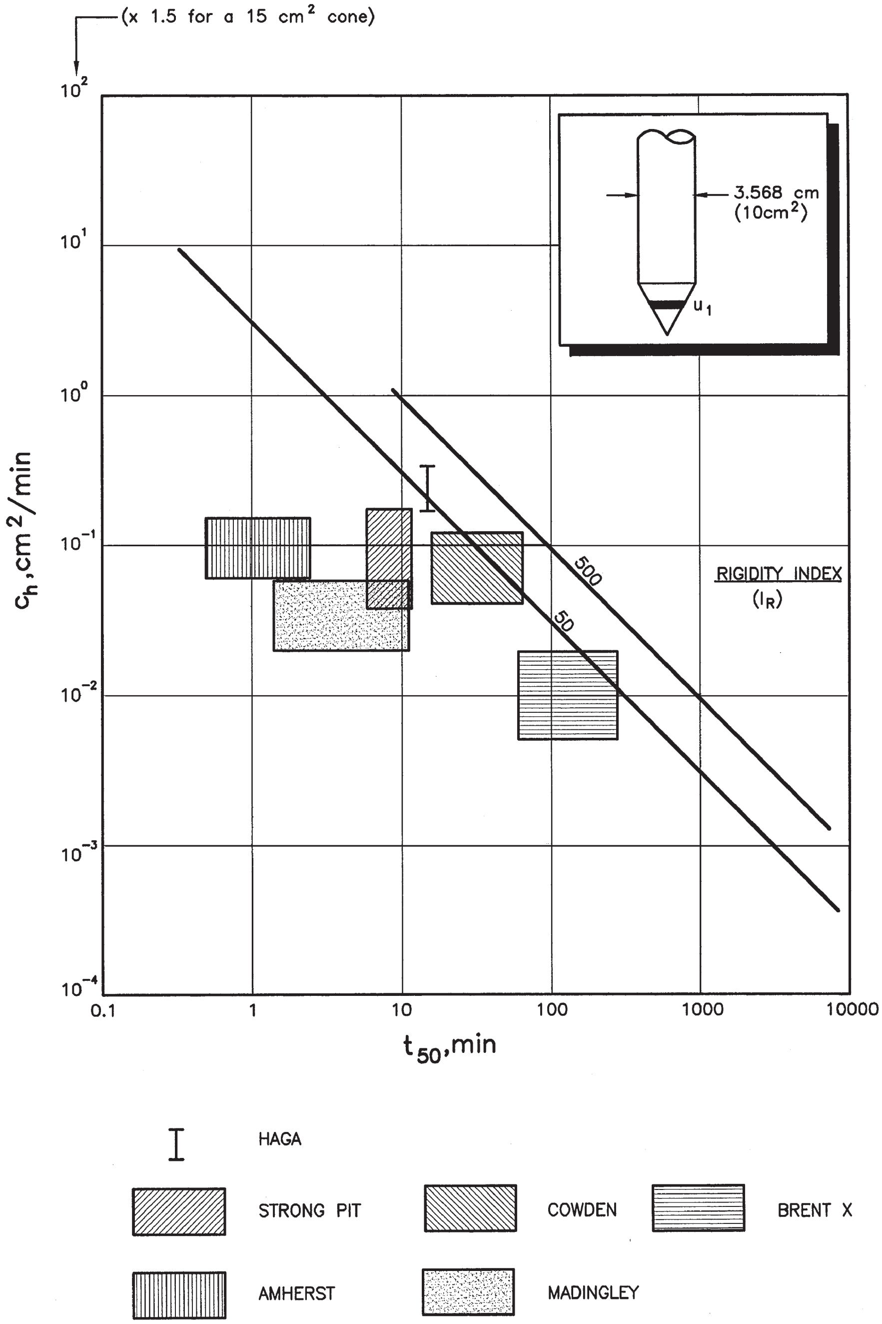
Evaluation of data from international sites

Dissipation tests using the cone penetrometer have been carried out in a variety of overconsolidated soils worldwide. The data have been summarized by Robertson et al. (1991). Typical results highlighting the different types of pore-pressure response described above are presented below.

The results of a u_1 dissipation in moderately overconsolidated Haga clay (BRE/NGI 1985) are presented in Fig. 11*a*. The pore pressure of about 875 kPa generated during penetration decreases immediately to 480 kPa after stopping penetration. Thereafter, the dissipation occurs as the excess pore pressure close to the cone dissipates laterally into the surrounding soil. This is similar to the type II response described earlier. The lightly overconsolidated glacial till at Cowden (OCR = 3) provides an example of a type III dissipation at the u_2 location (BRE/NGI 1985). The positive penetration pore pressure of 835 kPa increases after stopping penetration until a maximum value of around 980 kPa is reached some 10 s later (Fig. 11*b*). Correcting the dissipation curves for the maximum values and using the new zero times, as described earlier, allow the modified dissipation curve to be interpreted to obtain the overconsolidated coefficient of consolidation. The data from Brent Cross (OCR > 30) are also characteristic of a type III response for the u_2 location (Fig. 11*c*).

The Richards Island silty clay (OCR = 8) dissipation results presented by Campanella et al. (1986) are typical of a type III response for the u_3 location (Fig. 11*d*). For this case the penetration pore pressure is only marginally higher than the in situ equilibrium value, u_o ; if it had been lower than u_o, then this would have been a type IV dissipation response, similar to the result obtained at 200th Street in the Lower Mainland of British Columbia (Fig. 11*e*) where

Fig. 12. Comparison of t_{50} from corrected dissipation data in overconsolidated soils with c_h from laboratory consolidation tests using dissipation interpretation nomograph presented by Robertson et al. (1991).



the penetration pore pressure at the start of the dissipation (u_i = –90 kPa) is less than the equilibrium value (u_o = 16 kPa) in this moderately overconsolidated clay (OCR = 10). As the permeability of the soil increases, the type IV response changes to a type V response, where the negative excess generated during penetration dissipates to the equilibrium value. This type of response has been recorded in dilative silts and sandy silty clays (Fig. 11*f*).

All the dissipation data for overconsolidated soils discussed above were presented by Robertson et al. (1991), having been interpreted using the above correction techniques. These data and other results for overconsolidated soils presented by Robertson et al. are plotted in Fig. 12, using the c_h–t_{50} nomograph. In Fig. 12 the CPTU dissipation data are compared with c_h values obtained from laboratory consolidation tests. Data have been reported in the literature for a limited number of sites, where c_h has also been determined from back-analysis of field performance. The published values of c_h obtained from field performance are generally larger than laboratory values (Robertson et al. 1991; Jones and Rust 1993) and provide better agreement with the theoretical range obtained from the Teh and Houlsby (1988) solution.

Conclusions

This paper has presented a classification for the types of dissipation response recorded in overconsolidated, fine-grained soils during piezocone penetration tests. The different dissipation responses have been explained in terms of the unloading and redistribution that occur as the soil undergoes lateral displacement to permit advancement of the cone tip and shaft. The singularity at the shoulder of the cone tip gives rise to the unloading, as the soil no longer undergoes cavity-expansion type deformation, but rather shearing along the shaft–soil interface. The soil stiffness and strength, which can be considered to be dependent on the consolidation state of the soil, control the degree of unloading and redistribution, and consequently the extent of the modified pore-pressure response (compared to the type I response associated with normally consolidated soils). Also important in controlling the type of response is the location of the porous element used for measuring the pore-pressure variation with time.

Theoretical models do not exist at present that adequately model the unloading and redistribution of pore pressure which occur during penetration of a cone into stiff, overconsolidated, fine-grained soils. As a result, the above effects cannot be verified by numerical techniques and semi-empirical interpretation is required. Similar empirical and semi-empirical approaches are used to interpret CPT data in granular soils where stress rather than pore-pressure gradients occurs. However, the consistent dissipation pore pressure response types reported by major research centres worldwide indicate the validity of the reported behavior and the approach to interpretation.

Two data-presentation methods have been discussed for correcting the different types of pore-pressure response in overconsolidated soils. In one method, the data in log t space are normalized to the maximum post-penetration value and the time at which this maximum value occurs. In the second approach, a square root of time correction and extrapolation are used to correct the field data (see the Appendix). Once either of the above modifications has been made, the data can be interpreted using standard techniques available for normally consolidated soils. The Teh and Houlsby (1988) method has been used in this paper to interpret the data using the nomograph presented by Robertson et al. (1991).

The c_h values from CPTU generally fall into the range suggested by the laboratory data for the overconsolidated state for the in situ tests at the wide range of sites considered. The correction method proposed here appears to provide consistent estimates of c_h.

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Appendix: Classification used for the overconsolidation state

The following classification is used here for description of the soil overconsolidation state, with the overconsolidation ratio determined as OCR = $\sigma_{vm}'/\sigma_{vo}'$, where σ_{vm}' is the maximum vertical effective pressure the soil has experienced, and σ_{vo}' is the vertical effective stress presently acting in the ground on the soil.

Stress history	Description
OCR ≤ 1	Normally consolidated (NC)
1 < OCR ≤ 4	Lightly overconsolidated (LOC)
4 < OCR ≤ 10	Moderately overconsolidated (MOC)
10 < OCR ≤ 25	Heavily overconsolidated (HOC)
25 < OCR	Very heavily overconsolidated (VHOC)