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TECHNICAL NOTE

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Interpretation of Seismic Piezocone Results For the Estimation of Hydraulic Conductivity in Clays

ABSTRACT: Seismic piezocone penetration tests with dissipation phases provide sufficient data for outlining the geostratigraphy, soil strength, stiffness, and flow characteristics in fine-grained soils. The dissipation of excess pore water pressures with time can be monitored at regular intervals (e.g., at 1-m rod breaks) and provides information on the coefficient of consolidation (c_h). Monotonic and/or dilatory responses of Δu decay can be accommodated. The shear wave velocity (V_s) provides a measure of the small-strain stiffness (G_0), or surrogate evaluation of the constrained modulus (D'). A sampling of data from ten clay sites was used to assess the derived magnitude of hydraulic conductivity, as determined from $k = c_h\gamma_w/D'$.

KEYWORDS: clays, cone penetration, consolidation, dilatory response, dissipation, in-situ tests, hydraulic conductivity, modulus, permeability, piezocones, pore pressures, stiffness

Introduction

Characterization of the hydraulic conductivity (k) of a soil deposit is one of the most critical aspects of geoenvironmental engineering, because it determines the rate of flow of groundwater through the subsurface, which controls the advective transport of contaminating chemicals. Additionally, it is significant for geotechnical projects involving groundwater inflow into excavations and basements, as well as being important for studies of water resources, consolidation, and dewatering. Many high quality methods exist to quantify the hydraulic conductivity of a soil deposit, both in the laboratory and in the field. However, because the in-situ quantification of hydraulic conductivity using field tests can be time-consuming and expensive, an alternate in-situ method of determination of the hydraulic conductivity, based on the results of seismic piezocone tests (SCPTu), is proposed as a way to augment hydraulic conductivity data obtained from traditional in-situ and laboratory tests. The method supplements previous work that modeled pore water pressure dissipation behavior to obtain the coefficient of consolidation (c_h) (Torstensson 1977; Battaglio et al. 1981; Houlsby and Teh 1988; Burns and Mayne 1998), by using the measured shear wave velocity to estimate a constrained modulus for the soil deposit. The hydraulic conductivity can then be determined as a function of the unit weight of water, the coefficient of consolidation, and the constrained modulus.

Historically, a variety of methods have been proposed to evaluate the hydraulic conductivity from cone penetration testing. Schmertmann (1978) proposed an empirical correlation of hydraulic conductivity with piezocone dissipation data, based on the time for 50% and 90% dissipation of excess pore water pressure. Robertson et al. (1992) proposed an empirical correlation between the horizontal hydraulic conductivity and t_{50} observed in dissipation tests in clay soils, based on the values of t_{50} obtained with a Type-2 piezocone. Manassero (1994) performed piezocone tests in a cement-bentonite slurry wall in order to assess the construction quality and to evaluate the hydraulic conductivity of the barrier. The cone data were classified according to the method of Robertson et al. (1986) by choosing an equivalent soil type from that classification (for example, clay to clayey silt). The coefficient of consolidation was assessed from piezocone dissipation tests, using the method of Teh and Houlsby (1991), and the hydraulic conductivity was calculated based on the soil's coefficient of consolidation and constrained modulus. Parez and Fauriel (1988) presented a method for evaluation of the hydraulic conductivity of a soil deposit, based on correlation with t_{50} measured from Type-2 dissipation tests. The relationship was developed for soils ranging from gravel to clay, and the mean trend is approximately given as $k = (251 \cdot t_{50})^{-1.25}$, where k was in cm/s and t_{50} was in seconds. In order to estimate the hydraulic conductivity of the soil deposit, the existing methods require an evaluation of t_{50} from piezocone dissipation data. However, in cases where the pore pressure dissipation response is dilatory (increasing magnitude followed by a decrease to hydrostatic, or initial magnitude lower than hydrostatic followed by an increase to the hydrostatic value), the value of t_{50} is not defined (Fig. 1). The current paper proposes a technique, based on the results of a seismic piezocone test, to provide a methodology to estimate the hydraulic conductivity of a soil deposit for cases where the pore pressure exhibits either montonically decreasing or dilatory response.

Seismic Piezocone Penetration Testing

The seismic piezocone is a versatile, rapid, and relatively inexpensive tool for investigating soil deposits, and can provide measurements of soil strength and stiffness, in addition to consolidation and hydraulic conductivity parameters. The use of seismic piezocone testing to estimate hydraulic conductivity is not suggested as a replacement to traditional in-situ hydraulic conductivity testing, but as a source for supplementary information.

The seismic piezocone penetrometer is an electronic steel probe that provides four separate readings related to soil response. The

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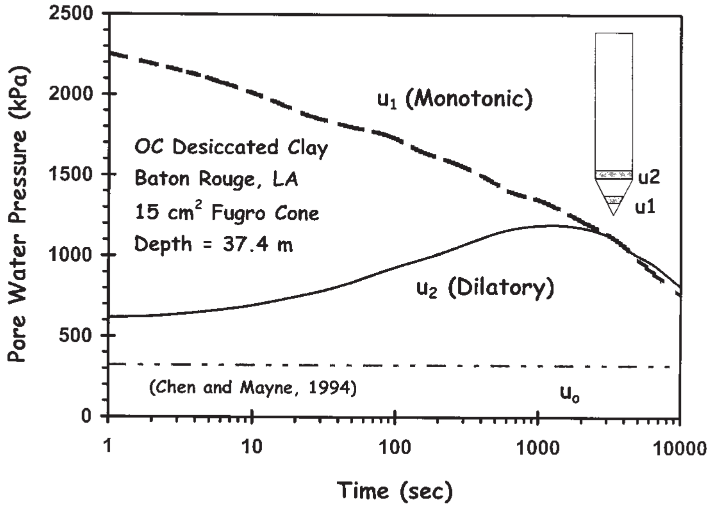

FIG. 1—*Monotonic and dilatory types of pore water pressure dissipation curves in overconsolidated deltaic clay (Chen and Mayne 1994).*

penetrometer is instrumented with strain gages to measure stresses exerted on the soil during penetration (q_c = tip resistance = stress exerted on cone tip and f_s = sleeve friction = stress exerted on cone sleeve). Filtered pressure transducers are included to measure the pore water pressures generated on the cone tip (u_1 or Type-1 pore pressure), on the cone shoulder (u_2 or Type-2 pore pressure), or behind the friction sleeve (u_3 or Type-3 pore pressure). The addition of an accelerometer or geophone to a cone penetrometer enables the direct measurement of the downhole shear wave velocity (V_s) with depth during the rod breaks. Seismic cone penetrometers are instrumented with either one or multiple accelerometers or geophones in the cone body to measure shear wave arrival times emanating from a distant source.

The geophysical portion of the seismic piezocone test is typically performed as a downhole test with the wave source generated at the surface (Campanella 1994). Surveys via the crosshole testing method (Standard Test Methods for Crosshole Seismic Testing, ASTM D4428/D4428M-00) are also possible (Baldi et al. 1989). During the pause in cone penetration while each successive rod is added, a seismic source is triggered and the travel time response is monitored on an oscilloscope that is connected through leads to the accelerometer or geophone within the cone body. The incremental distance over incremental time is used to calculate shear wave velocity, $V_s = \Delta x/\Delta t$, providing a pseudo-interval approach (Sully and Campanella 1995). A true-interval approach can be obtained if two accelerometers are located a fixed distance apart within the cone penetrometer (Burghignoli et al. 1991). Several tests, taking only about 15 s each for trigger and register, can be made at each depth for the pseudo-interval approach in order to verify repeatability of the measurements. A reverse strike may also be used for comparing polarized wave time records. Conventional applications of shear wave velocity data include the evaluation of dynamic stiffness for dealing with foundation vibration problems and the evaluation of site-specific amplification response spectra for earthquake events. Shear wave velocity can also be used to develop predictive relationships between V_s, depth, and the mass density of a soil deposit (Burns and Mayne 1996).

A single seismic piezocone sounding can provide all of the pertinent data necessary to evaluate the hydraulic conductivity of a soil deposit. By performing a downhole seismic test to obtain the shear wave velocity and a pore water pressure dissipation test to evaluate the coefficient of consolidation, the hydraulic conductivity for one-dimensional radial drainage can be determined through the following relationship:

$$k_h = \frac{c_h \gamma_w}{D'} \tag{1}$$

where k_h = the hydraulic conductivity for one dimensional horizontal drainage (L/T), c_h = the coefficient of consolidation (L^2/T), γ_w = the unit weight of water (M/L^3), and D' = the constrained modulus = $\Delta\sigma_z/\Delta\varepsilon_z$ from elastic theory (M/L^2). While the pore water drainage pattern during a piezocone dissipation test is quite complex, data from carefully instrumented pile sites indicate that the dominant direction of drainage is radial, away from the driven pile (Bjerrum and Johannessen 1961; Koizumi and Ito 1967; Randolph and Wroth 1979). Because the penetration of a piezocone has displacement and shear characteristics similar to that of a driven pile, it is assumed in this work that the drainage patterns for the two will be similar. Additionally, Tavenas et al. (1983) demonstrated that the horizontal hydraulic conductivity is only on average 4% higher than the vertical hydraulic conductivity for natural clays; consequently, the anisotropy in hydraulic conductivity can be assumed equal to approximately 1, except in highly stratified deposits and varved clay formations.

Examination of Eq 1 shows that both the coefficient of consolidation and constrained modulus need to be assessed. The coefficient of consolidation can be obtained from a pore water pressure dissipation curve performed during a pause in seismic piezocone testing, and constrained modulus can be evaluated from the shear wave velocity, as developed in this paper. The methodology presented here is a four-step procedure:

1. Evaluate the coefficient of consolidation from a pore pressure dissipation test using the method of Burns and Mayne (1998).
2. Measure the shear wave velocity using a downhole seismic piezocone test.
3. Evaluate the constrained modulus through an established correlation with the shear wave velocity.
4. Determine the one-dimensional hydraulic conductivity using Eq 1.

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Evaluation of the Coefficient of Consolidation from Piezocone Dissipation Testing

When a cone penetrometer is pushed into a fine-grained soil deposit, pore water pressure, typically with a magnitude that is larger than the hydrostatic value, is generated due to the physical displacement of soil and water and due to the shear interface between the soil and the cone body. The excess pore water pressure generated during a cone sounding in normally and lightly overconsolidated soils is positive; however, the magnitude of the pore water pressure can be less than hydrostatic or even negative in heavily overconsolidated and fissured soils due to the extremely large shear stresses involved. When the cone penetration is paused, the excess pore water pressure generated by insertion of the cone will return to the prevailing hydrostatic value if given enough time; the time required for return to the hydrostatic value is a function of the hydraulic conductivity and can be quite long in fine-grained soils. The change in the excess pore water pressure can be monitored by performing a dissipation test that measures pore water pressure as a function of time. For cases where the initial magnitude of pore water pressure is positive (in the u_1 or u_2 position), the dissipation curve exhibits consistently decreasing values; however, in cases involving overconsolidated geomaterials where the initial u_2 or u_3 magnitude is either positive or negative, the excess pressure can initially increase to a peak value and then subsequently decrease to the hydrostatic value, exhibiting dilatory behavior (Fig. 1).

During a dissipation test, the excess pore water pressures will decay as a function of a number of factors, including the coefficient of consolidation (c_h) of the soil deposit. A variety of models have been developed to evaluate c_h from piezocone dissipation testing using cavity expansion theory (Torstensson 1977; Battaglio et al. 1981), strain path method (Baligh and Levadoux 1986; Houlsby and Teh 1988), empirical methods (Sully and Campanella 1994), and other approaches (Tumay et al. 1982).

Methodology: Evaluation of c_h

The method of Burns and Mayne (1998), which was used to evaluate the coefficient of consolidation in this study, is based on a hybrid formulation from cavity expansion theory and critical state soil mechanics. A brief summary of the method is given here for clarity. In the formulation, the magnitude of the normal stress-induced pore water pressure is determined using cavity expansion, while the magnitude of the shear stress-induced pore water pressure is obtained from critical state soil mechanics. The model requires the effective stress friction angle, overconsolidation ratio, effective vertical stress, cone radius, rigidity index, and hydrostatic pore water pressure as input parameters in order to calculate the initial water pressure magnitudes at the beginning of the dissipation test. The magnitude of the generated pore water pressure is taken as the initial condition to develop an analytical solution to the consolidation equation. Boundary conditions include an impermeable boundary at the cone-soil interface, and no increase in the pore water pressure outside the zone influenced by spherical cavity expansion. The analytical solution is then used to evaluate the change in pore water pressure as a function of time, and the coefficient of consolidation can be determined by matching the model-evaluated dissipation curve with the field data through minimization of the sum of the squared errors (Santamarina and Fratta 1998). The second order differential equation can be solved either in a time-stepped finite difference mode or alternatively in closed-form. Rather than focus solely on one point from the dissipation, as is traditionally done with other approaches (i.e., t_{50}), the entire range of measured Δu with time are matched to produce c_h.

Methodology: Evaluation of the Constrained Modulus from Seismic Data

A database of clay soils from 13 different locations throughout the world with measurements of shear wave velocity (V_s) and constrained modulus (D') was compiled from data available in the literature (Mitchell et al. 1977; Lacasse and Lunne 1982; Holtz et al. 1985; Larsson 1986; Powell and Uglow 1988; Nash et al. 1992; Lambson et al. 1993; Brignoli et al. 1995; Burns and Mayne 1995; Jamiolkowski et al. 1995; Leroueil 1996). The database facilitated a correlation between: (1) the constrained modulus and the shear wave velocity (V_s), and (2) the constrained modulus and the small strain shear modulus, where $G_o = \rho_T \cdot V_s^2$ and ρ_T is the total soil mass density. A direct correlation between constrained modulus and shear wave velocity is shown in Fig. 2*a* ($n = 77$, $r^2 = 0.80$, *S.E.* $= 0.104$):

$$D' = 0.011 \cdot V_s^{2.43} \tag{2}$$

where D' is in kPa, V_s is in m/s, n = number of data sets, r^2 = coefficient of determination, and *S.E.* = standard error of the independent variable. While the data show a clear visual trend, it is important to note that there is a reasonable degree of scatter for the correlation with $r^2 = 0.80$. In all cases, application of the proposed relationship is conditional and should be based on site-specific calibration and applied with engineering judgment. Correlation with the shear wave velocity provides an estimate of the constrained modulus, and the conversion of shear wave velocity to the small-strain stiffness in terms of shear modulus (G_0) also facilitates a correlation of constrained modulus with shear modulus (Fig. 2*b*) ($n = 77$, $r^2 = 0.81$, *S.E.* $= 0.225$):

$$D' = 0.017 \cdot G_o^{1.12} \tag{3}$$

where D' and G_o are in kPa. Again, the correlation is somewhat scattered, with an $r^2 = 0.81$, so the correlation should be applied with caution.

The shear wave velocity data (m/s), depth z (meters), and total soil mass density (grams/cm^3) were converted to shear modulus through the following relationships (Mayne et al. 1999):

$$\rho_T = 1 + \frac{1}{0.614 + 58.7 \cdot \left(\dfrac{\log z + 1.095}{V_s}\right)} \tag{4}$$

$$G_o = \rho_T \cdot V_s^2 \tag{5}$$

While the loading mechanisms to measure constrained modulus and shear modulus are very different, the same material parameters influence both moduli, making the relation between the two rational.

Substitution of Eq 2 into Eq 1 yields the following relationship for hydraulic conductivity:

$$k_h = 90.9\,\frac{c_h \gamma_w}{V_s^{2.43}} \tag{6}$$

This form of the equation was used to evaluate hydraulic conductivity in ten clay deposits where reference values were available for verification.

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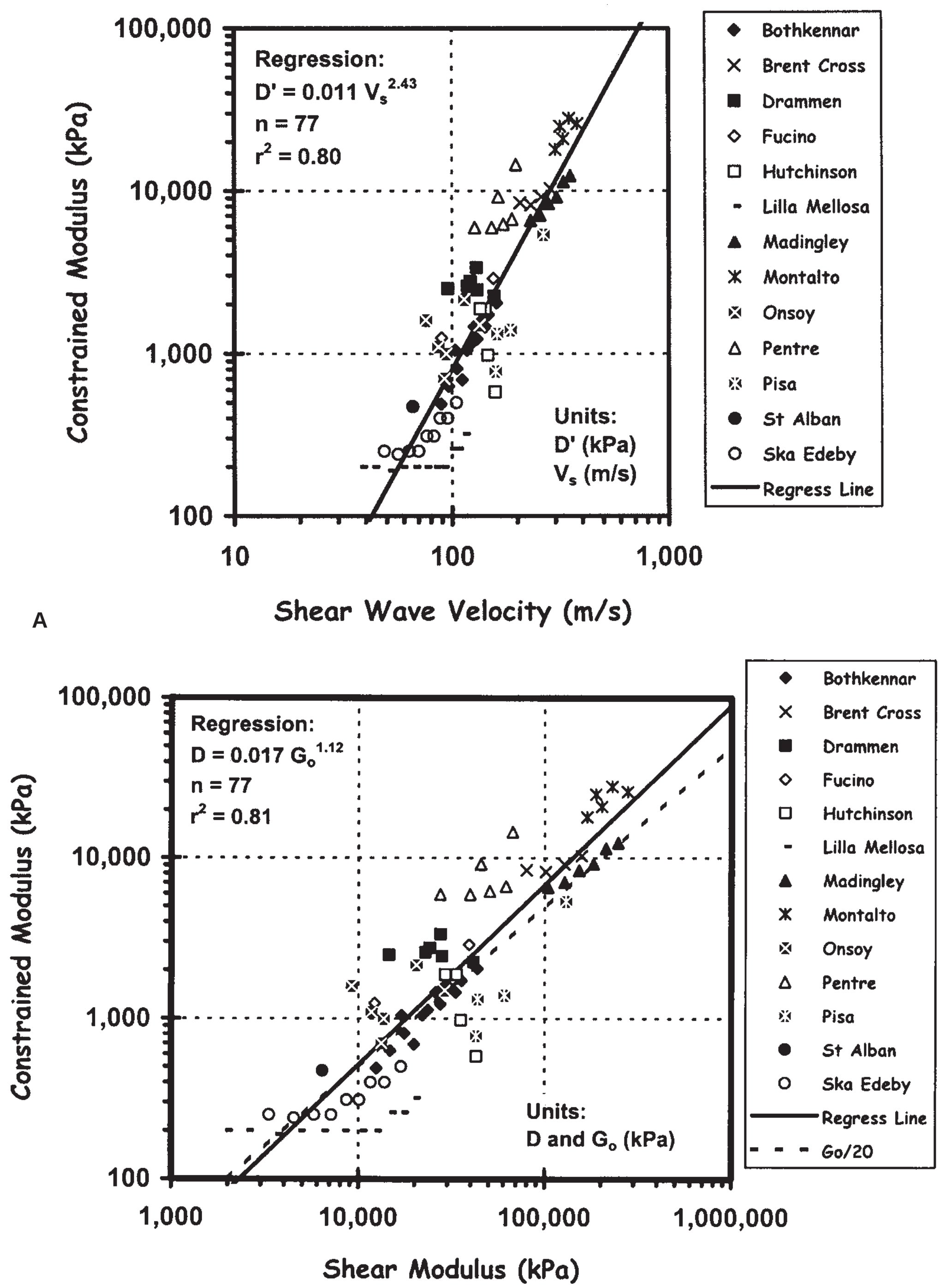

FIG. 2—*Constrained modulus correlations in clays: a) shear wave velocity; b) shear modulus.*

Results and Discussion

In order to assess the proposed approach, comparisons were made with the empirical method of hydraulic conductivity evaluation proposed by Parez and Fauriel (1988) and the hydraulic conductivity predicted using the shear wave velocity-constrained modulus correlation shown in Eq 2. A database of clay sites with known hydraulic conductivity, shear wave velocity, and piezocone dissipation tests was assembled for the evaluation (Table 1). The results of the comparison for the soft clays sites (Amherst (intact and noncrust), Bothkennar, Drammen, McDonald Farm, Onsøy, and St. Alban) are shown in Fig. 3. Only soft clay sites with monotonic dissipation records were used for comparison, due to the ambiguity in defining a value of t_{50} in dissipation tests that exhibit the dilatory behavior; that is, t_{50} is a necessary parameter for the Parez and Fauriel (1988) method of evaluation. While the two methods are based on very different approaches to evaluating the hydraulic conductivity, strikingly similar results are produced, with essentially identical predictions for soft clays at the Amherst, Bothkennar, and Onsøy sites. The proposed method gives better agreement with the laboratory measured values for the Drammen and McDonald Farm sites, while the method of Parez and Fauriel gives a better evalua-

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TABLE 1—*Database of clay sites for seismic piezocone evaluation of hydraulic conductivity.*

Test Site	Depth (m)	u_o (kPa)	σ_{vo}' (kPa)	OCR	ϕ'	q_t (kPa)	u_2 (kPa)	c_h[1] (mm^2/s)	V_s (m/s)	Predicted k (cm/s)	[2]Average Lab k_h (cm/s)	[2]Range Lab k_h (cm/s)	References
Amherst, MA (Noncrust)	12.2	120	73.2	1.8	33	690	449	0.83	145	4.1×10^{-7}	6.0×10^{-7}	3×10^{-7} to 1×10^{-6}	(DeGroot and Lutenegger 1994; Lally 1993; Lutenegger 1995; Martin and Mayne 1997)
Bothkennar, U.K.	12.0	107.8	96.2	1.4	33	898	499	0.2	130	1.3×10^{-7}	1.8×10^{-7}	4.8×10^{-8} to 3.3×10^{-7}	(Jacobs and Coutts 1992; Nash et al. 1992)
Drammen, Norway	19.5	179.0	121.0	1.1	34	1000	400	0.2	200	4.6×10^{-8}	1.0×10^{-8}	-	(Lacasse and Lunne 1982)
McDonald Farm, B.C.	20.0	181.5	178.5	1.1	35	1036	600	1.9	180	5.6×10^{-7}	4.0×10^{-7}	-	(Robertson et al. 1988; Sully 1991)
Onsøy, Norway	18.5	159.4	114.5	1.4	34	754	450	0.05	130	3.3×10^{-8}	1.3×10^{-7}	3×10^{-8} to 5×10^{-7}	(Lacasse and Lunne 1982)
St. Alban, Quebec	4.6	40.2	42.6	1.2	27	300	160	0.6	100	7.4×10^{-7}	3.0×10^{-7}	2×10^{-7} to 4×10^{-7}	(Roy et al. 1981; Roy et al. 1982)
Amherst, MA (Crust)	3.0	16.7	38.2	7.0	30.5	1369	80	0.4	215	7.7×10^{-8}	6.0×10^{-7}	3×10^{-7} to 1×10^{-6}	(DeGroot and Lutenegger 1994; Lally 1993; Lutenegger 1995; Martin and Mayne 1997)
Brent Cross, U.K.	12	115	117.8	31	20	2200	100	0.0067	285	6.5×10^{-10}	-	3×10^{-11} to 1×10^{-9}	(Burland and Hancock 1977; Lunne et al. 1986; Lunne et al. 1985)
Cowden, U.K.	17.2	95.0	283.4	3.4	24	898	575	0.2	186	5.4×10^{-8}	2.9×10^{-8}	1.93×10^{-8} to 4.44×10^{-8}	(Lehane and Jardine 1994; Lunne et al. 1985)
Madingley, U.K.[3]	5.8	37.2	72.8	35.0	26	2000	200	0.05	256	6.3×10^{-9}	1.4×10^{-9}	7.61×10^{-10} to 1.74×10^{-9}	(Coop and Wroth 1989; Lunne et al. 1986)

[1] from (Burns and Mayne 1998; Burns and Mayne *in review*)
[2](Robertson et al. 1992)
[3]80 mm Pile

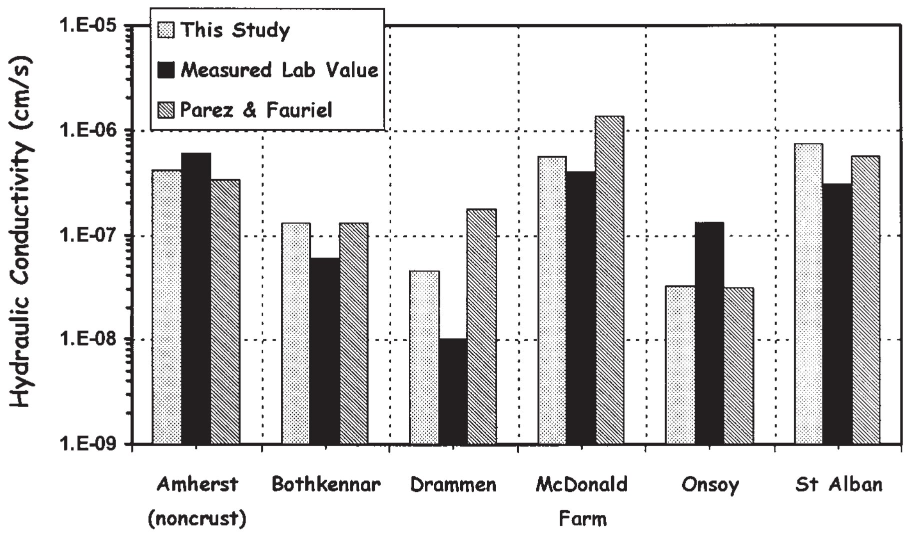

FIG. 3—*Comparison of model-evaluated hydraulic conductivity with the empirical method of Parez and Fauriel (1988) applicable to monotonic dissipations.*

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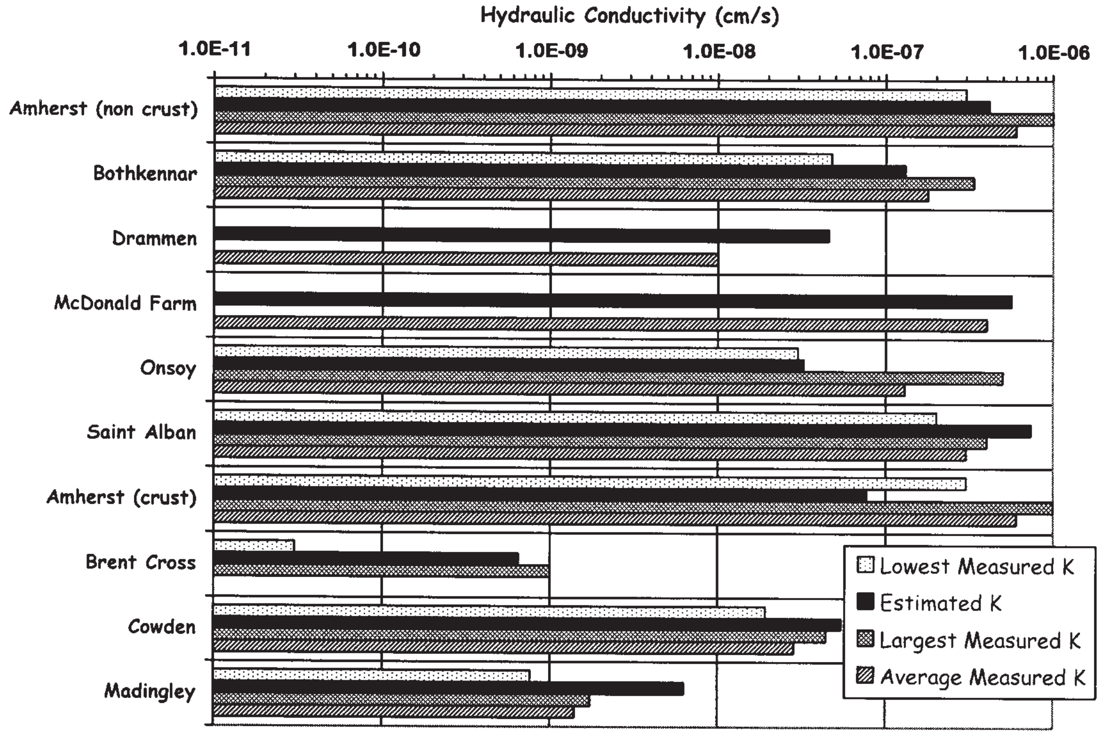

FIG. 4—*Lab-measured hydraulic conductivities versus SCPTu approach for monotonic and dilatory dissipation data.*

tion for the Saint Alban site. However, the differences in the predictions are relatively minor, especially considering that the Parez and Fauriel (1988) method is based on an empirical correlation solely with t_{50}, while the current model is based on independent evaluation of c_h, with measurement of V_s and correlation to constrained modulus (D').

While the Parez and Fauriel (1988) method provides good results for monotonically decaying pore water pressure dissipation in soft clays, it does not provide a methodology to determine hydraulic conductivity in stiff-to-hard clays that exhibit dilatory behavior. In order to further assess the proposed method, Eq 4 was used to evaluate the hydraulic conductivity for clay sites exhibiting both monotonic and dilatory pore water pressure decay. Site descriptions and data used for the evaluation are shown in Table 1, with the results of the prediction method shown graphically in Fig. 4. The average hydraulic conductivity reported for the sites, as well and the range of measured values are shown in the figure (Robertson et al. 1992). Note that only average values were reported for Drammen and McDonald Farm, while only a range of values was reported for Brent Cross.

For the most part, the proposed method tends to overpredict the laboratory-measured values of hydraulic conductivity; however, six estimates are within a factor of two times the average value reported. The remaining three sites with reported values of average hydraulic conductivity had estimates within a factor of 4.5 times the average. The estimate for four of the sites are within the range of reported hydraulic conductivities, while three estimates ranged between 1.3 and 3.6 times larger than the largest measured value and one estimate was 0.3 times the smallest value of hydraulic conductivity.

Conclusions

Using the SCPTu to measure the dissipation characteristics and shear wave profile of a soil deposit can provide the information necessary for the respective evaluation of the coefficient of consolidation and the constrained modulus, which in turn define the hydraulic conductivity. Comparison of the model evaluation with a database of 10 clay sites, composed of soft clays that exhibited monotonic pore pressure dissipation, as well as stiff clay sites that exhibited dilatory behavior, gave reasonable agreement with laboratory-determined values of hydraulic conductivity.

The proposed method is advantageous when compared to previous methods because piezocone dissipation curves that exhibit either monotonically decreasing or dilatory response (increase followed by decrease) can be accommodated with the same theoretical approach. While the proposed method can be useful for supplementing hydraulic conductivity data, critical applications will always require detailed, site-specific evaluation of the hydraulic conductivity.

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