In situ Measurement of Hydraulic Conductivity of Saturated Soils

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ABSTRACT: The hydraulic conductivity of saturated soilsvaries significantly from approximately 10^{-13} m/s for high plasticity clays to 1 m/s for clean, uniformly graded, coarse gravels. This very large range in possible values has resulted in the developed of numerous fieldmethods that cater to the soil type being tested and the anticipated hydraulic conductivity. The most common in situ method used in practice is the slug test as performed in open standpipe piezometers. Other methods include in situ dissipation tests and large scale pumping tests. This paper describes these various in situ measurement and analysis options and presents results obtained for a variety of soils including clays, silts, sands and gravels. The examples highlight the major influence of soil type on measured hydraulic conductivity values and also show the secondary influence of soil fabric and scale effects.

Keywords: hydraulic conductivity, in situ testing, slug tests, dissipation testing, pump tests

1. INTRODUCTION

The hydraulic conductivity (k) of soils is a key parameter that is required for analysis and design of numerous civil engineering works including: 1) seepage, 2) development of groundwater supplies, 3) consolidation, 4) drainage, 5) migration of contaminants, 6) flow through compacted landfill liners, etc. In porous media, such as soils, k is fundamentally based on Darcy's Law such that

$$q = kiA \tag{1}$$

where
q = rate of permeant flow (m^3/s),
k = hydraulic conductivity (m/s),
i = hydraulic gradient (m/m),
A = flow cross-sectional area (m^2).

When expressed in the form of Equation 1, the hydraulic conductivity (which is often referred to as the permeability or coefficient of permeability by geotechnical engineers) is a function of the soil physical characteristics (e.g., grain size and distribution) and permeant properties whereas the intrinsic permeability is only a function of the soil characteristics such that

$$q = K(\gamma/\mu)iA \tag{2}$$

where
K = intrinsic permeability (m^2),
γ = unit weight of permeant (kN/m^3),
μ = (absolute) viscosity of permeant (kN•s/m^2).

The term hydraulic conductivity is used in this paper, although all test results presented herein involved flow of water and the conversion between k and K is straight forward (e.g., K/k = 1.0×10^{-7} m•sfor water at 20°C). It is also common practice to convert the determined k values to a reference temperature such as 20°C or groundwater temperature (e.g., 10°C) by correcting for the viscosity of water at the test temperature to that of the selected reference temperature.

Possible k values for soils range from approximately 10^{-13} to 1 m/s which is a variation that is much greater than any other soil parameter. Because of this large range, no single measurement method is available that can accurately cover all possible values. Test equipment and procedures that are suitable for high k soils cannot be used for low k soils and vice versa. There are numerous in situ testing and laboratory methods, all of which have advantages and disadvantages. Scale effects are an important consideration in selecting a particular measurement method and the fact that some soils can be highly anisotropic is another important consideration.

Field techniques generally measure the horizontal hydraulic conductivity (k_h) and can involve pump tests or "permanent" installations such as predrilled or push-in open standpipe piezometers for conducting slug tests. Use of predrilled piezometers is very common although they can be time consuming to install and involve other problems including disposal of drill cuttings (if they are contaminated), positioning and alignment of the screen and proper construction of the sand pack and isolation seal. Push-in piezometers eliminate many of these problems and they are quicker to install but have the disadvantage of disturbing the soil during installation and hence altering the natural hydraulic response. In most clays and loose silts and sands they can be directly pushed in, while for denser soils they need to be hammered or vibrated in.

Other in situ tests such as the piezocone (CPTU) offer an indirect measurement of k_h through interpretation of the time rate of dissipation of excess pore pressures that are generated during penetration. The main advantage of this technique is that it can be relatively quick to perform if k_h of soil is not too low and hence allow detailed profiling in a short period of time. However, like push-in piezometers, the CPTU is a full displacement test and hence the results are influenced by disturbance of the soil during penetration. Furthermore, interpretation of the results requires assumptions to be made relative to soil compressibility and consolidation in order to compute k_h from the pore pressure dissipation data.

Given all the relative advantages and disadvantages of the many field and laboratory techniques available to estimate k, engineers are often faced with the difficult question of which technique(s) should be used to obtain appropriate values for design. This paper reviews commonin situ test equipment and interpretation methods available to measure the hydraulic conductivity of saturated soils including slug, dissipation, and pumping tests. Example results are presented from these tests conducted on various natural soils, most of which were glacially derived deposits.

2. IN SITU MEASUREMENT OF HYDRAULIC HEAD

Most field methods require measurement of the in situ hydraulic head which is commonly performed using an open standpipe piezometer. A piezometer is defined as a sealed device within the ground that responds to the pore water pressure around the location of the filter element and not at other elevations. An open standpipe piezometer can be placed in a predrilled borehole or pushed into the ground as shown in Figure 1. The key to installation in either case is that the filter element must be located at the depth of interest and be hydraulically sealed. In the case of placement within a predrilled borehole, bentonite is typically used as the seal material. A groundwater monitoring well is usually an open pipe equipped with a screened section that allows water to enter. They are typically designed for collection of groundwater samples but can also be used as a piezometer with the use of a short screen section that is hydraulically sealed.

The in situ hydraulic head can also be measured during a pause in piezocone testing as described in Section 4.1 but for silts and clays the time required to reach equilibrium conditions can be very lengthy and thus this method is not so common in practice in comparison to the widespread use of open standpipe piezometers.

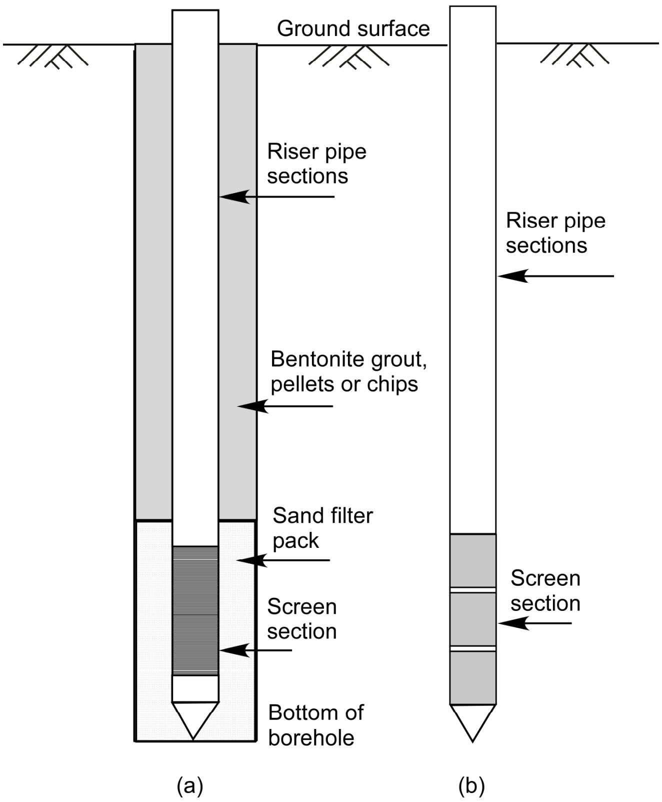

Figure 1 Schematic of a) open standpipe piezometer installed in borehole, b) push-in open standpipe piezometer(not to scale)

3. SLUG TESTS

3.1 Equipment and Test Procedures

Slug tests involve the displacement of an equilibrium hydraulic head condition within an open standpipe piezometer (e.g., Figure 1) and monitoring the recovery over time due to the head disturbance. There are large variations in possible test equipment and conditions. The "slug" can be achieved by placing and removing a metal rod into the piezometer, adding or removing water, using pneumatic pressure, etc. The choice largely depends on the expected response rate with metal slugs and use of water being common for most cases. More details on slug testing are presented by Butler (1997).

For conventional slug tests with an overdamped response (i.e., relatively slow recovery of the displaced hydraulic head), common interpretation theories include that of Hvorslev (1951), Bouwer and Rice (1976) and Cooper et al. (1967). In the straight forward Hvorslev method, the hydraulic conductivity is computed as

$$k = - 2.3am/F \qquad (3)$$

where
a = cross sectional area of standpipe (m^2),
m = slope of log head loss or gain versus time curve,
F = shape factor (m).

Chapius (1989) presents a detailed summary of available shape factors for variable geometric and flow direction considerations.

For sand and gravel aquifers with high hydraulic conductivity values (i.e., greater than 0.001 m/s) pneumatic testing is necessary (Figure 2). For such soils, recovery during a slug test typically follows an underdamped oscillatory response as the water level returns to equilibrium. The response time can be as fast as a few seconds depending on the soil hydraulic conductivity and geometry of the open standpipe piezometer. The rapid recovery of the displaced water prohibits the use of hand recordings and even low frequency portable transducer systems as are commonly used in lower permeability soils.In order to measure the oscillatory, underdamped response, the slug test must be initiated near instantaneously and the recording frequency must be high enough to accurately record the water level as it returns to equilibrium.

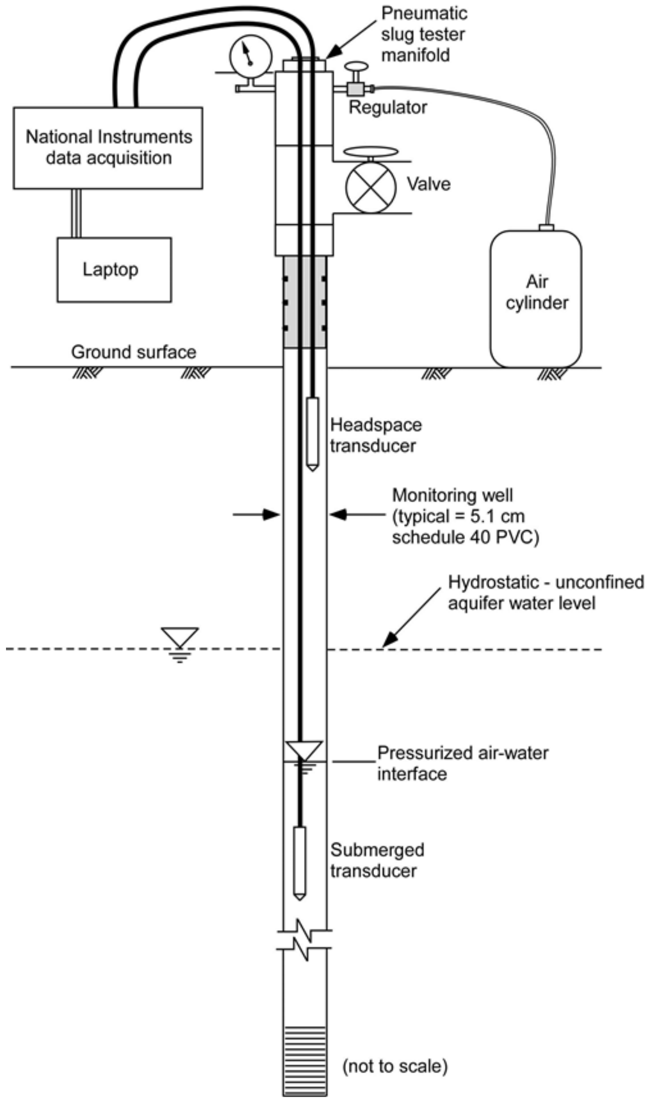

Figure 2 Schematic of pneumatic slug test system for testing of high hydraulic conductivity sands and gravels (from Dunaj et al. 2006)

The equipment shown in Figure 2uses a test manifold that is directly attached to the top of the riser pipe and is used for application and subsequent rapid release of air pressure. The air pressure is used to depress the water level by a prescribed constant amount (i.e., the 'slug'). The large ball value allows for near instantaneous release of the applied air pressure. Pressure transducers record the air pressure in the head space and water pressure within the water column inside the riser pipe during the test as the water level returns to equilibrium. The portable data acquisition system uses a PCMCIA 16-bit multifunction input/output analog to digital converter and Labview™ software for high speed processing control and real time data verification.

3.2 Example Results

3.2.1 Dense Glacial Till

Figure 3 plots a summary of slug test data collected from an array of 61 open standpipe piezometers installed in a dense glacial till drumlin located near Boston, MA, USA (Poirier et al. 2004). The general soil profile at the site consists of a weathered brown glacial till over an intact gray till. Both tills are of the same depositional unit and are very well graded and very dense. The tills contain all soil grain sizes from clay particles up to 1 m size boulders. The piezometers were installed at various depths ranging from near the ground surface down to the bottom of the deposit at approximately 30 meters. The tests were performed using aluminum slugs and portable data acquisition units consisting of pressure transducers that were placed in the wells prior to conduct of a test.

The large number of piezometers and slug tests performed at the test site provided an opportunity to analyze trends in k values due to factors such as interpretation method, test method, test repeatability over time, and till characteristics. It was found that hydraulic conductivity values estimated using the Hvorslev (1951) and Bouwer and Rice (1976) methods were approximately equal, there were no significant differences between k values determined from slug-in versus slug-out tests, and no variation in k values was observed over a four year period of conducting repeat slug tests in individual piezometers.

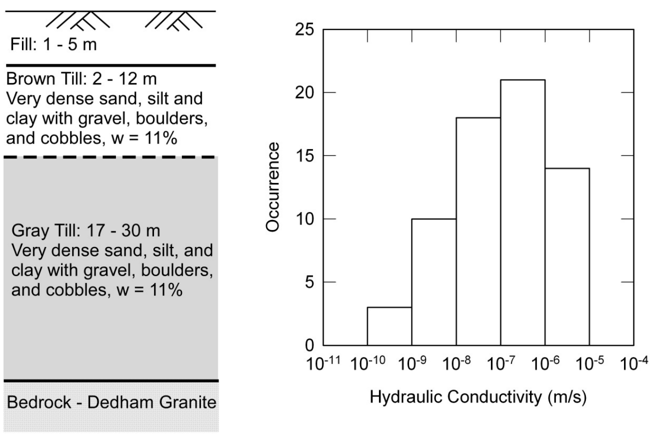

Figure 3 Soil profile for Boston, MA, USA, glacial till drumlin site and histogram of hydraulic conductivity data from slug tests in open standpipe piezometers (after Poirier et al. 2004)

Overall, the k values range 5 orders of magnitude from 10^{-10} to 10^{-5} m/s. Values generally increase with depth but no horizontal spatial trends were found. The largest difference in k is between the weathered and deep unweathered tills with close to two orders of magnitude difference in mean k values (≈ 1 x 10^{-8} vs. 2 x 10^{-6} m/s). Macrofeatures that were visible in test pits and dye infiltration tests are largely responsible for this difference. However, even within the unweathered gray till, the range in k values is large (3 orders of magnitude). This suggests a complex subsurface with randomly distributed preferential flow paths such as boulder fields, sand layers, and shear planes in the unweathered till. None of these potential features were indicated by any of the classification and index testing (water content, Atterberg Limits, grain size distribution, and SPT N values) which all suggested the site consisted of a uniform till layer.

3.2.2 Varved Clay

Figure 4 plots an example of a slug test conducted in a 50 mm diameter open standpipe piezometer in the varved clay deposit located at the University of Massachusetts (UMass) Amherst campus, MA, USA. This 25 meter thick lacustrine deposit consists of alternating silt and clay varves that range in thickness of a few millimeters near the top of the deposit to over several 100s of millimeters towards the bottom. The slug tests were performed by adding water to the piezometer as the 'slug' and the return to equilibrium conditions was monitored using a pressure transducer that was lowered into the piezometer below the equilibrium water level. Using the method of Horslev (1951) and the shape factor of Chapius (1989) for horizontal flow yields k_h = 2 x 10^{-8} m/s. This fairly high value for a 'clay' is due to the fact that the horizontal flow is largely dominated by the siltyvarves in the varved clay. For flow in the vertical direction the hydraulic conductivity is close to a factor of 10 less (DeGroot and Lutenegger 2003) because that flow is controlled by the clayey varves.

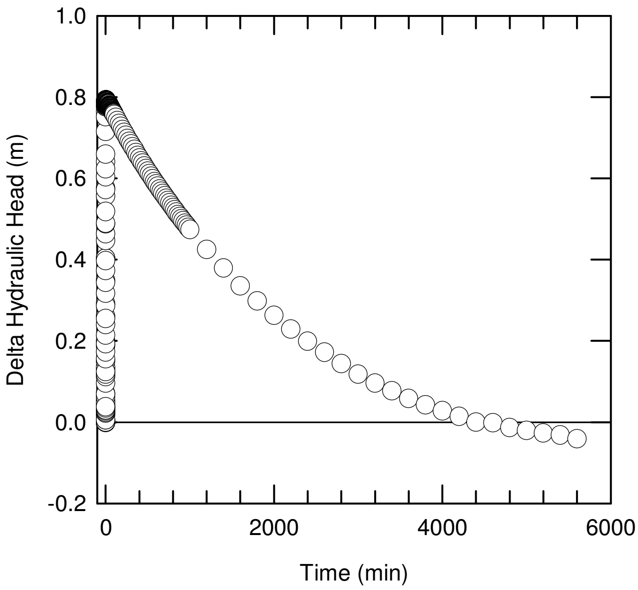

Figure 4 Example of slug test using added water as the slug conducted in a predrilled open standpipe piezometer in the UMass Amherstvarved clay deposit

In low hydraulic conductivity soils, recovery can take several days as shown for example in Figure 4, and in such cases the ambient or equilibrium hydraulic head can change due to environmental factors such as precipitation or evaporation. This is evident in the data presented in Figure 4 with the recovery data dropping below the initial equilibrium hydraulic head. In such cases, consideration of this change can be taken into account using for the example the theory presented by Ostendorf and DeGroot (2010).

Figure 5 plots a summary of tests conducted over the full depth of the varved clay deposit. The high value at a depth of 16 meters is in a zone within which the silt portion of the varves is very thick. The high values near the top of the deposit are those measured in the upper clay crust which is heavily desiccated and contains fissures and fractures thus increasing the hydraulic conductivity. In fact, tests conducted near the ground surface of such clay deposits are not only influenced by desiccation but also other features such as roots holes and small bores from earthworms and other organisms. Such macro features vary in size, shape and spatial distribution and hence scale effects (i.e., volume of soil being tested) are most likely to influence measured hydraulic conductivity values. Figure 6 presents results from a series of slug tests conducted at the UMass Amherst test site in predrilled piezometers with screens of different length to diameter ratios. Five piezometers were installed all with a nominal screen diameter of D = 76 mm but varying length (L) and all with the center of the screen located at a depth of 3.0 m, which is within the crust of the varved clay deposit. The results clearly show the influence of length to diameter ratio and suggest the greater influence of macro features as the length of the screen increases.The piezometer with the smallest L/D = 1 gives a k_h value that approaches that measured in a laboratory flexible wall permeametertest performed on a 76 mm diameter undisturbed Shelby tube sample with the specimen trimmed and oriented such that flow occurred parallel to the vertical direction of deposition (i.e., horizontal flow parallel to the varves).

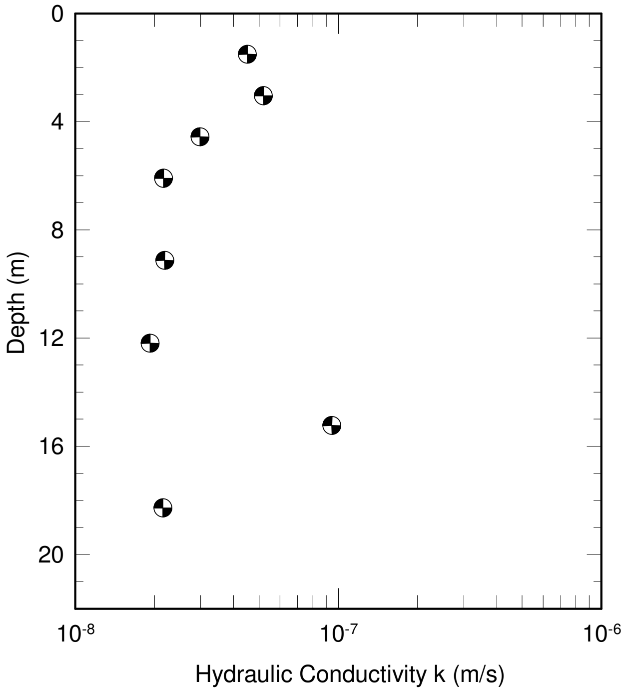

Figure 5 Hydraulic conductivity versus depth from slug tests performed in the UMass Amherstvarved clay deposit

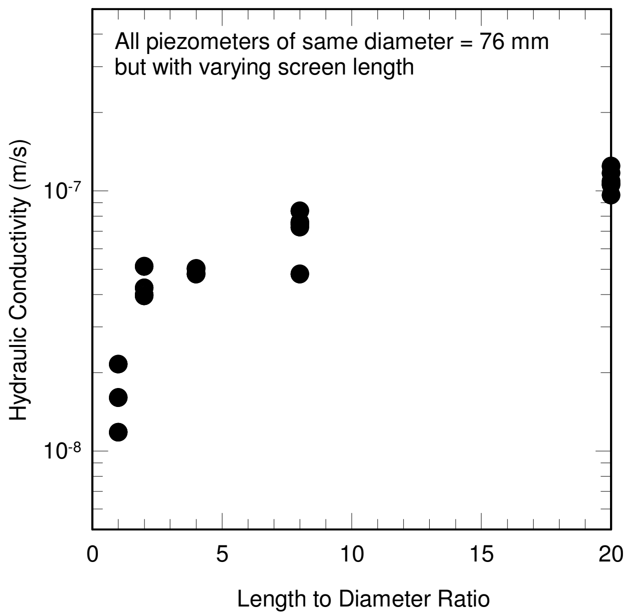

Figure 6 Influence of length to diameter ratioon measuredhydraulic conductivity from slug tests performed in open standpipe piezometers in the desiccated crust of the UMass Amherstvarved clay deposit (after DeGroot and Lutenegger 1994)

3.2.3 Gravelly Sand Deposit

Figure 7 presents results from pneumatic slug tests (using the equipment shown in Figure 2 with the submerged transducer at two different depths) performed in a glacial outwash gravelly sand aquifer located in southeastern MA, USA. The rapid and underdamped response is evident in the recorded data plotted in the Figure. Ostendorf et al. (2005) present a closed form slug test theory for high hydraulic conductivity soils that incorporates kinetic energy and casing friction. The solid line in Figures 7a and 7b display theory fit to the measured data showing that it captures the measured response very well. The resulting hydraulic conductivity values for these two tests equal k = 2.0 x 10^{-3} m/s.

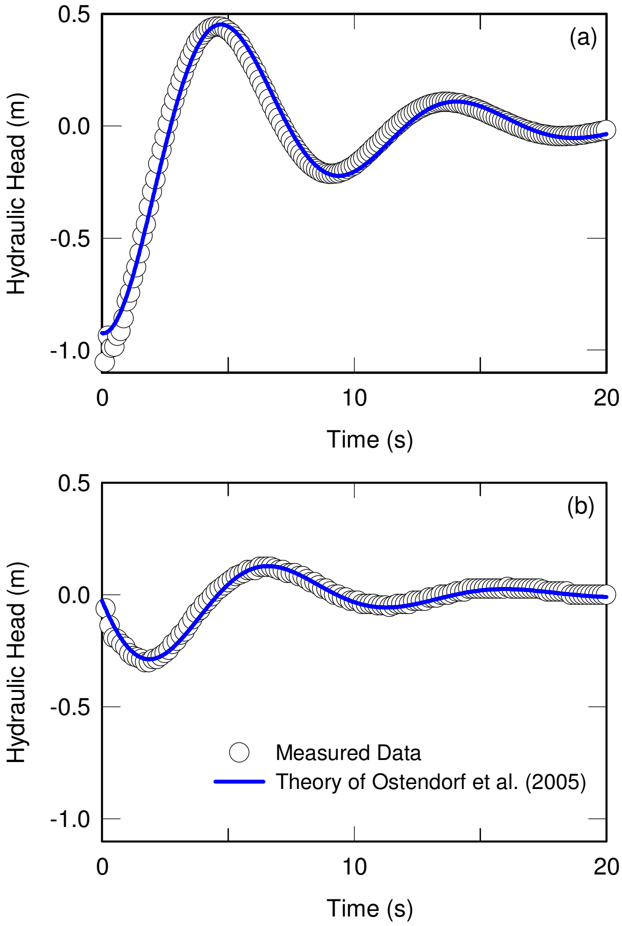

Figure 7 Results of pneumatic slug tests usinga 1 meter' amplitude 'slug' performed in a gravelly sand deposit with the submerged transducer located: a) 2 m below static water level, b) 20 m below static water level (after Ostendorf et al. 2005)

It is common practice for environmental investigations to install monitoring wells with long screen lengths. In such cases it is not possible to conduct conventional slug tests. To address this problem, Judge et al. (2008) developed a modified version of the pneumatic slug tester shown in Figure 2 for use in monitoring wells that are screened over their full length. The multilevel slug tester (MLST) uses a set of packers to isolate a 0.5 meter section within the well to conduct the slug test. Air pressure and pressure transducers are used to apply the slug and monitor the response over time. With this equipment tests can be conducted at various depths within a fully screened monitoring well.

The MLST system was used to conduct tests at various elevations within the 51 mm inside diameter of a continuously screened monitoring well at the gravelly sand deposit site. The well was approximately 25 meters long and constructed using 3 m long screened sections. Tests were typically conducted in intervals of 2.3 m starting from a depth of about 3 to 5 m beneath the static water table. Care was taken to avoid conducting tests at any of the approximately 0.3 m long solid casing joints that connected each of the 3 m screen sections. Tests were performed using air pressure to depress the static water level 1 m (i.e., the 'slug') and with the water pressure transducer at a shallow depth of 2 m below the static water level. During a sequence of full profile testing, tests were performed at multiple elevations while descending the well and in one case this profile was repeated by conducting tests at the same elevations while ascending back to the ground surface.

Figure 8 plots results from three sets of test profiles conducted in over a three month period. The range in hydraulic conductivity is about one order of magnitude from 0.0005 m/s to 0.005 m/s. It is noted that the difference in the hydraulic conductivity between descending versus ascending the well (November tests) is small at all test locations.The data plotted in Figure 8 highlight the significant advantage of using a MLST system in a continuously screened monitoring well. The fine resolution of data (with depth) in Figure 8 would require significantly greater cost and effort using single screened, depth specific, monitoring wells.

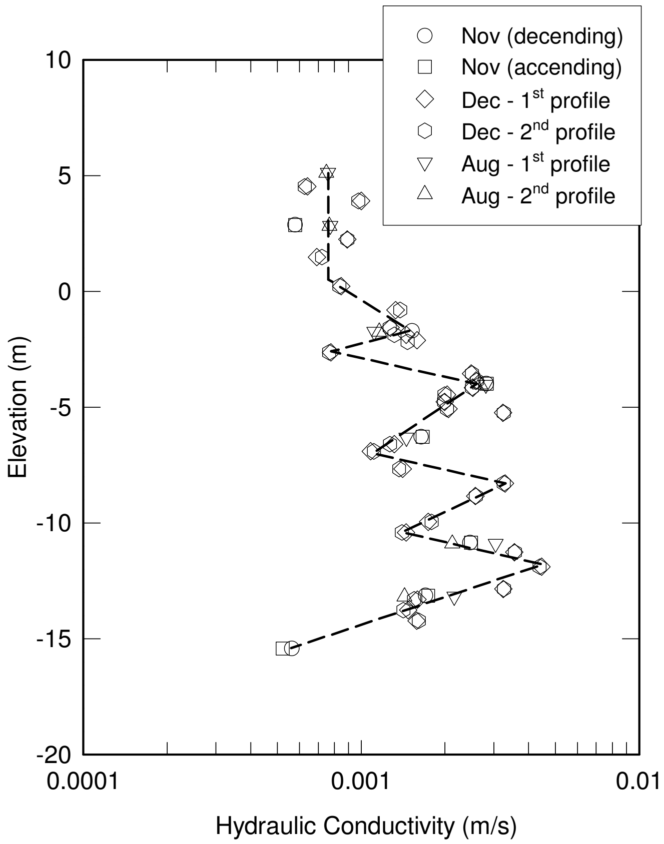

Figure 8 Hydraulic conductivity versus depth measured using the multilevel pneumatic slug test equipment in a continuously screened monitoring well installed in a gravelly sand deposit (from Judge et al. 2008)

Overall the data plotted in Figure 8 are somewhat scattered in the upper 5 m after which there appears to be four zones within which the hydraulic conductivity sharply increases and then decreases in a near linear fashion. The dashed line in Figure 8 is a visual interpretation of this zigzag pattern. These data suggest the existence of layers within the deposit with higher hydraulic conductivity. It is likely these are sections of the deposit that contain more coarse grained sand and gravel particles. Visual observation of split spoon samples and subsequent grain size data indicate the existence of such layers but the poor quality of the split spoon samples (e.g., often little to no recovery) did not allow for these observations to be quantified.

Layers of coarser grained particles and thus higher hydraulic conductivity is consistent with the geologic history of the deposit. This glacial fluvial deposit is composed of stratified sands and gravels deposited by glacial melt water streams. The information in Figure 8 is valuable for contaminant transport predictions. This site in question is the location of a highway drainage detention pond and receives large quantities of runoff in the winter months that are contaminated with deicing agents. Conventional practice is to model the subsurface movement of the contaminated water as a uniform plug flow whereas the data shown in Figure 8 indicate that significant fingering at various depths is the more likely scenario.

4. IN SITU PUSH PROBES

4.1 Piezocone Dissipation

4.1.1 Test Methods and Data Interpretation

Piezocone penetration in low to medium hydraulic conductivity soils generates excess pore pressures that will dissipate once penetration is paused. Monitoring of this dissipation using the piezocone pore water pressure transducer provides data that can be used to estimate the horizontal coefficient of consolidation and in turn the horizontal hydraulic conductivity. The pore pressure is most commonly measured with the filter element just behind the shoulder of the cone (i.e., the u_2 position) as shown in Figure 9a. For low to medium overconsolidation ratio (OCR) clays the u_2 pore pressure during penetration is typically well in excess of the equilibrium (often hydrostatic) pore pressure whereas in stiff high OCR clays it is often less than equilibrium. In either case dissipation results in return of the positive or negative shear induced pore pressure back to equilibrium.

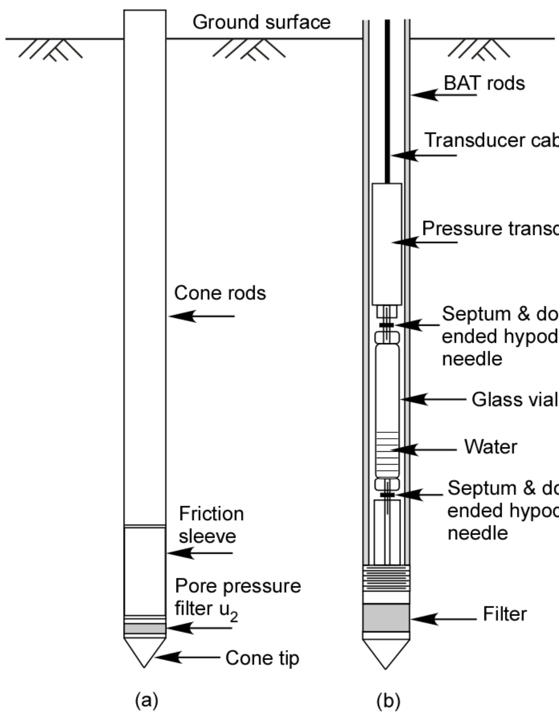

Figure 9 Schematic of a) Piezocone, b) BAT probe (not to scale)

CPTU dissipation testing is fairly well established in practice (e.g., Lunne et al. 1997), especially in the case for low to medium OCR clays for which the dissipation behavior and interpretation of the results is relatively straight forward. The practical challenge is that it can take a long to very long time for full dissipation to occur (e.g., high plasticity clays); in some cases greater than 24 hrs. As a result, interpretation methods have been developed that use part of the dissipation curve to estimate the hydraulic conductivity allowing the dissipation test to be cut short. Figure 10a plots an example of dissipation data for a low plasticity clay located in northeastern Massachusetts, USA for which the dissipation test was stopped after 40 minutes so as to continue on with the CPTU sounding. It is evident that full dissipation would have taken many more hours.

Figure 9b presents the same data but now in terms of the normalized pore pressure which is computed as

$$U_n = (u_t - u_0)/(u_i - u_0) \qquad (4)$$

where
U_n = normalized pore pressure,
u_t = pore pressure at any time t (kPa),
u_0 = in situ equilibrium pore pressure at the test depth (kPa),
u_i = initial pore water pressure during the dissipation test (kPa).

A common interpretation procedure is that of Teh and Housby (1991) which uses the time t_{50} required to reach 50% dissipation (i.e., U_n = 0.5 as shown for example in Figure 9b) to estimate the horizontal coefficient of consolidation (c_h) and k_h is estimated using Terzaghi's theory of consolidation such that

$$k_h = c_h \gamma_w m_h \qquad (5)$$

where
k_h = horizontal hydraulic conductivity (m/s),
c_h = horizontal coefficient of consolidation (m^2/s),
γ_w = unit weight of water (kN/m^3),
m_h = horizontal coefficient of volume change (m^2/kN).

Values of $m_h$are typically estimated from the CPTU tip resistance as described by Lunne et al. (1997).

Approximate CPTU based estimates of k can also be made using the soil behavior charts of Robertson (1990) and the Lunne et al. (1997) suggested range of k for each zone in those charts.

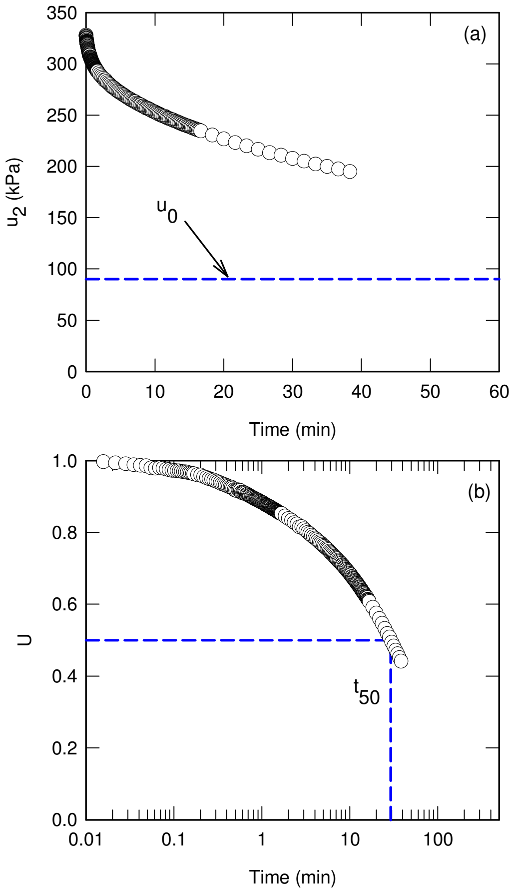

Figure 10 Partial CPTU u_2 dissipation data for a low plasticity clay

4.1.2 CPTU Dissipation – Example Results

Varved Clay

Figure 11 plots a series of dissipation tests conducted at various depths in the varved clay deposit at the UMass Amherst campus. Interpretation of these data using Teh and Housby (1991) via Equation 5 results in k_h values that range from $1.0x10^{-9}$ to $1.0x10^{-8}$ m/s. These data are plotted in Figure 11 together with the slug test results from Figure 5. In addition,k values determined in the laboratory from flexible wall permeameter tests on vertically (k_v) and horizontally (k_h) oriented specimens trimmed from fixed piston Shelby tube samples are also plotted.

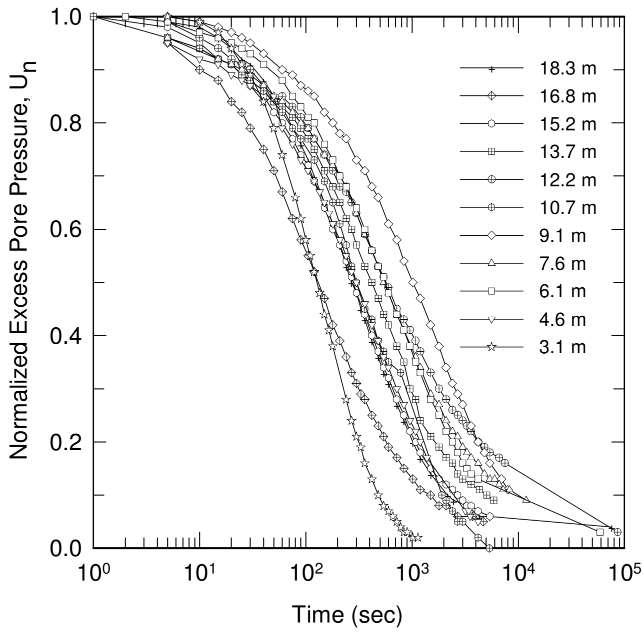

Figure11 Plots of CPTU pore pressure dissipation for various test depths at the UMass Amherst varved clay test site (from DeGroot and Lutenegger 1994)

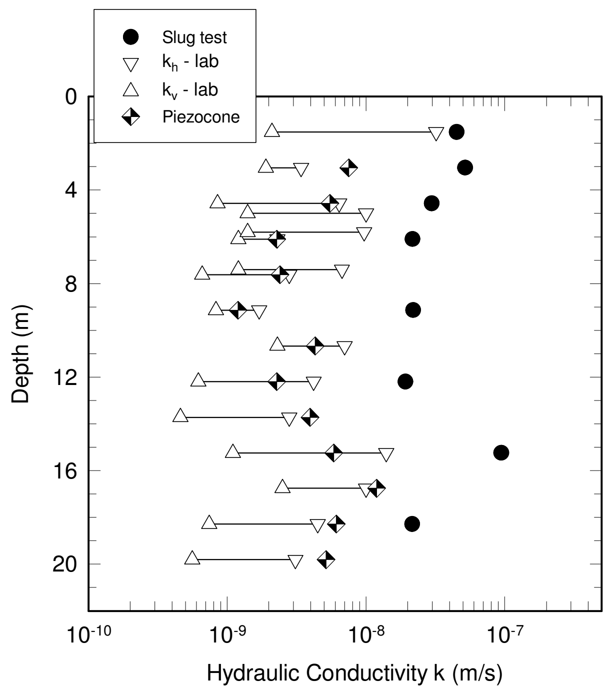

Figure 12 In situ and laboratory measurement of hydraulic conductivity for the UMass Amherst varved clay (from DeGroot and Lutenegger 2003)

The data presented in Figure 12 provide an opportunity for comparing various hydraulic conductivity measurement methods for a highly anisotropic soil. The highest values of k were measured by slug tests conducted in the open standpipe piezometers and lowest values by the small size laboratory flexible wall specimens with vertical flow. Flow in the open standpipe piezometers involved a much larger volume of soil and presumably incorporated more in situ macrofeatures than the small scale laboratory specimens. The piezocone tests (data from Figure 11) in this case are somewhat similar to the horizontal flow laboratory flexible wall specimens. Piezocone dissipation tests involve a smaller volume of soil than the in situ slug tests and also alter the natural in situ soil state by remolding/smearing of the soil adjacent to the piezocone surface during penetration. As expected for this varved clay, the hydraulic conductivity measured with flow perpendicular to the varves gave lower values than for flow parallel to the varves based on the laboratory flexible wall tests. The corresponding anisotropy ratio ($r_k = k_h/k_v$) for the flexible wall tests ranges from approximately 2 to 14 with an average of 6 whereas the ratio of the slug tests to the flexible wall tests with vertical flow represents the highest anisotropy ratioand ranges from 20 to 85 with an average of 35.

Glaciolacustrine Deposit

CPTU soundings in soft clays such as the varved clay deposit described in the previous section always results in positive excess pore pressures and hence dissipation curves of the type shown in Figure 11. The situation is more complicated for stiff, high OCR clays and dense silts which generate negative (i.e., below u_0) u_2 pore pressures. Lunne et al. (1997) presents some examples of such and Burns and Mayne (1998) present an interpretation method for when a dilatory dissipation response is measured. Partial drainage during penetration, which can occur in 'intermediate soils' such as silts is another complicating factor and is the subject of current research (e.g., DeJong et al. 2012).

Figure 13 presents a CPTU profile that highlights some of these issues. It was performed in a glaciolacustrinedeposit located in eastern Massachusetts, USA that consists of uniform sand (approximately down to 10 meters) over a highlystratified silt that contains numerous thin interbedded layers of silt, sand, and clay (with an increase in the presence of clay layers with depth). This layering was visually confirmed from inspection of undisturbed fixed piston tube samples. The CPTU q_{net} data show large variations in the lower 10 meters and the u_2 data show especially large variations over very small changes in depth.

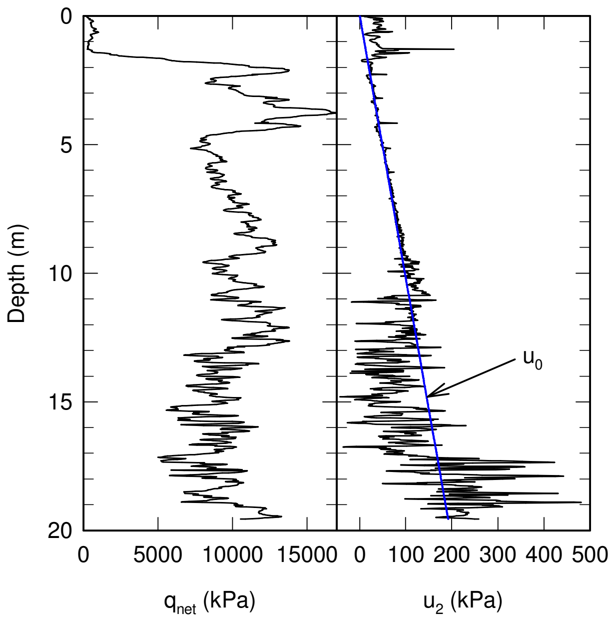

Figure 13 CPTU q_{net} and u_2 data versus depth of highly stratified glaciolacustrinedeposit in eastern Massachusetts

Dissipation tests were conducted approximately every meter starting below a depth of 10 meters. Figure 14 plots two examples, one at a depth of 14.7 m (Figure 14a) and one at a depth of 19.1 m (Figure 14b). The 14.7 meter test is in the zone of the deposit in which u_2 is well below hydrostatic whereas the 19.1 meter test is in the zone with more clayey layers and hence $u_2$in this case starts well above hydrostatic. The dissipation behavior is complicated since large shear induced u_2 pore pressure generated in the dense silt and silt layers are influence by positive excess pore pressures generated just in front of the cone tip and presumably rapidly migrate towards the u_2 position once the push is stopped and dissipation starts (e.g., Figure 14a). Furthermore, the highly stratified nature of the deposit means that pore pressure dissipation is not fully dictated by the soil layer at the u_2 location during dissipation. The data in Figure 14b show high initial excess u_2 at the start of dissipation, indicative of a clay layer, but the dissipation is very rapid ($t_{50} \approx 5$ sec) and presumably the excess pore pressure is readily dissipating to the adjacent silt and sand layers.

As a practical matter the key issue for such a highly stratified soil deposit such as that shown in Figure 13is whether the design application involves vertical or horizontal flow. In spite of the variations in the dissipation response within the depth interval of 10 to 20 m as shown for example in Figure 14, the practical reality is that dissipation takes place relatively quickly. The t_{50} values are all quite small (in the range of 10 to 20 s)and imply that the soil is effectively draining as a sandy silt or silty sand with k approximately equal to 10^{-7} to 10^{-6} m/s (Mayne 2007). Such values would be relevant for horizontal drainage whereas vertical drainage in the deeper part of the deposit with more clay layers (between 17 and 20 m depth) is likely to have much lower hydraulic conductivity values. Laboratory flexible wall permeameter test performed on undisturbed fixed piston Shelby tube samples resulted in $k_v = 10^{-8}$ to 10^{-7} m/s for samples collected in the depth interval of 17.5 to 21 m.

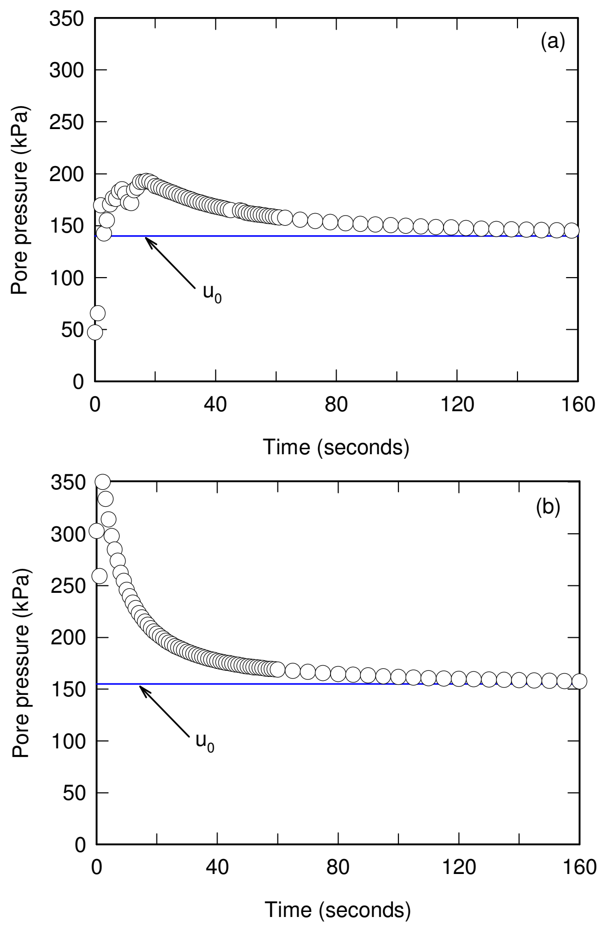

Figure 14 CPTU dissipation data for soil profile shown in Figure 12 at depths of a) 14.7 m and b) 19.1 m

4.2 BAT PROBE

The BAT probe, as shown schematically in Figure 9b, was originally developed by Torstensson (1984) for collection of hermetically sealed groundwater samples. The BAT system uses an evacuated glass vial that collects a ground water sample upon piercing a rubber septa with a hypodermic needle (which is done by lowering a set of weights down the center of the BAT rods when the probe has reached the depth of interest). It was not long thereafter that the BAT system was modified to accommodate a pressure transducer as shown in Figure 8b. This then allowed the BAT probe to be used to monitor the pressure in the glass vial during the inflow of water if using an evacuated vial or during outflow of water if the vial is first filled with water and pressurized at the ground surface before placing down the BAT rods. Schellingerhout (2000) developed a series of equations that uses the volume of the vial and the pressure-time response data via Boyle's Law and Hvorslev's slug test equation (Equation 3) to estimate the hydraulic conductivity. Wilson and Campanella (1997) and Campanella (2008) describe a modification to the BAT probe using Swagelok quick-connect valves instead of the small bore hypodermic needle that restricted flow in sands. Campanella (2000) also noted that while in clay soils performing flow-in or flow-out test makes no difference, it is very important to only perform flow-out tests in silty, sandy soils to avoid migration of fines clogging the filter element as would occur during flow-in tests and result in an incorrect lower hydraulic conductivity.

5. AQUIFER (PUMP) TESTING

Of all in situ test methods, aquifer (pump) testing involves the largest volume of soil and is typically used to evaluate the potential yield of drinking water aquifers (i.e., storativity S and transmissivity T). The test method requires a pumping well and several piezometers placed at varying radial distances from the pumping well. The piezometers, which are most often open standpipe, are used to monitor the hydraulic head drawdown of the aquifer during pumping. The subsurface conditions, test conditions, and measured data control model selection and data analysis. Variations include: pumping conditions (steady state, transient, well penetration), aquifer properties (confined, unconfined, leaky, homogeneous, etc.), and measured hydraulic response data sets (temporal, spatial). The classical Theis (1935) solution models the transient response of a homogeneous, isotropic, confined aquifer for radial flow towards a pumping well. Estimates of T and the aquifer thickness b are used to compute k equal to T/b. Other models were developed that vary the Theis model assumptions allowing for analysis of more complex conditions. One example includes that of Jacob (1946) for the steady state response of a leaky aquifer, i.e., an aquifer with an overlying low hydraulic conductivity aquiclude that leaks water into the much more permeable aquifer). Through the use of a cascade (multiple time and length scales) of hydraulic models, global and location specific (i.e., at piezometer screens) hydraulic conductivity values can be estimated from aquifer testing data.

Figure 15 presents an example of steady state piezometer data collected over a two year period for a drinking water supply well located in a sand and gravel aquifer in eastern Massachusetts (MA), USA. The aquifer is hydraulically protected by an overlying 10 m thick silt aquiclude, which in turn is overlain by a contaminated unconfined sand aquifer. The data clearly indicate hydrostatic conditions within the upper unconfined aquifer. The main water supply aquifer receives recharge laterally over a significant distance from the supply well while the lower hydraulic conductivity aquiclude prevents draw-in of contaminated water from the upper unconfined aquifer. These data were modeled by Ostendorf et al. (2009) using Jacob's (1946) solution for steady well flow in a leaky aquifer as represented by the solid line in Figure 15. These data give a global estimate of the aquifer transmissivity (T = 0.008 m^2/s) which for an approximate 10 m aquifer thickness implies a k_h value for the aquifer of 8.0 x 10^{-4} m/s and a global aquiclude vertical hydraulic conductivity k_v = 2.3 x 10^{-10} m/s. The aquiclude low k_v value in this case implies that aquiclude leakage to the aquifer is negligible and confirms that it is hydraulically protecting the aquifer from the contaminated upper unconfined aquifer.

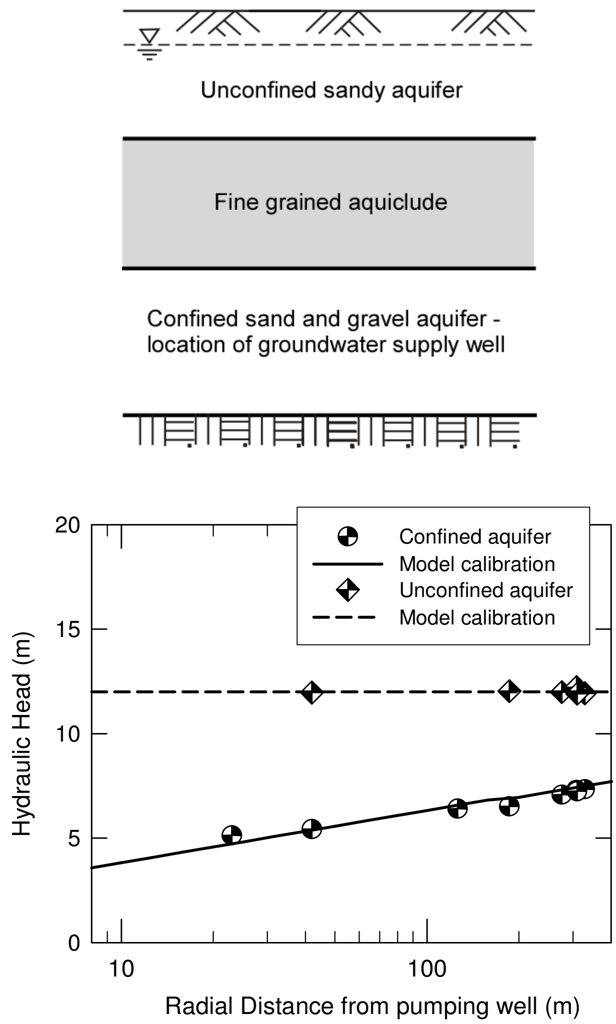

Figure 15 Soil profile and steady state hydraulic head in open standpipe piezometers located in an upper shallow unconfined aquifer and a lower confined aquifer located in eastern Massachusetts (from Ostendorf et al. 2009)

Examination of individual piezometer transient response during episodic well pump shut-downs can yield local estimates of aquifer and aquiclude hydraulic conductivity. In this case the Theis (1935) model for demand driven pump testing is used to analyze the aquifer data and Figure 16 plots an example of this for two piezometers located at varying radial distances from the pumping well. The location specific estimates of k_h yield values of 6.4 x 10^{-4} to 1.2 x 10^{-3} m/s in comparison to the global aquifer estimate of 8.0 x 10^{-4} m/s from the steady state data.

6. SUMMARY OF METHODS

Table 1 presents a summary of the methods described in the previous sections and highlights aspects of the test procedure, advantages, and disadvantages. Also included for reference are comments on laboratory testing with more details given in Daniel (1994). Ideally, comprehensive site characterization programs should combine in situ and laboratory test methods. This allows for measurements at large scale via in situ tests and under well controlled test conditions via laboratory tests, but at a smaller scale. For high hydraulic conductivity soils, large scale pump tests are a good but often expensive option. Slug tests are quite versatile as they can be conducted in a wide range of soils provided open standpipe piezometers can be and are properly installed. Laboratory testing of high hydraulic conductivity coarse grained soils rely on collected samples being reconstituted to representative conditions (with the most important variable being density) in the laboratory. For low hydraulic conductivity fine grained soils, flexible wall testing of representative good quality undisturbed samples is the best option followed by either direct or indirect measurement of k via the either incremental load or constant rate of strain consolidation tests.

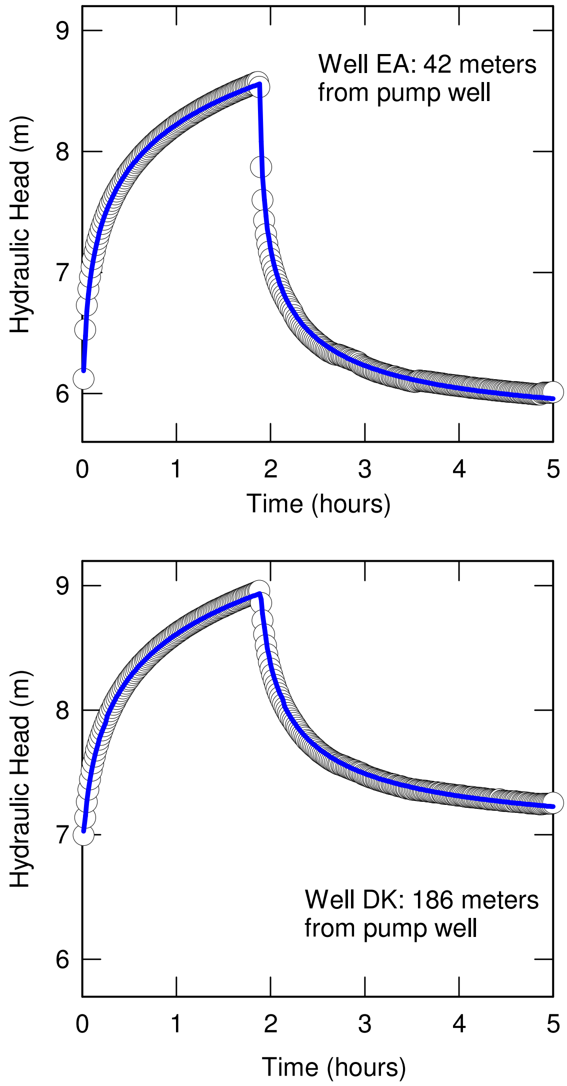

Figure 16 Observed (symbols) and calibrated (line) hydraulic head in two open standpipe piezometers during a cyclic pump test in a confined aquifer in eastern MA (after Ostendorf et al. 2009)

7. CONCLUSIONS

There are numerous in situ and laboratory methods available for measurement of the hydraulic conductivity of soils, which can vary from approximately 10^{-13} m/s for high plasticity clays to 1 m/s for clean, uniformly graded, coarse gravel. The selection of appropriate equipment and test procedures should depend on the anticipated hydraulic conductivity. For in situ testing, slug tests conducted in open standpipe piezometers offer the greatest versatility whereas pump tests involved the largest volume of soil. Accurate laboratory testing depends greatly on the ability to collect representative undisturbed test samples. This is generally not possible for coarse grained soils and laboratory testing for such soils relies on accurate reconstitution of test samples. For fine grained soils, collection of good quality undisturbed samples is possible and testing of such samples in flexible wall permeameters or indirectly via consolidation testing are good options. Scale effects and anisotropy are important considerations and selected test methods for a project should take these factors into consideration.

Table 1 Summary of evaluation of in situ and laboratory test methods for measurement of hydraulic conductivity of saturated soils (afterDeGroot and Ostendorf 2010)

Test Method	**Field or Lab**	**Remarks**
Pump test	Field k_h	- captures large scale features - can be time consuming and expense to perform - need array of piezometers to monitor hydraulic head response - deposit thickness needs to be known
Slug test	Field k_h	- need open stand pipe piezometer, ideally with short screen length (say < 1.5 m) - can also serve as a groundwater monitoring well - underdamped response requires more sophisticated field methods - changes in background head influence long term tests in low k soils
CPTU dissipation test	Field k_h	- conveniently conducted during a CPTU sounding. - can take long for low hydraulic conductivity soils (> 1 day) - analysis of dissipation data results in estimate of c_h and need to estimate soil compressibility in order to compute k_h - works best in homogenous soils with dissipations times > 1 min
Rigid wall permeameter	Lab k_v	- direct measurement of k under well controlled laboratory boundary conditions - used mostly for reconstituted granular soils (and compacted soils) - cannot back pressure saturate
Flexible wall permeameter	Lab k_v or k_h	- direct measurement of k under well controlled laboratory boundary conditions - can test undisturbed tube samples of fine grained soils and oriented specimens for vertical flow (k_v) or horizontal flow (k_h) relative to direction of deposition - can back pressure saturate and control effective stress state

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