Groundwater Investigations

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Subsurface Investigations
— Geotechnical Site Characterization

Reference Manual






National Highway Institute

CHAPTER 6.0

GROUNDWATER INVESTIGATIONS

6.1 GENERAL

Groundwater conditions and the potential for groundwater seepage are fundamental factors in virtually all geotechnical analyses and design studies. Accordingly, the evaluation of groundwater conditions is a basic element of almost all geotechnical investigation programs. Groundwater investigations are of two types as follows:

- Determination of groundwater levels and pressures and
- Measurement of the permeability of the subsurface materials.

Determination of groundwater levels and pressures includes measurements of the elevation of the groundwater surface or water table and its variation with the season of the year; the location of perched water tables; the location of aquifers (geological units which yield economically significant amounts of water to a well); and the presence of artesian pressures. Water levels and pressures may be measured in existing wells, in boreholes and in specially-installed observation wells. Piezometers are used where the measurement of the ground water pressures are specifically required (i.e. to determine excess hydrostatic pressures, or the progress of primary consolidation).

Determination of the permeability of soil or rock strata is needed in connection with surface water and groundwater studies involving seepage through earth dams, yield of wells, infiltration, excavations and basements, construction dewatering, contaminant migration from hazardous waste spills, landfill assessment, and other problems involving flow. Permeability is determined by means of various types of seepage, pressure, pumping, and flow tests.

6.2 DETERMINATION OF GROUNDWATER LEVELS AND PRESSURES

Observations of the groundwater level and pressure are an important part of all geotechnical explorations, and the identification of groundwater conditions should receive the same level of care given to soil descriptions and samples. Measurements of water entry during drilling and measurements of the groundwater level at least once following drilling should be considered a minimum effort to obtain water level data, unless alternate methods, such as installation of observation wells, are defined by the geotechnical engineer. Detailed information regarding groundwater observations can be obtained from ASTM D 4750, “Standard Test Method For Determining Subsurface Liquid Levels in a Borehole or Monitoring Well” and ASTM D 5092 “Design and Installation of Groundwater Wells in Aquifers”.

6.2.1 Information on Existing Wells

Many states require the drillers of water wells to file logs of the wells. These are good sources of information of the materials encountered and water levels recorded during well installation. The well owners, both public and private, may have records of the water levels after installation which may provide extensive information on fluctuations of the water level. This information may be available at state agencies regulating the drilling and installation of water wells, such as the Department of Transportation, the Department of Natural Resources, State Geologist, Hydrology Departments, and Division of Water Resources.

6.2.2 Open Borings

The water level in open borings should be measured after any prolonged interruption in drilling, at the completion of each boring, and at least 12 hours (preferably 24 hours) after completion of drilling. Additional water level measurements should be obtained at the completion of the field exploration and at other times designated by the engineer. The date and time of each observation should be recorded.

If the borehole has caved, the depth to the collapsed region should be recorded and reported on the boring record as this may have been caused by groundwater conditions. Perhaps, the elevations of the caved depths of certain borings may be consistent with groundwater table elevations at the site and this may become apparent once the subsurface profile is constructed (see Chapter 11).

Drilling mud obscures observations of the groundwater level owing to filter cake action and the higher specific gravity of the drilling mud compared to that of the water. If drilling fluids are used to advance the borings, the drill crew should be instructed to bail the hole prior to making groundwater observations.

6.2.3 Observation Wells

The observation well, also referred to as piezometer, is the fundamental means for measuring water head in an aquifer and for evaluating the performance of dewatering systems. In theory, a "piezometer" measures the pressure in a confined aquifer or at a specific horizon of the geologic profile, while an "observation well" measures the level in a water table aquifer (Powers, 1992). In practice, however, the two terms are at times used interchangeably to describe any device for determining water head.

The term "observation well" is applied to any well or drilled hole used for the purpose of long-term studies of groundwater levels and pressures. Existing wells and bore holes in which casing is left in place are often used to observe groundwater levels. These, however, are not considered to be as satisfactory as wells constructed specifically for the purpose. The latter may consist of a standpipe installed in a previously drilled exploratory hole or a hole drilled solely for use as an observation well.

Details of typical observation well installations are shown in Figure 6-1. The simplest type of observation well is formed by a small-diameter polyvinyl chloride (PVC) pipe set in an open hole. The bottom of the pipe is slotted and capped, and the annular space around the slotted pipe is backfilled with clean sand. The area above the sand is sealed with bentonite, and the remaining annulus is filled with grout, concrete, or soil cuttings. A surface seal, which is sloped away from the pipe, is commonly formed with concrete in order to prevent the entrance of surface water. The top of the pipe should also be capped to prevent the entrance of foreign material; a small vent hole should be placed in the top cap. In some localities, regulatory agencies may stipulate the manner for installation and closure of observation wells.

Driven or pushed-in well points are another common type for use in granular soil formations and very soft clay (Figure 6-1b). The well is formed by a stainless steel or brass well point threaded to a galvanized steel pipe (see Dunnicliff, 1988 for equipment variations). In granular soils, an open boring or rotary wash boring is advanced to a point several centimeters above the measurement depth and the well point is driven to the desired depth. A seal is commonly required in the boring above the well point with a surface seal at the ground surface. Note that observation wells may require development (see ASTM D 5092) to minimize the effects of installation, drilling fluids, etc. Minimum pipe diameters should allow introduction of a bailer or other pumping apparatus to remove fine-grained materials in the well to improve the response time.

Local or state jurisdictions may impose specific requirements on "permanent"observation wells, including closure and special reporting of the location and construction that must be considered in the planning and installation. Licensed drillers and special fees also may be required.

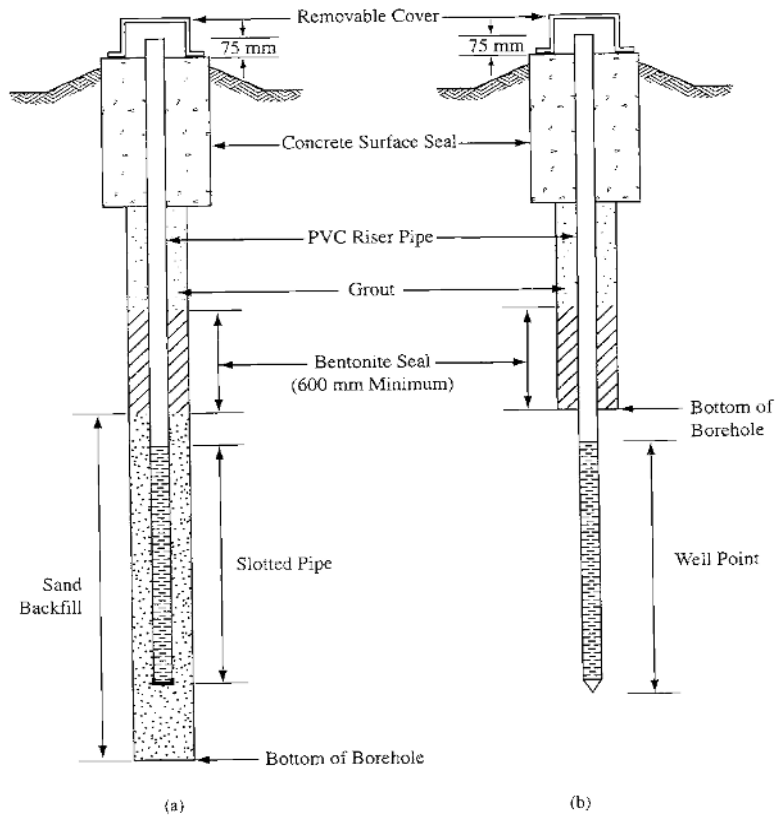

Figure 6-1. Representative Details of Observation Well Installations. (a) Drilled-in-place Stand-Pipe Piezometer, (b) Driven Well Point.

Piezometers are available in a number of designs. Commonly used piezometers are of the pneumatic and the vibrating wire type. Interested readers are directed to Course Module No. 11 (Instrumentation) or Dunnicliff (1988) for a detailed discussion of the various types of piezometers.

6.2.4 Water Level Measurements

A number of devices have been developed for sensing or measuring the water level in observation wells. Following is a brief presentation of the three common methods that are used to measure the depth to groundwater. In general, common practice is to measure the depth to the water surface using the top of the casing as a reference, with the reference point at a common orientation (often north) marked or notched on the well casing.

Chalked Tape

In this method a short section at the lower end of a metal tape is chalked. The tape with a weight attached to its end is then lowered until the chalked section has passed slightly below the water surface. The depth to the water is determined by subtracting the depth of penetration of the line into water, as measured by the water line in the chalked section, from the total depth from the top of casing. This is probably the most accurate method, and the accuracy is useful in pump tests where very small drawdowns are significant. The method is cumbersome, however, when taking a series of rapid readings, since the tape must be fully removed each time. An enameled tape is not suitable unless it is roughened with sandpaper so it will accept chalk. The weight on the end of the tape should be small in volume so it does not displace enough water to create an error.

Tape with a Float

In this method, a tape with a flat-bottomed float attached to its end is lowered until the float hits the water surface and the tape goes slack. The tape is then lifted until the float is felt to touch the water surface and it is just taut; the depth is then measured. With practice this method can give rough measurements, but its accuracy is poor. A refinement is to mount a heavy whistle, open at the bottom, on a tape. When it sinks in the water, the whistle will give an audible beep as the air within it is displaced.

Electric Water-Level Indicator

This battery operated indicator consists of a weighted electric probe attached to the lower end of a length of electrical cable that is marked at intervals to indicate the depth. When the probe reaches the water a circuit is completed and this is registered by a meter mounted on the cable reel. Various manufacturers produce the instrument, utilizing as the signaling device a neon lamp, a horn, or an ammeter. The electric indicator has the advantage that it may be used in extremely small holes.

The instrument should be ruggedly built, since some degree of rough handling can be expected. The distance markings must be securely fastened to the cable. Some models are available in which the cable itself is manufactured as a measuring tape. The sensing probe should be shielded to prevent shorting out against metal risers. When the water is highly conductive, erratic readings can develop in the moist air above the actual water level. Sometimes careful attention to the intensity of the neon lamp or the pitch of the horn will enable the reader to distinguish the true level. A sensitivity adjustment on the instrument can be useful. If oil or iron sludge has accumulated in the observation well, the electric probe will give unreliable readings.

Data Loggers

When timed and frequent water level measurements are required, as for a pump test or slug test, data loggers are useful. Data loggers are in the form of an electric transducer near the bottom of the well which senses changes in water level as changes in pressure. A data acquisition system is used to acquire and store the readings. A data logger can eliminate the need for onsite technicians on night shifts during an extended field permeability test. A further significant saving is in the technician’s time back in the office. The preferred models of the data logger not only record the water level readings but permit the data to be downloaded into a personal computer and, with appropriate software, to be quickly reduced and plotted. These devices are also extremely useful for cases where measurement of artesian pressures is required or where data for tidal corrections during field permeability tests is necessary.

6.3 FIELD MEASUREMENT OF PERMEABILITY

The permeability (k) is a measure of how easily water and other fluids are transmitted through the geomaterial and thus represents a flow property. In addition to groundwater related issues, it is of particular concern in geoenvironmental problems. The parameter k is closely related to the coefficient of consolidation (c_v) since time rate of settlement is controlled by the permeability. In geotechnical engineering, we designate small k = coefficient of permeability or hydraulic conductivity (units of cm/sec), which follows Darcy's law:

$$q = k \cdot i \cdot A \tag{6-1}$$

where q = flow (cm^3/sec), i = dh/dx = hydraulic gradient, and A = cross-sectional area of flow.

Laboratory permeability tests may be conducted on undisturbed samples of natural soils or rocks, or on reconstituted specimens of soil that will be used as controlled fill in embankments and earthen dams. Field permeability tests may be conducted on natural soils (and rocks) by a number of methods, including simple falling head, packer (pressurized tests), pumping (drawdown), slug tests (dynamic impulse), and dissipation tests. A brief listing of the field permeability methods is given in Table 6-1.

The hydraulic conductivity (k) is related to the specific (or absolute) permeability, K (cm^2) by:

$$K = k\mu/\gamma_w \tag{6-2}$$

where μ = fluid viscosity and γ_w = unit weight of the fluid (i.e., water). For fresh water at T = 20°C, μ = 1.005E-06 kN-sec/m^2 and γ_w = 9.80 kN/m^3. Note that K may be given in units of darcies (1 darcy = 9.87E-09 cm^2). Also, please note that groundwater hydrologists have confusingly interchanged k $\Leftrightarrow$ K in their nomenclature and this conflict resides within the various ASTM standards. The rate at which water is transmitted through a unit width of an aquifer under a hydraulic gradient i = 1 is defined as the transmissivity (T) of the formation, given by:

$$T = k \cdot b \tag{6-3}$$

where b = aquifer thickness.

The coefficient of consolidation (c_v for vertical direction) is related to the coefficient of permeability by the expression:

$$c_v = k \cdot D'/\gamma_w \tag{6-4}$$

where D' = ($1/m_v$) = constrained modulus obtained from one-dimensional oedometer tests (i.e., in lieu of the well-known e-log σ_v' curve, the constrained modulus is simply $D = \Delta\sigma_v'/\Delta\varepsilon_v$). In conventional one-dimensional vertical compression, c_v is often determined from the time rate of consolidation:

$$c_v = T H^2/t \tag{6-5}$$

where T = time factor (from Terzaghi theory), H = drainage path length, and t = measured time. For field permeability, it may be desirable to distinguish between vertical (c_v) and horizontal consolidation (c_h).

TABLE 6-1.

FIELD METHODS FOR MEASUREMENT OF PERMEABILITY

Test Method	***Applicable Soils***	***Reference***
Various Field Methods	Soil & Rock Aquifers	ASTM D 4043
Pumping tests	Drawdown in soils	ASTM D 4050
Double-ring infiltrometer	Surface fill soils	ASTM D 3385
Infiltrometer with sealed ring	Surface soils	ASTM D 5093
Various field methods	Soils in vadose zone	ASTM D 5126
Slug tests.	Soils at depth	ASTM D 4044
Hydraulic fracturing	Rock in-situ	ASTM D 4645
Constant head injection	Low-permeability rocks	ASTM D 4630
Pressure pulse technique	Low-permeability rocks	ASTM D 4630
Piezocone dissipation	Low to medium k soils	Houlsby & Teh (1988)
Dilatometer dissipation	Low to medium k soils	Robertson et al. (1988)
Falling head tests	Cased borehole in soils	Lambe & Whitman (1979)

6.3.1 Seepage Tests

Seepage tests in boreholes constitute one means of determining the in-situ permeability. They are valuable in the case of materials such as sands or gravels because undisturbed samples of these materials for laboratory permeability testing are difficult or impossible to obtain. Three types of tests are in common use: falling head, rising head, and constant water level methods.

In general, either the rising or the falling level methods should be used if the permeability is low enough to permit accurate determination of the water level. In the falling level test, the flow is from the hole to the surrounding soil and there is danger of clogging of the soil pores by sediment in the test water used. This danger does not exist in the rising level test, where water flows from the surrounding soil to the hole, but there is the danger of the soil at the bottom of the hole becoming loosened or quick if too great a gradient is imposed at the bottom of the hole. If the rising level is used, the test should be followed by sounding of the base of the hole with drill rods to determine whether heaving of the bottom has occurred. The rising level test is the preferred test. In those cases where the permeability is so high as to preclude accurate measurement of the rising or falling water level, the constant level test is used.

Holes in which seepage tests are to be performed should be drilled using only clear water as the drilling fluid. This precludes the formation of a mud cake on the walls of the hole or clogging of the pores of the soil by drilling mud. The tests are performed intermittently as the borehole is advanced. When the hole reaches the level at which a test is desired, the hole is cleaned and flushed using clear water pumped through a drill tool with shielded or upward-deflected jets. Flushing is continued until a clean surface of undisturbed material exists at the bottom of the hole. The permeability is then determined by one of the procedures given below. Specifications sometimes require a limited advancement of the borehole without casing upon completion of the first test at a given level, followed by cleaning, flushing, and repeat testing. The difficulty of obtaining satisfactory in situ permeability measurements makes this requirement a desirable feature since it permits verification of the test results.

Data which must be recorded for each test regardless of the type of test performed include:

1. Depth from the ground surface to groundwater surface both before and after the test,
2. Inside diameter of the casing,
3. Height of the casing above the ground surface,
4. Length of casing at the test section,
5. Diameter of the borehole below the casing,
6. Depth to the bottom of the boring from the top of the casing,
7. Depth to the standing water level from the top of the casing, and
8. A description of the material tested.

Falling Water Level Method

In this test, the casing is filled with water, which is then allowed to seep into the soil. The rate of drop of the water surface in the casing is observed by measuring the depth of the water surface below the top of the casing at 1, 2 and 5 minutes after the start of the test and at 5-minute intervals thereafter. These observations are made until the rate of drop becomes negligible or until sufficient readings have been obtained to satisfactorily determine the permeability. Other required observations are listed above.

Rising Water Level Method

This method, most commonly referred to as the "time lag method" (US Army Corps of Engineers, 1951), consists of bailing the water out of the casing and observing the rate of rise of the water level in the casing at intervals until the rise in the water level becomes negligible. The rate is observed by measuring the elapsed time and the depth of the water surface below the top of the casing. The intervals at which the readings are required will vary somewhat with the permeability of the soil. The readings should be frequent enough to establish the equalization diagram. In no case should the total elapsed time for the readings be less than 5 minutes. As noted above, a rising level test should always be followed by a sounding of the bottom of the hole to determine whether the test created a quick condition.

Constant Water Level Method

In this method water is added to the casing at a rate sufficient to maintain a constant water level at or near the top of the casing for a period of not less than 10 minutes. The water may be added by pouring from calibrated containers or by pumping through a water meter. In addition to the data listed in the above general discussion, the data recorded should consist of the amount of water added to the casing at 5 minutes after the start of the test, and at 5-minute intervals thereafter until the amount of added water becomes constant.

6.3.2 Pressure ("Packer") Test

A test in which water is forced under pressure into rock through the walls of a borehole provides a means of determining the apparent permeability of the rock, and yields information regarding its soundness. The information thus obtained is used primarily in seepage studies. It is also frequently used as a qualitative measure of the grouting required for reducing the permeability of rock or strengthening it. Pressure tests should be performed only in holes that have been drilled with clear water.

The apparatus used for pressure tests in rock is illustrated schematically in Figure 6-2a. It comprises a water pump, a manually-adjusted automatic pressure relief valve, pressure gages, a water meter, and a packer assembly. The packer assembly, shown in Figure 6-2b, consists of a system of piping to which two expandable cylindrical rubber sleeves, called packers, are attached. The packers, which provide a means of sealing off a limited section of borehole for testing, should have a length at least five times the diameter of the hole. They may be of the pneumatically, hydraulically, or mechanically expandable type.

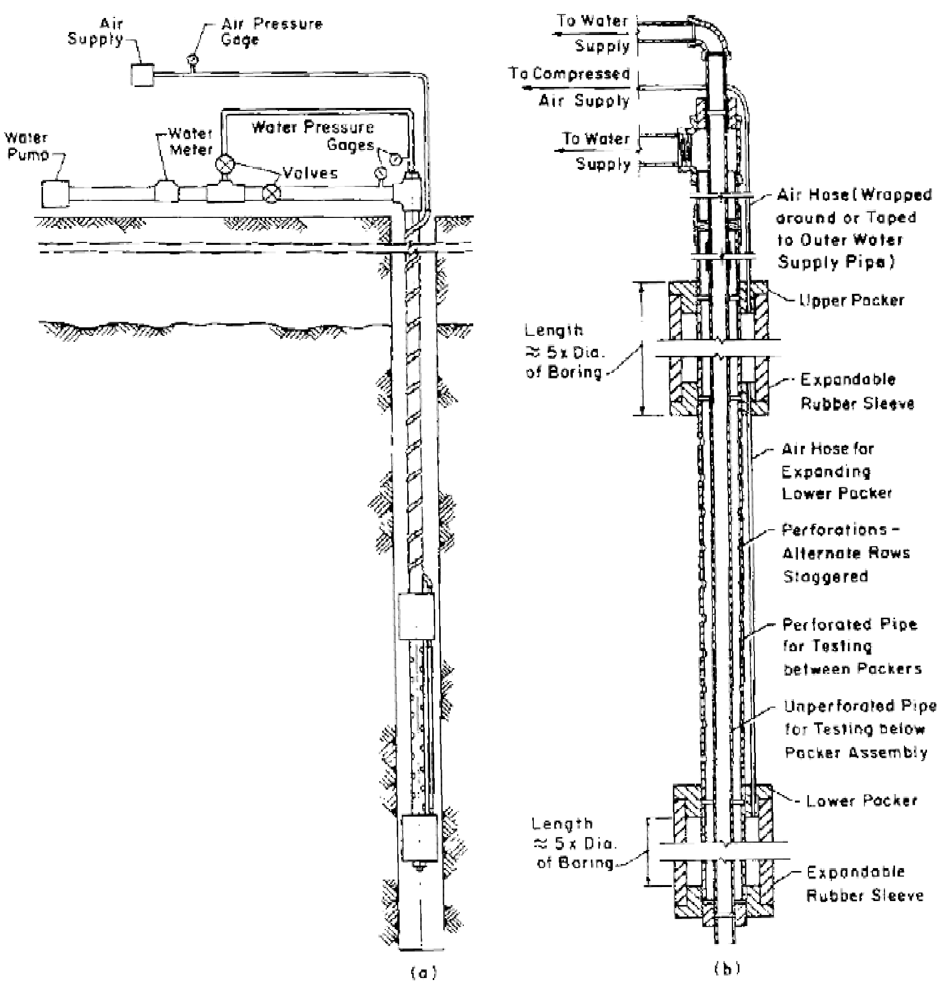

Figure 6-2. Packer-Type Pressure-Test Apparatus for Determining the Permeability of Rock. (a) Schematic Diagram; (b) Detail of Packer Unit. (Lowe and Zaccheo, 1991)

Pneumatic or hydraulic packers are preferred since they adapt to an oversized hole whereas mechanical packers may not. However, when pneumatic/hydraulic packers are used, the test apparatus must also include an air or water supply connected, through a pressure gage, to the packers by means of a high-pressure hose as shown in Figure 6-2a. The piping of the packer assembly is designed to permit testing of either the portion of the hole between the packers or the portion below the lower packer. Flow to the section below the lower packer is through the interior pipe; flow to the section between the packers is provided by perforations in the outer pipe, which have an outlet area two or more times the cross-sectional area of the pipe. The packers are normally set 0.6, 1.5 or 3 m apart and it is common to provide flexibility in testing by having assemblies with different packer spacing available, thereby permitting the testing of different lengths of the hole. The wider spacings are used for rock that is more uniform; the short spacing is used to test individual joints that may be the cause of high water loss in otherwise tight strata.

The test procedure used depends upon the condition of rock. In rock that is not subject to cave-in, the following method is in general use. After the borehole has been completed it is filled with clear water, surged, and washed out. The test apparatus is then inserted into the hole until the top packer is at the top of the rock. Both packers are then expanded and water under pressure is introduced into the hole, first between the packers and then below the lower packer. Observations of the elapsed time and the volume of water pumped at different pressures are recorded as detailed in the paragraph below. Upon completion of the test, the apparatus is lowered a distance equal to the space between the packers and the test is repeated. This procedure is continued until the entire length of the hole has been tested or until there is no measurable loss of water in the hole below the lower packer. If the rock in which the hole is being drilled is subject to cave-in, the pressure test is conducted after each advance of the hole for a length equal to the maximum permissible unsupported length of the hole or the distance between the packers, whichever is less. In this case, the test is limited, of course, to the zone between the packers.

The magnitudes of these test pressures are commonly 100, 200 and 300 kPa above the natural piezometric level. However, in no case should the excess pressure above the natural piezometric level be greater than 23 kPa per meter of soil and rock overburden above the upper packer. This limitation is imposed to insure against possible heaving and damage to the foundation. In general, each of the above pressures should be maintained for 10 minutes or until a uniform rate of flow is attained, whichever is longer. If a uniform rate of flow is not reached in a reasonable time, the engineer must use his/her discretion in terminating the test. The quantity of flow for each pressure should be recorded at 1, 2 and 5 minutes and for each 5-minute interval thereafter. Upon completion of the tests at 100, 200 and 300 kPa the pressure should be reduced to 200 and 100 kPa, respectively, and the rate of flow and elapsed time should once more be recorded in a similar manner.

Observation of the water take with increasing and decreasing pressure permits evaluation of the nature of the openings in the rock. For example, a linear variation of flow with pressure indicates an opening that neither increases nor decreases in size. If the curve of flow versus pressure is concave upward it indicates that the openings are enlarging; if convex, the openings are becoming plugged. Detailed discussion for interpretation of pressure tests is presented by Cambefort (1964). Additional data required for each test are as follows:

1. Depth of the hole at the time of each test,
2. Depth to the bottom of the top packer,
3. Depth to the top of the bottom packer,
4. Depth to the water level in the borehole at frequent intervals (this is important since a rise in water level in the borehole may indicate leakage around the top packer. Leakage around the bottom packer would be indicated by water rising in the inner pipe).

5. Elevation of the piezometric level,
6. Length of the test section,
7. Radius of the hole,
8. Length of the packer,
9. Height of the pressure gage above the ground surface,
10. Height of the water swivel above the ground surface, and
11. A description of the material tested.

The formulas used to compute the permeability from pressure tests data are (from *Earth Manual*, US Bureau of Reclamation, 1960):

$$k = \frac{Q}{2\pi\, LH}\ln\left(\frac{L}{r}\right) \quad for\ L \geq 10r \tag{6a}$$

$$k = \frac{Q}{2\pi\, LH}\sinh^{-1}\left(\frac{L}{2r}\right) \quad for\ 10r > L > r \tag{6b}$$

where, k is the apparent permeability, Q is the constant rate of flow into the hole, L is the length of the test section, H is the differential head on the test section, and r is the radius of the borehole.

The formulas provide only approximate values of k since they are based on several simplifying assumptions and do not take into account the flow of water from the test section back to the borehole. However, they give values of the correct magnitude and are suitable for practical purposes.

6.3.3 Pumping Tests

Continuous pumping tests are used to determine the water yield of individual wells and the permeability of subsurface materials in situ. The data provided by such tests are used to determine the potential for leakage through the foundations of water-retaining structures and the requirements for construction dewatering systems for excavations.

The test consists of pumping water from a well or borehole and observing the effect on the water table by measuring the water levels in the hole being pumped and in an array of observation wells. The observation wells should be of the piezometer type. The depth of the test well will depend on the depth and thickness of the strata to be tested. The number, location, and depth of the observation wells or piezometers will depend on the estimated shape of the groundwater surface after drawdown. Figure 6-3 shows a typical layout of piezometers for a pumping test. As shown in Figure 6-3, the wells should be located on the radial lines passing through the test well. Along each of the radial lines there should be a minimum of four wells, the innermost of which should be within 7.5 m of the test well; The outermost should be located near the limits of the effect of drawdown, and the middle wells should be located to give the best definition of the drawdown curve based on its estimated shape.

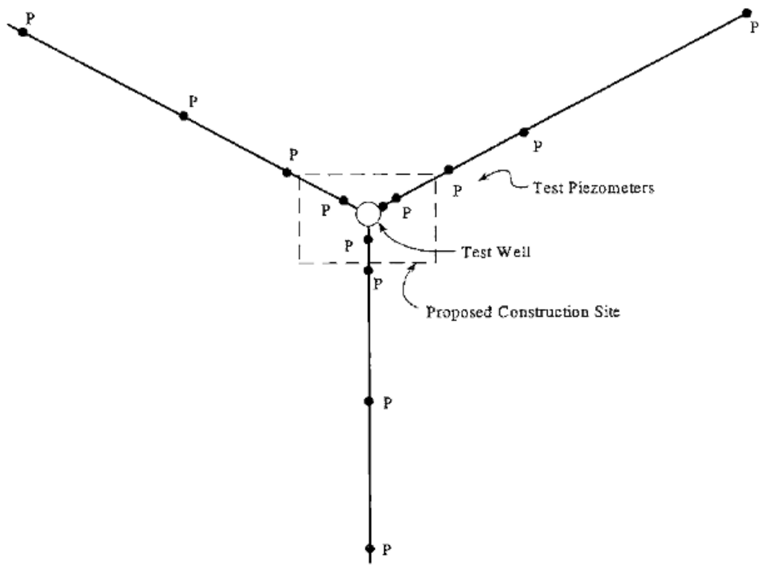

Figure 6-3. A General Configuration and Layout of Piezometers for a Pumping Test.

The pump used for these tests should have a capacity of 1.5 to 2 times the maximum anticipated flow and should have a discharge line sufficiently long to obviate the possibility of the discharge water recharging the strata being tested. Auxiliary equipment required include an air line to measure the water level in the test well, a flow meter, and measuring devices to determine the depth to water in the observation well. The air line, complete with pressure gage, hand pump, and check valve, should be securely fastened to the pumping level but in no case closer than 0.6 m beyond the end of the suction line. The flow meter should be of the visual type, such as an orifice. The depth-measuring device for the observation well may be any of the types described in Section 6.2.

The test procedure for field pumping tests is as follows: Upon completion of the well or borehole, the hole is cleaned and flushed, the depth of the well is accurately measured, the pump is installed, and the well is developed. The well is then tested at 1/3, 2/3 and full capacity. Full capacity is defined as the maximum discharge attainable with the water levels in the test and observation wells stabilized. Each of the discharge rates is maintained for 4 hours after further drawdown in the test and observation well has ceased, or for a maximum of 48 hours, whichever occurs first. The discharge must be maintained constant during each of the three stages of the test and interruptions of pumping are not permitted. If pumping should accidentally be interrupted, the water level should be permitted to return to its full non-pumping level before pumping is resumed. Upon completion of the drawdown test, the pump is shut off and the rate of recovery is observed.

The basic test well data which must be recorded are:

1. Location, top elevation and depth of the well,
2. The size and length of all blank casing in the well,
3. Diameter, length, and location of all screen casing used; also the type and size of the screen opening and the material of which the screen is made,
4. Type of filter pack used, if any,
5. The water elevation in the well prior to testing, and
6. Location of the bottom of the air line.

Information required for each observation well are:

1. Location, top elevation, and depth of the well,
2. The size and elevation of the bottom of the casing (after installation of the well),
3. Location of all blank casing sections,
4. Manufacturer, type, and size of the pipes etc.
5. Depth and elevation of the well and
6. Water level in the well prior to testing.

Pump data required include the manufacturer's model designation, pump type, maximum capacity, and capacity at 1800 rpm. The drawdown test data recorded for each discharge rate consist of the discharge and drawdowns of the test well and each observation well at the time intervals shown in Table 6-1.

TABLE 6-2.

TIME INTERVALS FOR READING DURING PUMPING TEST

Elapsed Time	Time Interval for Readings
0-10 min	0.5 min
10-60 min	2.0 min
1-6 hour	15.0 min
6-9 hour	30.0 min
9-24 hour	1.0 hour
24-48 hour	3.0 hour
>48 hour	6.0 hour

The required recovery curve data consist of readings of the depth to water at the test location and observation wells at the same time intervals given in Table 6-2. Readings are continued until the water level returns to the prepumping level or until adequate data have been obtained. A typical time-drawdown curve is shown in Figure 6-4. Generally, the time-drawdown curve becomes straight after the first few minutes of pumping. If true equilibrium conditions are established, the drawdown curve will become horizontal.

Field drawdown tests may be conducted using 2 or more cased wells and measuring the drop in head with time. A submersible pump at a central well is used for the drawdown and the head loss at two radial distances may be measured manually or automated via pore pressure transducers. Sowers (1979) discusses the details briefly for two cases: (1) an unconfined aquifer over an impervious layer and (2) artesian aquifer. If the gradient of the drawdown is not too great ($< 25°$ slope), then the head loss in the drawdown well may be used itself (r_1 = well radius) and only two cased wells are necessary.

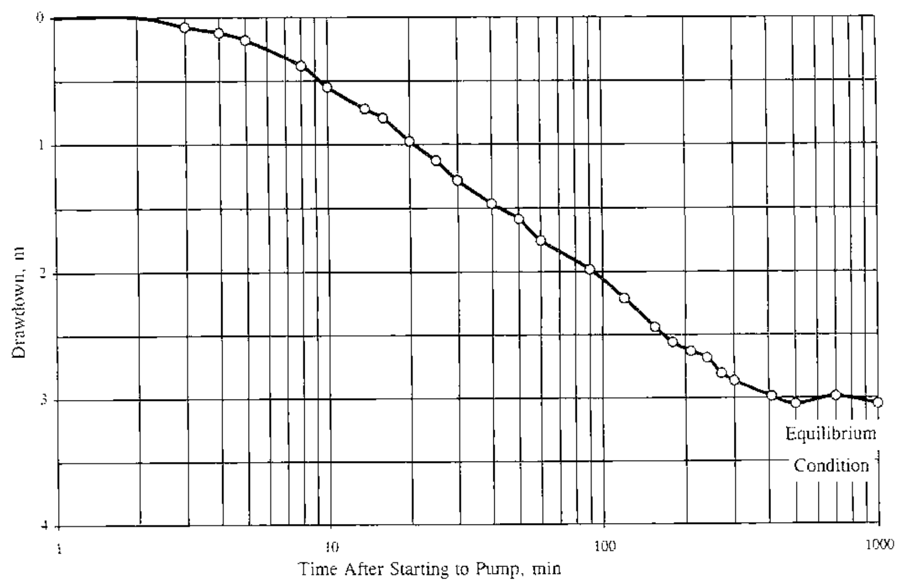

Figure 6-4. Drawdown in an Observation Well Versus Pumping Time (Logarithmic Scale).

For the case of measured drawdown pressures in an unconfined aquifer (shown in Figure 6-5), the permeability (k in cm/s) of the transmitting medium is given by:

Unconfined: $$k = \frac{q \ln(r_2/r_1)}{B\,[(h_2)^2-(h_1)^2]} \qquad (6\text{-}7)$$

where q = measured flow with time (cm^3/s), r = radial distance (cm), and h = height of water above the reference elevation (cm).

For a confined aquifer where an impervious clay aquiclude caps the permeable aquifer, the permeability is determined from:

Confined: $$k = \frac{q \ln(r_2/r_1)}{2Bb\,(h_2-h_1)} \qquad (6\text{-}8)$$

where b = thickness of the aquifer (Figure 6-6).

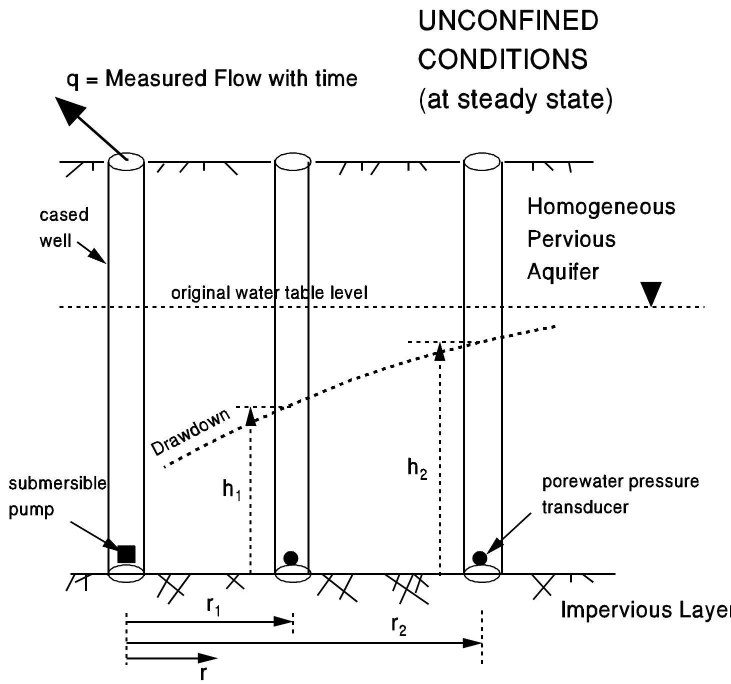

Figure 6-5. Definitions of Terms in Pumping Test Within an Unconfined Aquifer.

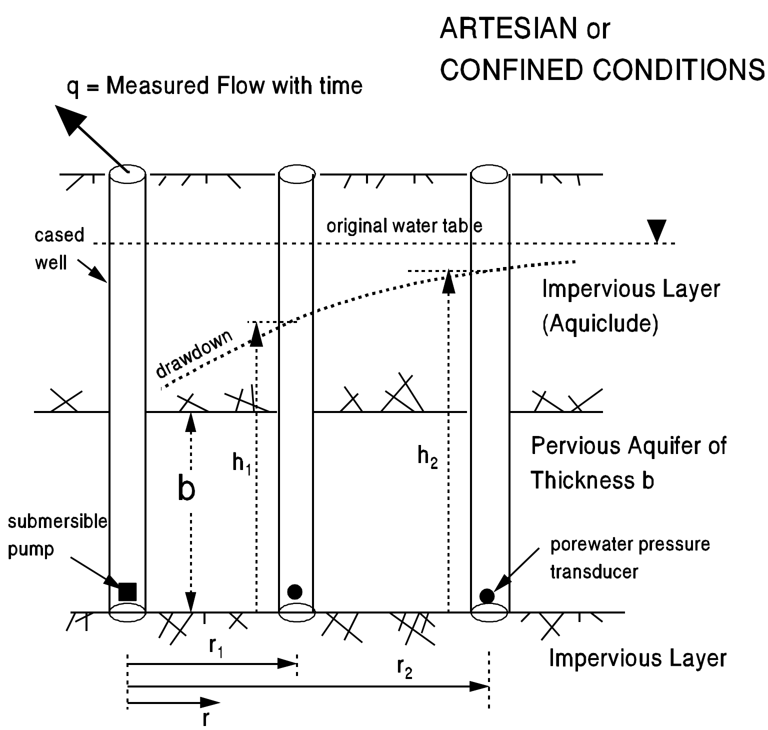

Figure 6-6. Definitions of Terms in Pumping Test Within a Confined Aquifer System.

6.3.4 Slug Tests

Using mechanical slug tests (ASTM 4044) in which a solid object is used to displace water and induce a sudden change of head in a well to determine permeability has become common in environmental investigations. Figure 6-7 presents the slug test procedure. It is conducted in a borehole in which a screened (slotted) pipe is installed. The solid object, called a "slug", often consists of a weighted plastic cylinder. The slug is submerged below the water table until equilibrium has been established; then the slug is removed suddenly, causing an "instantaneous" lowering of the water level within the observation well. Finally, as the well gradually fills up with water, the refill rate is recorded. This is termed the "slug out" procedure.

The permeability, k, is then determined from the refill rate. In general, the more rapid the refill rate, the higher the k value of the screened sediments.

It is also possible to run a "slug in" test. This is similar to the slug out test, except the plastic slug is suddenly dropped into the water, causing an "instantaneous" water level rise. The decay of this water level back to static is then used to compute the permeability. A slug in and slug out test can be performed on the same well.

Alternatively, instead of using a plastic slug, it is possible to lower the water level in the well using compressed air (or raising it using a vacuum) and then suddenly restore atmospheric pressure by opening a quick-release valve.

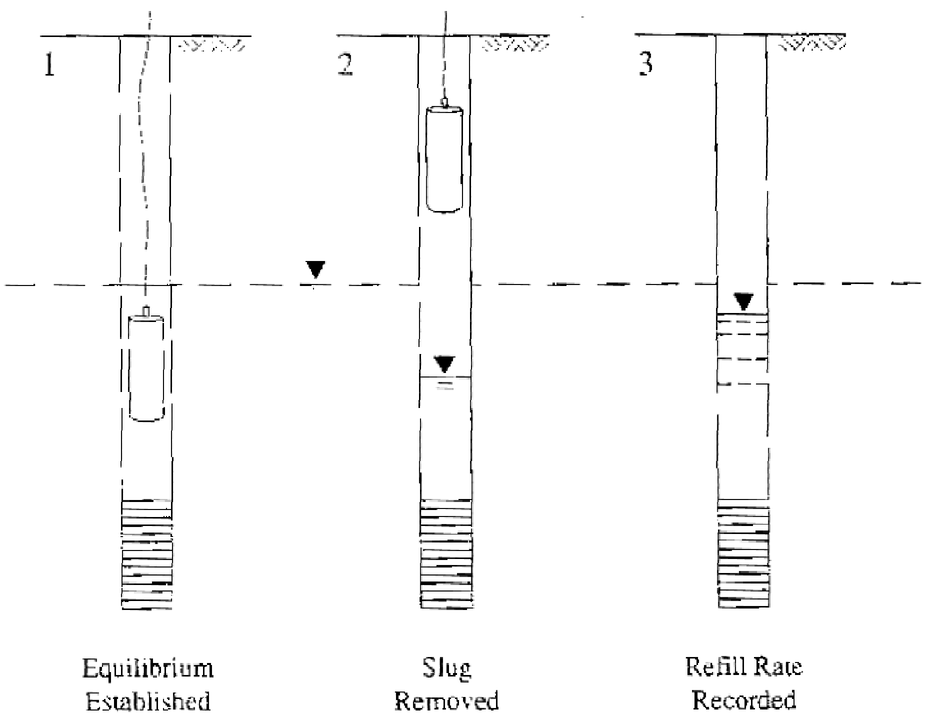

Figure 6-7. General Procedure for Slug Test in as Screened Borehole.

With either method, a pressure transducer and data logger are used to record time and water levels. In instances where water-level recovery is slow enough, hand-measured water levels (see Section 6.2) are adequate. Once, the data have been collected, drawdown is graphed versus time, and various equations and/or curve-matching techniques are used to compute permeability.

Much of the popularity of these tests results from the ease and low cost of conducting them. Unfortunately, however, slug tests are not very reliable. They can give wrong answers, lead to misinterpretation of aquifer characteristics, and ultimately, improper design of dewatering or remediation systems. Several shortcomings of the slug tests may be summarized as follows (Driscoll, 1986):

1. Variable accuracy: Slug tests may be accurate or may underestimate permeability by one or two orders or magnitude. The test data will provide no clue as to the accuracy of the computed value unless a pumping test is done in conjunction with slug tests.
2. Small zone of investigation: Because slug tests are of short duration, the data they provide reflect aquifer properties of just those sediments very near the well intake. Thus, a single slug test does not effectively integrate aquifer properties over a broad area.
3. Slug tests cannot predict the storage capacity of an aquifer.
4. It is difficult to analyze data from wells screened across the water table.
5. Rapid slug removal often causes pressure transients that can obscure some of the early test data.
6. If the true static water level is not determined with great precision, large errors can result in the computed permeability values.

Therefore, it is crucial that a qualified hydrogeologist assesses the results of the slug tests and ensures that they are properly applied and that data from them are not misused. Although the absolute magnitude of the permeability value obtained from slug tests may not be accurate, a comparison of values obtained from tests in holes judiciously located throughout a site being investigated can be used to establish the relative permeability of various portions of the site.

6.3.5 Piezocone Dissipation Tests

In a CPT test performed in saturated clays and silts, large excess porewater pressures (Δu) are generated during penetration of the piezocone. Soft to firm intact clays will exhibit measured penetration porewater pressures which are 3 to 6 times greater than the hydrostatic water pressure, while values of 10 to 20 times greater than the hydrostatic water pressure will typically be measured in stiff to hard intact clays. In fissured materials, zero or negative porewater pressures will be recorded. Regardless, once penetration is stopped, these excess pressures will decay with time and eventually reach equilibrium conditions which correspond to hydrostatic values. In essence, this is analogous to a push-in type piezometer. In addition to piezometers and piezocones, excess pressures occur during the driving of pile foundations, installation of displacement devices such as vibroflots for stone columns and mandrels for vertical wick-drains, as well as insertion of other in-situ tests including dilatometer, full-displacement pressuremeter, and field vane. How quickly the porewater pressures decay depends on the permeability of the surrounding medium (k), as well as the horizontal coefficient of consolidation (c_h), as per equation 6-4. In clean sands and gravels that are pervious, essentially drained response is observed at the time of penetration and the measured porewater pressures are hydrostatic. In most other cases, an initial undrained response occurs that is followed by drainage. For example, in silty sands, generated excess pressures can dissipate in 1 to 2 minutes, while in contrast, fat plastic clays may require 2 to 3 days for complete equalization.

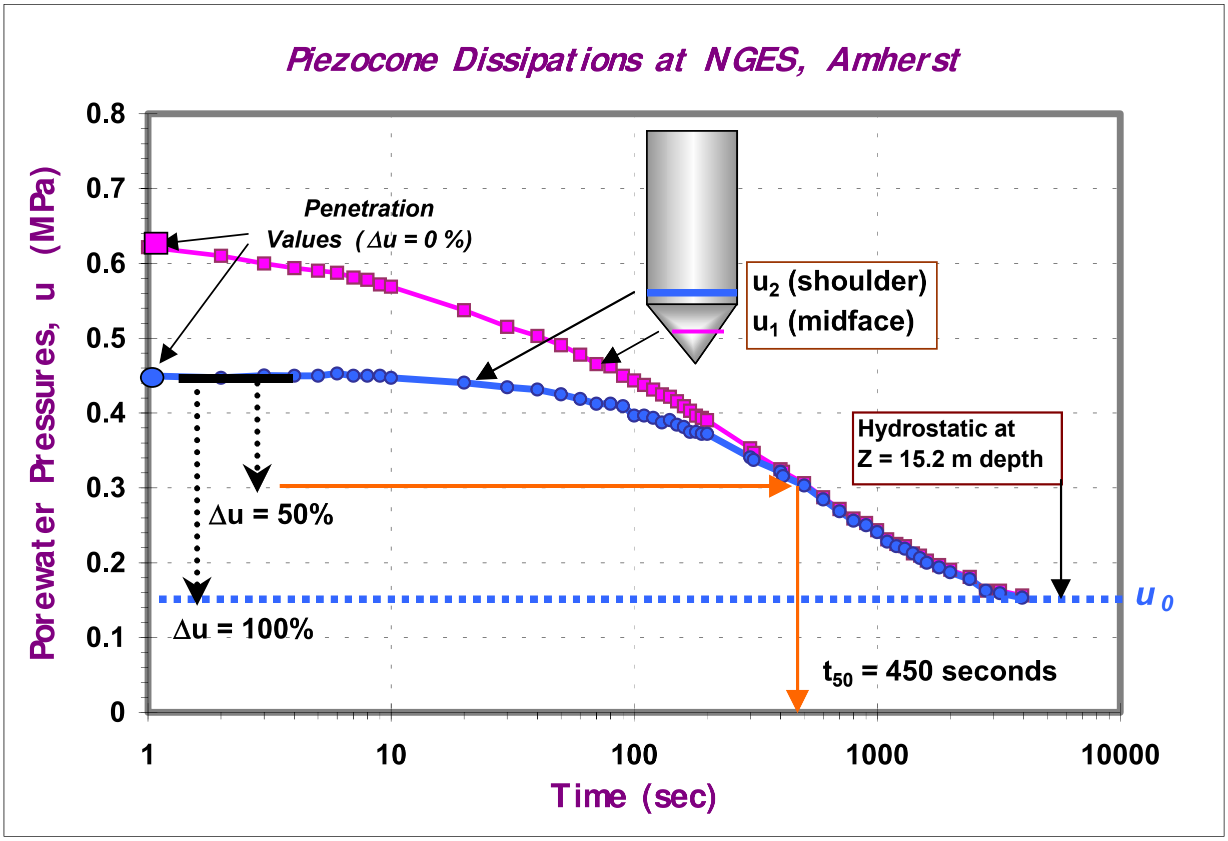

Figure 6-8. Porewater Pressure Dissipation Response in Soft Varved Clay at Amherst NGES.
(Procedure for t_{50} determination using U_2 readings shown)

Representative dissipation curves from two types of piezocone elements (midface and shoulder) are presented in Figure 6-8. These data were recorded at a depth of 15.2 meters in a deposit of soft varved silty clay at the National Geotechnical Experimentation Site (NGES) in Amherst, MA. Full equalization to hydrostatic conditions is reached in about 1 hour (3600 s). In routine testing, data are recorded to just 50 percent consolidation in order to maintain productivity. In this case, the initial penetration pressures correspond to 0 percent decay and a calculated hydrostatic value (u_0) based on groundwater levels represents the 100 percent completion. Figure 6-8 illustrates the procedure to obtain the time to 50 percent completion (t_{50}).

The aforementioned approach applies to soils that exhibit monotonic decay of porewater pressures with logarithm of time. For cases involving heavily overconsolidated and fissured geomaterials, a dilatory response can occur whereby the porewater pressures initially rise with time, reach a peak value, and then subsequently decrease with time.

For type 2 piezocones with shoulder filter elements, the t_{50} reading from monotonic responses can be used to evaluate the permeability according to the chart provided in Figure 6-9. The average relationship may be approximately expressed by:

$$k\ (cm/s) \quad \approx \quad \left(\frac{1}{251 \cdot t_{50}}\right)^{1.25} \tag{6-9}$$

where t_{50} is given in seconds. The interpretation of the coefficient of consolidation from dissipation test data is discussed in Chapter 9 and includes a procedure for both monotonic and dilatory porewater pressure behavior.

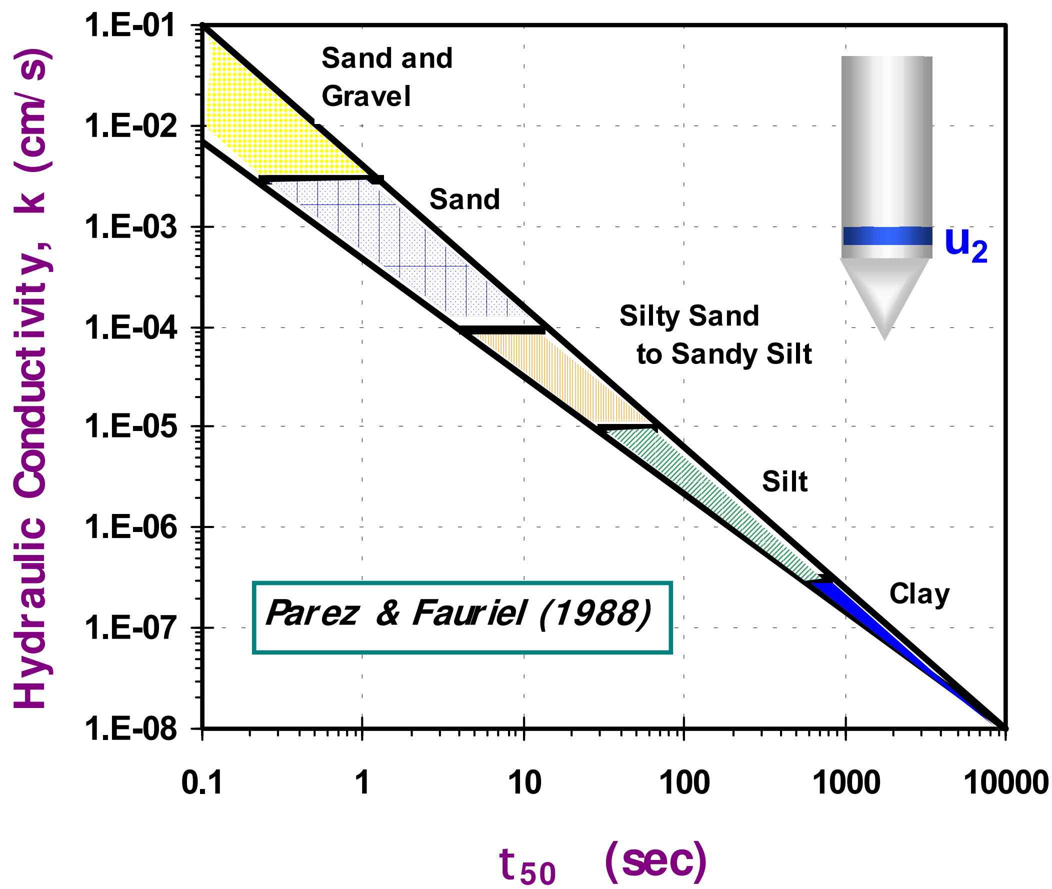

Figure 6-9. Coefficient of Permeability (k = Hydraulic Conductivity) from Measured Time to 50% Consolidation (t_{50}) for Monotonic Type 2 Piezocone Dissipation Tests
(from Parez & Fauriel, 1988).