COMMONWEALTH OF MASSACHUSETTS

DEPARTMENT OF ENVIRONMENTAL PROTECTION

STANDARD REFERENCES FOR MONITORING WELLS

SECTION 5.1 WATER-LEVEL MEASUREMENTS

SECTION 5.1
WATER-LEVEL MEASUREMENTS

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5.1 WATER LEVEL MEASUREMENTS

5.1-1 PURPOSE

Accurate water-level measurements are essential data in any hydrogeologic investigation. Water-level measurements are taken to determine the elevation of the potentiometric surface in a monitoring well, observation well or piezometer at a particular point in time. Single-event measurements, multiple-time measurements, or continuous-time measurements may be taken. Water-level data can be used to determine the following:

- Water levels prior to water quality sampling
- Horizontal and vertical ground water gradients
- Aquifer characteristics from measurements during slug and pump tests
- Aquifer response to rainfall, barometric and tidal influences
- Aquifer response to pumping or other outside influences
- Direction of ground water flow under pumping and non-pumping conditions
- Local and regional changes in ground water levels

5.1-2 GENERAL CONSIDERATIONS

5.1-2.1 Measuring Point

A measuring point for all water-level measurements must be established and consistently maintained as a reference point on a monitoring well. The reference point should be stable and have a professionally surveyed elevation. The top of the well casing (riser) should always be used as the permanent reference point. The top of the riser is preferred over the top of protective casing because the protective casing is more susceptible to movement through settling, heaving, or displacement by impact. The reference points for both the top of riser and protective casing should be indicated with a permanent mark or notch to ensure consistent measurements. Reference points must be related to Mean Sea Level (see Section 5.5) to ensure correlation between sites.

In addition to measuring the depth to water from the top of the well riser, it is recommended that one measure the difference between the top of riser and top of the protective casing. If changes are noted with time, it is an indication that one of the reference points has moved. If there has been considerable activity (such as construction or filling) or a change is detected in the distance between the riser and protective casing reference marks, a re-survey is the only sure way of knowing that the elevations are still accurate.

5.1-2.2 Records

Manual water-level measurements should be recorded on a water-level data sheet similar to that shown on Figure 5.1-1. The unique well number, date, time, and depth to water should be recorded for each measurement. Measurements should be recorded in feet and tenths and hundredths of a foot, not inches and fractions.

The form should be drafted so that there is room for both permanent and temporary data. Permanent data include such items as unique well identification number, geographical coordinates, site address, location of measuring point, surveyed elevation of the measuring point, depth to the bottom of the well, type of well screen, length of screened interval, presence or absence of contamination, inside diameter of the well screen and riser, and hydraulic conductivity of the formation opposite the well screen. Temporary data include observations about the condition of the well, such as, volatile organic analyzer (VOA) readings, the measurement of the depth of the water level in the well, the elevation of the water level (MSL), the type of measuring device used, the data and time the readings were taken, and the name of the person taking the measurements. In addition, recording the difference in elevation between the top of riser and protective casing is recommended to verify that the reference marks are stable.

5.1-3 INSTRUMENTS

5.1-3.1 Weighted Tape (Plunker)

A plunker usually consists of a small weighted metal cylinder with a concave undersurface. When this concave surface hits the water, it produces a "plopping" sound. By lowering the plunker in the well with a gentle up-and-down motion, the water surface can be determined. Usually the plunker is attached to a 100-foot steel or fiberglass measuring tape. A direct reading of the depth to water can be obtained if the tape has been shortened a distance equal to the length of the plunker.

If a permanent adjustment has not been made to the tape, it is not possible to obtain a direct measurement of the water level. With an unadjusted tape a compensating calculation must be made each time, to add the distance between the tape and the end of the plunker to the depth measured directly from the tape. The accuracy of this method is approximately 0.05-0.1 foot. If a steel tape is used, the weight of the plunker should be adjusted to offset the weight of a long tape.

Advantages

- Simple to operate.
- Simple and inexpensive to construct; can be dedicated to a well.
- Generally unaffected by most ground water contaminants.
- With tape modification it provides a direct reading of depth to water.

Disadvantages

- o Not suitable for deep measurements (i.e., over 100 feet).
- o Not suitable when ambient noise levels are high (e.g., pumps or drill rigs operating nearby).
- o Not suitable if the well contains dampening substances (e.g., high percentage of sediments or viscous liquids).
- o Unadjusted tape is a potential source of error.
- o Very difficult to hear "plop" when the top of water is in the screened section.
- o Not suitable for determining thickness of floating fluid.
- o Fiberglass tapes may stretch, providing inconsistent readings.
- o Unless treaded with teflon the fiberglass tape may stick to casing.

5.1-3.2 Chalked Tape

This method is not recommended in wells that are also being used for water quality sampling due to the fact that the chalk may introduce impurities into the well.

A steel or fiberglass tape coated with chalk with a small diameter weight attached to the end can be used to obtain water-level measurements in a well. The lower 3 to 4 feet of the tape is rubbed with chalk, and the tape is lowered into the well until the lower part is submerged and an even foot mark is at the measuring point on the well. The length of the wetted section of the chalked tape is subtracted from the total tape measurement to obtain the depth to water. The accuracy of the chalked tape device is on the order of 0.05 foot.

Chemically sensitive chalks or coatings can be used to determine the presence and thickness of fluids other than water, such as gasoline. These substances can be spread on a coated steel tape and the depth and thickness of the substances can be determined from the color change on the tape.

Advantages

- o Simple, easy to operate.
- o Not subject to mechanical or electrical failure.

Disadvantages

- o Dripping water and condensation can result in erroneous readings.
- o Chalk may introduce unacceptable impurities into the water.
- o Requires subtracting the wetted length for total measurement - not a direct reading method.

5.1-3.3 Electrical Tapes

Electrical water-level tapes are based on the principle that once the probe (consisting of two unconnected wires located on the end of the tape) is immersed, an electrical circuit is completed and a buzzer and/or a light is activated. Electrical water-level tapes are usually marked in one- to five-foot intervals. Therefore, the intermediate distance must be measured with a ruler to determine the actual depth to water. A few instruments recently introduced and commercially available are fully marked, allowing for a direct measurement in feet or meters. Accuracy of this method is approximately 0.05 feet.

Advantages

- o Small diameter (¼- to ½-inch) cable and probe is capable of measurements in small diameter piezometers.
- o Relatively simple to operate.
- o Multiple readings during slug and pump tests are possible without removing tape from the well.
- o The individual tape length is the only depth limitation.
- o Background noise is not a problem.

Disadvantages

- o Dripping water or condensation on the sides of the well riser can result in erroneous readings.
- o The tape may become kinked and will not hang straight in a well, producing inaccurate readings.
- o If the tape requires a manual measurement between markings, it is subject to error; it may not be suitable where fast measurements are required.
- o The instrument is subject to electrical malfunction (e.g., dead batteries or cable breaks); also, one-foot markings may shift, resulting in inaccurate readings.
- o Not suitable for wells with PCBs or other di-electric fluids.

5.1-3.4 Transducer

A pressure-sensitive transducer can be used to measure water levels in a well. Transducers displace water when they are lowered into a well. One must be sure to allow adequate time for the water level to equilibrate. The pressure transducer produces an electrical signal (voltage or amps) proportional to the height of the water column above the transducer. The pressure is recorded in pounds per square inch (psi) which can then be converted to feet of water. A display meter, data logger, recorder, or similar instrument must be used to interpret the transducer signal. The output may be displayed on a meter, recorded on a chart, or fed directly into computer memory for later data reduction. Transducers are particularly suitable for slug tests or pump tests where frequent, rapid readings or long-term measurements are desirable. The accuracy of the water-level measurements depends on the type of data logging instrument used, and the psi range of the transducer (e.g., 10, 25, or 50 psi). Accuracy, usually 0.1 percent of full scales, ranges from 0.01 to 0.2 feet for most common transducer ranges. The data generated is referenced to the elevation (i.e., depth below a reference mark) of the transducer. The position (i.e., depth or elevation of the transducer) must be calculated for each well or testing event.

Advantages

- Continuous and rapid readings possible.
- Can operate remotely for long periods of time.
- Can provide for direct access to the data on a computer.
- Length of the individual transducer cable and the transducer pressure range is the only depth limitation.

Disadvantages

- Equipment is expensive.
- Operation is moderately complex and sophisticated.
- Subject to electronic failure or data transmission loss.
- Some instruments are not weatherproof and require special protection from inclement weather.
- Subject to reduced accuracy if not checked regularly and calibrated properly.
- May not be resistant to certain chemicals.
- Voltage must be held constant.
- Will give inaccurate readings if not vented to compensate for barometric pressure changes.

- o Use over long periods must account for changes in atmospheric pressure.

- o Probe constants are determined for water, which has a specific gravity of 1.0. False readings may be obtained or compensating adjustments must be made for substances with a specific gravity less than or greater than 1.

- o If a data logger is dedicated to a particular well, it may be conspicuous and encourage vandalism.

5.1-3.5 Acoustic Well Probe

Acoustic well probes operate on the same theory as sonar or similar sonic depth-finding devices. Acoustic well probes use ultrasonic sensors or transducers to transmit a signal and record the amount of time it takes for the reflected signal to return to the sensor. This information can then be translated into depth to water. Accuracy reportedly ranges from 0.5 to 1.0 feet for different models commercially available. This level of accuracy may be unacceptable for many projects.

Advantages

- o Probes do not contact liquid; therefore, they are particularly suitable for highly contaminated environments, or highly viscous contaminants.

- o Eliminates the potential for cross-contamination between wells.

Disadvantages

- o Equipment is expensive, relatively new and untested.
- o Operation is moderately complex and sophisticated.
- o Accuracy may be influenced by temperature changes.
- o Accuracy may not be adequate for many applications.

5.1-3.6 Continuous Water-level Chart Recorder

A continuous chart recorder can be used to record water levels over periods of time ranging from 4 hours to 32 days. Typically, the recorder consists of a float mechanism attached to a drum chart recorder. The relative level of the float is recorded on the chart for the time period specified. The final chart is a plot of relative water levels versus time.

Recently, quartz clocks have replaced the original key-wound clocks in these recorders, increasing the reliability of the instrument. Continuous chart recorders were originally designed for surface water monitoring, such as stream gauging, but they have been adapted to ground water monitoring by the use of small-diameter floats. More sophisticated systems are capable of translating the data into digital information and transmitting the data to a distant receptor.

Advantages

- Provides almost continuous water-level record.
- Accurate from 0.01 to 0.05 feet.
- Relatively simple to operate.
- Recognized as a well-proven method used by USGS.

Disadvantages

- Subject to mechanical problems, particularly in cold weather.

- Requires box or compartment attached to well.

- Floats and cable move with water-level fluctuations, and may become stuck or lodged in well.

- It is subject to extraneous interference from vibrations caused by trains, earth tides, etc.

- Large water-level fluctuations may be difficult to interpret, particularly near pumping wells due to overlapping impacts on water levels.

- The recorder is conspicuous, and may be vandalized. It needs a protective cover.

5.1-3.7 Interface Probes

Interface probes consist of a small probe attached to the end of a coated tape that includes an optical liquid sensor and an electrical conductivity probe to differentiate between water and non-polar liquids (i.e., hydrocarbons). The probe transmits a signal up the tape to the reel, where an audible alarm emits a tone: a continuous tone for hydrocarbons and an oscillating tone for water. A direct reading of the depth of free product and of the water level can be made from the tape. The interface probe can also be used to measure water levels where floating hydrocarbons do not occur. The accuracy of this method is approximately 0.05 feet. It is advisable to cross-check the measured hydrocarbon thickness by retrieving a sample of the product and observing its thickness in a clear bailer.

Advantages

- Permits measurement of the depth and thickness of separate phase liquids, as well as water levels.

- Useful for water-level measurements alone.

- Direct reading possible.

Disadvantages

- o Battery-operated and subject to possible electrical malfunctions.
- o If the signal or light is not on the reel, but on the probe, it may be difficult to hear the tone or see the light.
- o Probe diameter is 1¼ inches; may be too large for some piezometers; could become lodged in riser.
- o Probe may be affected by decontamination solutions.
- o For high viscosity fluids (i.e., No. 6 fuel oil), accurate readings may be difficult to obtain.
- o Difficult to decontaminate.

5.1-4 METHODOLOGY FOR MEASURING WATER LEVELS

Water level measurements are so important in interpreting site hydrogeology that a clear, concise, well-ordered methodology is imperative. The following checklist is offered to ensure consistant and accurate data.

1. Prior to going into the field, check the measuring equipment to be sure that it is working properly and that it is in good repair. Also, prior to undertaking field work, the equipment should be decontaminated. Carry extra equipment and batteries to eliminate lost time in the event of equipment loss or malfunction.

2. Prior to entering the field, fill out the field forms with the permanent well data, such as well number, depth to the bottom of the well, and elevation of the permanent measuring point. Bring a current site map showing the location and identification numbers of all wells.

   When collecting measurements, it is useful to bring along previous water-level data. Comparison of the current measurement to previous measurements can help identify anomalous readings or misread well numbers.

3. Unlock the padlock in the hasp. Remove the protective cap from the well. Check that the I.D. number on the cap is the same as the one entered on the permanent record. Record any unusual sounds, odors, staining, damage, or other observations in the "Remarks" column. Be alert for evidence of vandalism or tampering.

- o Well risers should be vented during installation by drilling a small hole into the casing below the depth of the seated cap (see Section 4.3). This will permit air and gas to escape during water level fluctuations. If a popping or sucking sound is heard when the cap is removed, the well is probably not vented properly. If the well is not adequately vented, the water level may take a while to stabilize once the cap is removed, especially in low permeability materials.

- o If the well has been completed in contaminated ground water, appropriate health and safety protection and procedures must be utilized (see Section 2.3). Generally, an OVA is used to monitor for volatile organics immediately upon opening the well.

4. First check for the measuring point. Holding the instrument at this point obtain a water-level reading with the measuring device. Record the actual reading obtained from the instrument - do not correct or convert data in your head. Repeat the measurement again to confirm the reading. Under ideal conditions the measurements would be taken using two different kinds of instruments. The reported depth would be the average of these two readings. Remember to record the time of the measurement.

5. Measure and record the difference in elevation between the reference marks on the top of well riser and protective casing.

6. Measure and record the depth to the bottom of the well.

7. Remove the instrument from the well. If the well is located at a site where contamination is suspected or known to be present, the measuring instrument must be completely decontaminated before taking another measurement in another well (see Section 6.5).

8. Replace cap and secure the well.

9. Once the measurements are complete, translate the water level depth readings into elevations (NGVD).

5.1-5 PROBLEMS AND POSSIBLE SOLUTIONS

5.1-5.1 Cross-contamination

Where contaminated groundwater exists, care must be taken to avoid cross-contamination of wells caused by contaminated water-level measuring instruments. Adequate decontamination procedures or dedicated instruments should be used to avoid this problem. It is advisable to start with the cleanest wells and work progressively to the more contaminated wells. Use historical data to determine this order.

5.1-5.2 Water/Floating Fluids

If immiscible fluids with a specific gravity that is less than 1 are encountered in a monitoring well, special procedures may be required to obtain a free product/water-level measurement. Instruments, such as an interface probe, are available that will measure the water/product levels. In the case of highly contaminated ground water or non-aqueous phase liquids, special dedicated instruments may be required for water level measurements. Where immiscible, floating fluids are encountered, measurements of both fluid levels should be recorded during each measuring event. Without correction water level contour maps prepared on the basis of a 2-phase liquid surface will always contain some unavoidable errors.

5.1-5.3 Flowing Artesian Wells

In order to obtain accurate measurements in flowing artesian wells, the water level must be stabilized. This is generally accomplished by adding an additional section of riser pipe onto the well to stabilize the flow so as to permit the measurement of a water surface below the top of the added riser or by using a pressure gauge. If additional riser pipe is added, a new measuring point for the well must be established, documented, and reported. Water pressure gauges can be adapted to fit over the top of the riser and measure the artesian pressure at the wellhead. The elevation or height above ground can be calculated from the pounds per square inch (psi) reading of the instrument. If flowing artesian conditions are anticipated, wells can be constructed to allow for the necessary riser additions or the use of a pressure gauge. If artesian conditions are anticipated, the surveyor conducting the original survey should be asked to establish a permanent reference datum that can be used as a reference point for future changes.

Measurement of water levels in flowing artesian wells may be impossible to obtain if the water column freezes above the ground.

5.1-5.4 Cyclic External Factors Affecting Water Levels

Water levels may be influenced by any combination of pumping, barometric and tidal influences. In general, tidal and pumping influences produce the most extreme deviations from undisturbed conditions. If tidal influences are a possibility at a site, ideally it is advisable to monitor the water levels in selected wells for a full 28-day tidal cycle to determine the significance of this factor. This can be roughly approximated by taking just two measurements: one at high tide and one at low tide in the middle of a 28-day tidal cycle. Nearby pumping wells can also significantly alter natural water-level elevations. If interfering pumping conditions are encountered, water levels should be measured during periods of pumping and non-pumping conditions when the water level has stabilized. A continuous water-level recorder is preferred for this type of monitoring. If anomalous measurements are obtained or pumping is occurring, these factors need to be evaluated. All water level measurements should include the time that the measurement was obtained. Use of the military or 24-hour time designation will eliminate the possibility of confusing A.M. with P.M. readings.

5.1-5.5 Non-cyclic External Factors Affecting Water Levels

Trenches in which underground utilities such as water, gas, sewer, and transmission pipelines are laid, disturb the natural permeability of the soil, increasing its hydraulic conductivity. At times ground water infiltrates directly into underground vaults and pipes. Taken collectively these features may represent line sinks or sources - places where ground water will tend to discharge or recharge preferentially. If a site has a large number of monitoring wells closely spaced together, the effects of these will be readily apparent on a map of the potentiometric surface. If the density of the wells is less, the water levels may produce anomalous readings that defy interpretation.

5.1-5.6 Dropping Something into a Well

Caution should be taken to avoid dropping objects into a well. Pencils, keys, eyeglasses, and other loose objects easily drop into wells, but are not so easily retrieved. All measuring instruments should be connected to something larger than the diameter of the well riser to avoid dropping these down a well. Measuring instruments should have a diameter small enough so that they fall freely in the well. This will avoid lodging the device in the riser, thereby obstructing further measurements or sampling. All measuring tapes or cables should be in good repair, free from breaks or splits that could result in separation of a cable within a well. Fishhooks and string may sometimes successfully retrieve lost items. Extreme care must be taken when taking water-level measurements in pumping wells to prevent entanglement of the measuring instrument on downhole equipment.

ADDITIONAL REFERENCES

Barcelona, M.J., et al., 1983, A guide to the selection of materials for monitoring well construction and groundwater sampling: Urbana, Illinois, Illinois Water Survey, 78 p.

____________, 1985, Practical guide for groundwater sampling: Urbana, Illinois, Illinois Water Survey, ISWS Contract 374, 93 p.

Driscoll, F., 1986, Groundwater and wells: St. Paul, Minnesota, Johnson Div., 1089 p.

Heath, R.C., Basic ground water hydrology: USGS Water Supply Paper 2220, 84 p.

SECTION 5.1
WATER-LEVEL MEASUREMENTS

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Water Level Measurements

Project Number ________ Project Location ______________________ Date ____________

Measuring Instrument ______________ Field Person ______________________________

Well Number	Elevation Top of Well Riser (M. S. L.)	Distance Between Top of Well Riser and Rim of Protective Casing (Last)	(Today)	Depth to Water from Top of Well Riser	Elevation Water Level (M.S.L.)	Time

Source: MADEP

Figure 5.1-1

Example of a Water-level Record Sheet.

SECTION 5.2
IN-SITU HYDRAULIC CONDUCTIVITY TESTS

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5.2 IN-SITU HYDRAULIC CONDUCTIVITY TESTS

5.2-1 PURPOSE

In-situ tests to determine hydraulic conductivity (K) should be performed in nearly all hydrogeologic investigations. Hydraulic conductivity can be determined from both field and laboratory tests. Laboratory tests, such as permeameter tests and grain-size analysis, are discussed in Section 3.8 Laboratory Tests of Soil and Rock. Packer tests and pumping tests are additional field techniques that can be used to obtain in-situ data about hydraulic conductivity. The applications and requirements of these tests are presented in Section 5.3 Pumping Tests and Section 5.4 Packer Tests. This Standard Reference (SR) provides guidelines for the measurement of in-situ hydraulic conductivity using boreholes or monitoring wells. These tests are often referred to as "slug" or "permeability" tests. This SR presents a discussion of the various applications, test procedures, and data interpretation methods that can be used.

As shown on Table 5.2-1, hydraulic conductivity values for natural soil and rock materials vary over more than 10 orders-of-magnitude. At many sites, the point-specific hydraulic conductivity varies spatially (i.e., vertically and horizontally) by several orders-of-magnitude. The need to define the variability of the hydraulic conductivity (K) will depend on the objectives of the investigation. For water-supply investigations, minor variations in hydraulic conductivity may not be important. In contaminant transport modeling, however, any variation and spatial distribution of hydraulic conductivities may be of prime importance in estimating rates and directions of contaminant movement. Knowledge of the spatial variation can be especially valuable in interpreting the depositional environment to extend prediction of the contaminant movement outside the area tested.

When comparing various techniques for measuring hydraulic conductivity, it is important to recognize the scale of the test measurement and how that relates to the objectives of the investigation. Laboratory tests can be used to measure hydraulic conductivity of cohesive undisturbed samples that range in vertical length from a couple of inches to a few feet. By comparison, pumping tests may evaluate the entire aquifer thickness, a sample on the order of tens to several hundred feet in thickness. In many cases it is advisable to perform several types of tests, at different environmental scales, in order to raise the level of confidence in data interpretation.

In-situ measurements of hydraulic conductivity have the following advantages and disadvantages as an aquifer characterization method:

Advantages

- This method allows the estimation of in-place hydraulic conductivity.

- The methodology eliminates the problems associated with collection of undisturbed samples and the effects of testing apparatus associated with laboratory tests.

- Tests can be performed quickly in the field and at low cost. The drilling of observation wells and multi-day pumping tests are not required.

- At contaminated sites, treatment of contaminated discharged water is minimized.

- Unlike conventional pumping tests, in-situ tests allow for the estimation of the hydraulic conductivity of discrete zones within an aquifer.

Disadvantages

- Only the hydraulic conductivity in the immediate vicinity of the well or borehole is measured; this may not be representative of the average hydraulic conductivity of the aquifer or even the tested zone.

- Because test data analysis requires many simplifying assumptions, the hydraulic conductivity values are generally accurate only to an order-of-magnitude.

- Unlike pumping tests, the storativity parameter (S) or specific yield (Sy) usually cannot be determined.

Measurements of hydraulic conductivity are of fundamental importance in almost all hydrogeologic investigations. Hydraulic conductivity values are used:

- To estimate rates of ground water flow.

- To estimate responses of aquifers to applied stresses, such as pumping.

- To estimate the rate of movement of various chemicals in tested subsurface zones.

- To identify zones favorable for development of ground water resources.

- To estimate soil or rock transmissivity where pumping tests are not feasible due to extremely low permeabilities or highly contaminated ground water.

- To construct and calibrate ground water flow models.

5.2-2 THEORY OF FLOW THROUGH SATURATED POROUS MEDIA

Hydraulic conductivity is a measure of the ease of flow of a specific fluid through a specific porous medium. Hydraulic conductivity was first described by the empirical relationship known as Darcy's Law, which states that the flow rate (Q) through a given cross-section of porous media (A) is directly proportional to the hydraulic gradient (dh/dl) and the hydraulic conductivity (K). Hydraulic conductivity is also known as the coefficient of permeability. The two-dimensional expression of Darcy's Law is:

$$Q = K\,A\,(dh/dl)$$

where,

- Q = flow or discharge (volume/time)
- A = cross-sectional area (length squared)
- dh/dl = hydraulic gradient (length/length; dimensionless)
- K = hydraulic conductivity (length/time)

Hydraulic conductivity is a function of the properties of the fluid and the properties of the porous medium. The fluid properties that influence hydraulic conductivity are the fluid density and viscosity. The influencing properties of the medium are porosity, particle size, shape, distribution, and sorting. Consequently, K will vary for the same fluids in different geologic materials and for different fluids (e.g., water and oil) in the same geologic materials.

The terms permeability and hydraulic conductivity are often used interchangeably. This can be confusing. In the strictest usage, permeability, or intrinsic permeability, is the property of the porous solid material through which a fluid is moving. Intrinsic permeability is generally expressed in units of darcys, and is represented by the letter "k" (lower-case). The relationship between hydraulic conductivity and intrinsic permeability is expressed in the following equation:

$$K = k\frac{\rho\,g}{\mu}$$

where,

- K = hydraulic conductivity (length/time)
- k = intrinsic permeability of the porous media (length squared)
- ρ = density of fluid (mass/length)
- g = acceleration of gravity (length/time)
- μ = dynamic viscosity of fluid (mass/length time)

This equation becomes important when dealing with fluids having a specific gravity greater or less than 1. Typical values for hydraulic conductivity and intrinsic permeability for a wide spectrum of geologic media are presented in Table 5.2-2.

Intrinsic permeability is a property of the porous media regardless of the fluid it contains. It is defined by the equation:

$$k = C\, d^2$$

where,

k = intrinsic permeability (length squared)
C = a 'shape' factor (dimensionless)
d = mean grain-size diameter or effective grain diameter (length squared)

When water is the only fluid in question, hydraulic conductivity values are often used to compare variations in the intrinsic permeability of the media. It should be kept in mind that hydraulic conductivity will vary if either the properties of the fluid or the media change. Large differences in water temperature will also affect hydraulic conductivity values. Fluids, such as creosote, that have densities different from water will also affect the hydraulic conductivity values. For additional information on hydraulic conductivity, the reader is referred to any introductory text on ground water or hydraulics, such as Freeze and Cherry (1979) or Fetter (1988).

5.2-3 IN-SITU TESTS AND TEST PROCEDURES

5.2-3.1 Test Conditions

In-situ tests to determine hydraulic conductivity can be conducted either in open boreholes as drilling proceeds or in monitoring wells after they are installed and developed. In-situ tests can be divided into two types: 1) variable-head tests and 2) constant-head tests. If monitoring wells are used for water-quality sampling, it is often preferable to perform a rising-head test to avoid the introduction of water that does not originate within the formation. Prior to testing, all monitoring wells should be developed to a point that further development does not result in a noticeable increase in water yield. Variable-head and constant-head tests can be performed in either an open borehole or a screened well.

5.2-3.1.1 Borehole Tests

Two types of borehole tests can be performed during the advancement of cased boreholes. A flush-bottom test makes use of only the bottom cross-sectional area of a borehole, while an open hole test is conducted on a section of uncased hole. A flush-bottom test is one in which casing is advanced to the bottom drilling depth; it provides an estimate of the vertical hydraulic conductivity. On the other hand an open-hole test generally provides an estimate of the horizontal hydraulic conductivity, because normally the side-wall area of an open hole is much greater than its bottom area. If open-hole tests are to be performed in poorly consolidated formations, it may be helpful to fill the casing with clean coarse sand for the desired test interval and then carefully pull back the casing to just below the top of the sand. The hydraulic conductivity of the sand must be significantly greater than the hydraulic conductivity of the formation, and not impose resistance to water movement. This technique will help to prevent collapse of the borehole walls during exposure of the test zone. Flush-joint casing should be used in boreholes where in-situ tests are planned. If field tests are planned in cased holes, care should be taken during drilling to minimize disturbance and smearing on the borehole walls. In many cases, augered holes are not suitable for open borehole tests due to the unavoidable smearing and disturbance of the soil and borehole walls. Flush-bottom tests run inside an open-end auger flight also may give erroneous data due to the excessive water leakage through auger flight joints.

5.2-3.1.2 Tests in Monitoring Wells

The test procedures for monitoring wells are essentially the same as for boreholes, except that the test zone is pre-determined by the location of the screen and sand pack. Figure 5.2-1 shows the typical design of a monitoring well and the configurations of measurements pertinent to hydraulic conductivity tests. The total length of the test zone (L) should be determined as the total saturated length of the sand pack, including all of the screened interval. Monitoring well tests provide estimates of horizontal hydraulic conductivities.

The maximum hydraulic conductivity that can be measured in any given well may be limited by the hydraulic conductivity of the sand pack and by the open area of the screen.

5.2-3.2 Variable-head Tests

Variable-head tests are performed by causing a sudden ("instantaneous") rise or drop of the static water level in a borehole or well casing or riser. This deviation from the initial head (static water level equated to zero) is termed excess head.

5.2-3.2.1 Falling-head Test

A falling-head test is initiated by either quickly injecting water into the well or, preferably, by displacing the standing water upward with a tube-shaped slug to create an excess head. The drop in water level with time is measured as the excess head declines to zero (the static water level). For a given well construction, the rate at which the water level drops is controlled by the formation characteristics, provided that the well was properly constructed. A plot of head data (H-x) versus time data ($t\text{-}t_o$) is used to calculate the hydraulic conductivity. Figure 5.2-1 shows, schematically, a falling-head test.

Falling head tests should not be performed in wells where the screened interval straddles the water table. To measure hydraulic conductivity, the well screen must be placed entirely within the saturated portion of the aquifer. If the top of the screen is at, or close to, the present water table, rising head tests should be performed.

5.2-3.2.2 Rising-head Test

A rising-head test is quite similar to a falling-head test except that the water is suddenly displaced downward in the casing or riser pipe. In this case, the dissipation of negative excess head will be measured. The immediate water-level rise (x-H) is plotted against time after the depression is initiated ($t-t_o$), and these data are used to calculate the hydraulic conductivity. Figure 5.2-1 also illustrates a rising-head test.

5.2-3.2.3 Requisite Data

The location of the well screen or test zone with respect to the geologic materials shown on a boring log must be known. In order to calculate the hydraulic conductivity, the following information must be recorded:

H - static water level prior to start of test.

r - radius of the inside of riser pipe.

R - radius of the bottom of the casing or, if more appropriate, one-half the effective diameter of the borehole.

L - length of the zone below the casing in a open hole test, or the length of the screen and saturated filter pack for monitoring wells.

Ho - initial excess head at time t = 0 (to).

h - the amount (length) of positive or negative excess head that is created in the well or borehole.

($x, x, x...x_n$) - water-level head measurements at various times (t).

(t, t2, t3..tn) - elapsed times corresponding to the times when water-level (head) measurements are made.

These relationships and related measurements are illustrated in Figure 5.2-1.

5.2-3.2.4 General Test Methods

The following procedure presents a general method for performing a slug test. There are a number of variations to this general method developed for specific hydrogeological conditions. The variations presented in the following sections should be researched before the test is performed.

(a) Slug Injection or Withdrawal

The height of water displacement caused by slug injection or withdrawal must be accurately known in order to calculate the hydraulic conductivity based on the head difference at different time intervals. The preferred technique of creating water displacement upward for a falling-head test is to suddenly lower a weighted cylindrical solid beneath the static water level. By knowing the volume of the cylinder, the height of rise in water level (H_o) can be calculated for any diameter well. The technique of quickly pouring water in the well ("slugging"), commonly used in the past, gives significantly less accurate values of hydraulic conductivity in moderately to highly permeable media. The slug should be injected as quickly as possible. It is important to remember that a falling-head test should not be conducted when the initial static head in the well is below the top of the screened zone or the test will be affected by unsaturated conditions in the filter pack and the aquifer.

To create a downward displacement of the water column with respect to static level, either of two techniques is recommended. If water quality integrity is of concern or if the formation is believed to be highly permeable, the well can be pressurized with compressed air at the well head. At $t = 0$, the pressurized plumbing is suddenly vented and kept open. The other technique is to lower a weighted cylinder just below the static level and, upon water-level equilibration to static, remove the cylinder quickly. This procedure follows naturally after the cylinder-displacement rising-head test described above. Under certain circumstances and if the formation is not very permeable, a bailer or well pump may be satisfactorily used to remove a measurable volume of water over a period of several seconds.

(b) Equipment for Water-level Measurements

Once the excess head has been produced by either lowering or raising the water level, the altered water level will decay to its initial static level at a rate directly related to formation permeability. Measurements of the water levels at regularly timed intervals after the start of the test should be recorded. Figure 5.2-2 is an example of a field form that can be used to record data when conducting a variable-head test in a borehole.

If the formation materials are relatively homogeneous, the water level will recover at a logarithmic rate, with rapid recovery occurring during the early part of the test. In highly permeable formations, the measurement of water levels at frequent intervals is particularly critical during the early part of the test. The frequency of measurement is the same as that discussed in Section 5.3-5.3 and shown in Table 5.3-1.

Various types of equipment can be used to take water-level measurements, including "plunkers," chalked tapes, electric water-level meters, interface probes, and pressure transducers. Section 5.1 Water-level Measurements contains information on various types of water-level measuring equipment. The selection of the measuring equipment should be based on the anticipated rate of recovery. In highly permeable sands and gravels, water levels may recover almost immediately, making accurate manual measurements impossible. A pressure transducer, an instrument that can record the hydrostatic pressure of the column of water above the transducer (i.e., water levels) at an extremely rapid rate, is required for tests under these types of conditions.

(c) Duration of the Test

The length of time required to obtain sufficient test data is dependent on the volume of the slug (i.e., initial height of excess head), the hydraulic conductivity of the formation, and the configuration of the test zone. If the test zone is relatively permeable, the water level should be monitored until it returns to the initial static level.

If the test data are being collected manually, excess head versus time data should be plotted on a semi-log plot as the test proceeds. In order to calculate a valid hydraulic conductivity, a portion of these data should plot as a straight line on a semi-log plot. Once a sufficient number of readings are obtained for determining the straight-line fit, the test can be terminated. A second test (i.e., duplicate test) in each well or borehole is recommended as a check on the accuracy and reproducibility of the results.

Using the best preliminary estimate of hydraulic conductivity (K), the analytical equations discussed in this section can be used to estimate the amount of time required for an in-situ field test. If the hydraulic conductivity is quite variable at the site, the sequence of tests should proceed from the lowest permeability to the highest to allow adequate time for recovery of formations with low values of hydraulic conductivity.

5.2-3.2.5 Test Procedure for Boreholes and Monitoring Wells

The following procedures can be used for in-situ hydraulic conductivity tests conducted in either temporarily cased boreholes or in finished monitoring wells. Due to the stable borehole condition of a monitoring well, tests conducted in a temporarily cased borehole presents more potential for hole condition irregularities, and the procedure is more demanding. Steps 1 through 7 are used in borehole tests, while only Steps 3, 4, 5, and 6 are necessary when a test is run in a monitoring well.

1. Advance casing to the desired depth.

2. Carefully wash out all the material to the bottom of the casing until the wash water remains clear. If an open-hole test is desired, carefully pull back the casing to an appropriate depth. Just prior to pulling back, the casing can be filled with coarse sand to prevent collapse of the borehole walls when the casing is pulled back. The water level in the casing must be maintained at or above the static water level to prevent collapse of soil into the borehole or the movement of soil up the casing. This is particularly important during the removal of the drilling rods. Measure the depth of the hole to determine if any voids were created below the bottom of the casing. If a void is found, lower

the casing below the void and carefully repeat the drilling and washing procedure.

3. Measure the static water level from the top of the casing; confirm this reading with a second measurement five minutes later.

4. To conduct the test, fill the casing with clean water to the desired height above the static water level or lower the water level to the desired depth. Typically, excess heads of 5 to 20 feet are used. The choice of the length of the imposed head will depend on the depth to the static water level, the permeability of the formation, and the amount of time allowed for the test. The greater the excess head imposed during the test, the faster the rate at which initial recovery will occur.

5. Obtain water-level measurements at the prescribed time intervals until the water level stabilizes or adequate recovery data have been obtained.

6. Check for leakage during the test; air bubbles rising in the casing may indicate leakage around casing joints. Also, check for water flowing between the outside of the casing and the ground surface, indicating a leaky seal between the casing and borehole walls.

7. After the test has been completed, measure the depth of the borehole to determine if any caving has occurred during the test. If caving has occurred, the test-interval length (L) existing at the end of the test should be used in the calculations.

5.2-3.3 Constant-Head Test

In a constant-head permeability test, water is added to the well or borehole at a rate sufficient to keep a constant water level in the well. The water-level reference point usually is the top of the well casing. Constant-head tests are only suitable for permeable soils such as sands and gravels. Figure 5.2-3 is a schematic of a constant-head test in a monitoring well, showing the measurements needed.

5.2-3.3.1 Requisite Data

In order to calculate the hydraulic conductivity, the following information should be obtained and recorded:

H - static water level, measured prior to the start of the test.

r - radius of the riser pipe.

L - .... the length of the zone below the casing in a borehole or the length of the screened interval in a monitoring well including the full distance from the bottom to the top off he filter pack.

$(Q, Q_2,...Q_n)$ - flow measurements at various times (t).

$(X, X_2,...X_n)$ - water-level measurements at various times (t).

5.2-3.3.2 General Test Methods.

(a) Constant Flow

The constant flow rate required to maintain a selected excess head elevation has to be experimentally determined, and then maintained for a short period of time. Generally, a flow meter is connected to the pump discharge line that goes into the well to monitor the flow. It is important that the flow meter be calibrated, especially when low flows are used. The rate of injection should begin low and systematically be increased until a steady rate of flow is established. In some cases an anti-surge device should be placed in the supply line near the pressure gauge to obtain steady flow readings. The water level should be closely monitored at some reference point, preferably at or near the top of the casing.

(b) Duration of the Test

The test should be run until a steady flow rate is maintained for at least 15 minutes to one-half hour.

5.2-3.3.3 Test Procedures for Boreholes and Monitoring Wells.

The procedure described below applies to boreholes. For monitoring wells only Steps 3 and 4 are necessary.

1. Advance casing to the desired depth

2. Carefully wash out all the material to the bottom of the casing until the wash water remains clear. The water level should be maintained at or above the static water level to prevent squeezing of soil into the casing. This is particularly important during removal of the drilling rods. Measure the depth of the hole to determine if a void was created below the bottom of the casing.

If an open-hole test is desired, bump or pull back the casing to the desired depth.

3. Measure the static water level from the top of the casing.

4. Fill the riser with clean water and maintain the water-level at the top of the riser, or at some fixed elevation, by pumping water in at the experimentally determined, appropriate constant rate. The volume of water entering the casing should be measured with a flow meter and recorded at regular time intervals, such as every minute, to determine if a stable flow rate has been achieved and is being maintained. If the target excess head elevation is below the top of the casing and is not visually confirmable, water-level measurements should also be recorded each time the flow is measured. An example of a field form for recording constant-head test data is shown in Figure 5.2-4.

5.2-4 DATA ANALYSIS

Several methods are available for analyzing data obtained from in-situ hydraulic conductivity tests. Most methods incorporate graphical techniques, such as semi-log and log-log plots, to evaluate the data and select values for the calculations.

When evaluating these tests, the calculated hydraulic conductivity should be compared to an expected hydraulic conductivity based on the formation characteristics. (See Table 5.2-1.) Potential sources of error that can affect in-situ hydraulic conductivity tests include:

- Leaky casing or riser joints
- A low permeability skin on the borehole wall formed during drilling
- Uncertainty about the initial head
- Failure to allow the pressure transducer to stabilize
- Stress release around the borehole
- Incorrect readings
- Bridging of seals
- Entrapped air in the sandpack or formation
- Anisotropy of the formation
- Sandpack or screen permeability limitations
- Partial-penetration effects (saturated zone within the aquifer is not fully screened)
- Fractures
- Multi-phase fluids

The analysis of well or borehole hydraulic conductivity test data is based on modifications of the Thiem equation for steady-state conditions and the Theis equation for transient conditions. A few of the more commonly used analytical methods are summarized in this section. Inherent in all these methods are several simplifying assumptions concerning the aquifer properties (i.e., homogeneity, isotropy) and the test methods (instantaneous water-level change). When selecting a particular analytical method, it is important to consider the basic assumptions that underlie the mathematical expressions. In many cases it may be advisable to evaluate the data using several methods and examine the range of hydraulic conductivities that are obtained. When reporting a calculated in-situ hydraulic conductivity, the analytical method(s) used should always be referenced.

5.2-4.1 Analysis of Variable Head Test Data

In this section three analytical methods are presented that are commonly used to evaluate variable head test data. Additionally, a large number of methods developed for specialized conditions can be found in the literature. More sophisticated analytical methods than those discussed here may need to be applied under certain test conditions. In particular, the reader may want to apply the method of Bouwer and Rice (1976) for unconfined aquifers with wells that are either partially or completely penetrating the aquifer.

5.2-4.1.1 Hvorslev Time-lag Method

The Hvorslev method (Hvorslev, 1951) is based on a modification of the Thiem equation for steady-state flow. This method includes the following assumptions or conditions:

- The aquifer has unconfined conditions
- The aquifer is homogeneous and isotropic
- The aquifer has infinite areal extent
- The soil and water are incompressible
- Steady-state conditions
- The change in water level is instantaneous
- The test zone partially or fully penetrates the aquifer
- Effects of aquifer storage are assumed to be small and are ignored

The Hvorslev method is based on the following equation:

$$K = \frac{r^2 \ln(mL/R)}{2LT_o}$$

where,

K = hydraulic conductivity (length/time)
r = radius of riser or casing (length)
m = a transformation ratio to allow for some anisotropy in the vertical direction (dimensionless)
where,

$$m = \sqrt{K_h/K_v}$$

and,

K_h = horizontal hydraulic conductivity (length/time)
K_y = vertical hydraulic conductivity (length/time)

L = length of test zone (length)
R = effective radius of borehole or test zone (length)
T_o = lag time value, or the time at which $\ln(h/H_o)$ equals 0.37 on the head versus time data plot (time)

The value of T_o is determined as follows:

The excess head (h), normalized by dividing by the initial excess head (H_o), is plotted on the log scale against corresponding values of time (t) on the arithmetic scale of semi-log paper. The value of T_o is determined graphically as that value where the normalized head equals 0.37, as shown on Figure 5.2-5.

If the head versus time data deviates significantly from a linear plot, it may indicate bad test data or that the assumptions of the equation are not met. In this case, this method will not provide a reliable value for hydraulic conductivity.

Hvorslev developed this method for a variety of borehole or well configurations by including a "shape factor" in the equation. Figure 5.2-6 presents the borehole or well geometry for various shape factors (page 1 of 2) and their corresponding equations (page 2 of 2).

5.2-4.1.2 Bouwer and Rice Method

A method of analysis quite similar to that of Hvorslev's was developed by Bouwer and Rice (1976). Their equation for calculating hydraulic conductivity is identical to the Hvorslev equation except that the term $\ln(mL/R)$ is replaced by $\ln(r_e/R)$. Thus the equation is:

$$K = \frac{r_c^2 \ln(r_e/R)}{2Lt} \ln \frac{Y_o}{Y_t}$$

where,

K = hydraulic conductivity (length/time)
L = length of the test zone (length)
r_c = casing radius (length)
r_e = effective horizontal radius over which the instantaneous slug (Ho) is dissipated (length)
R = radius of borehole in test zone (length)
t = selected time since the slug was initiated (time)
Y_o = initial (t = 0) change in head (length)
Y_t = value of excess head (h) at selected time (t) (length)

The assumptions or conditions that apply to the Hvorslev method also apply to this method. The benefit of this technique is that the calculated values are based on a more vigorous estimate of the radius of influence of the test than those derived from the Hvorslev equation. This appraisal is based on the fact that instead of using a best estimate of the effective radius of the borehole or test zone, as the Hvorslev method does, Bouwer and Rice have developed a graphical procedure to calculate the effective distance from the borehole that is affected by the slug.

The graphical procedure involves determining the values of two coefficients (A and B) that appear as separate curved plots in Figure 5.2-7. These values, which are functions of the value of L/R that is specific to a well geometry, are used in the following equation to derive ln (r_e/R):

$$\ln (r_e/R) = \left[\frac{1.1}{\ln(H/R)} + \frac{A+B \ln [(D-H)/R]}{L/R} \right]^{L}$$

where,

A = a coefficient that is a function of (dimensionless)
B = a second coefficient that is a function of L/R (dimensionless)
r = effective horizontal radius (length)
H = distance from the water table to the bottom of the open test zone of the well (length)
L = length of the test zone (length)
D = saturated aquifer thickness (length)
R = radius of borehole in test zone (length)

In applying the above equation, Bouwer and Rice (1976) determined that the following conditions should be observed.

1) If 1n [D-H/R] is greater than 6, this term should be set equal to 6 in the above equation for determining ln (r_e/R).

2) If D equals H (implying that the open test zone fully penetrates the aquifer), the following equation should be used to determine ln (r_e/R):

$$\ln (r_e/R) = \left[\frac{1.1}{\ln(H/R)} + \frac{c}{L/R} \right]^{-1}$$

where,

c = a third coefficient that is a function of L/R (dimensionless)

and, all other terms are as previously defined.

In Figure 5.2-7, if the open test zone fully penetrates the aquifer being tested, only the "C" curve is read.

To implement this method, the field data are plotted as h on the log scale versus t on the arithmetic x-axis. A best fit straight line is drawn through the data points. Figure 5.2-7 is consulted to obtain the coefficients necessary to compute 1ln (r_e/R). The value of Y_o is determined by the intersection of the fitted straight line with the zero point on the X-axis. For a selected time (t), the corresponding value of Y_t on the Y-axis is read. All of these values are substituted into Bouwer and Rice's hydraulic conductivity equation to compute K.

5.2-4.1.3 Cooper et al. Type-Curve Matching.

Cooper et al. (1967) developed and Papadopulos et al. (1973) extended a type-curve matching method, based on the Theis equation, with the following assumptions or conditions:

- The aquifer is confined
- The aquifer is homogenous and isotropic
- Aquifer has infinite horizontal extent
- The change in water level is instantaneous
- Transient flow conditions (non-steady state) exist in the immediate proximity of the well
- The well fully penetrates the aquifer
- Aquifer can have limited vertical extent
- The aquifer has a uniform aquifer thickness

The values for the transmissivity (T) and hydraulic conductivity (K) can be calculated from the equations:

$$T = \frac{r_c^2}{t} (Tt/r_c)^2 \quad \text{and} \quad K = T/b$$

where,

T	=	transmissivity near the well (length squared/time)
Tt/r_c^2	=	a time parameter: value is usually selected as 1.0 on the type-curve overlay at the match point
r_c	=	radius of the casing or riser (length)
t	=	time determined from match point (time)
K	=	hydraulic conductivity near the well (length/time)
b	=	aquifer thickness (length)

This method involves plotting the head and time data on semi-log paper and determining the best data fit to one in a family of type-curves that are plotted at the same scale. An example of the type-curves is shown on Figure 5.2-8. The normalized head values are plotted on the vertical, arithmetic scale, and the corresponding time values are plotted on the horizontal, log scale. The field data curve is then superimposed on the type-curve set. With the arithmetic axes coincident, the data plot is translated horizontally to a position where the field plot best fits the type-curve. Once the match point is determined, the time value (t) is read off the plot. Typically, a value of 1.0 is chosen for Tt/r_c^2 to simplify the calculation.

Transmissivity (T) is then calculated by solving the above equation in terms of r and t units. The value for hydraulic conductivity can be calculated by dividing the calculated transmissivity by the aquifer thickness.

5.2-4.1.4 Nguyen and Pinder Slug Test Method.

The Nguyen and Pinder method (Nguyen and Pinder, 1984) incorporates factors that account for the effects of wellbore storage and partially penetrating wells. Although it is a little more complex than the other methods that have been described, the Nguyen and Pinder method is especially suitable for conditions of low hydraulic conductivity where wellbore storage effects can be a problem. The following assumptions are inherent in this method:

- The aquifer is either confined or unconfined conditions
- The aquifer is homogeneous and isotropic
- Aquifer has infinite areal extent
- Steady-state conditions must exist around the well
- The change in water level is instantaneous
- A well either fully or partially penetrates the aquifer

This method utilizes a semi-log plot of the head (dh/dt) versus inverse time data (1/t), similar to the Hvorslev semi-log method. In addition, a log-log plot of (h/H_o) versus time data (t) is prepared. The slopes of these plots, C_3 for the log-log and C_4 for the semi-log plots respectively, are used to calculate the hydraulic conductivity according to the Nguyen and Pinder equation:

$$K = \frac{R2C_3}{4C_4 L}$$

where,

K = hydraulic conductivity near the well (length/time)
R = radius of the wellbore (length)
C_3 = value obtained from slope of semi-log plot (length/time)
C_4 = value obtained from slope of log-log plot (length)
L = test zone length (length)

5.2-4.2 Analysis of Constant-Head Test Data

The analysis of constant-head test data is based on a modification of Darcy's Law: Q = KA(dh/dl), described in Section 5.2-2. The analytical equation presented here requires the following simplifying assumptions:

- The aquifer is confined
- The aquifer is homogenious and isotropic
- The well screen is not at the upper or lower aquifer boundary
- There is a constant flow rate of water into or from the well

The following equation (Hvorslev, 1951) can be used to calculate the hydraulic conductivity (K) from a constant-head test when the screened section is installed in uniform soil away from soil boundaries (Figure 5.2-6, Case G):

$$K = \frac{Q_s \ln \left[\frac{L}{2r} \sqrt{1 + \frac{L^2}{r^2}} \right]}{2\pi L h}$$

where,

K = hydraulic conductivity (length/time)
Q_s = stabilized flow rate required to maintain a constant head (length cubed/time)
L = test zone length (length)
r = radius of riser or casing (length)
h = excess head, as height above static water-level (length)

Other well/aquifer geometries and associated equations for computing hydraulic conductivity are given in Figure 5.2-6 (page 2 of 2). In situations where the top of the well casing is not the established constant head elevation during the test, as shown in Figure 5.2-3, measurements of water level must be made to maintain a constant head. The measured distance, x, is subtracted from H to determine h; the excess head, as depicted in Figure 5.2-4.

REFERENCES

Bouwer, H., and Rice, R.C., 1976, A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells: Water Resources Research, v. 12, no. 3, pp. 423-428.

Davis, S.N., 1969, Porosity and permeability in natural materials: in Flow through porous media, ed. R.J.M. DeWeist: New York, NY, Academic Press, pp. 53-89.

Cooper, H.H., Jr., Bredehoeft, J.D., and Papadopoulos I.S., 1967, Response of a finite-diameter well to an instantaneous charge of water: Water Resources Research, v. 3, no. 1, pp. 263-269.

Fetter, C.W., Jr. 1988, Applied hydrogeology: Columbus, OH, Merrill Publishing Co., 491 p.

Freeze, R.A., and Cherry, J., 1979, Groundwater: Englewood Cliffs, NJ, Prentice Hall, 604 p.

Hvorslev, M.J., 1951, Time lag and soil permeability in groundwater observations: Vicksburg, MS, U.S. Army Corps of Engineers Waterways Experimental Station Bulletin 36, 50 p.

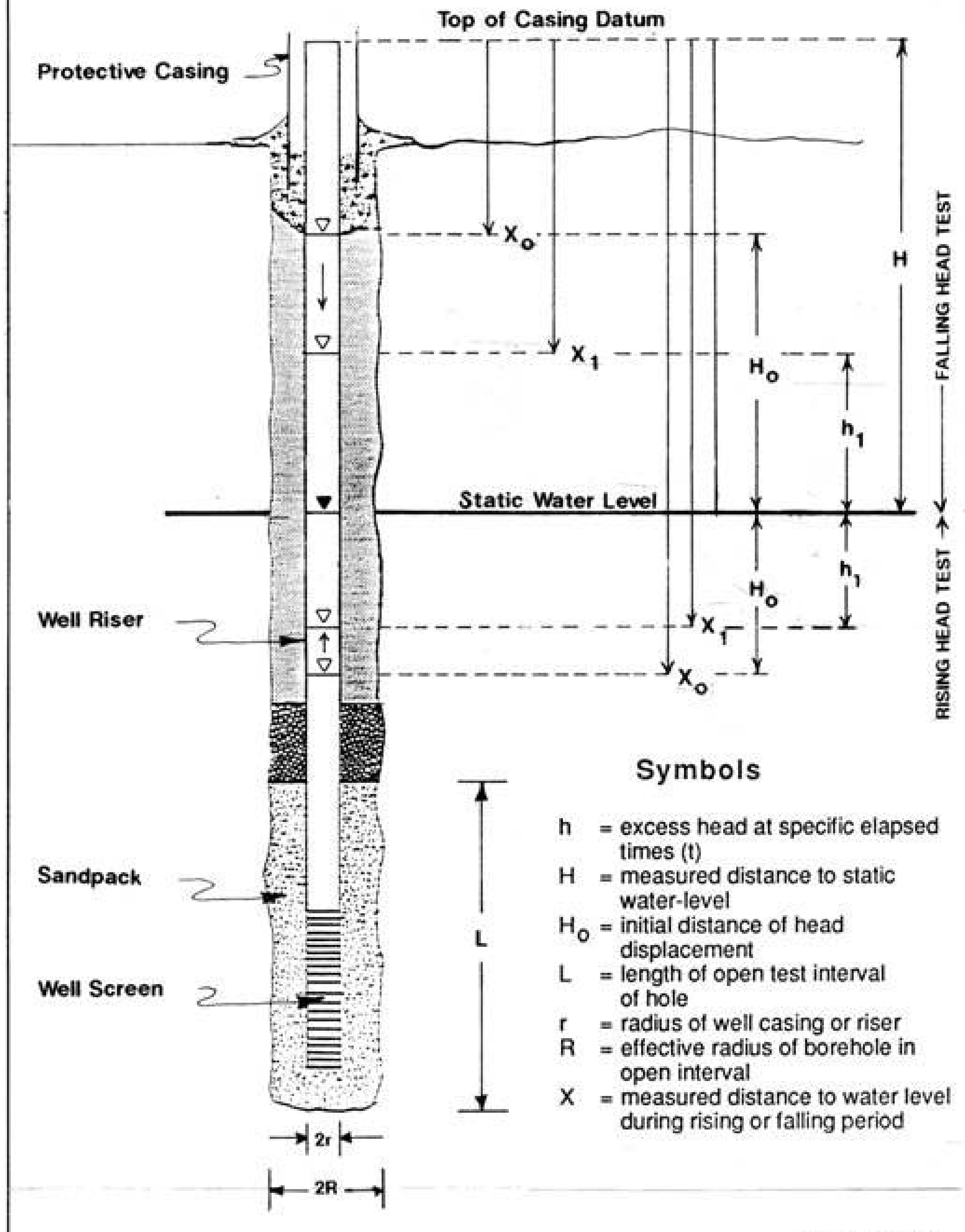
Nguyen, V., and Pinder, G.F., 1984, Direct calculation of aquifer parameters in slug tests analysis: Water Resources Monograph No. 9, pp. 222-239.

Papadopulos, I.S., Bredehoeff, J.D., and Cooper, H.H., Jr., 1973, On the analysis of "slug test" data: Water Resources Research, v. 9, no. 4, pp 1087-1089.

SECTION 5.2
IN-SITU HYDRAULIC CONDUCTIVITY TESTS

LIST OF FIGURES

LIST OF TABLES



Source: MADEP

Figure 5.2-1

Well Construction Features and Measurement Configurations for Variable-head Tests.

BOREHOLE PERMEABILITY TEST
Variable-head Test

BORING NO. ________ WELLPOINT ☐ STANDPIPE ☐ TEST NO. ________

TYPE OF TEST: FALLING HEAD ☐ RISING HEAD ☐

PROJECT ________ SITE/LOCATION ________

INSPECTOR ________ DATE ________ CHECKED BY ________ DATE ________

TIME ________ GROUND ELEVATION ________ REFERENCE ELEVATION ________

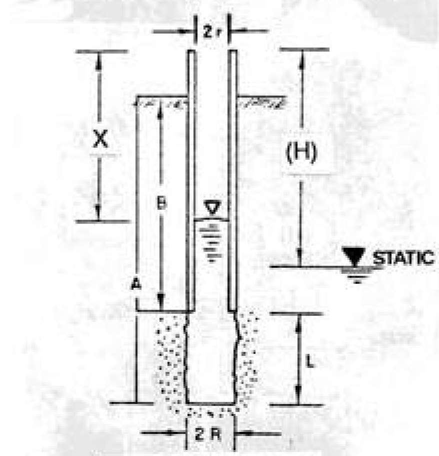
CASING I.D. ________ CASING O.D. ________

DEPTH OF BORING (A) ________ DEPTH OF TOP OF TEST SECTION (B) ________

DEPTH OF GROUNDWATER TABLE (H) ________ LENGTH OF TEST SECTION (L) ________

TYPE OF MATERIAL IN TEST ZONE (USC OR OTHER) ________

COMMENTS ________



h = H-X (falling head)
or
= X-H (rising head)
H_0 = H-X_0 (falling head)
or
X_0-H (rising head)
X_0 = X at t=0
A, B, H&L are defined above

TIME	ELAPSED TIME (t)	h/H_o	WATER DEPTH (x)	ACTIVE HEAD (h)
	0	1.0		

Source: MADEP

Figure 5.2-2

Example of a Field Form for Variable-head Tests.

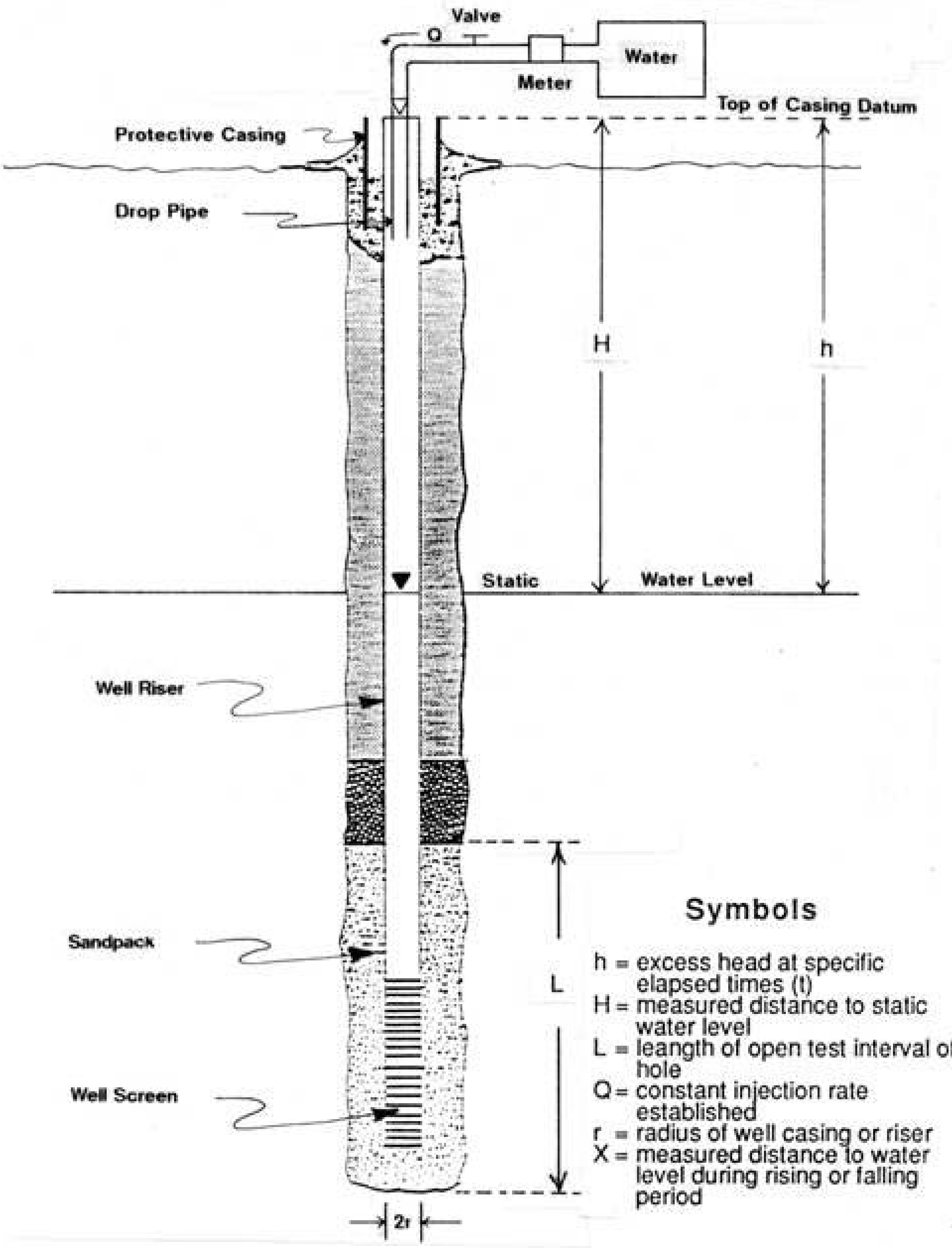


Figure 5.2-3

Source: MADEP

Well Construction Features and Measurement Configurations for Constant-head Tests.

BOREHOLE PERMEABILITY TEST
Constant-head Test

BORING NO. ______ WELLPOINT ☐ STANDPIPE ☐ TEST NO. ______

PROJECT ______ SITE/LOCATION ______

INSPECTOR ______ DATE ______ CHECKED BY ______ DATE ______

TIME ______ GROUND ELEVATION ______ REFERENCE ELEVATION ______

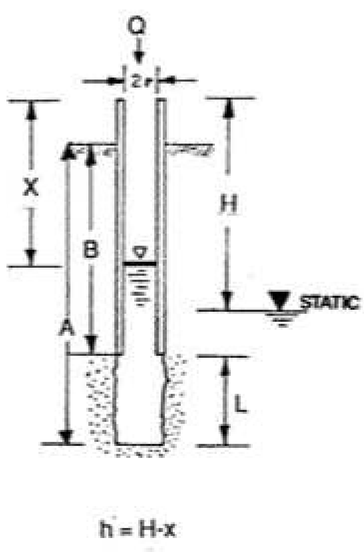
CASING ID ______ CASING O.D. ______ FLOW METER NO. ______

DEPTH OF BORING (A) ______ DEPTH OF TOP OF TEST SECTION (B) ______

DEPTH OF GROUNDWATER TABLE (H) ______ LENGTH OF TEST SECTION (L) ______

TYPE OF MATERIAL IN TEST ZONE (USC OR OTHER) ______

COMMENTS ______



h = H-x

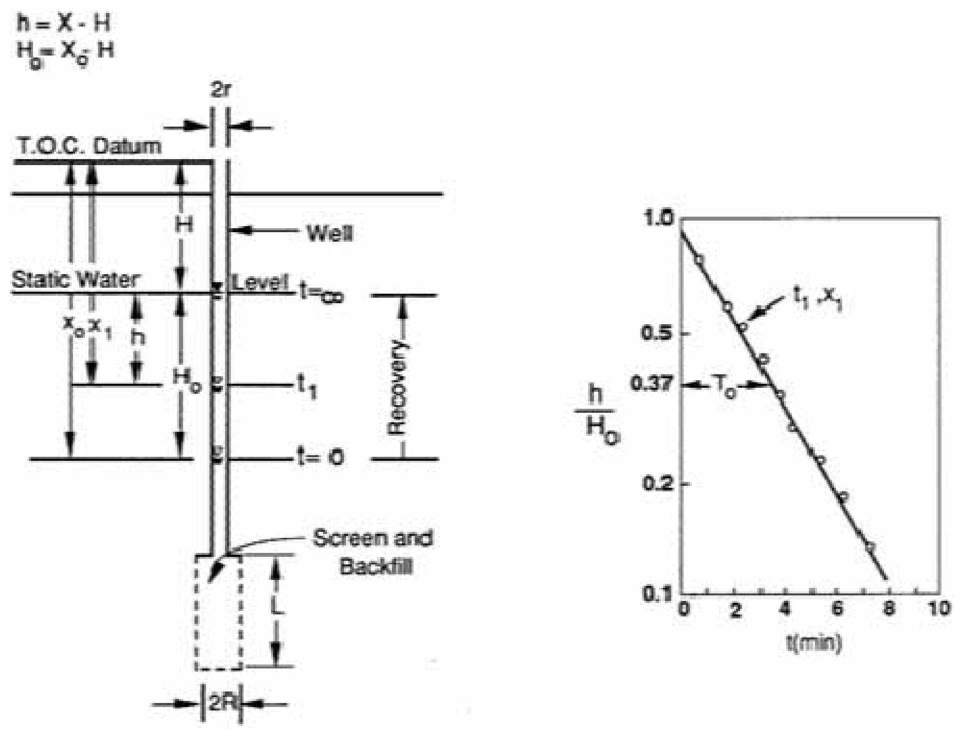
A, B, H&L are defined above

ELAPSED TIME (t) (min)	FLOWMETER READINGS (gallons)	INJECTION RATE (Q) (gal/min)	WATER DEPTH (X)	ACTIVE HEAD (h)

Figure 5.2-4

Source: MADEP

Example of a Field Form for Constant-head Tests.



(a) Test Conditions **(a) Data Plot**

Symbols

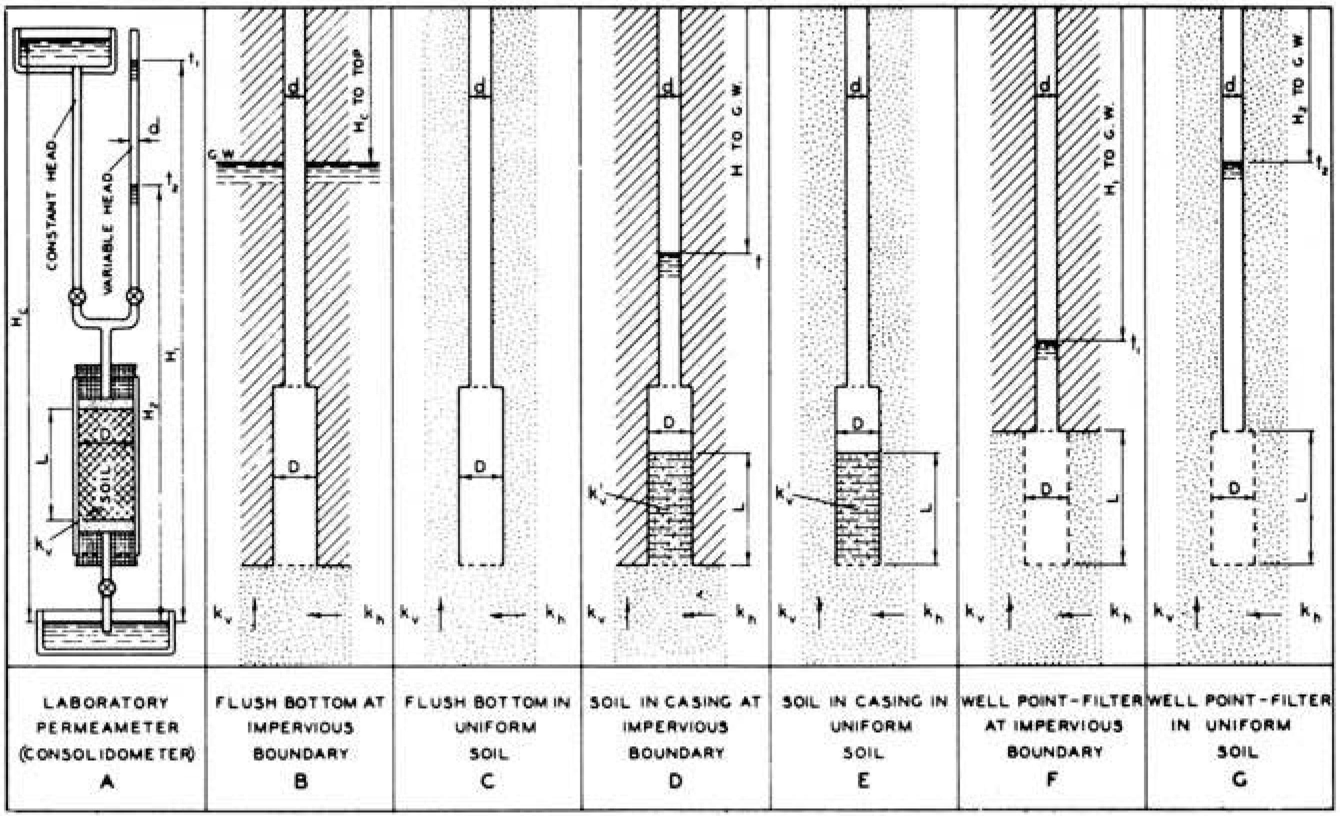
h = excess head at specfiic elapsed times (t)
H = initial distance of head displacement
L = length of open test interval of hole
r = radius of well casing or riser
R = effective radius of borehole in open interval
t = elapsed time corresponding to x values
T_o = lag-time value at $h/H_o = 0.37$
X = measured distance to water level during rising or falling period

T_o = normalized water level measurement at a specific time(t)

Source: After Freeze and Cherry (1979)

Figure 5.2-5

Basic Hvorslev Time-log Method for In-situ Head Tests.



Source: Hvorslev (1951)

Figure 5.2-6
Page 1 of 2
Hvorslev Shape Factors for Hydraulic Conductivity Testing for Various Borehole Conditions.

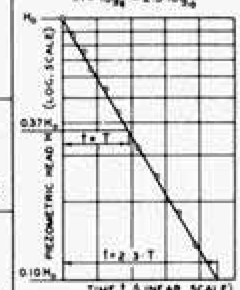
CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D\,(t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8\,(t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D\,(t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11\,(t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ FOR $d = D$
D	$k_v' = \frac{4 \cdot q\left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 H_c}$	$k_v' = \frac{d^2\left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m}\right] + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
E	$k_v' = \frac{4 \cdot q\left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 H_c}$	$k_v' = \frac{d^2\left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot T}$ $k_v' = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
F	$k_h = \frac{q \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM. INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, CM^3/SEC.
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k_v' = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM., CM/SEC.
m = TRANSFORMATION RATIO

$k_m = \sqrt{k_h \cdot k_v}$ $m = \sqrt{k_h / k_v}$

$\ln = \log_e = 2.3 \log_{10}$



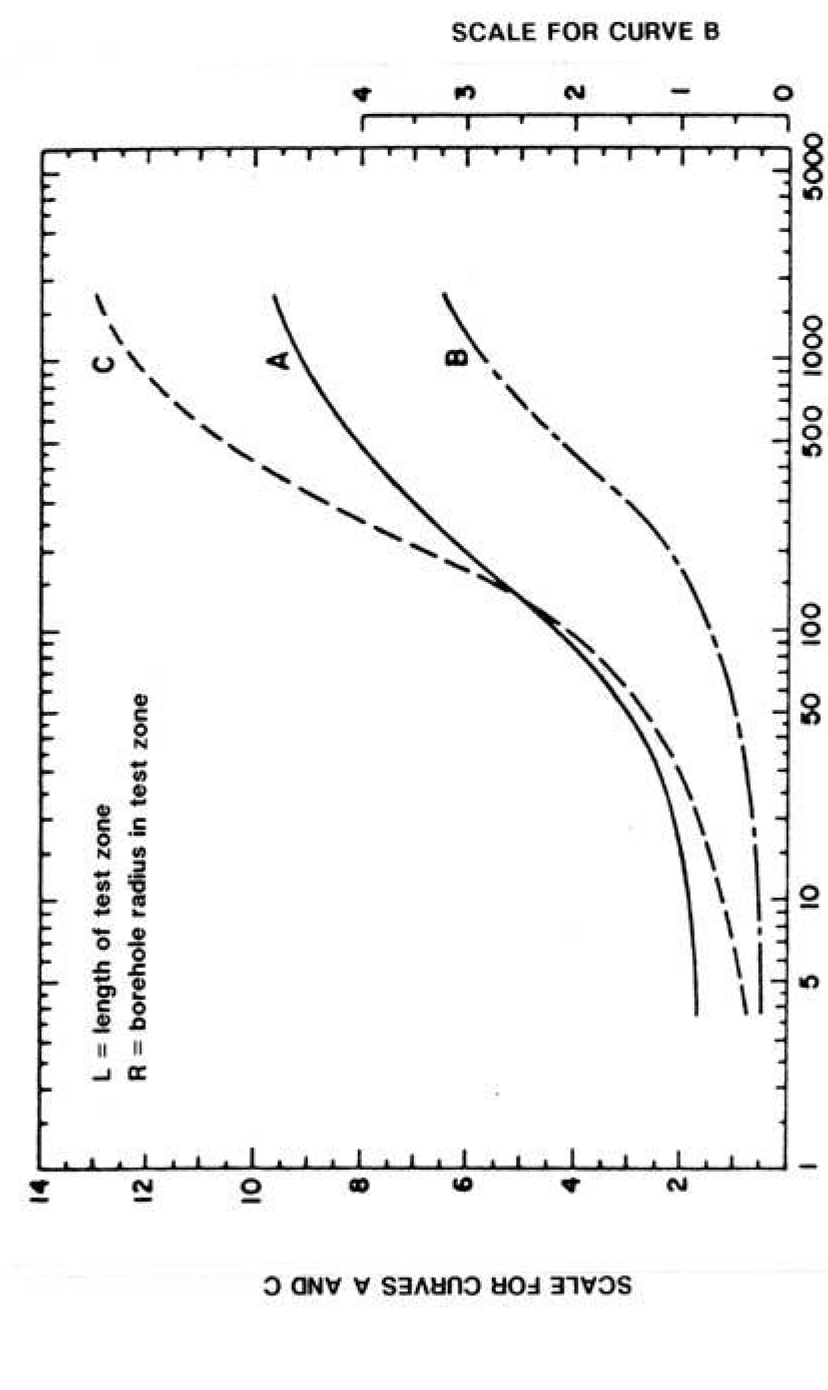
DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) - NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL - NO SEDIMENTATION OR LEAKAGE - NO AIR OR GAS IN SOIL, WELL POINT, OR PIPE - HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

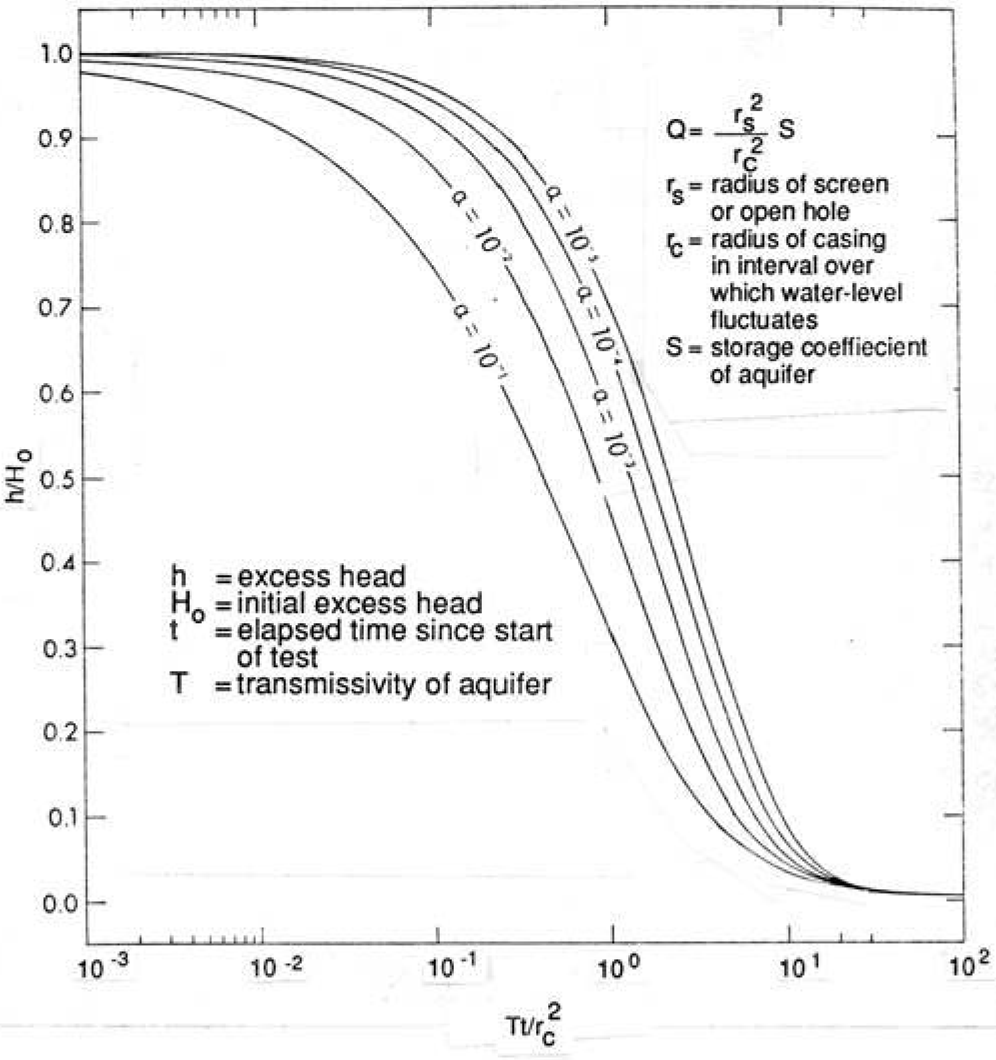
Source: Hvorslev (1951)

Figure 5.2-6
Page 2 of 2
Hvorslev Equations for Hydraulic Conductivity Testing for Various Borehole Conditions.



Source: Bouwer and Rice (1976)

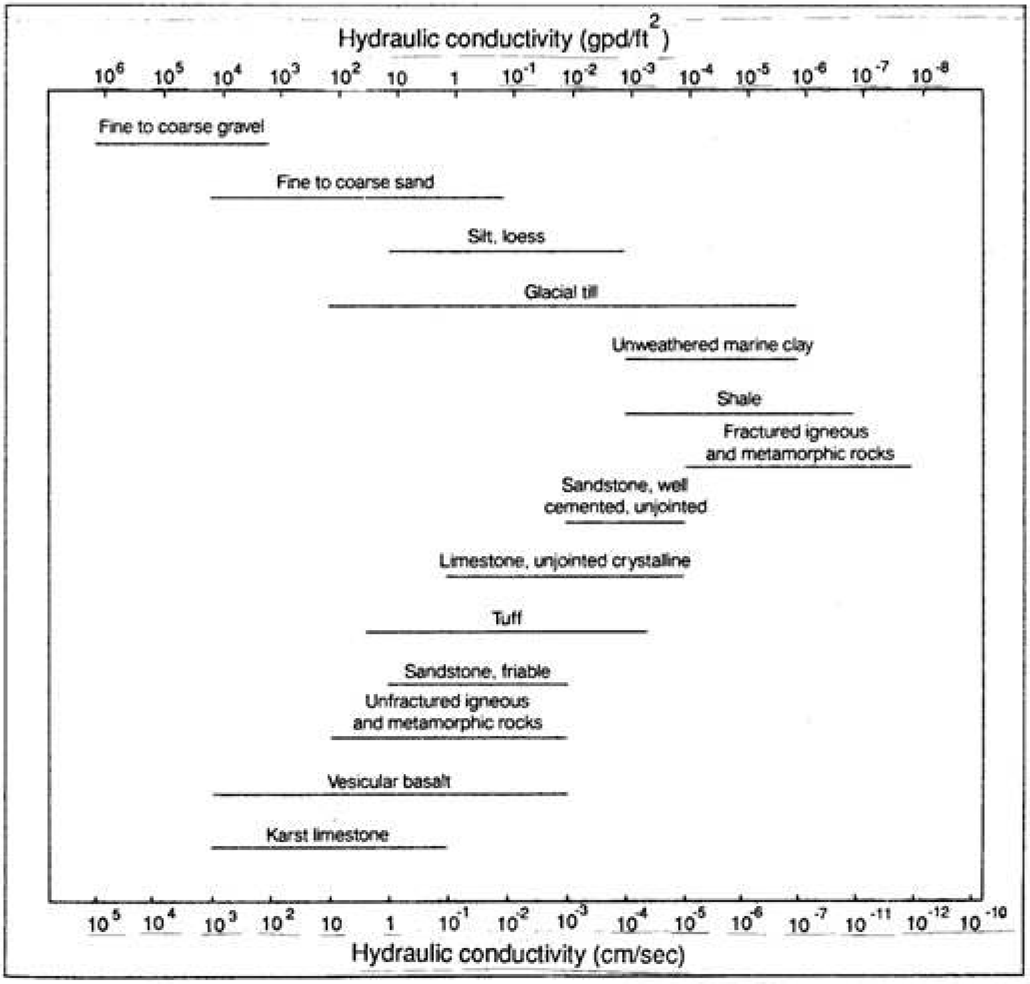
Figure 5.2-7
Graphic Determination of Bouwer and Rice's Coefficients for Calculating Effective Radius Term ln(r_e/R).



Source: After Cooper et al. (1967)

Figure 5.2-8

Type-curves for Slug Tests of Confined Aquifer Wells, Considering Wellbore Storage.



Source: After Driscoll (1986)

Table 5.2-1

Ranges of Hydraulic Conductivity for Natural Soils and Rocks

Material	ϕ Porosity (%)	K Hydraulic Conductivity (cm/sec)	K Hydraulic Conductivity (ft/day)	K Permeability (darcys @ 20 c)
Granite, dense	0.3	1.7×10^{-9}	4.9×10^{-6}	2.0×10^{-6}
Granite, fractured	1.2	2.3×10^{-5}	6.6×10^{-2}	2.7×10^{-2}
Quartzite, dense	0.6	1.6×10^{-9}	4.6×10^{-6}	1.9×10^{-6}
Schist, highly-weathered, clay-rich	48	2.7×10^{-5}	7.5×10^{-2}	3.1×10^{-2}
Schist, fractured and partly weathered	5	1.2×10^{-3}	3.4	1.4
Basalt, dense	7.7	1.2×10^{-8}	3.4×10^{-5}	1.4×10^{-5}
Tuff, friable	36	1.2×10^{-6}	3.4×10^{-3}	1.4×10^{-3}
Conglomerate, highly-lithified	17.3	4.2×10^{-7}	1.2×10^{-3}	4.9×10^{-4}
Sandstone, medium-grained	15.6	6.5×10^{-5}	1.8×10^{-1}	7.6×10^{-2}
Shale, compacted	21	3.5×10^{-9}	9.8×10^{-6}	4×10^{-6}
Limestone, dense	10.1	6.6×10^{-6}	1.9×10^{-2}	7.7×10^{-3}
Clay, marine	48.5	1.4×10^{-8}	3.9×10^{-5}	1.6×10^{-5}
Sand, medium-grained	42.9	1.6×10^{-2}	44.3	18.2
Sand, medium to coarse-grained	37.4	2.4×10^{-2}	66.9	27.5
Sand, fine-grained	40.1	1.3×10^{-3}	3.6	1.5
Silt, sandy	39.4	3.2×10^{-5}	9.2×10^{-2}	3.8×10^{-2}
Silt, loess, fine-grained	50	2.8×10^{-4}	7.9×10^{-1}	0.33
Gravel, fine-grained, some sand	32.1	7.6×10^{-2}	216	89

Note: With water at 20°C, material having one darcy permeability will have a hydraulic conductivity of 0.74 meters/day which is equivalent to 2.43 feet/day.

Source: Davis et al. (1969)

Table 5.2-2

Examples of Values for Hydraulic Conductivity, Intrinsic Permeability, and Associated Porosities

COMMONWEALTH OF MASSACHUSETTS

DEPARTMENT OF ENVIRONMENTAL PROTECTION

STANDARD REFERENCES FOR MONITORING WELLS

SECTION 5.3 PUMPING TESTS

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SECTION 5.3 PUMPING TESTS

5.3-1 PURPOSE

This Standard Reference (SR) describes various applications, design criteria, test procedures, and data interpretation methods for aquifer pumping tests. Pumping tests involve the pumping of a fluid (generally water) from a well while monitoring the water-level decline (drawdown) over time in the pumping well and surrounding observation wells.

Generally, pumping tests require a relatively large expenditure of funds, manpower and time. Failure to adequately design and execute a test program can severely compromise the data. This section is intended to provide some general procedures that should help to minimize data collection problems and errors. These procedures are meant as general guidelines only; specialized pumping test programs may require specific procedures. For example, pumping tests conducted for public water supply sources require that the test be designed and carried out in accordance with the guidelines and requirements of the Division of Water Supply. Guidelines and Policies for Public Water Systems (revised 1989) should be consulted concerning the specific test design requirements and procedures for water supply sources in Massachusetts.

5.3-2 APPLICATIONS

Until quite recently pumping tests were primarily undertaken to determine the suitability of a well and/or aquifer for water supply purposes. More recently, pumping tests have been conducted for a variety of other reasons, such as to obtain a better overall understanding of a hydrogeologic system or to design and evaluate the effectiveness of an aquifer remediation program. A list of some of the more common pumping test applications is presented below.

5.3-2.1 Water Supply Studies

- To determine sustained well yield and specific capacity.
- To determine pump sizing parameters.
- To calculate aquifer properties in order to estimate aquifer system storage, recharge rate, long-term drawdown, and potential interference with other production wells or aquifer boundaries.
- To define wellhead protection zones and recharge areas for aquifer protection purposes.

5.3-2.2 Hydrogeologic Studies

- To define aquifer characteristics such as transmissivity, hydraulic conductivity, and storativity.
- To identify local boundaries of an aquifer system.
- To provide calibration data for ground water flow models.
- To estimate dewatering requirements for excavations.
- To predict the rise in the water table as the result of a dam and associated impoundments.
- To predict the feasibility and impact of injection wells.

5.3-2.3 Contaminant Studies

- To perform chemical time-series sampling to evaluate the temporal and spatial variability of contaminants in ground water.
- To estimate capture zones of existing or proposed extraction wells and evaluate their effectiveness in intercepting and removing contaminated ground water, or non-aqueous phase liquids.
- To design remedial pumping schemes for plume containment through hydraulic controls.
- To evaluate the effectiveness of alternative remedial pumping schemes.

5.3-3 GENERAL CONCEPTS

In this section several general concepts and terms are presented that relate to the design and evaluation of pumping tests. Individuals involved in the design, implementation and analysis of pumping test data must have a working knowledge of the fundamentals of ground water hydrology. For additional information, the reader should consult one of the many texts covering well hydraulics such as Bear (1979), Driscoll (1986), Fetter (1988), Freeze and Cherry (1979), and Todd (1980).

5.3-3.1 Types of Aquifers

5.3-3.1.1 Confined Aquifer

A confined aquifer occurs when the ground water is confined by an overlying, less permeable geologic unit under pressure that is greater than atmospheric pressure. As shown on Figure 5.3-1, a confined aquifer is bounded above and below by relatively impermeable strata. The level of the water in a well that penetrates the confined aquifer is called the potentiometric (or piezometric) surface; it represents the hydrostatic pressure level of the water in the well. By definition, the water level or hydraulic head in a confined aquifer is always at or greater than the elevation of the bottom of the upper confining layer. During a pumping test in which the aquifer remains under confined conditions, the volume of water removed from storage in the aquifer is derived from the expansion of water due to the decrease in hydrostatic pressure and the compression of the aquifer matrix; discharge is not derived from gravity drainage.

5.3-3.1.2 Unconfined Aquifer

In an unconfined aquifer the water in the aquifer is at atmospheric pressure. The upper boundary of this potentiometric surface is often referred to as the water table; it is shown on Figure 5.3-2. In an unconfined aquifer the water removed from storage during a pumping test beyond the initial few minutes of pumping is the result of gravity drainage from the saturated material. The water level change resulting from water being removed from storage may not occur instantaneously; it may show a delayed response. This phenomenon, known as delayed yield, is frequently observed in pumping tests in unconfined aquifers.

5.3-3.1.3 Leaky Aquifer

A leaky aquifer is a fully saturated (semi-confined) aquifer that is bounded above and below by a less permeable layer, one or both of which transmit water to the pumped aquifer. (As the most permeable layer does not actually leak itself, the term "leaky aquifer" encompasses the above and/or below leaky layers, called aquitards.) When pumped, the water comes from both horizontal flow of water released by pressure reduction in the permeable layer and vertical flow (leakage) downward or upward from aquitards. Figure 5.3-3 is a diagram of a leaky aquifer receiving leakage from the overlying aquitard.

5.3-3.2 Aquifer Conditions

5.3-3.2.1 Steady-state (Equilibrium) Conditions

Under steady-state flow conditions the magnitude and direction of the flow velocity at any location in the aquifer is constant with time. For such a condition to exist in an aquifer, there can be no change in the water level or potentiometric surface over time. Consequently, in order to achieve steady-state conditions during a pumping test, the rate of recharge to the aquifer must become equal to the rate of withdrawal. Although in the strictest sense this condition is rarely achieved during a pumping test of several days to several weeks, long, steady-state conditions are often assumed in situations where the changes in head over time are so small that they can be considered negligible. In practice, drawdown changes of less than about 0.1 foot over 24 hours can often be attributed to factors other than a lack of balance between discharge of the well and recharge to the aquifer.

5.3-3.2.2 Non-steady-state Conditions

Non-steady-state conditions, also called transient or non-equilibrium conditions, occur when the potentiometric surface is changing over time. In a pumping test, non-steady flow conditions occur from the start of the test until steady-state conditions are achieved. Thus, analysis of most pumping tests involves the application of only transient analytical methods (solutions).

5.3-3.3 Aquifer Properties

5.3-3.3.1 Transmissivity

Transmissivity (T) is defined as the rate at which water of the prevailing kinematic viscosity is transmitted through a unit width of the aquifer under a unit hydraulic gradient. Figure 5.3-4 shows a conceptual representation of transmissivity and hydraulic conductivity (K). Transmissivity is a function of the properties of the fluid, the porous media, and the thickness of the saturated porous media. Transmissivity is equal to an integration of the hydraulic conductivities across the saturated part of the aquifer perpendicular to the flow direction. Transmissivity (T) is related to hydraulic conductivity as follows:

$$T = K\,b$$

where,

T = aquifer transmissivity (length/time)
K = average hydraulic conductivity of the saturated zone (length/time)
b = average thickness of the saturated zone (length)

The alternative English units for transmissivity are gpd/ft, which can be reduced to ft^2/day by dividing by 7.48.

The concept of aquifer transmissivity assumes horizontal flow through the aquifer that may be violated where vertical hydraulic gradients and vertical hydraulic conductivity are larger than these same components horizontally. Because transmissivity is directly proportional to both the aquifer thickness (b) and the hydraulic conductivity (K), it differs from one aquifer to another and from place to place within a single aquifer.

5.3-3.3.2 Storativity and Specific Yield

Storativity, also called the storage coefficient, is a dimensionless aquifer parameter. It is defined as the volume of water an aquifer releases or takes into storage per unit surface area of the aquifer per unit change in head. Figure 5.3-5 is an illustration of water released from storage in confined and unconfined aquifers resulting from a unit decline in head. The magnitude of the storage coefficient depends on aquifer type (i.e., whether it is confined or unconfined). In a confined aquifer the aquifer remains saturated and the amount of water released during pumping is related to the thickness or the aquifer, compressibility of the aquifer structure and the expansion of the pore water. Consequently, the storage coefficient is related to the elasticity of the aquifer material and that of the fluid. Storage coefficients for confined aquifers range from 0.001 to 0.00001 (Walton, 1988).

In unconfined aquifers, the primary source of water is from gravity drainage of the pore spaces due to a decline in head. This dimensionless storage parameter is called the specific yield; it is related to the effective porosity of the porous media. In pumping-test analyses of unconfined aquifers, the terms storage coefficient and specific yield are often used synonymously. Values for specific yield normally range from 0.01 to 0.3 (Walton, 1988). For crystalline rock aquifers, storativity may be as low as the 10^{-3} range. The storage coefficient and specific yield must be quantified to determine the amount of water available from an aquifer.

5.3-3.4 Terminology

The concepts relating to the description of a natural ground water flowfield caused by a pumping well are taken from Morrissey (1987). Morrissey's terminology that accompanies his figures will be used throughout this chapter.

5.3-3.4.1 Cone of Depression

The cone of depression is the geometric solid included between the water table or other potentiometric surface after a well has begun discharging and the hypothetical position the water table or other surface would have had if there had been no discharge by the well (Theis, 1935). This depression in the water table or other potentiometric surface can be visualized as a flared, cone-shaped geometric solid (see Figures 5.3-1, -2 and -3). The maximum drawdown occurs at the pumping well. For a given aquifer, the cone of depression increases in depth and extent with increasing time until steady-state flow is reached. Drawdown at any point at a given time is directly proportional to the pumping rate and inversely proportional to aquifer transmissivity and aquifer storativity, with transmissivity exerting the greater influence.

5.3-3.4.2 Area of Influence

The area of influence of a pumping well is the land that directly overlies and has the same horizontal extent as the part of the water table or other potentiometric surface that is perceptibly lowered by the withdrawal of water (Meinzer, 1923).

Ground water in porous media flows radially to a well from all directions. Under ideal aquifer conditions of homogenity and isotropy, and essentially zero natural flow gradient, ground water at any given distance from a pumping well flows at an equal rate towards the pumping well through imaginary concentric cylinders about the well (Figure 5.3-6). Under these conditions the area of influence is circular and the velocity is inversely related to the radial distance. In reality, the area of influence is elliptical but irregularities may occur due to aquifer inhomogeneity. These patterns apply to both confined and unconfined aquifers.

The size and shape of the area of influence is determined by the slope of the pre-pumping water table or potentiometric surface, by the pumping rate, by the transmissivity of the aquifer and its variations, and by the degree and distribution of vertical leakage from aquitards.

5.3-3.4.3 Zone of Contribution

The zone of contribution of a pumping well is defined by Morrissey (1987) as the volumetric portion of an aquifer from which ground water flow is diverted to a pumping well. The zone of contribution can be visualized as a three-dimensional volume of aquifer as depicted in cross-section and plan view in Figure 5.3-7. It is sometimes called the capture zone.

5.3-3.4.4 Contributing Area

The contributing area of a pumping well is defined by Morrissey (1987) as the land area that has the same horizontal extent as that part of an aquifer, or adjacent areas, from which ground water flow is diverted to the pumping well. The contributing area for a pumping well can be visualized as a two-dimensional bullet-shaped area on the land surface, as shown in Figure 5.3-7(b).

Morrissey (1987) lists a number of factors that have been shown to affect the area that contributes flow to a pumping well. Among these factors are:

- Well discharge rate and duration of pumping period.
- Aquifer transmissivity.
- Aquifer storage coefficient or specific yield.
- Proximity of the pumping well to aquifer boundaries.
- Spatial and temporal variations in aquifer transmissivity and/or storage coefficient.
- Spatial and temporal variations in aquifer recharge.
- Partial penetration by the pumping well.
- The presence of extensive confining layers.

One must be careful not to treat the contributing area and area of influence as identical. These areas can be the same only under the hypothetical circumstances where the pre-pumping water table is perfectly flat and all aquifer properties are uniform within the area of influence. When the pre-pumping water table has a gradient, as it does under nearly all natural conditions, the contributing area to the well will be distorted so that it extends a greater distance on the upgradient side and a lesser distance on the downgradient side. Figure 5.3-7, based on Morrissey (1987), illustrates this point.

The equilibrium water-table configuration and natural flow directions in the aquifer are shown in Figure 5.3-8(a). The drawdown and area of influence for steady-state pumping conditions are shown in Figure 5.3-8(b). (The area of influence is herein defined as that area where drawdowns caused by pumping are 0.1 feet or greater.) Theoretically, very small drawdowns will extend to the boundaries of the aquifer even though they might not be detectable by field measurements. The difference between the areas of influence and contribution for the hypothetical conditions portrayed are clearly shown on Figures 5.3-8(b) and (c).

5.3-3.4.5 Boundary Conditions

The presence of boundary conditions can have a major effect on the response of an aquifer to pumping. There are three basic types of aquifer boundaries that are significant to the interpretation of pumping tests: (1) impermeable boundaries, (2) constant-head boundaries, and (3) infinite boundaries. Impermeable boundaries, also known as no-flow boundaries, consist of very low permeability features, such as buried bedrock valley walls. Constant-head boundaries are sources of unlimited amounts of water; a river is a line source of constant head (see Figure 5.3-8). An infinite boundary, or open boundary, lies at a remote distance from the pumping well and does not affect the drawdown caused by a pumping well.

5.3-4 DESIGN CONSIDERATIONS

A well-conceived pumping test design will help ensure that adequate data are collected during the test to permit reliable calculations of the necessary aquifer parameters and a technically sound prediction of the long-term response of the aquifer to pumping. Recommended references covering design considerations are Stallman (1971), Kruseman and De Ridder (1983), and Walton (1988).

5.3-4.1 Objectives of the Pumping Test

It is important to clearly define the objectives of the pumping test prior to installing observation wells, pumps, or expensive instruments. The objective(s) of the pumping test, in addition to budgetary constraints, will determine the number and location of observation wells, the duration of the test, and the number and type of water samples collected for analyses during the test. If the objective of the test is merely to determine the yield of a well, then observation wells may not be required. On the other hand, if the purpose of the test is to define the zone of contribution (i.e., capture zone) around a water-supply well because of concern about contamination, the test may require the installation of a number of observation wells and a longer pumping period.

In contaminated aquifers, monitoring wells may already have been installed. Because existing monitoring wells are usually not suitable to serve as a pumping well, a new pumping well may be required to sustain a sufficient level of stress on the aquifer. Depending upon the locations and screened depths of existing wells, additional monitoring wells also may be needed for collection of useful pumping test data.

5.3-4.2 Pre-test Conceptual Model

A conceptual model of the aquifer should be developed as part of the pumping test design. The aquifer type (confined, unconfined, or leaky) and geometry should be generally, if not specifically, known prior to the test. Boundary conditions such as the location of lakes, streams, and valley walls should be considered as potential sources of recharge or diminished flux, and outlined on a map of the test area.

5.3-4.3 Pre-test Response Prediction

Prior to pumping it is often helpful to estimate the response of the aquifer to a set of assumed conditions to aid in the effective placement of observation wells, as well as to determine the test duration and anticipated drawdowns. Assumed values of aquifer parameters (i.e., hydraulic conductivity, transmissivity, and storativity) can be used in simple analytical equations to predict the gross aquifer response to pumping at a specified rate. From these estimates optimum distances and locations for monitoring wells can be selected.

5.3-4.4 Long- and Short-term Tests

If a long-term pumping test is planned, it is advisable to perform a short-term, step-drawdown test on the discharge test well to determine the most appropriate pumping rate, evaluate the well efficiency, and observe the response in the observation wells. The short-term, step-drawdown pumping test will provide an estimate of the magnitude of the drawdown and the rate of response in the pumping and observation wells during the long-term test. It is important to utilize a pumping rate that will adequately stress the system and produce a measurable response in the aquifer and observation wells. An additional consideration, when working at contaminated sites, is to minimize the amount of contaminated water that is discharged from the pumping well to reduce the problems and cost associated with treatment and disposal of the pumped water.

5.3-5 CONSTANT-RATE PUMPING TESTS AND PROCEDURES

There are two common types of aquifer pumping tests: (1) constant-rate pumping tests and (2) step-drawdown tests. These tests are discussed in this and the following subsections. The constant-rate test is discussed in more detail than the step-drawdown because much of the information presented about a constant-rate test can be applied to a step-drawdown test.

A constant-rate pumping test consists of pumping a well at a constant discharge rate for an extended period of time and measuring the water-level response in the pumping well and surrounding observation wells. A constant-rate test may last for only a few hours (short-term) or it may be conducted for a period of several days, weeks, or months (long-term). Reliable estimates of aquifer transmissivity and storativity can usually be computed from constant-rate pumping test data. Depending on the duration of the test and the conditions in the aquifer, aquifer boundaries (including leakage) and stratigraphic boundaries generally can be identified.

5.3-5.1 Test Operating Requirements

5.3-5.1.1 Selection of a Pump

Accurate control and monitoring of the pump discharge rate is essential during a pumping test. The rule of thumb is that the pumping rate should not deviate more than ±10% during the test (Stallman, 1971). Selection of the properly-sized pump will help achieve this objective. Ideally, the pump should operate at 1/2 to 3/4 of its rated capacity but not at the maximum rate, inasmuch as it is difficult to stabilize the flow at maximum capacity. A valve should be placed between the pump and the discharge line to regulate the flow. If the valve is kept partially closed, the back pressure will help the pump operate more smoothly. The proper pump size can be selected based on the results of a short-term step-drawdown test.

5.3-5.1.2 Selection of the Pumping Rate

Selection of the correct pumping rate is dependent on the objectives of the test and the aquifer conditions. In general, a constant-rate pumping test is conducted at a rate greater than the anticipated pumping rate of a water supply or extraction well in order to maximize the information collected over the relatively short test period. The maximum practical drawdown is often used to anticipate the optimum pumping rate for a given length of test. The response of the aquifer to a short-term stress is used to extrapolate the effects of long-term pumping at a reduced rate. In certain cases, such as the design of a remedial pumping program or two-phase product recovery program, use of a pumping rate similar to the anticipated rate of withdrawal is more desirable.

5.3-5.1.3 Measuring the Pump Discharge

Monitoring of the pump discharge can be performed in several ways. Selection of the most appropriate method depends on the expected pumping rate and the scope of the test. Although timed bucket measurements or flow meter measurements can be used, their range of accuracy is usually quite limited. A more reliable method of discharge measurement is the circular orifice weir method. As shown on Figure 5.3-9, a length of pipe with an orifice plate on the end is attached to the pump discharge. A small piezometric tube, called a manometer, taps into the side of this pipe and is attached to a measuring rule. The pump discharge is monitored by maintaining a specific water level in the manometer throughout the test. The height of the water level in the manometer is related to the discharge rate and the size and type of circular orifice. Tables for manometer/orifice discharge relationships can be found in the literature.

It should be pointed out that as the water level in the well declines due to pumping, the work required of the pump increases. Because of the increased lift required of the pump to discharge at the land surface, the discharge rate may decline significantly. In order to maintain a constant pumping rate, the discharge valve must be adjusted during the test to compensate for this effect. Thus, the test should begin with the control valve not fully opened.

5.3-5.1.4 Discharge

The water from the pumping test should be discharged into an area where it will not affect the pumping test results. This is especially important for pumping tests conducted in shallow, unconfined aquifers. It is important to select a discharge line with a diameter large enough to eliminate the potential for back pressure on the outlet of the orifice plate, as this will affect the manometer reading. If the pumping test water is contaminated, appropriate arrangements must be made for its storage and treatment or disposal. In some cases the discharge can be treated on-site and discharged to the ground; under other circumstances off-site disposal may be required. In some instances a DWPC or EPA permit may be required for a pump test discharge. In all cases, DEP requires that the discharge options be evaluated and that the preferred alternative be approved by the Department.

5.3-5.1.5 Observation Wells

(a) Size Considerations

Observation well diameters should be small enough to prevent time lags in the drawdown response. Generally observation wells ranging in diameter from 2- to 4-inches are used, depending on the permeability of the aquifer. Low permeability aquifers, and particularly aquitards, may require small diameter observation wells, such as 1-inch or 3/4-inch.

(b) Placement

A major factor influencing the size and rate of expansion of a cone of depression is whether an aquifer is confined or unconfined. In general, drawdown in an unconfined aquifer will expand gradually and slowly from a pumping well. By comparison, confined aquifer drawdown will occur quite rapidly, forming a comparatively steep cone of depression around a pumping well; the rate of expansion depends on the transmissivity of the aquifer.

If the aquifer is strongly anisotropic, the distance beyond which the flow can be assumed to be horizontal has been described by Walton (1988).

$$r = 2b\sqrt{K_h/K_v}$$

where,

r = distance between observation and pumping wells (length)

b = average thickness of the saturated zone (length)

K_h = average horizontal hydraulic conductivity of the saturated zone (length/time)

K_v = average vertical hydraulic conductivity of the saturated zone (length/time)

Therefore, within this proximity to the pumping well, the screened interval of the observation wells should be placed at the same elevation as the screened interval of the pumping well in order to negate the effects of vertical flow.

If confined or leaky aquifer conditions are expected, placement of two or more observation wells in vertically adjacent strata may be desirable. The horizontal placement (i.e., distance from the pumping well) of the observation wells will depend on the test objective(s) and the type of aquifer conditions. If possible, observation wells should be placed to allow for both time-drawdown and distance-drawdown calculations; they should also be located close to known or suspected aquifer boundaries to determine their characteristics.

The optimal distances that observation wells should be located from the pumping well can be examined through the application of the Theis equation, using best estimates for aquifer transmissivity and storativity. At a minimum, two close-in locations and two distant locations should be monitored. The close-in wells are generally at least 1.5 times the aquifer thickness from the pumped well if the pumped well has a short screen relative to aquifer thickness. This criterion should eliminate the need to analyze the observation water-level data with partial penetration type-curves. As a general rule, distant observation wells should be located such that, at the anticipated pumping rate, drawdowns will be greater than total fluctuations expected from other causes so as not to be masked. Usually, at least 0.5 feet of drawdown at the end of a test is desirable.

If determination of anisotropy and/or the area of influence is of interest, observation wells may be placed radially around the pumping well to define the shape and size of the cone of depression at different times during the test. If a moderate to steep potentiometric gradient (greater than approximately 0.005 ft/ft) is known to exist in a specific orientation, observation wells should be located along two perpendicular lines intersecting at the production well. One line should be aligned with the general direction of groundwater flow. If possible, the wells should be located at 1 times and 10 times the distance from the pumping well in at least three of the four directions from the well along the lines (Fetter, 1988).

In some cases, existing monitoring or domestic wells are used to monitor the water-level response during a pumping test. When reviewing data from such wells, however, the well diameter, screen length and elevations of the screened interval should be considered to determine their impact on the water-level response at the well. In addition, previous stresses on the aquifer, mounding effects, or drawdowns due to extraneous pumping offsite, must be considered.

(c) Hydraulic Communication

The degree of hydraulic response (i.e., "sluggishness") must be known at least qualitatively for each observation well to be monitored. Generally the information is available readily for most newly completed wells, but often the hydraulic communication between well and aquifer is totally unknown for older wells. Wells that respond sluggishly to rapid changes in aquifer head (or confined pressure) may not provide a valid, drawdown or recovery plot.

Slug tests can be performed on a well to establish its degree of hydraulic response, assuming that aquifer permeability is approximately known or can be estimated within an order-of-magnitude. Many of the references cited in this section describe how to conduct slug tests. The procedure is also described in Section 5.2 In-situ Hydraulic Conductivity Tests.

5.3-5.2 Pre-test Procedures

Before beginning the actual pumping test the following procedure should be followed to ensure getting the maximum amount of data from the test:

1. Install observation wells at appropriate locations. Geologic logs of the borings should be prepared to aid in the definition of aquifer conditions and interpretation of the pump test data.

2. Measure the water levels in the pumping well and the observation wells prior to the pump test. The period of pre-test monitoring should be equal to or longer than the anticipated duration of the pumping test. This pre-test monitoring will help to

identify the effects of barometric, tidal, or man-made influences on water levels. If nearby production wells are influencing water levels in the pumping or observation wells, it will be difficult to interpret the water-level response during the pump test. Consequently, prior to conducting a pumping test, efforts should be made to identify and eliminate controllable disturbances such as other pumping wells. It is recommended that water levels be monitored with a continuous water-level recorder, such as a float/chart recorder or a datalogger and transducer. See Section 5.1 Water-level Measurements for more information on the types of water-level measuring devices available.

3. Perform a short-term or step-drawdown test to determine the optimum pumping rate to be used during the test. Guidelines and procedures for conducting a step-drawdown test are outlined in subsection 5.3-6.

4. Set the pump discharge at the desired pumping rate. Shut off the pump and allow the water level in the pumping well to return to static conditions. Adjust the equipment to appropriate rates prior to the start of the test to minimize the amount of irregularity in the flow rate occurring in the early part of the actual test. Failure to stabilize the flow at the long-term pumping rate during the first 30 seconds to 2 minutes may jeopardize data analysis.

5.3-5.3 Test Procedures for a Constant-Rate Pumping Test

The following procedure should be followed to ensure consistency when conducting the actual pumping test:

1. Record meteorological data, particularly noting rainfall before, during, and after the test.

2. Just prior to the start of the pumping test, measure the static water levels in the pumping well and observation wells.

3. Synchronize watches.

4. Start the pump and, if adjustments are necessary, stabilize the flow rate as rapidly as possible.

5. Measure water levels in the pumping well and nearby observation wells at decreasingly frequent intervals. Table 5.3-1 presents recommended minimum time intervals for measuring water levels during a constant-rate pumping test. At least 10 measurements should be obtained over each log cycle; during the first minute, measurement frequency requires an automated technique. (It is also desirable to collect automated readings during the first-few hours of a test.)

For wells where measurements are made manually, a reasonable attempt should be made to adhere to the Table 5.3-1 schedule. In cases where electronic instruments are used to store digital readings, the frequency of measurement may significantly exceed this schedule during some log cycles.

6. During the pumping test, frequently monitor the pump discharge to make certain that a constant discharge is maintained. Adjust the discharge pipe valve as necessary.

7. Record field-measured water levels on appropriate forms. Examples of aquifer test forms are presented in Figures 5.3-10 and 5.3-11. If automatic recording instruments are used, manual measurements should be collected periodically to check the instrument data. Information about each well should also be recorded on the same sheet. Important data to be recorded in a field book or on the above-referenced forms are listed below:

    - Description of measuring equipment
    - Well diameter (ID)
    - Screened interval (MSL)
    - Nature of soil or rock around screen
    - Radial distance from the pumping well
    - Static water-level prior to pumping
    - Comments on activities or events that might affect the pumping test
    - Presence of a second fluid phase

8. Plot the drawdown versus elapsed-time data. Generally, semi-log plots are prepared but arithmetic plots are sometimes appropriate. If type-curve matching is necessary, log-log plots also should be prepared. Plotting the drawdown data during the test allows frequent re-evaluation of test duration for needed extensions, preliminary estimates of aquifer transmissivity and storativity, as well as early identification of aquifer boundaries, malfunctioning equipment, or improper data collection procedures. Field analysis of trends and slope changes on data plots will indicate when adequate data have been collected and the test can be terminated.

9. Turn off the pump.

10. Measure all water levels during well recovery. Ideally, water levels during recovery should be measured at the same time intervals as during drawdown. Recovery measurements should be collected until water levels stabilize or attain a 98 percent return to pre-test levels or for the duration of the withdrawal test whichever is less.

5.3-6 STEP-DRAWDOWN OR VARIABLE RATE TEST

A step-drawdown test is similar to a constant-rate test except that the pumping rate is systematically increased in a series of several steps of equal duration. The basic requirements of the constant-discharge test should be maintained for each step, including maintaining a constant pumping rate during each step of the test, and obtaining frequent water-level measurements in the pumping well and observation wells.

Generally, step-drawdown tests are conducted during a single day with each pumping step consisting of a 1-hour to 2-hour period. Consistent time intervals permit easy comparison of the drawdown data. It is desirable, but not critical, that the water level in the pumping well be allowed to recover to its static condition before starting the next discharge step of the test.

Step-drawdown tests are used to determine the specific capacity of a pumping well, optimum pumping rates, and the percentage of turbulent and laminar flow occurring at a pumping well. Under ideal, laminar-flow conditions, the drawdown in a pumping well is directly proportional to the discharge (Driscoll, 1986). If the flow is not entirely laminar, meaning that some turbulent flow also occurs, the drawdown will be proportional to the discharge rate raised to some power. Analytical equations have been developed to estimate the percentage of laminar versus turbulent flow from pumping wells (Driscoll, 1986). From such analyses a long-term test discharge rate can be selected that will avoid excessive turbulent flow.

5.3-7 DATA ANALYSIS

There are numerous methods available to evaluate pumping test data that utilize analytical equations, graphical techniques, numerical techniques, and/or computer-assisted techniques. These methods all depend on several different, but simplifying, assumptions. The art and skill in analyzing pumping tests requires the application of the appropriate analytical techniques to the specific aquifer and test conditions. Unfortunately, the analytical solutions of many drawdown and recovery tests are not unique. Correct evaluation requires a good understanding not only of the test response, but also of the geology in the test area and the hydrologic boundaries and anomalies.

When reviewing pumping test data, it is important to evaluate the simplifying assumptions of the method being applied to assure that it fits the aquifer and test conditions. Incorrect application of analytical methods can produce apparently realistic values of transmissivity and storativity, but may not provide reliable estimates of long-term aquifer response to pumping. At some sites, aquifer properties, such as transmissivity, can vary by as much as a factor of 10 from one location to another. Commonly, transmissivity, as determined from observation well data, will vary by a factor of 2 to 3.

Due to the large number and complexity of the various methods for analyzing pumping test data, only a few will be presented here for illustrative purposes. Table 5.3-2 is a compilation of some commonly used methods to analyze aquifer pumping test data, along with the basic assumptions inherent in each method and the source. Readers should refer to the sources directly in order to correctly apply the appropriate method. An excellent reference on this subject has been written by Kruseman and DeRidder (1983). The basic assumptions that should be determined and a description of analytical methods are discussed briefly below.

5.3-7.1 Basic Assumptions

Prior to selecting a specific method for analyzing the data, the following basic test conditions should be identified:

- o Aquifer type (confined, unconfined, or leaky).
- o Aquifer conditions (homogeneity, degree of isotropy, limits of areal extent, and natural flow gradient).
- o Well characteristics (screen fully or partially penetrating the aquifer; possibility of wellbore or casing storage effects).
- o Test type (constant-rate or step-drawdown).
- o Test termination status of aquifer conditions (steady-state, non-steady-state).
- o Boundaries (type and location).
- o Seasonal trend effects.
- o Weather effects (prior to test, during pumping, during recovery).

Once these variables have been described, an appropriate method can be selected for analyzing the pumping test data.

5.3-7.2 Analysis of Pumping Test Data

Analysis of pumping test data usually incorporates graphical data plots of time versus drawdown or distance versus drawdown and the application of analytical equations. There are basically two general types of methods used to evaluate well data: (1) type-curve matching methods, and (2) analytical solutions based on best linear fits derived from data plots. If possible, more than one specific method should be used to calculate aquifer parameters. If the aquifer and test conditions meet the applicability requirements of more than one method, alternative methods should produce reasonably close values of transmissivity and storativity.

In some cases, drawdowns may need to be corrected for various well effects such as partial penetration, barometric change, antecedent trends, aquifer dewatering, etc. References such as Todd (1980), Walton (1988), Stallman (1971), or Kruseman and DeRidder (1983) should be consulted.

5.3-7.2.1 Type-Curve Matching

Type-curves have been developed for a number of aquifer conditions, including confined, unconfined and leaky aquifers, delayed-yield effects for unconfined aquifers, steady-state and non-steady-state conditions, and effects of partial penetration. Pumping test data can be plotted and compared to a variety of type-curves in order to assess the conditions prevailing during the test. Although the type-curve method is somewhat subjective, it does allow for evaluation of complex aquifer responses that is not possible with simple analytical methods. A number of investigators have published type-curve matching methods, the most often referenced being Theis (1935), Hantush (1964), Cooper-Jacob (1946), Boulton (1954), Neuman (1975) and Neuman et al. (1984) (see Table 5.3-2).

Several computerized methods of curve matching are available for use when analyzing pumping test data, but the programs must be used with extreme caution because analysis of an aquifer test is not unique. Various combinations of aquifer conditions can yield the same drawdown response during a given test. Leaky aquifer effects may appear very similar to recharge boundary conditions. Consequently, a large amount of hydrologic judgment must be used when interpreting the fit, or match, of aquifer type curves to field drawdown data. Depending on the sophistication of the computer program and the user, the program may simplify or normalize the data so that important subtle response characteristics are obscured. Therefore, it is best to use carefully prepared plots of the test data and first perform the matching exercise manually in order to evaluate the various combination of aquifer conditions that might occur to produce a particular plot.

The general procedure for type-curve matching is described below:

1. The drawdown versus time data are plotted on a standard 3x5 cycle log-log graph.

2. The plot is overlain on the appropriate type-curve and a match-point is determined.
3. The match-point coordinates are taken from the plot and used in the calculation of the aquifer parameters.

Two examples of a type-curve matching method are presented in the Appendices. Figure A-1 illustrates the matching technique and solved equations for transmissivity and storativity for a Theis aquifer condition. A common aquifer condition of leakage through an aquitard that has negligible storativity is illustrated by a field example from Walton (1962) in Figure B-1.

5.3-7.2.2 Analytical Solution

Methods using analytical solutions consist of plotting the test data on semi-log or log-log graph paper and determining the slope of the plot or some other parameter, such as the y-intercept. This value is then used in the appropriate equation to calculate the aquifer parameters. These methods are less subjective than the type-curve method, but cannot be used if the data are affected by delayed yield or non-steady, leaky conditions. The most commonly used analytical solution is the Jacob Straight-line method, which is presented in Appendix C. An example of the application of the Jacob solution, showing solved equations for transmissivity and storativity, appears in Figure C-1.

REFERENCES

Bear, J., 1979, Hydraulics of groundwater: New York, NY, McGraw-Hill, 569 p.

Boulton, N.S., 1954, Unsteady radial flow to a pumped well allowing for delayed yield from storage: International Association of Scientific Hydrology Publication 37, pp. 472-477.

__________, and Streltsova, T.D., 1976, The drawdown near an abstraction well of large diameter under non-steady conditions in an unconfined aquifer: Journal of Hydrology, v. 30, pp. 29-46.

Cooper, H. H., Jr, and Jacob, C.E., 1946, A generalized graphical method for evaluating formation constants and summarizing well field history: American Geophysical Union, Transactions, v. 27, no. 4, pp. 526-534.

DEP, Division of Water Supply, 1989, Guidelines and policies for public water systems: Boston, MA.

Driscoll, F.G., 1986, Ground water and wells, 2nd ed.: St. Paul, MN, Johnson Div., 1089 p.

Fetter, C.W., Jr., 1988, Applied hydrogeology: Columbus, OH, Charles Merrill Publishing Co., 488 p.

Freeze, R.A., and J.A., Cherry, 1979, Groundwater: Englewood Cliffs, NJ, Prentice-Hall, 604 p.

Hantush, M.S., and Jacob, C.E., 1955, Non-steady radial flow in an infinite leaky aquifer: Transactions of the American Geophysical Union, v. 36, no. 1, pp. 95-100.

Hantush, M.S., 1961, Drawdown around a partially penetrating well, Journal of the American Society of Civil Engineers, v. 87, no. HY4, pp. 83-98.

_____________, 1964, Hydraulics of wells: in Advances in Hydroscience, V.T. Chow, editor, New York, NY, Academic Press, pp. 281-442.

Kruseman, G.P., and De Ridder, N.A., 1983, Analysis and evaluation of pumping test data, 3rd ed.: Wageningen, The Netherlands, Institute of Land Reclamation and Improvement, Bulletin 11, pp. 41-49.

Lohman, S.W., 1979, Groundwater hydraulics: Washington, DC,USGS Professional Paper 708, 70 p.

REFERENCES
(continued)

Meinzer, O.E., 1923, The occurrence of groundwater in the United States, with a discussion on principles: Washington, DC, USGS Water Supply Paper 489, Government Printing Office, 321 p.

Morrissey, D.J., 1987, Estimation of the recharge area contributing water to a pumped well in a glacial-drift, river-valley aquifer: U.S. Geological survey Open-file Report 86-543, 60 p.

Murray, W.A., 1987, Discussion of "An alternate procedure for analyzing aquifer tests using the Theis nonequilibrium solution", by O.L. Frank: Ground Water, v.25, no. 5, p. 616-617.

Neuman, S.P., 1975, Analysis of pumping test data from anisotropic unconfined aquifers considering delayed gravity response: Water Resources Research, v.11, no.2, pp. 329-342.

__________, Walter, G.R., Bentley, H.W., Ward, J.J., and Gonzale, D.D., 1984, Determination of horizontal aquifer anisotropy with three wells: Ground Water, v. 22, no. 1, pp. 66-72.

Sen, Zekai, 1988, Dimensionless time-drawdown plots of late aquifer test data: Ground water, v. 26, no. 5, pp. 615-618.

Stallman, R.W. 1971, Aquifer test design, observations, and data analysis: USGS Techniques of Water Resources Investigations, Book 3, Chapter B1.

Streltsova, T.D., 1974, Drawdown in compressible unconfined aquifer: Proceedings, American Society of Civil Engineers, Journal of Hydraulics Division, v. 100, no. HY11, pp. 1601-1616.

Theis, C.V., 1935, The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using ground water storage: Washington, DC, Transactions, American Geophysical Union, pp. 518-524.

Thiem G., 1906, Hydrogische methoden: Leipzig, Germany, Gebhardt, 56 p.

Todd, D.K., 1980, Groundwater hydrology: New York, NY, John Wiley, 535 p.

Walton, W.C., 1962, Selected analytical methods for well and aquifer evaluation: Illinois State Water Survey Bulletin No. 49, Urbana, Illinois, 81 p.

__________, 1988, Groundwater pumping tests: design and analysis: Chelsea, MI, Lewis Publishers, Inc., 201 p.

REFERENCES
(continued)

Way, S.C., and McKee, C.R., 1982, In-situ determination of three-dimensional aquifer permeabilities: Ground Water, v. 20, no. 5, pp. 594-603.

Weeks, E.P., 1969, Determining the ratio of horizontal to vertical permeability by aquifer test analysis: Water Resources Research, v. 5, no. 1, pp. 196-214.

Witherspoon, P.A., Javandel, I., Neuman, S.P., and Freeze, R.A., 1967, Interpretation of aquifer gas storage conditions from water pumping tests: New York, NY, American Gas Association, 273 p.

ADDITIONAL REFERENCES

Bouwer, H., 1978, Groundwater hydrology: New York, NY, McGraw-Hill, 479 p.

Hantush, M.S., and Jacob, C.E., 1960, Modification of the theory of leaky aquifers: Journal of Geophysical Research, v. 65, no. 11, p. 3713-3725.

Jacob, C.E., 1946, Radial flow in a leaky artesian aquifer: Transactions of the American Geophysical Union, v. 27, no. 2, pp. 198-205.

__________, 1963. Determining the permeability of water-table aquifers: in Bentall, Ray, complier, methods for determining permeability, transmissibility, and drawdown: USGS Water Supply Paper 1536-I, pp. 245-271.

Papadopulos, I. S., 1967, Drawdown distribution around a large-diameter well: Proceedings, Ground Water Symposium, American Water Resources Association, pp. 157-167.

U. S. Bureau of Reclamation, 1977, Ground water manual: Washington, D.C., U.S. Government Printing Office, 480 p.

Walton, W. C., 1970, Groundwater resource evaluation: New York, NY McGraw Hill, 644 p.

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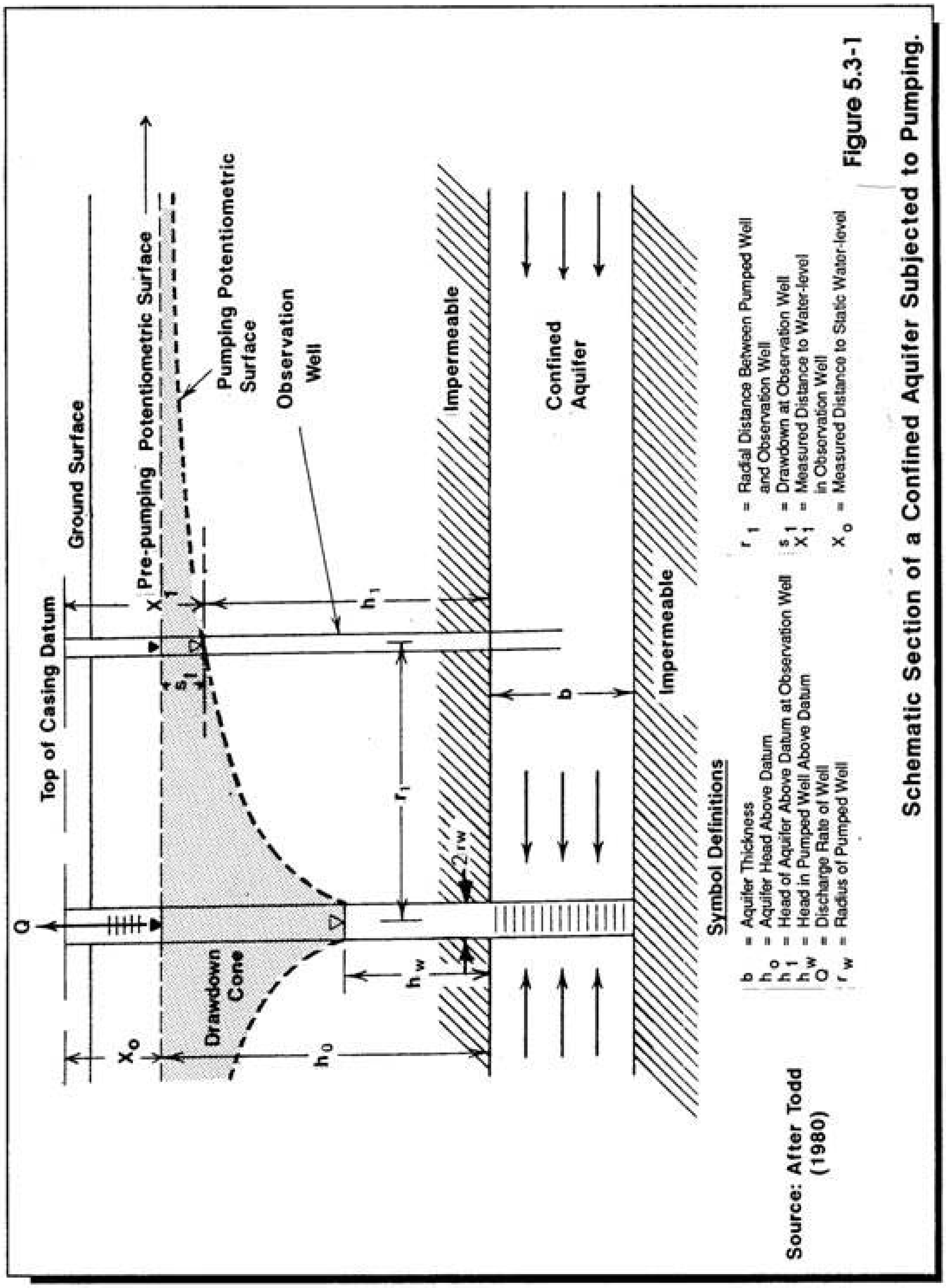
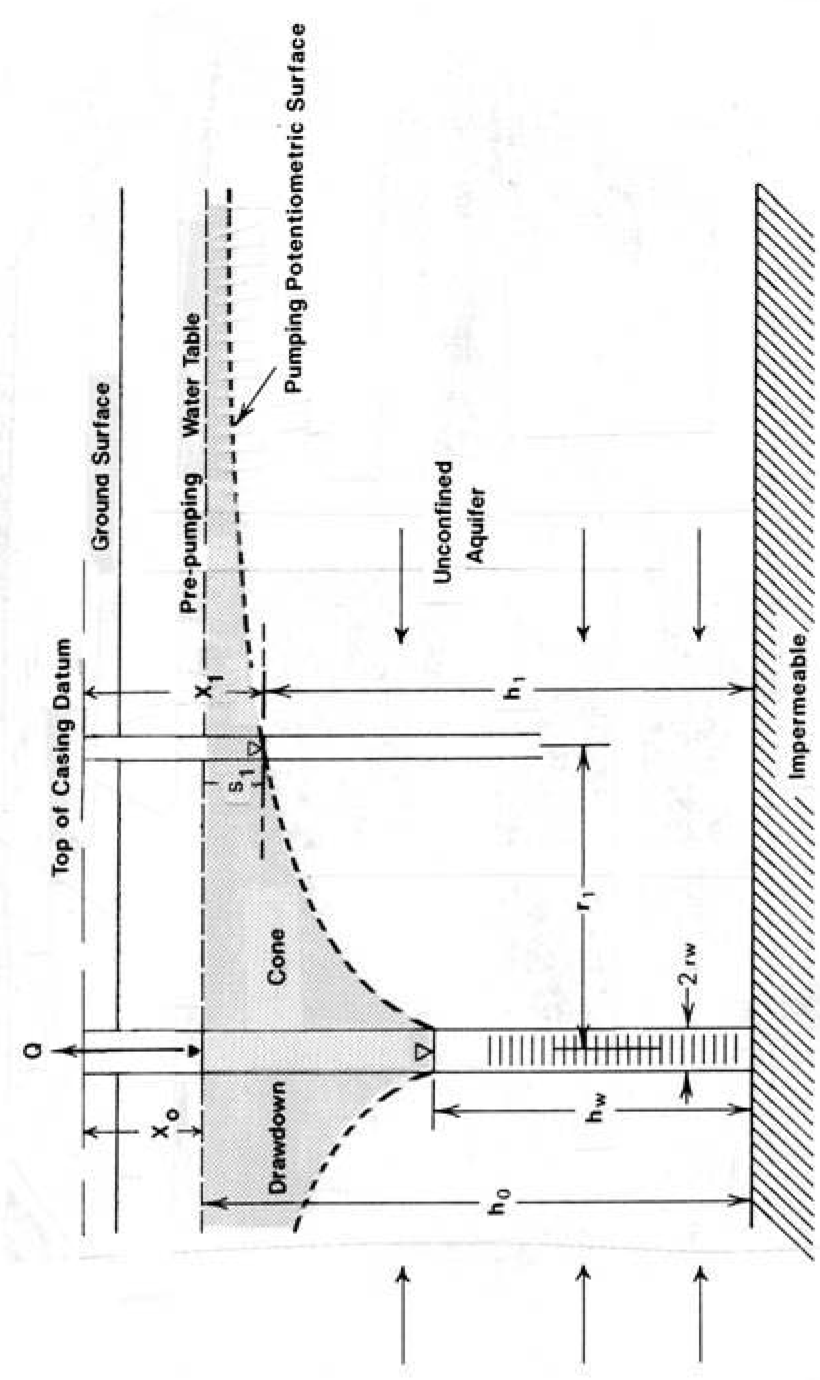


Figure 5.3-1

Schematic Section of a Confined Aquifer Subjected to Pumping.



Note: See Fig. 5.3-1 for Symbol Definitions

Source: After Todd (1980)

Figure 5.3-2

Schematic Section of an Unconfined Aquifer Subjected to Pumping.

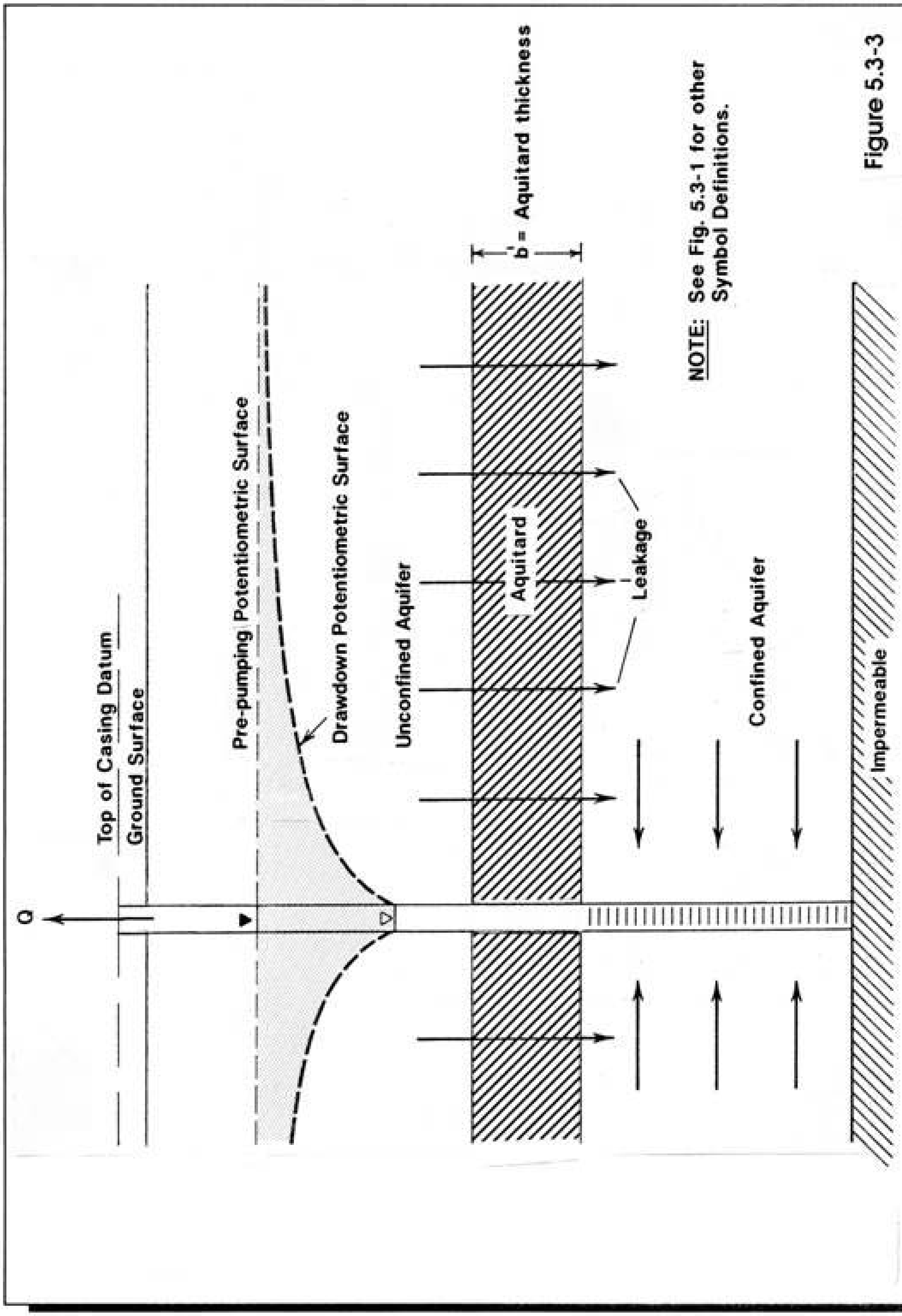


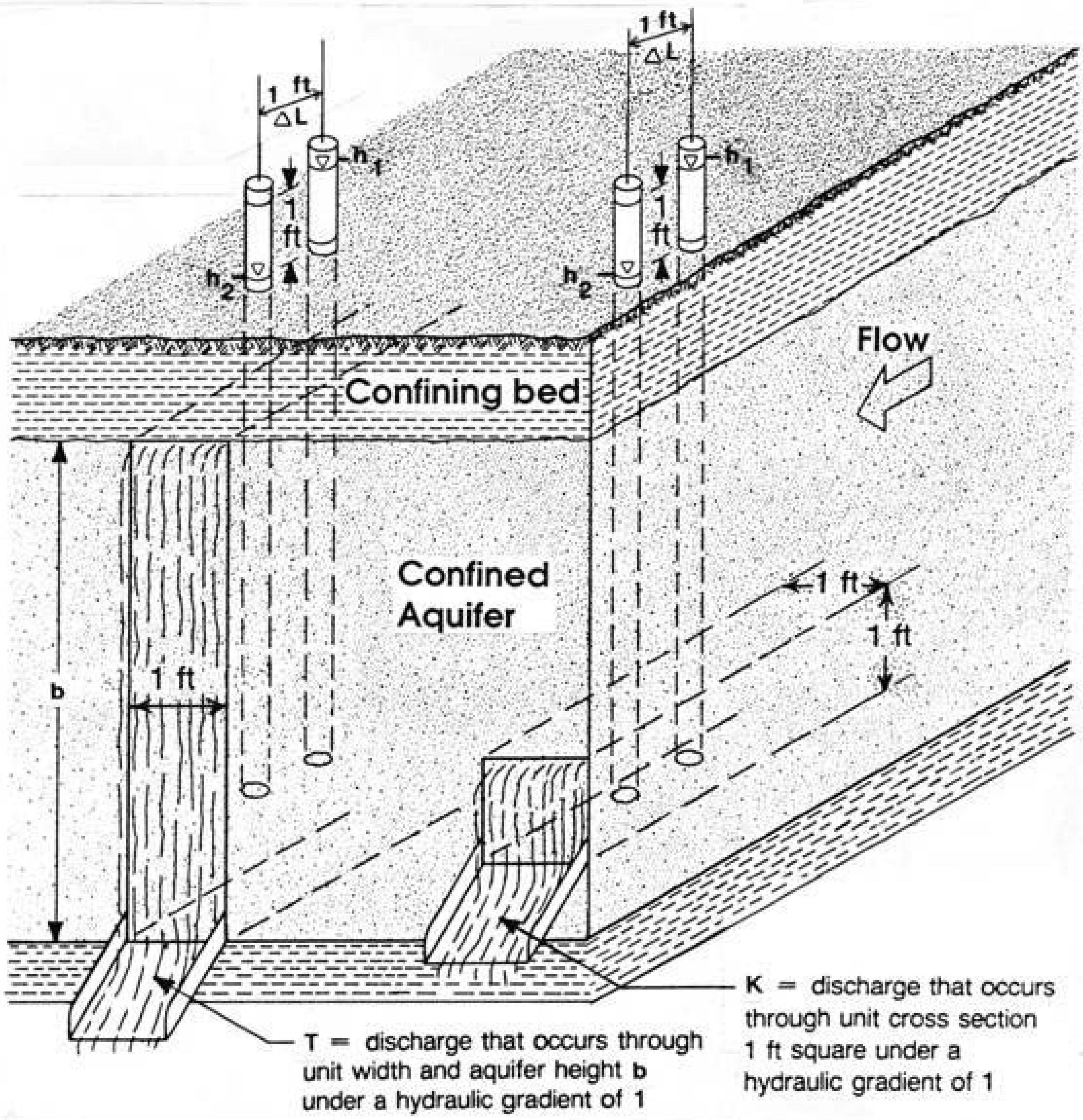
Figure 5.3-3

Schematic Section of a Leaky Aquifer Subjected to Pumping.

Source: After Todd (1980)

b = Total Aquifer Thickness
h_1 = Potentiometer Level in Upgradient Well
h_2 = Potentiometer Level in Downgradient Well
K = Hydraulic Conductivity of Aquifer
ΔL = Distance Between Upgradient and Downgradient Wells
T = Transmissivity of Aquifer

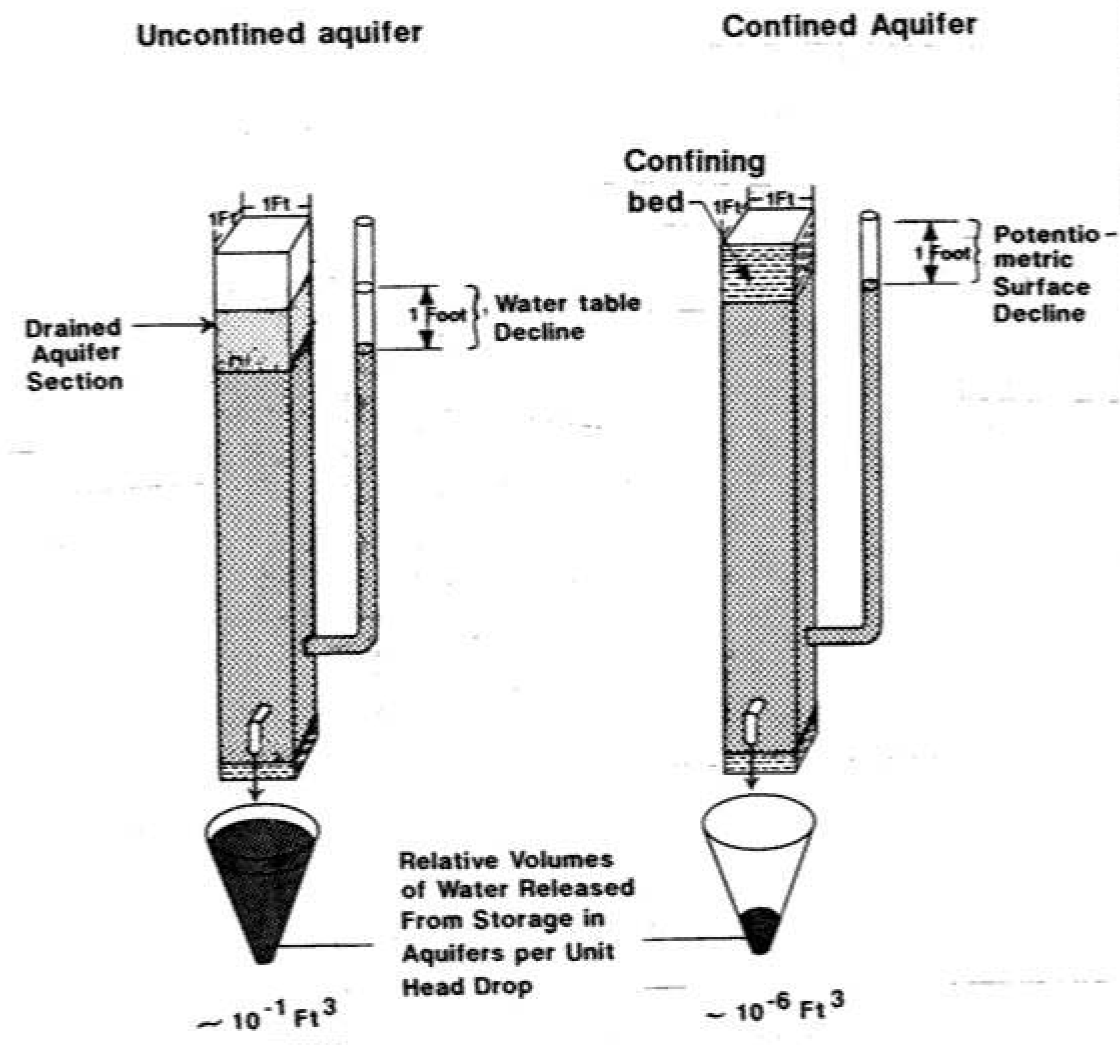
$$\frac{dh}{dL} = \text{Hydraulic Gradient} = \frac{h_1 - h_2}{\Delta L}$$



Source: After Driscoll (1986)

Figure 5.3-4

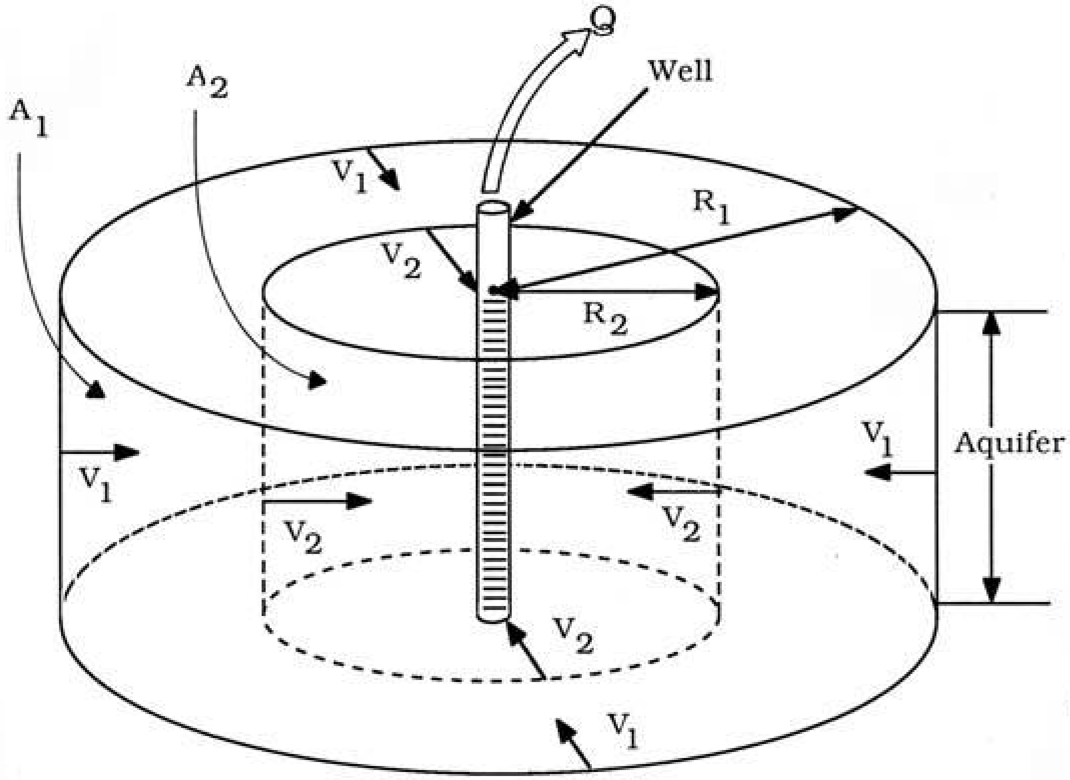
Conceptual Representation of Transmissivity and Hydraulic Conductivity for a Confined Aquifer.



Source: After Driscoll (1986)

Figure 5.3-5

Conceptual Representation of Storativity in Unconfined and Confined Aquifers.

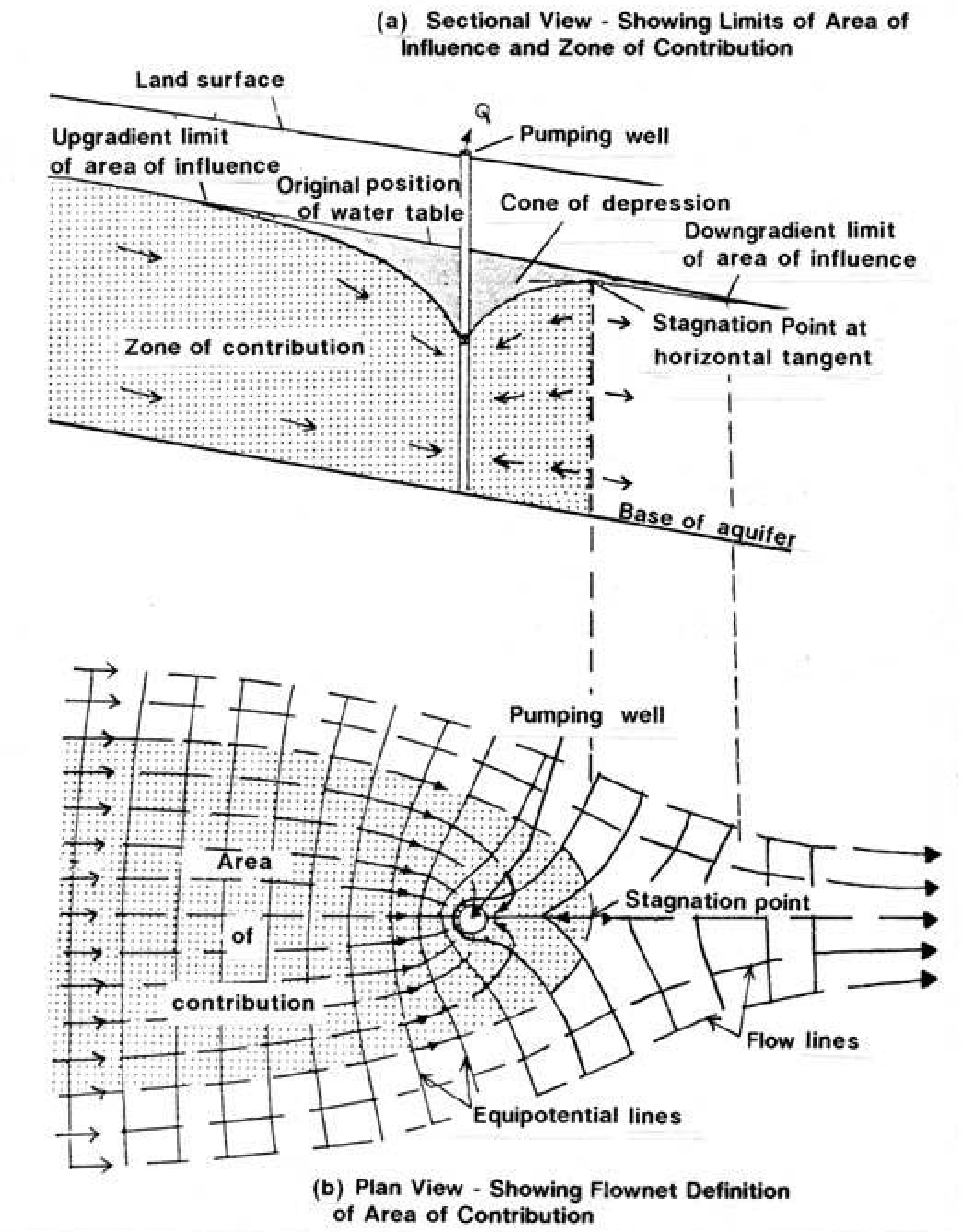


Note: Per Title Conditions, all V_1s are Identical

	Radius (R)		Cylinder Area (A)		Ground Water Velocity (V)
If,	$R_1 = 2R_2$	**then,**	$A_1 = 2A_2$	**and,**	$V_2 = 2V_1$

Figure 5.3-6

Convergent Uniform Radial Flow in a Homogeneous, Isotropic Aquifer Due to Pumping of a Fully Penetrating Well.



Note: Under this development of the cone of depression, groundwater downgradient of the stagnation point will not be drawn to the well and discharged

Source: After Morrissey (1987)

Figure 5.3-7

Diagram of Groundwater Flow Around a Pumping Well.

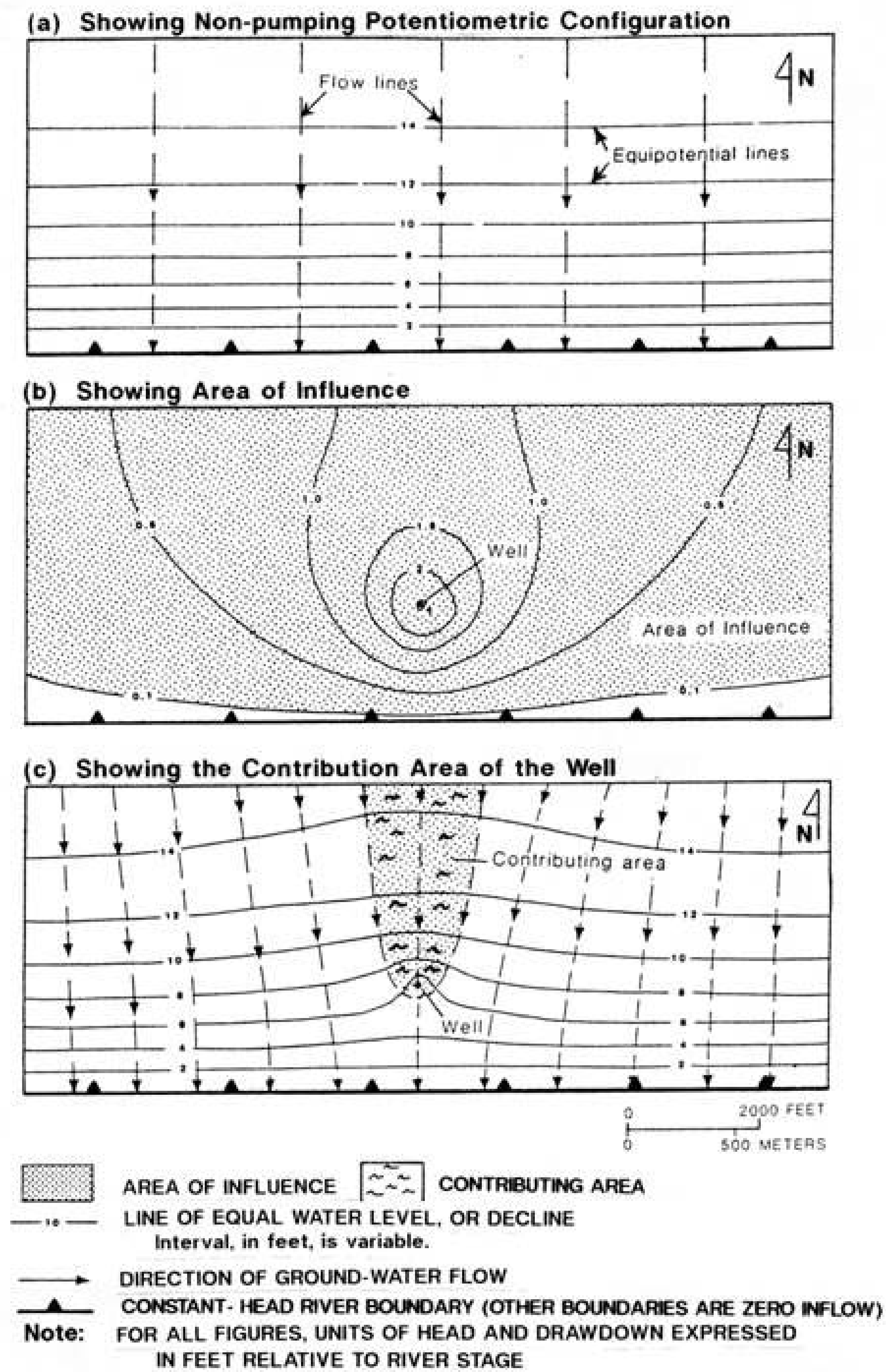


Figure 5.3-8

Source: After Morrissey (1987)

Flow Diagrams for a Hypothetical Aquifer for Steady-state Pumping.

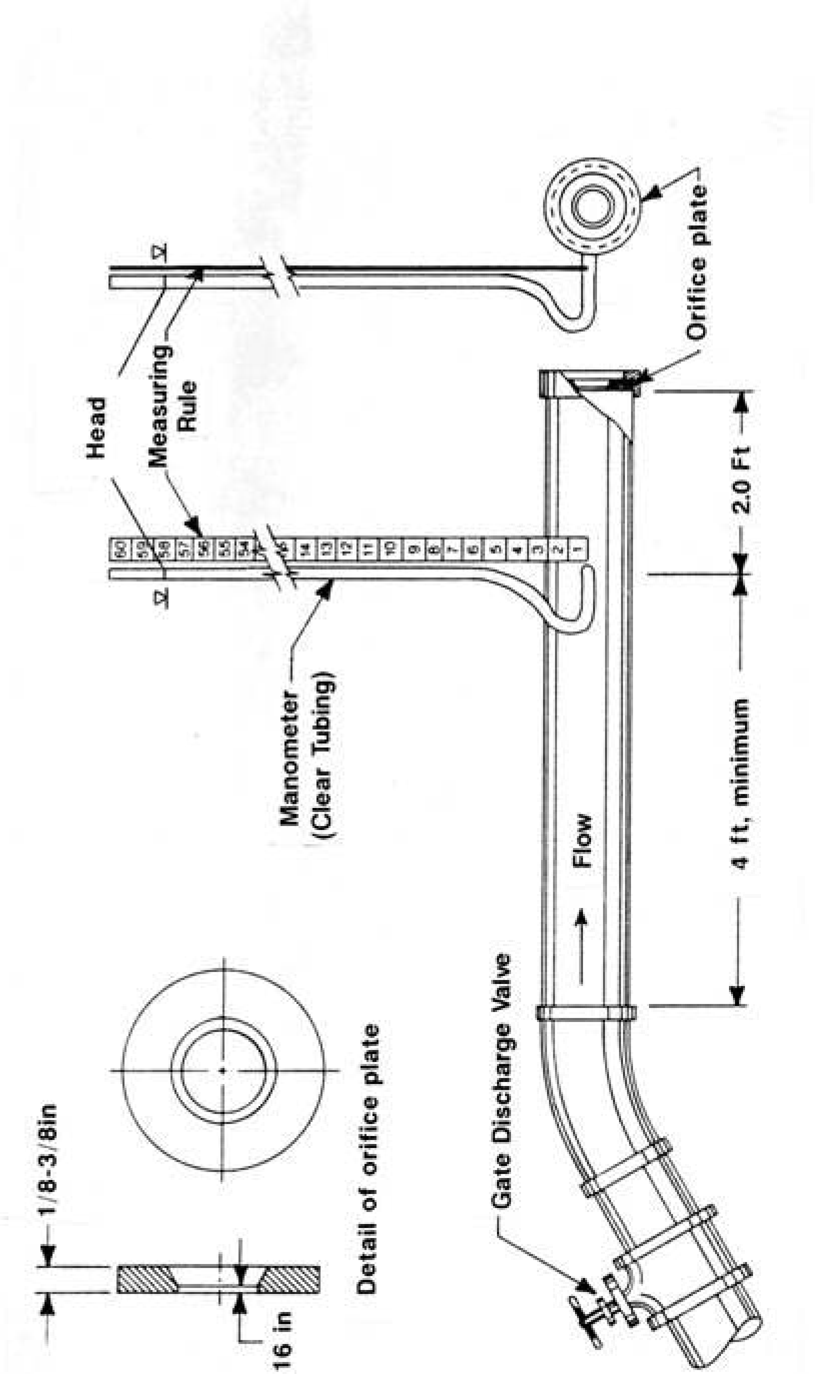


Figure 5.3-9
Diagram Illustrating the Circular Orifice Weir Method for Measurment of Well Discharge.

Source: After Driscoll (1986)

CONSTANT-RATE PUMPING TEST DATA

Date ________________ Site ________________________________ Well ________________________

Radial Distance from Pump (r) __________(ft) Screened Depth ______________ Static WL (x_o) ______________

Measuring Point Used ____

Elapsed Time (t) (min)	Distance to WL (x) (ft)	Drawdown $(x-x_o)$ (ft)	Barometric Reading (inches-Hg)	Corrections to Drawdown (ft)	Drawdown Corrected (s) (ft)	$\frac{t}{r^2}$ $\frac{min}{ft^2}$	Pumping Rate (gal/min) and Remarks

See Figures 5.3-1 for Symbol Definitions

Source: ABB-ES

Figure 5.3-10

Example of an Aquifer Test Data Form for Recording Water-level Drawdown.

PUMPING TEST RECOVERY DATA

Date ______________ Site ______________________________ Well ______________

Radial Distance from Pump (r) ________ (ft) Screened Depth ______________ Static WL (x_o) ______________

Measuring Point Used ______________________________

Elapsed Time (t) (min)	Time Since Pump Off (t') (min)	t/t'	Distance to WL (x) (ft)	Residual Drawdown ($x-x_o$) (ft)	Barometric Reading (inches-Hg)	Corrections to Residual Drawdown (ft)	Residual Drawdown Corrected (s') (ft)	Remarks

See Figures 5.3-1 for Symbol Definitions

Source: ABB-ES

Figure 5.3-11

Example of an Aquifer Test Data Form for Recording Water-level Recovery.

Time After Pumping Started (minutes)	Measurement Frequency
0-1.5	15 seconds
1.5-5	30 seconds
5-15	1 minute
15-50	5 minutes
50-120	10 minutes
120-240	20 minutes
240-500	40 minutes
500-1000	1 hour
>1000	4 hours

Note: More frequent measurements at the start of the test is desirable

Source: Adapted from Walton (1987)

Table 5.3-1

Minimum Recommended Time Intervals for Measuring Water Levels During a Constant-rate Pumping Test

General Flow Conditions*	Method Name (citation)	Inherent Assumptions**	No. of Monitored Wells Required	Parameters Determined **	Solution Technique	Data Plot
UNE, CNE	Theis (1935)	a-f, h	1/(2 or more)	T, S, boundaries	curve-match	log-log s vs, t (or s vs. t/r^2)
USS, UNE, CSS, CNE	Cooper-Jacob (1946)	a-h	1/(2 or more)	T,S, boundaries	analytical	semi-log s vs. t (or s vs. r)
CSS	Thiem (1906)	a-c,e	2 or more	T	anlaytical	semi-log s vs. r
USS	Thiem (1906)-Dupuit	a-c,e,h-j	2 or more	T	anlaytical	semi-log s vs. r
UNE	Boulton (1954)	a-d,h,m	1	T,S,K'	curve-match	log-log s vs. t
UNE	Papadopulous (1967)	a-c,e,f,h	1(pumped)	T,S ?	curve-match	log-log s vs. t
UNE, CNE	Theis Recovery (1935)	a-h	1	T,S, boundaries	anlaytical	semi-log s vs. t/t'
UNE, CNE	Weeks (1969)	a,b,d,h,k-m,p	3 or more	$T,K_r/K_z,S$	curve-match	semi-log s vs. t
UNE	Streltsova (1974)	a,b,d,h,k-m,p	2	T,S	curve-match	log-log s vs. t
UNE	Boulton and Streltsova (1976)	a,b,h,k-m,p	2	T,S	curve-match	log-log s vs. t
SSS, SNE	Hantush and Jacob (1955), Walton (1962)	a-e,k,n,p	2 or more	T,S,K'/b'	curve-match	log-log s vs. t
SNE	Hantush (1964)	a-e,p	1 or more	T,S,K',S'	curve-match	log-log s vs. t/r^2
SNE	Witherspoon et al (1967)	a-d,k,p	2	K'/S'	curve-match	log-log s vs. t @ equal r
SNE, CNE	Way and McKee (1982)	a-d,k,p	3 obs	K_x,K_y,K_z,S	curve-match	log-log s vs. t
CNE	Neuman et al (1984)	a-d,f,p	3 or more	K_x,K_y,S	analytical	semi-log s vs. t
CNE	Hantush (1961)	a,b,d-f,p	1	K,S	curve-match	log-log s vs. t
UNE	Neuman (1975)	a-d,h,m	1	$T,S, K_z/K_r$	curve-match	log-log s vs. t

* Codes: USS = unconfined steady-state, UNE = unconfined non-equilibrium, SSS = semi-confined steady-state, SNE = semi-confined non-equlibrium, CSS = confined steady-state, CNE = confined non-equilibrium

** See explanation of symbols on Page 2 of 2.

Notes: The references cited are included in the references or additional references of section 5.3

Source: MADEP, After Stallman (1971)

Table 5.3-2
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Commonly-used Methods for Analyzing Aquifer Pumping Test Data (Porous Media)

Explanation of Symbols

Assumptions

a = constant pumping rate
b = homogeneous, infinite aquifer
c = pumping and/or observation wells screened full aquifer thickness
d = pumping well has and infinitesimal diameter (neglectable well storage)
e = aquifer is isotropic
f = no delayed drainage, or vertical leakage into or out of aquifer
g = u <0.01 to 0.1
h = drawdown (or recovery) is corrected for changing saturated thickness unless drawdown is small in comparison to initial saturated thickness
i = potentiometric surface is horizontal prior to pumping
j = flow in aquifer is horizontal
k = observation well is partially penetrating
l = $r < 1.56\,(K_h/K_z)1/2$
m = principal; permeabilities are oriented parallel to coordinate axis
n = leakance is proportional to head variation
p = other limiting conditions

Parameters

b' = thickness of aquitard
K = hydraulic conductivity of aquifer (horizontal)
Kr = radial hydraulic conductivity
Kv = vertical hydraulic conductivity
Kx = hydraulic conductivity x-coordinate direction
Ky = hydraulic conductivity in y-coordinate direction
Kz = hydraulic conductivity in z-coordinate direction
K' = hydraulic conductivity of aquitard (vertical)
r = radial distance between pumped well and observation well
s = measured residual drawdown for a given r and t
s' = measured residual drawdown for a given r and t after pumping ceased
S = storativity (storage coefficient) of aquifer
S' = storativity (storage coefficient) of aquitard
t = total elapsed time since pumping began
t' = elapsed time since pumping ceased
T = transmissivity (transmissivity) of aquifer

Source: MADEP, after Stallman (1971)

Table 5.3-2
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Commonly-used Methods for Analyzing Aquifer Pumping Test Data (Porous Media)

APPENDICES

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APPENDIX A

Application of the Theis Curve-matching Method

The transmissivity (T) and storativity (S) of an aquifer can be determined by this method if the following conditions are satisfied, or if it is assumed that these conditions are not seriously violated.

- o The aquifer is homogenous and isotropic
- o The aquifer boundaries are beyond the edge of the drawdown cone of the pumping well during the entire test period
- o The discharging well penetrates the full thickness of the aquifer
- o The well diameter is not large enough to cause casing storage effects at the test discharge rate
- o No vertical leakage of water occurs from aquitards either overlying or underlying the pumped aquifer

Using the type-curve matching procedure given in Section 5.3-7.2.1, aquifer transmissivity is calculated as:

$$T = \frac{114.6\,Q}{s}\,W(u)$$

where,

T = aquifer transmissivity (gpd/ft)

Q = average pumping rate in gallons per minute (gpm)

s = drawdown read from data plot corresponding to the selected match point (feet)

$W(u)$ = the exponential integral called the well function of u, which is usually selected during type-curve matching to be a value of 1 on the Y-axis of the Theis-curve plot (dimensionless)

Aquifer storativity is calculated for each observation well plot (but not the pumped well) of either drawdown or recovery as:

$$S = \frac{T\,t\,u}{1.87\,r^2}$$

where,

- S = aquifer storativity (dimensionless)
- T = transmissivity calculated using the above equation (gpd/ft)
- t = time read from data plot corresponding to the selected match point (converted to days)
- r = radial distance from pumping well to observation well (feet)
- u = the independent variable of the function W(u), which is usually s

If drawdown versus time data are available for two or more observation wells located at differing distances from the pumped well, the data should be plotted on the X-axis as t/r. This procedure removes the variable distance factor, and will cause drawdowns of all wells to lie along a single positioning of theTheis-curve, if the Theis conditions given above are satisfied.

Figure A-1 gives an example of the application of the Theis method with solved equations for transmissivity and storativity. In this case, a moderately fractured and weathered dolomite aquifer is confined by the overlying clayey residuum. Due to the thinness of the fractured zone (about 20 ft) and the relatively high aquifer storativity, drawdown did not reach the observation well at a distance of 300 feet until about 200 minutes pumping at 10 gpm. The wandering of data points along the Theis-curve (Figure A-1) is typical of tests conducted where the aquifer possesses a minor degree of geologic heterogeneity.

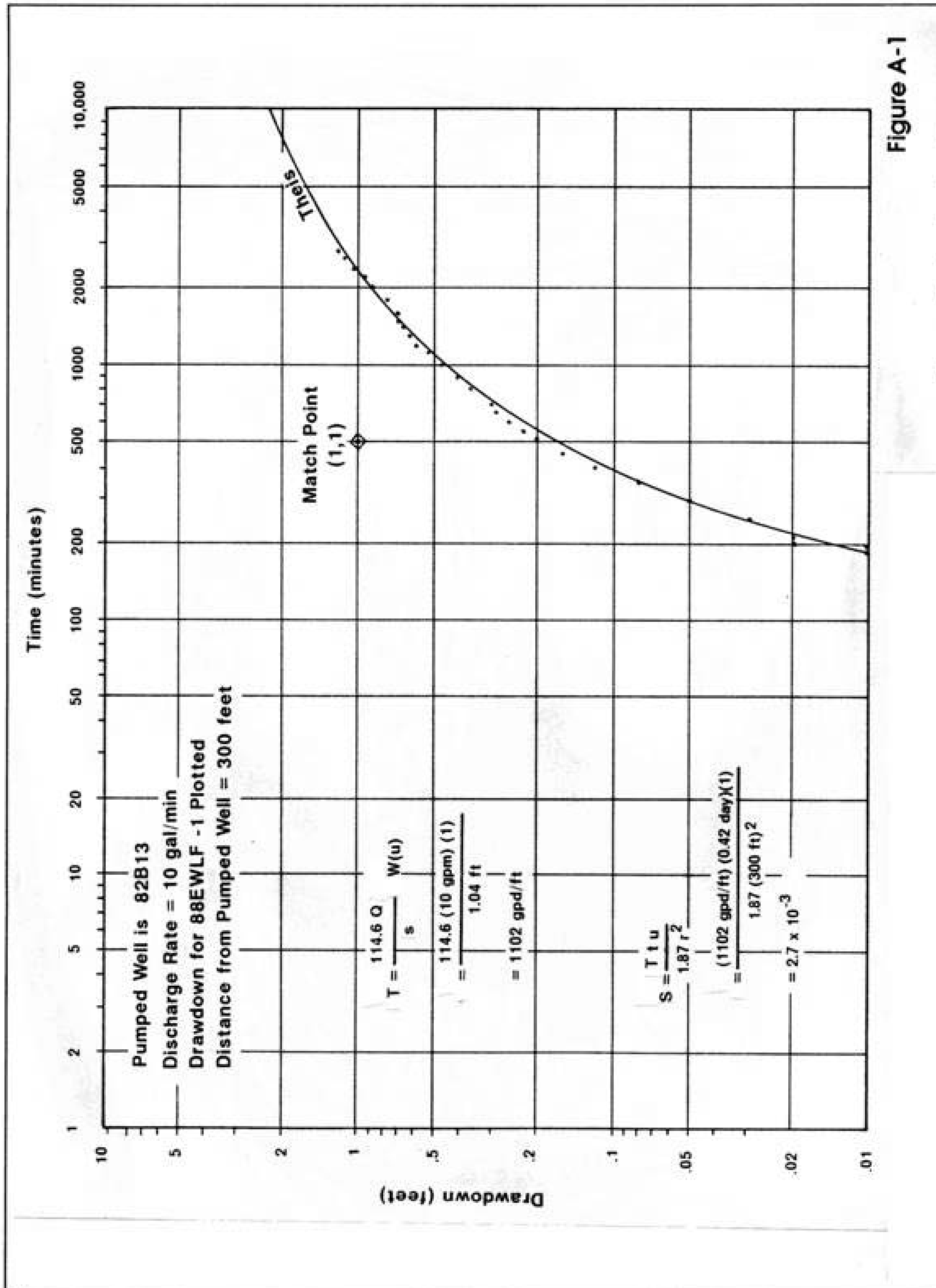


Figure A-1

Example Analysis of Pumping Test Data with Theis' Curve Method.

Source: ABB-ES

APPENDIX B

Application of Walton's Type-curves for a Leaky Semi-confined Aquifer without Storage in the Aquitard

One of the more common reasons that drawdown data do not plot along the Theis curve is that a significant quantity of water being pumped from a well comes from vertical leakage from adjacent aquitards. An aquitard is a geologic unit that immediately overlies or underlies an aquifer, and partially confines the aquifer. Aquitards possess a high enough vertical hydraulilc conductivity to allow leakage of water into the aquifer under hydraulic gradients.

Families of type-curves have been developed for two different leaking aquitard conditions: (a) leakage without water derived from aquitard storage, and (b) leakage of water derived exclusively from aquitard storage. Condition (a) generally occurs when the aquitard is relatively thin and/or relatively permeable, and is in contact with a second aquifer (called a source bed). Condition (b) occurs when the aquitard is relatively thick and has very low hydraulic conductivity compared to the pumped aquifer. The leakage-with-storage response may also appear on data plots of short duration pumping tests if actual geologic conditions are between (a) and (b). Lohman (1979) advised that "thorough knowledge of the geology, including the character of the confining beds, should indicate in advance which of the two leaky aquifer type-curves to use, or whether to use the Theis type-curve for non-leaky aquifers."

The analytical procedure to interpret drawdown measured at an observation well follows nearly the same data-plotting and curve-matching routine as for the Theis method. Data are prepared and plotted as discussed in Section 5.3-7.2.1. The field data plot is fitted to the most appropriate curve in the family of type-curves, and a match point corresponding to the type-curve coordinates of unity (if possible) is determined. The equations for calculating transmissivity and storativity are the same as those given in Appendix A (Theis method). W(u) and u essentially become leaky well function parameters, with the match point location on the data plot giving s and t values that reflect leakage.

The example shown in Figure B-1 is for leaky condition (a), where a 63-foot thick aquifer is overlain by a 20-foot thick aquitard that is, in turn, overlain by a second aquifer. It is assumed that recharge to the source bed maintains a constant head in this bed, balancing loss of water due to vertical leakage to the pumped aquifer.

The drawdown plot in Figure B-1 has been matched with a specific leakage curve (r/B = 0.2). All curves in this family of type curves go flat (no increase in drawdown with time) when the drawdown cone expands radially far enough so that the total rate of leakage equals the constant discharge rate of the well. This condition occurs at about 200 minutes in the example. The Theis-curve is shown in its appropriate position for the given match point to illustrate the increasing difference of drawdowns between a non-leaky and a leaky analysis with elapsed pumping time.

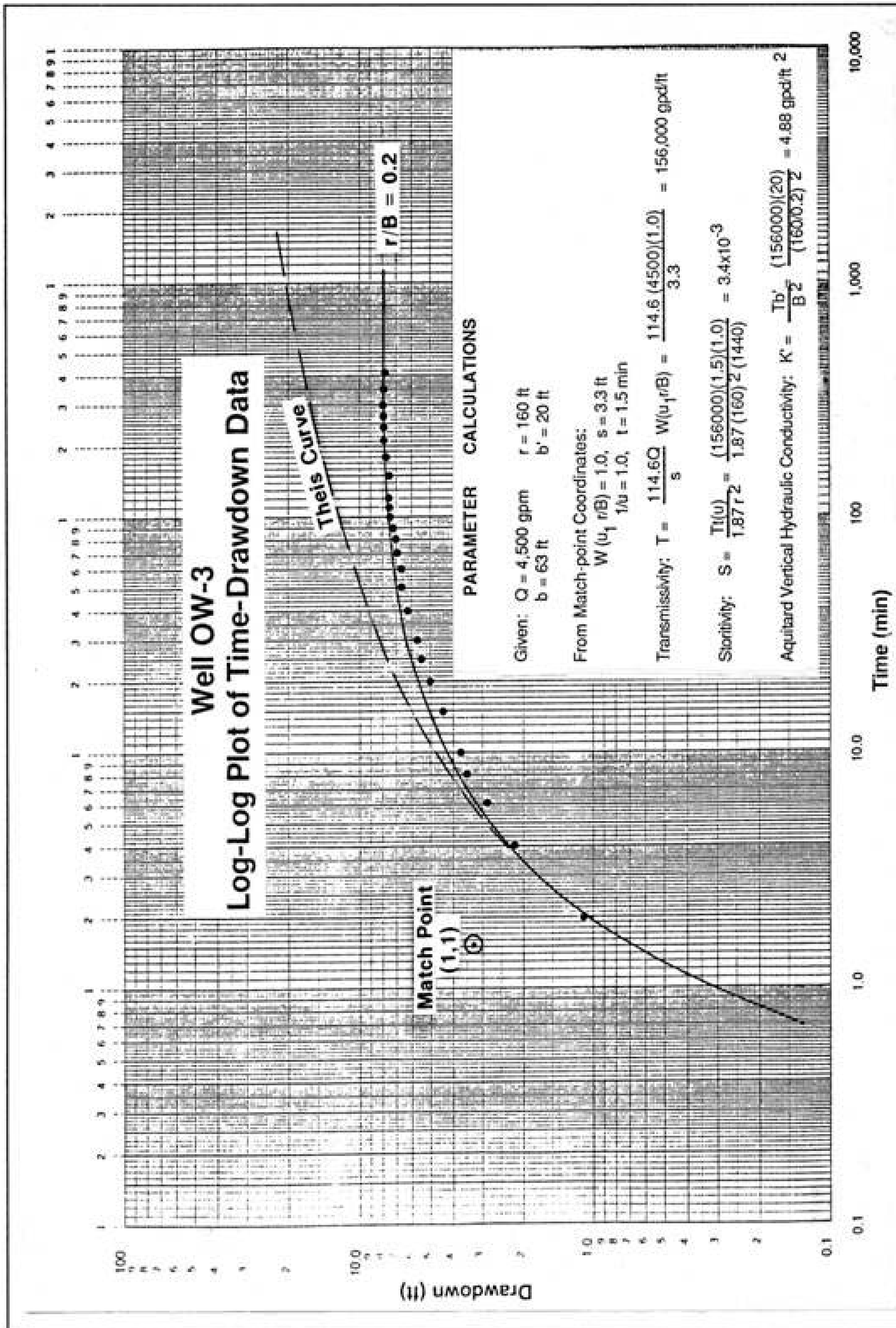
The vertical hydraulic conductivity (K') of the aquitard can be calculated in leaky aquifer analyses once the transmissivity and fitted type-curve are known. For leaky conditions with no aquitard storage, the equation for K' is:

$$K' = \frac{T\,b'}{B^2}$$

where,

K' = vertical hydraulic conductivity of aquitard (gpd/ft^2)
b' = aquitard thickness (feet)
B = leakage factor, derived by dividing the radial distance (r) by the numerical value of the matched curve (feet)
T = aquifer transmissivity (gpd/ft)

Figure B-1 shows this calculation for the above field example.



NOTE: See Text for Definitions
Source: ABB-ES

Figure B-1
Example of Analysis of Pumping Test Data Using Walton's Method for a Leaky Aquifer with Negligible Aquitard Storage.

APPENDIX C

Application of Jacob's Straight-line Method

Jacob's method is based on the modification of Theis' equation and is represented by the following equation:

$$s = \frac{264\ Q}{T} \log \frac{0.3T\ t}{r^2 S}$$

where,

- s = drawdown in the aquifer at a point corresponding to r and at time of t (feet)
- Q = test discharge rate (gallon per minute)
- T = aquifer transmissivity (gallon per day/foot)
- t = time since pumping began at rate Q (minutes)
- r = radial distance from observation well to pumping well (feet)
- S = aquifer storativity (dimensionless)

A plot of drawdown (s) versus time (t) on semi-logarithmic paper (with t on the logarithmic scale) should form a straight line having a slope $\Delta s/\log t$ with an absolute value equal to 264 Q/T. When this line is extended until it intercepts the time-axis (i.e., where s = 0), the time interception point is termed t_o.

By constructing the best-fit straight line through the data points on the semi-logarithmic plot, the values of transmissivity (T) and storativity (S) may be computed as:

$$T = \frac{264\ Q}{\Delta s}$$

$$S = \frac{0.3T\ t_o}{r^2}$$

Where,

- Δs = drawdown over one log cycle along straight line (feet)
- t_o = time-axis intercept of straight line (minutes)

and, all other terms are as defined above.

Use of the Jacob method has been prolific among pumping test analysts because of its simplicity, general applicability to both confined and unconfined aquifers, and dependence on late-time test data rather than early time data, which normally are much more susceptible to inaccuracies in field procedures. However, this method must be used with caution as it is invalid for some of the commonly encountered physical constructions of aquifers and their flow conditions (Sen, 1988).

The basic underlying assumptions for use of the Jacob method are:

1) the well discharge (Q) is held at a constant rate throughout the test (a variance of 10 percent may jeopardize interpretation).

2) the pumped well is open to the full thickness of a homogeneous, isotropic, and uniformly thick aquifer.

3) well casing storage is negligible at the test discharge rate, or the initial time duration of affected drawdown or recovery data is so short that it does not cause inappropriate straight-line fitting.

4) discharge from the well (and recovery of the drawdown cone) is by water derived exclusively from storage in the aquifer (i.e., no vertical leakage from underlying or overlying aquitards).

Conditions under assumptions 1-3 are usually controllable by the investigator and should be readily testable. An equation to calculate the elapsed pumping time (t_c) when casing storage becomes negligible is:

$$t_c = \frac{0.6\,(d_c - d_p)}{Q/s}$$

where,

d_c = inside diameter of well casing (inches)

d_p = outside diameter of pump column pipe (inches)

Q/s = specific capacity of the well (gpm/ft of drawdown) at time t_c

The Jacob method will not give correct values for transmissivity and storativity if the plotted data that are fitted to a straight line are affected by the cone of depression encountering aquifer boundaries, either barrier or recharge types. Barrier boundaries will cause the slope of the plotted data to increase, while a recharge boundary (such as a river) will cause the slope to decrease. Vertical leakage will result in the data plot becoming convex for a variable period of time followed by another variable time span during which a nearly straight-line trend is likely.

If leakage is present, the initial straight-line plot segment usually will not conform to Jacob method requirements, and the later straight-line segment is always invalid.

A specific mathematical requirement for valid application of the straight-line method, cited by many authors, is that u (called a dimensionless time factor) be less than about 0.01 for the data points fitted to a straight line. Theis (1935) derived u as:

$$\mu = \frac{r^2 S}{1 - 4 t T}$$

where (in consistent English units),

r = radial distance of observation well from pumping well (feet)
S = aquifer storativity (dimensionless)
T = time (elapsed) of data point being tested (days)
T = aquifer transmissivity (feet squared per day)

In practice, it has been found that u may reach about 0.1 before this technique becomes invalid (Murray, 1987).

Sen (1988) points out that non-Darcian turbulent flow near the well will invalidate a Jacob analysis of drawdown data that follow a straight line on semi-log plots. In his article, Sen gives a rigorous data-manipulation procedure to check the validity of the Jacob method for any data set. Although the Sen test could be performed regularly, it is recommended that when any analysis is being performed on data collected in a geologic setting that potentially may deviate from the Jacob-method requirements, a log-log, type-curve analysis also be performed. Commonly, the two approaches complement and/or corraborate interpretations of one another.

An example of the Jacob straight-line method applied to pumping test recovery data is given in Figure C-1 (Appendix). Instead of simply plotting drawdown versus elapsed time as would be done for data collected during pumping, residual drawdown is plotted against total elapsed time since pumping began divided by elapsed time since pumping stopped (t/t'). Residual drawdown is the difference between the water-level elevation corresponding to any given t' and the pre-pumping static elevation.

In Figure C-1, recovery data were not collected long enough for the water level to return to the pre-pumping static level. However, the fitted straight-line is extended to zero residual drawdown where a value is read for t/t', and subsequently a t' value can be determined for use in Jacob's storativity equation.

Recovery analyses often give more reliable values of transmissivity and storativity than drawdown analyses. When pumping rates are low due to low permeability geologic materials typical of many contamination sites, the effect of variable discharge at low rates during a test is minimized by analyzing recovery data with the Jacob method. The average pumping rate for the entire period of pumping must be used in Jacob's equation for transmissivity.

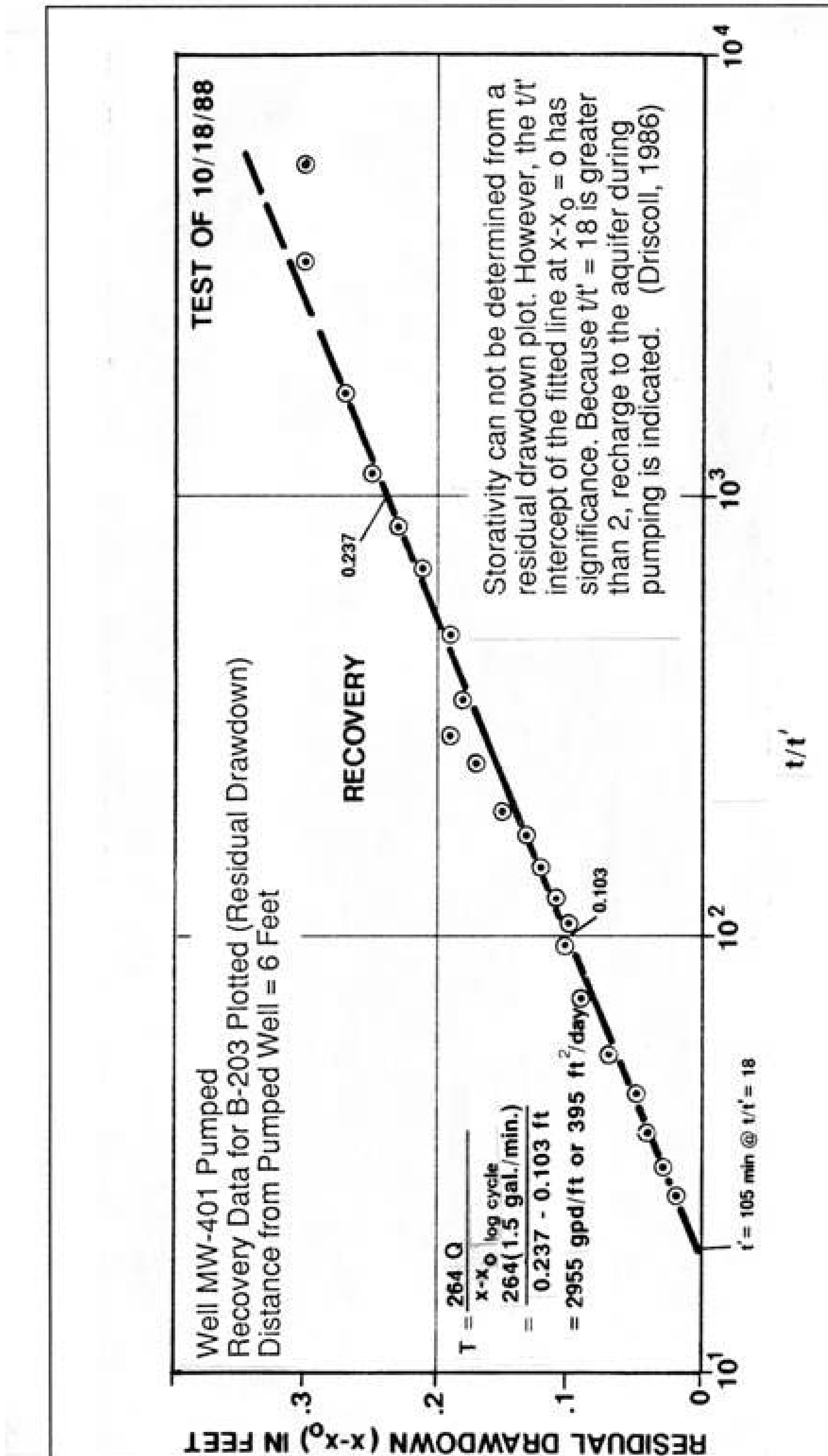


Figure C-1

Example of Jacob's Straight-line Analytical Solution.

Source: ABB-ES

COMMONWEALTH OF MASSACHUSETTS

DEPARTMENT OF ENVIRONMENTAL PROTECTION

STANDARD REFERENCES FOR MONITORING WELLS

SECTION 5.4 PACKER TESTS

SECTION 5.4 PACKER TESTS

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5.4 PACKER TESTS

5.4-1 PURPOSE

Water pressure tests or "packer tests" are in-situ tests performed to measure the permeability of a specific zone in a bedrock borehole. Water pressure tests are used to estimate bedrock permeabilities for hydrogeologic studies and in estimating grouting and dewatering requirements for construction purposes.

Packer tests may be done during the advancement of the borehole or after drilling is completed. Packer tests are usually conducted in NQ/NX-size (3-inch) boreholes, but can be conducted in boreholes of a larger size. The test involves placing expandable packers, either mechanical or pneumatic, in a borehole. A pneumatic packer assembly is preferred because it is easier to use and provides a more positive seal. A section of the borehole, usually five feet in length, is sealed off with the packers. Water is then pumped through the zone between the packers at a known pressure. The rate of flow into the formation is measured with a flow meter. The permeability of the test zone is calculated using the data obtained in the test.

5.4-2 METHODOLOGY

The following methodology was designed to present the general requirements of a bedrock packer test. It is advisable to consult additional references before actually performing this type of test.

1. Flush the borehole with clean water to remove cuttings. Measure the depth of the borehole, and check for caving. Be sure that an adequate reserve of water is available to avoid running out of water during a test.

2. Determine the test zone. The test section length should be a minimum of 5 times the diameter of the borehole. Avoid placing the packer in a zone of fractured rock or in the bottom of the casing because leakage will occur. Keep the rock core or drilling logs handy to refer to during the test.

3. Maintain the test pressures below what is commonly referred to as the Maximum Water Pressure (Pmax). This should avoid the chance of hydrofracturing (loosening) the rock mass. Pmax is determined by the following formula:

$$Pmax = (H_1)\ (1\ psi/ft)$$

(Note: in highly fractured rock this should not exceed 0.75 psi/ft.)

where,

H_1 = depth in feet from ground surface to the bottom of the upper packer

During test operations the water pressures are observed at the gauge. The Maximum Gauge Pressure (GPmax) is calculated by the following formula:

$$Gpmax = (H_1+H_3)\ (1\ psi/ft) - (H_1-H_2)\ (.43\ psi/ft)$$

where,

H_1 = depth in feet from ground surface to the bottom of the upper packer

H_2 = depth in feet from ground surface to the static water level

H_3 = height in feet of pressure gauge above ground surface

The depth and height variables (H_1, H_2 and H_3) are shown on Figure 5.4-2.

When significant flow rates are encountered during the test the gauge pressure may need to be increased to compensate for system pressure loss due to frictional head loss. This is an unusual situation.

4. To ensure that the packer system is not leaking, test it prior to the start of the actual permeability test. This can be done by installing the packer in a piece of steel casing and conducting the test as if it were being done in the borehole. The water pressure must not exceed the Packer Inflation Pressure (see Step #5, below). Calibration for a particular test assembly can be obtained on site by laying the system out on the ground and pumping water through the system while collecting the data as if the test were being performed in-situ. Check the hose for leaks. Check the water meter to assure that it is working properly.

5. Determine the Packer Inflation Pressure (PIP), by performing the following steps:

Step 1 - Establish Minimum Inflation Pressure (MIP) (i.e., the pressure required to inflate the packers in the casing so that they can no longer be pushed or pulled through the casing)

Step 2 -Establish the Static Head Pressure (Ps) in psi at the test depth by the following calculation:

$$Ps = (H_1 - H_2)\ (0.43\ psi/ft)$$

where,

H_1 and H_2 are as above

Step 3 - Make sure the Packer Inflation Pressure (PIP) equals the Minimum Inflation Pressure (MIP) plus the Static Head Pressure plus the Maximum Gauge Pressure (Gpmax) of the test zone between the packers. This is sometimes written as follows:

$$PIP = MIP + Ps + Gpmax$$

6. Determine the static water level in the borehole prior to the installation of the packer.

7. Assemble and install the packer equipment in the borehole. Measure each rod to top of coupling as it goes into the hole. Be sure rods are tightened to prevent leakage at the joints; teflon tape may be helpful. Number the rods for easy tracking of the packer location for sequential tests. Lower the equipment to the location of the deepest test. Figures 5.4-1 and 5.4-2 depict configurations for mechanical and pneumatic packer tests.

8. Before performing the first test, bleed air out of the lines by forcing water through the packer system assembly before the packers are inflated. Inflate both packers to the required packer pressure. Double packers are usually spaced five feet apart, but spacing can be varied to meet specific test requirements.

9. Before starting the test, review the Packer Test Data Sheets (Figure 5.4-3) and record the following:

   o Test number
   o Test section (i.e., length)
   o Hole size
   o Height of pressure gauge above ground surface
   o Ground surface elevation
   o Depths to rock surface, ground water, bottom of boring, bottom of upper packer, and to top of lower packer

10. Conduct the bedrock packer test in three stages:

    Step 1 - 1/2 Gpmax

    Pump water into the system and record observations of gauge pressure and water meter at 30-second intervals for at least three to five minutes after a constant rate of flow is reached.

    Step 2 - Full Gpmax

    Pump water into system and record observations of gauge pressure and water meter at 30-second intervals for at least three to five minutes after a constant rate of flow is reached.

    Step 3 - Full Gpmax plus 20 psi increase on the Packer Inflation Pressure

    Increase Packer Inflation Pressure by 20 psi. Pump water into the system and record observations of gauge pressure and water meter at 30-second intervals for at least three to five minutes after a constant rate of flow is reached. The results of Steps 2 and 3 should be similar. If they are not, Step 3 should be repeated, increasing the Packer Inflation Pressure by an additional 20 psi. This is done to check for leakage past the packers.

    For all test steps, record water levels in the casing during the test. If the water level rises or bubbles appear during the test, the packers may not be sealed and the test results may be suspect. Measurements of doubtful accuracy must be noted, along with a description of the questionable aspects. If possible, testing should be continued until accurate data is obtained. It may be necessary to move the packer assembly a short distance to obtain an adequate seal.

11. If leakage of water from the packed section into the surrounding rock is so great that the Gpmax cannot be reached, run the pump at its full capacity with the bypass valve closed. Record the volume of water pumped into the test section and the associated pressure readings at timed intervals. This data will give a minimum value of the rock permeability.

12. Upon completion of the test, deflate the packers and move to the next test depth. Complete log sheets (see Figure 5.4-3).

13. The same test methodology may be used with a single packer. Single packer tests are conducted either as the borehole is advanced or after the entire borehole has been completed. With this test configuration the bottom of the borehole takes the place of the second packer.

5.4-3 COMPUTED ROCK MASS PERMEABILITY

Compute the rock mass permeability. Additional data required for each test are as follows:

(1) depth of hole at time of each test;
(2) depth to bottom of top packer;
(3) depth to top of bottom packer;
(4) depth to water level in borehole at frequent intervals;
(5) elevation of potentiometric level;
(6) length of test section;
(7) radius of hole;
(8) length of packer;
(9) height of pressure gauge above ground surface;
(10) height of water swivel above ground surface; and
(11) description of material tested.

Item (4) is important since a rise in water level in the borehole may indicate leakage from the test section or an interconnected bedrock fracture pattern. A sketch of the test equipment arrangement showing the relative portions of the components should be made for each configuration used. (See Figure 5.4-3, page 3 of 3.)
The formulas used to compute the permeability from pressure test data are:

$$K = (Q/2\pi LH)\ \ln(L/r)$$

When $L > 10r$ (the above formula is used when the length is greater then ten times the radius)

and,

$$K = (Q/2\pi LH)\ \sinh^{-1}(L/2r)$$

When $10r > L > r$ (the above formula is used when the length is greater than the radius but less the ten times the radius)

where,

K = permeability
Q = constant rate of flow into the hole
L = length of the test section
H = differential head on the test section
r = radius of the borehole

It should be noted that when the test is conducted above the water table H is the distance from the water pressure gauge to the middle of the test section. When the test is below the water table H is the distance from the gauge to the static water level.

While the above formula is most often used with a double packer arrangement, it also applies for use with a single packer. With a single packer the length of the test section (L) is not fixed (as with the double packer arrangement) and is equal to the distance from the bottom of the packer to the bottom of the hole.

These formulas provide only approximate values of K since they are based on several simplifyin0g assumptions and do not take into account the flow of water from the test section back to the borehole (U.S. Bureau of Reclamation, 1977).

However, they give values of the correct order of magnitude and are suitable for practical purposes. Table 5.4-1 (Haley and Aldrich, 1977) provides a general grouping of rock mass permeability.

5.4-4 PROBLEMS AND POSSIBLE SOLUTIONS

There are a number of possible problems that may develop while performing a bedrock packer test. Several of the most common problems and their possible solutions are outlined below.

1. Packers move up out of the hole at the start of the test.

   Occasionally, particularly in low permeability rocks, the packer assembly may lift out of the hole due to the water pressure. Observers should stay clear of the top of the borehole to avoid injury. It may be helpful to deflate and re-inflate the packers to obtain a more positive seal in the borehole. Also, the rig drive head can be placed over the top of the swivel to help to hold the packers in place during the testing.

2. Excessive amounts of water are pumped into the formation.

   In certain types of hydrogeologic or contaminant investigations, large quantities of water should not be pumped into the aquifer as this may impact local ground water quality and movement. If this is a concern, packer tests should be avoided. Alternatively, falling or rising head tests may be performed or geophysical borehole data may be obtained.

3. The packers jam in the borehole.

   Packers may become caught in the borehole for two reasons: 1) caving of the formation around the packers, or 2) failure of the packers to deflate. In the latter case, it is generally advisable to re-inflate and deflate the packers a second time to try to remedy the problem. Forcibly removing the packers from the hole should be avoided as they may become permanently lodged or damaged. In some instances it may be helpful to pump water through the system to help lubricate the equipment for removal. Packer tests in soft, broken or cavernous formations should always be attempted with great caution.

4. Water meter malfunctions.

Water meters are sensitive instruments and are subject to malfunctions due to clogging by debris or mechanical failure. It is important to check the water meter prior to use to be certain that it is working properly. Generally, it is best to place the water meter in a horizontal position, particularly for low flow measurements. It is also important to determine what the units of the meter dial are prior to use, as they are often poorly marked. Discharging water from the meter into a container of known volume (e.g., 5-gallon bucket or a 55-gallon drum) and comparing this to the metered volume provides a reasonably accurate check.

REFERENCES

Haley and Aldrich Inc., 1977, Manual of field procedures, procedure no. 27, water pressure test (rock): Cambridge, MA.

U.S.Bureau of Reclamation, 1977, Ground water manual: Denver, CO, U.S. Government Printing Office, 783 p.

ADDITIONAL REFERENCES

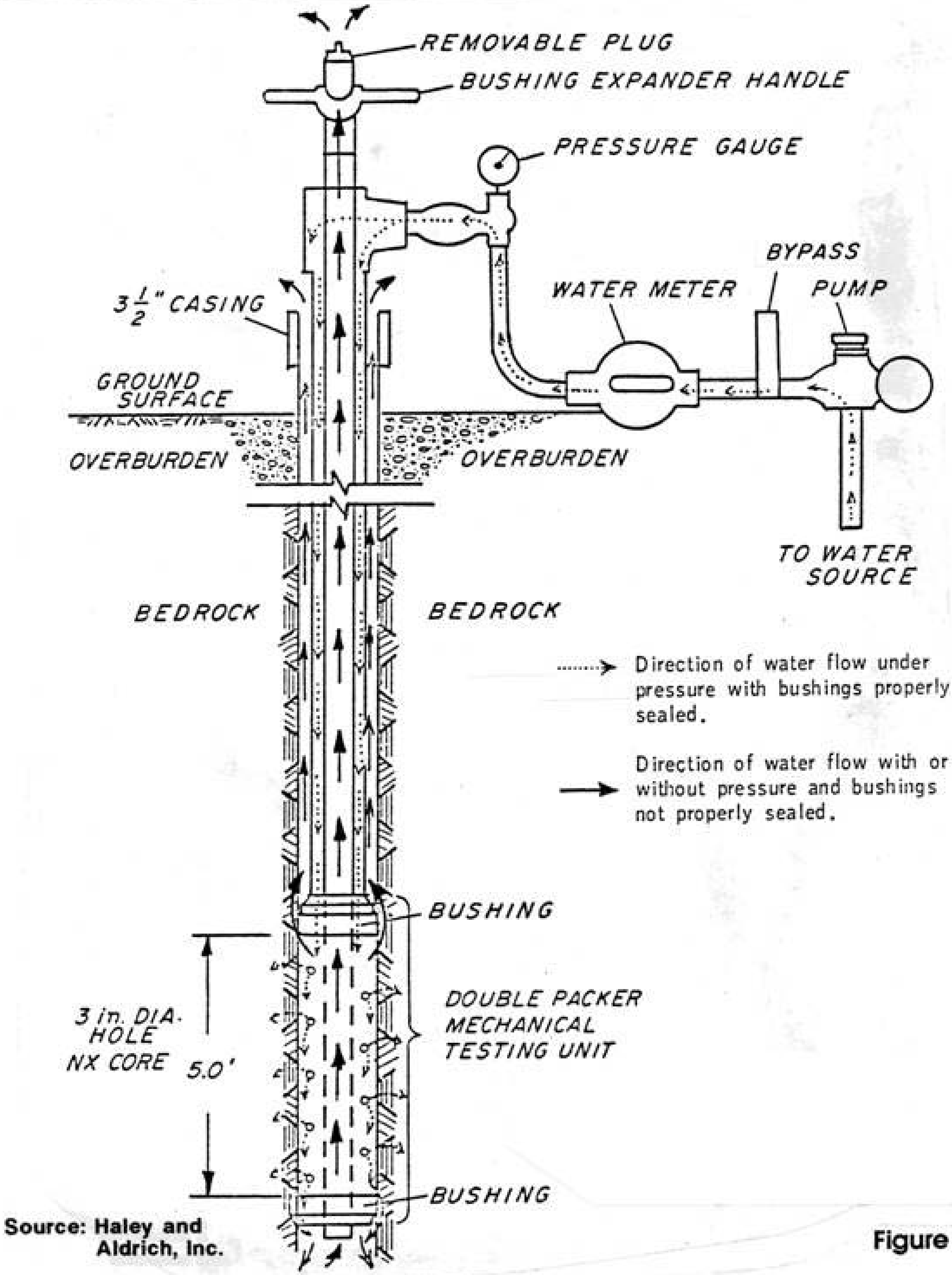
U.S.Bureau of Reclamation, 1974, Earth manual, Washington: DC, U.S. Government Printing Office, 810 p.

Winterkorn, H., F., and Fang, H., 1975, Foundation engineering handbook: New York, NY, Van Nostrand Reinhold Company, 751 p.

SECTION 5.4
PACKER TESTS

LIST OF FIGURES

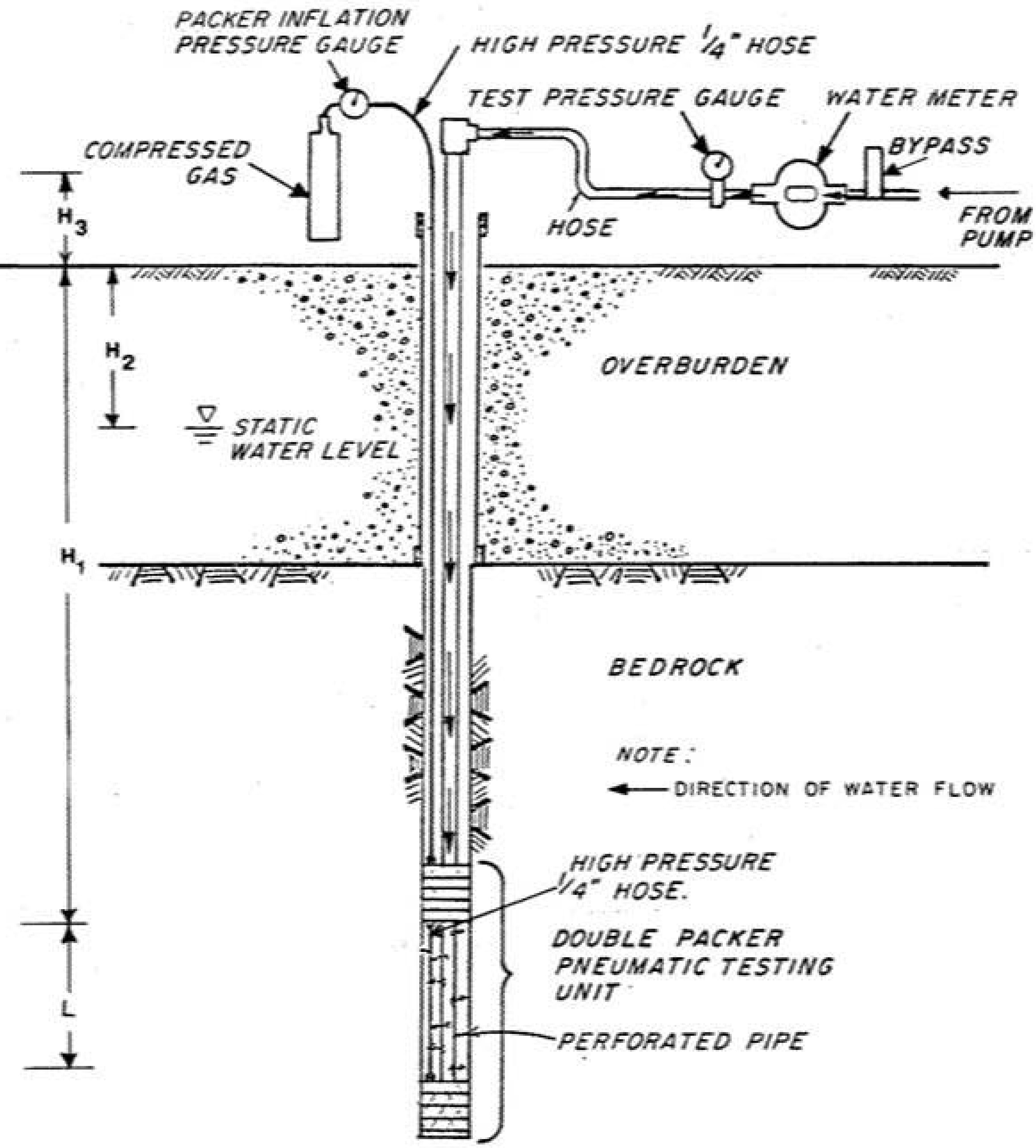
LIST OF TABLES



Source: Haley and Aldrich, Inc.

Figure 5.4-1

Schematic of Mechanical Packer Pressure Test and Related Equipment.



Source: Haley and Aldrich, Inc.

Figure 5.4-2

Schematic of Pneumatic Packer Test and Related Equipment.

WATER PRESSURE TEST

HOLE NO. | TEST NO.

PROJECT:
CLIENT:
CONTRACTOR:

JOB NO.
SHEET NO.
LOCATION:
ELEVATION:
DATE START:
DATE FINISH:
DRILLER:
INSPECTOR:
GEOLOGIST:

	PACKER SYSTEM	WATER METER	WATER GAUGE	SURGE CHAMBER
TYPE				
MFG.				
MODEL NO.				

M.G.P. = (0.566 to 1.0) x 2
COMPUTED MAX GAUGE PRESS: (MGP)
COMPUTED INTERNAL FRICTION:

ROCK TYPE: HOLE SIZE
RECOVERY (%)
R Q D (%)

DEPTHS: (All Distances Measured From Ground Surface In Feet)

TO TOP OF ROCK
TO TOP LOWER PACKER
TO BOTTOM OF BORING
TO BOTTOM UPPER PACKER (2)
TO WATER TABLE
LENGTH OF TEST SECTION
HEIGHT OF WATER PRESSURE GAUGE ABOVE GROUND SURFACE

TIME	ELAPSED TIME (MIN)	PACKER PRESSURE (PSI)	GAUGE PRESSURE (PSI)	METER READING (GALS)	VOLUME OF FLOW (GALS/MIN)	REMARKS

Figure 5.4-3
Page 1 of 3
Example of Packer Test Data Sheet.

Source: Weston Geophysical

WATER PRESSURE TEST

Project______________________ Boring No. _____________ Test No. _____________

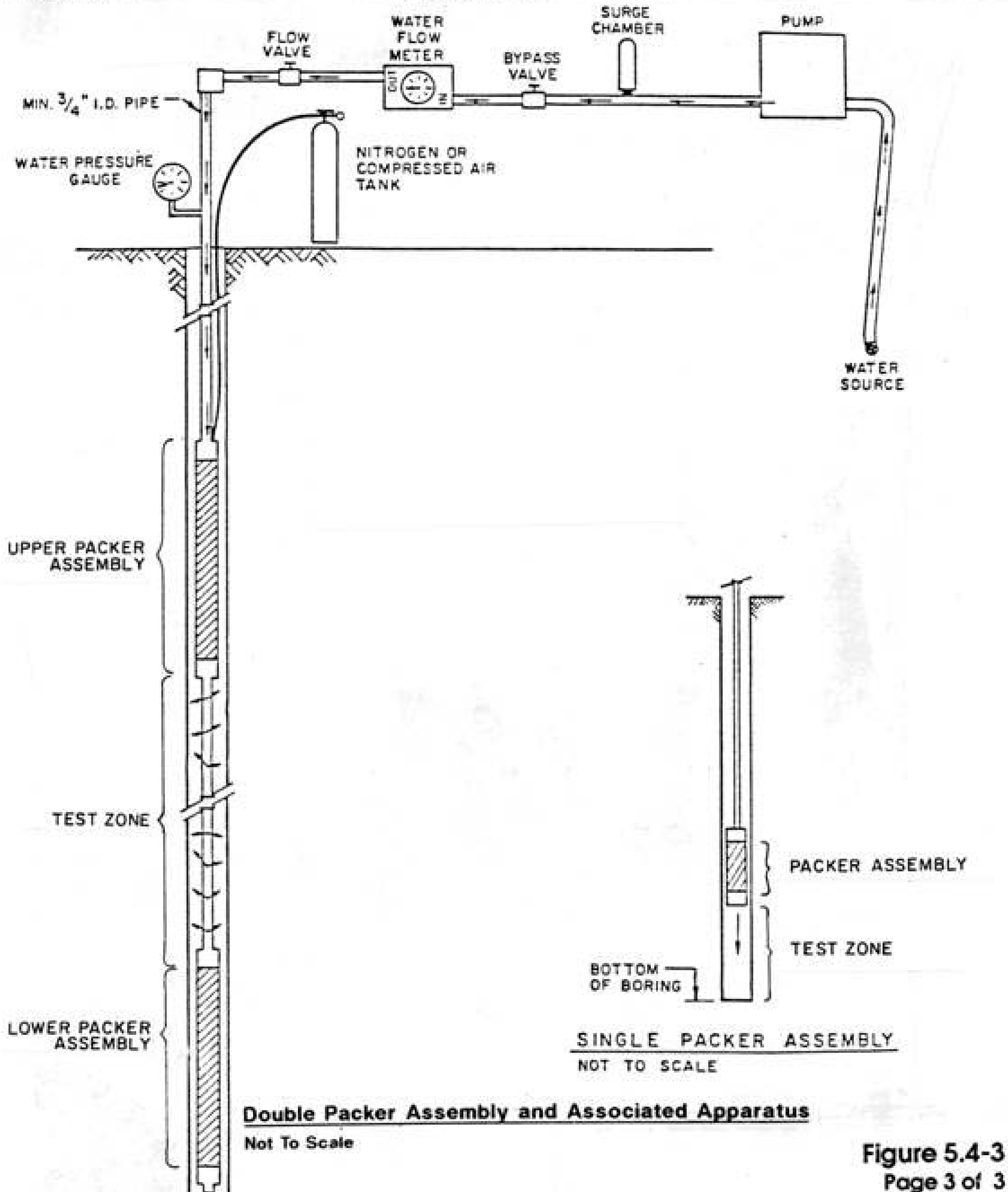
TIME	ELAPSED TIME (MIN)	PACKER PRESSURE (PSI)	GAUGE PRESSURE (PSI)	METER READING (GALS)	VOLUME OF FLOW (GALS/MIN)	REMARKS

Source: Weston Geophysical

Figure 5.4-3
Page 2 of 3
Example of Packer Test Data Sheet.

SITE / LOCATION ____________ PROJECT NO. ____________ DATE ____________ BORING NO. ____________

INSPECTOR ____________ APPROVED BY ____________



Double Packer Assembly and Associated Apparatus

Not To Scale

Figure 5.4-3
Page 3 of 3
Example of Packer Test Data Sheet.

Source: Weston Geophysical

Description	Range
Very low (equivalent to clay)	Less that 1×10^{-7} cm/sec
Low (equivalent to silt)	1×10^{-5} to 1×10^{-7} cm/sec
Medium (equivalent to fine sand)	1×10^{-4} to 1×10^{-5}cm/sec
High (equivalent to sand)	1×10^{-2} to 1×10^{-4} cm/sec
Very high (equivalent to clean sand or gravel)	More than 1×10^{-2} cm/sec

Table 5.4-1
General Grouping of Rock Mass Permeability

COMMONWEALTH OF MASSACHUSETTS

DEPARTMENT OF ENVIRONMENTAL PROTECTION

STANDARD REFERENCES FOR MONITORING WELLS

SECTION 5.5 SURVEYING AND DATUM PLANES

SECTION 5.5
SURVEYING AND DATUM PLANES

TABLE OF CONTENTS

5.5 SURVEYING AND DATUM PLANES

5.5-1 PURPOSE

The purpose of this section is to present the minimum requirements for establishing horizontal and vertical surveying control for exploration programs that will satisfy the requirements of the Commonwealth's new computerized Geological Information System (G.I.S.). Accurately surveyed locations of explorations are a key element in the evaluation of all field data and are necessary for the preparation of geologic profiles and the interpretation of vertical and horizontal ground water flow directions. The accuracy of measurements and established elevations is particularly important when ground water gradients are low, as errors may easily lead to misinterpretation of the direction of ground water flow. The survey is usually performed after the explorations have been completed. Explorations and land features requiring accurate horizontal and vertical control are:

- Borings
- Test pits and trenches
- Monitoring wells and piezometers
- Geophysical surveys
- Surface water and drainage features
- Buildings and underground tanks

5.5-2 METHODOLOGY

The project manager shall go over the survey program with the survey chief to be sure that all requirements are understood and that the survey crew is alerted to potential site hazards. The following criteria should be met for <u>all</u> survey programs.

1. The survey is to be performed by registered professional land surveyors or civil engineers.

2. The survey shall be accurately performed to a precision of 0.01 foot for vertical control and 1.0 foot for horizontal locations.

3. Horizontal control is to be tied into either the USGS grid or the UTM grid coordinate system. Mean Sea Level (NGVD, 1929) should be used as the vertical datum.

4. Elevation precision to be obtained at monitoring wells and piezometers shall be:

- o Top lip of protective casing without cover (0.01 foot); this point should only be used for vertical control and not for water-level measurements
- o Top of monitoring well riser pipe (0.01 foot); a permanent reference point should be marked on the top of the riser to be used as the measuring point for all water-level measurements

5. Mark clearly a permanent site benchmark at the site on the most stable nearby feature and note its location on survey maps.

6. The surveyor should submit, as part of the survey report, a copy of all original field notes, including a description of the measuring points at all monitoring wells to make certain that the elevation has been assigned to the correct point.

7. Survey information needs to be reviewed carefully with respect to horizontal and vertical determinations. Survey errors may often be caught by using relative distances between wells or noting apparent anomalies in water levels or flow directions. The survey should proceed in a manner that closes out the loop so that one can detect errors. Check to see that all survey traverses have been closed on the original benchmark or reference point to within acceptable limits.

5.5-3 PROBLEMS AND POSSIBLE SOLUTIONS

5.5-3.1 Previous Use of a Datum Other Than Mean Sea Level

Many times a parcel of land contains a previously established permanent benchmark on-site to which all vertical elevations have been referenced. Such an arbitrary local datum may not provide any specific information about its relationship to the USGS datum Mean Sea Level (MSL), the standard National Geodetic Vertical Datum (NGVD) of 1929. An arbitrary datum, when used, should be designated by the letters L.D., for Local Datum; in the past, this designation often has been omitted. In other cases, a standard city-wide local datum is used; this carries a known and published relationship to the USGS datum.

Surveys at all sites subject to DEP review shall be referenced to Mean Sea Level. Due to the requirements of the Department's computerized mapping program, the Department can only accept information that is expressed in terms of the USGS datum. Fortunately, Massachusetts is liberally endowed with reference elevation benchmarks. Therefore, it should not be too difficult to convert a local datum to the USGS standard.

5.5-3.2 Weather Conditions

Inclement conditions increase the chance for errors in identification, measurement, and recording. Surveyors need to take extra time to assure proper identification of all monitoring wells surveyed, to guarantee ice-and snow-free surface elevation shots, and to carefully record survey data despite adverse conditions. Obtaining stable tripod set-ups may be more difficult under these conditions. Sightings should use shorter distances than under more favorable conditions. Warm, sunny days generate heat waves that may present problems for optical instruments.

5.5-3.3 Work at Hazardous Waste Sites

Surveyors need to be made aware of hazardous site conditions and potential exposures. Surveyors should have been enrolled in a health monitoring program for any sites which require personal protection above Level D (see Section 2.3 Health and Safety). Note that anticipated risks to surveyors would be expected to be less than for those engaged in collecting samples or in subsurface explorations. However, potential surface contact with hazardous materials should be pointed out and appropriate protective equipment worn and used. Surveyors shall also be made aware of other site activities and procedures for evacuation in case there is a release elsewhere on-site which triggers implementation of site evacuation or other contingency plans.