Chapter 6
Aquifer-Test Data Collection and Analysis

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Overview

Various texts interchangeably use the terms "pumping tests" and "aquifer tests" to describe a variety of hydraulic tests and, in practice, groundwater professionals also generally use the terms interchangeably. For consistency, in this chapter the term "aquifer testing" covers all types of tests conducted to assess the performance of the well-aquifer system and the aquifer itself. Pumping tests—as defined in this chapter—are conducted to assess the capabilities of the pump, and such information can be obtained from pump manufacturers.

Aquifer testing includes tests used to estimate the performance characteristics of a well (screen and aquifer in the immediate vicinity of the well), and also to determine the hydraulic properties of the aquifer. For a well-performance test, flow rate and drawdown are recorded within the pumping well to enable calculation of the specific capacity (which is equal to flow divided by drawdown). These data provide a measure of the productive capacity of the completed well. Specific-capacity data collected periodically over the operational life of the well provide insight as to whether blockage of the screen or aquifer is occurring. Specific-capacity data also provide information needed for selection of pumping equipment.

The primary purpose of aquifer testing is to provide data from which aquifer properties, such as transmissivity and storage coefficient, can be calculated. Aquifer tests also are used to assess the impacts of boundaries on well production. The results of aquifer tests provide fundamental input data that can be used in calculations that predict drawdowns in an area (due to pumping).

Pumping a well lowers the water level in both the well and the surrounding geologic materials. The change (drop) in water level from the static or pre-pumping level is termed "drawdown." The greatest drawdown occurs at the pumping well and dissipates as distance from the well increases. Under ideal conditions, the distribution of drawdown around the pumping well assumes a conical shape—often referred to as a cone of depression. Figure 6.1 provides a diagram of the processes associated with pumping a well, and also shows the static water level (SWL) and pumping water level (PWL).

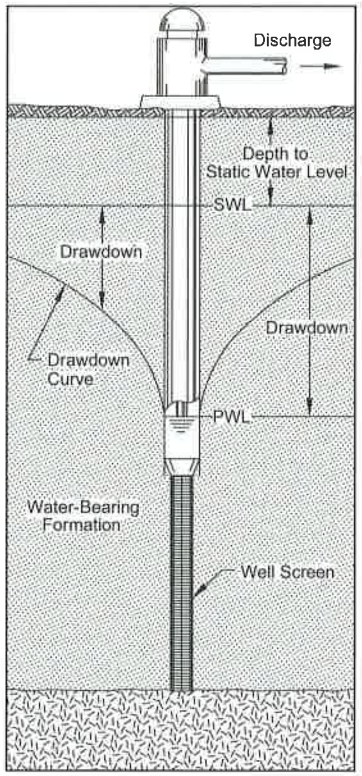

Figure 6.1. Processes associated with pumping a well.

A common type of test is the constant-rate aquifer test, which consists of pumping a well at a specific rate for a length of time and recording the drawdown in both the pumping well and in nearby observation wells at specific times. A constant-head variable-rate test commonly is run when a naturally flowing well in a confined aquifer is shut-in long enough for the head to recover, and then the well is opened and allowed to flow.

Step-drawdown tests commonly are conducted prior to undertaking a long-term constant-rate test, and are used to assess a long-term pumping rate and to estimate laminar versus non-laminar well losses within the aquifer and well. A step-drawdown test also is useful in assessing the appropriate pump for the well. These types of tests are described in more detail in this chapter.

Measurements required for aquifer tests include the:

- Static water levels just before the test is started;
- Time since the pump started;
- Pumping rate;
- Pumping levels or dynamic water levels at various intervals during the pumping period in both the pumping well and observation wells (if installed);
- Time of any change in discharge rate;
- Time the pump stopped; and
- Water levels during recovery.

Measurements of water levels after the pump was stopped (recovery) are extremely valuable for verifying the aquifer properties calculated during the pumping phase of the test. Most hydraulic testing of water-supply or irrigation wells is conducted in the pumping well without the use of an observation well or piezometer to measure drawdown occurring as a result of pumping. Single-well tests can lead to erroneous interpretations of aquifer characteristics due to head losses occurring within the pumping well. As is discussed in this chapter, the use of an observation well for measuring water levels is very advantageous.

This chapter emphasizes constant-rate, variable-rate, and step-drawdown tests because they are the most common types of aquifer tests conducted using large-capacity wells and aquifers having high transmissivity.

This chapter only provides an overview of concepts of delayed yield and boundaries. Single-well tests (or "slug tests") are not discussed because they primarily are used to test small-diameter wells installed in geologic materials that have relatively low to moderate hydraulic conductivities. Some excellent references on slug tests or aquifer testing include Butler (1998) and Kruseman and de Ridder (1992).

Conducting an Aquifer Test

Prior to performing an aquifer test, the following topics should be addressed.

- Objectives of testing
- Budget
- Site logistics (access, discharge of water)
- Availability and types of equipment for pumping and measurements of flow rates and water levels
- Conceptual hydrogeologic model of site based on available background information and geologic exploration

The objectives and associated budget should determine the type of test to be performed. If the objective of testing merely is to assess the type of pump to be installed in a well, for example, then a step-drawdown test that spans the probable range of pumping rates could suffice. Conversely, if the objective is to assess the long-term impacts to water levels in a fractured bedrock terrain due to mining, then aquifer tests might have to be conducted for several weeks and water levels measured in multiple observation wells or piezometers that are distributed around the proposed mine. The availability of flow and water-level measuring equipment, together with the number of field personnel available, dictate the frequency and accuracy of measurements.

It cannot be overemphasized that background hydrogeologic information should be used to synthesize a conceptual hydrogeologic model of the area. The conceptual model should include comprehensive information about recharge and discharge areas; locations and thicknesses of aquifers and aquitards; preliminary estimates of hydraulic properties based on background literature and experience; and the locations of boundaries (such as surface-water bodies and changes in stratigraphy). The conceptual hydrogeologic model influences the length of test (i.e., a test performed under confined conditions generally is shorter than one done in unconfined conditions). The locations and design of observation wells also are influenced by the conceptual model. For example, the impacts of boundaries on pumping can be assessed by placing observation wells between the pumping well and the suspected boundary.

Estimates of the magnitude and timing of drawdown can be made based upon information obtained from the drilling and development of the well through the use of air-lift tests. Air-lift tests are discussed in Appendix 6.A (DVD). Geologic data obtained from drilling and sampling in conjunction with estimates of hydraulic properties from air-lift tests can be used to refine the preliminary conceptual hydrogeologic model. Calculations can be made based upon the model and then used to design the aquifer test and to determine expected drawdown. Using the estimates of drawdown, the type of water-level measurement device and timing of measurements can be established. As noted, considerations of equipment used are based on budget constraints and data requirements for each project. Prior planning and experimentation with equipment by the field personnel during preliminary testing can eliminate potential errors that could occur during the actual aquifer test.

Aquifer tests do not produce accurate data unless the tests are conducted by systematically and accurately recording the duration, flow rates, and water levels over time. Certain preliminary steps should be taken to assure the reliability of test data recorded during the actual aquifer test. Before a constant-rate test is to be performed, for instance, a step-drawdown test can be conducted in the pumping well to assess the following.

- Maximum anticipated drawdown (for most aquifer tests, a major portion of the drawdown occurs in the first few hours of pumping)
- The volume of water produced at certain pumping rates and drawdown
- Flow measurement methods
- Whether the discharge from the pump is piped far enough away to avoid recharge
- Whether the observation wells are located so that they exhibit sufficient drawdown to produce usable data

The quality of drawdown data recorded during a constant-rate aquifer test depends on the following.

- Maintaining a constant flow rate during the test
- Accurately measuring the drawdown in the pumping well and in one or more properly placed observation wells, if installed
- Recording drawdown measurements at appropriate time intervals

- Determining how changes in barometric pressure and other loadings or stresses affect drawdown data
- Continuing the test for sufficient time for its objectives to be met

Maintaining Discharge

It is vital that uninterrupted drawdown data be obtained once the constant-rate aquifer test commences. As such, the pump and power unit should be capable of operating at a constant rate for the length of the test. In cases where the observation wells must be located at considerable distances from the pumped well, or where it is critical to assess boundary conditions, the pump must be capable of operating for at least several days or even weeks. Pump failure during the test can significantly complicate the analysis.

The pumping rate should be measured accurately and recorded periodically. Control of the pumping rate during testing requires both a device for accurately measuring the discharge of the pump and a convenient means for adjusting the rate to keep it as nearly constant as possible. A valve in the discharge line of the pump provides the best control. The discharge pipe and the valve should be sized so that the valve is from one-half to three-fourths open when pumping at the desired rate. Minor changes in motor or pump output cause less fluctuation of the discharge when the pump is working against the back pressure or head developed in a partially closed valve. Changing the pumping rate by controlling the pump speed (in the case of a turbine pump) generally is unsatisfactory, and using a valve—such as a gate valve (Figure 6.2)—is preferred for adjusting the flow rate.

Figure 6.2. Gate valve (Danfoss Flomatic Corp.).

Flow Measurement Methods

Calibrated Container

A simple and accurate method for determining the pumping rate is to observe the time required to fill a container of known volume or calibrated container. If it takes 30 sec to fill a 55-gal (0.2-m^3) barrel, for example, then the pump is delivering 110 gpm (600 m^3/day). This method is practical only for measuring relatively low pumping rates, however, and requires near-constant attention.

Flow Meter

A commercial water meter is more reliable for measuring large discharges. On a totalizing flow meter, the dials or digital readout on the meter show the total volume discharged through the meter up to the time of observation. Subtracting two readings taken at a set time apart gives the pumping rate. This meter perhaps is the easiest apparatus to use. Its only disadvantage is the unavoidable delay in obtaining values at the start of the test, when the pumping rate is being adjusted to the desired level.

Instantaneous readings can be made using instantaneous flow meters (Figure 6.3). To obtain accurate readings, an inline flow meter must be located in a straight section of pipe. It is recommended that a minimum of 10 pipe diameters upstream and 5 diameters downstream of the meter be straight and have no obstructions.

Figure 6.3. In-line flow meter (Kobold, Inc.).

Orifice Weir

A circular orifice weir is a device commonly used to measure the discharge rate from a high-capacity pump. Figure 6.4 shows a typical orifice weir.

The orifice is a round hole—with clean, square edges—in the center of a circular steel plate. The plate must be 1/16 in (1.6 mm) thick around the circumference of the hole; it is fastened against the outer end of a level discharge pipe so that the orifice is centered on the pipe. The end of the pipe must be cut squarely so that the plate is vertical. The bore of the pipe should be smooth and free of any obstruction that could cause abnormal turbulence. The discharge pipe must be straight and level for a distance of at least 6 ft (1.8 m) before the water reaches the orifice plate. The piezometer (manometer or transducer) tube must be placed exactly 24 in (610 mm) from the end of the pipe. The pipe wall is tapped midway between top and bottom with a ⅛-in (3.2-mm) or ¼-in (6.4-mm) hole that is exactly 24 in (0.6 m) away from the orifice plate.

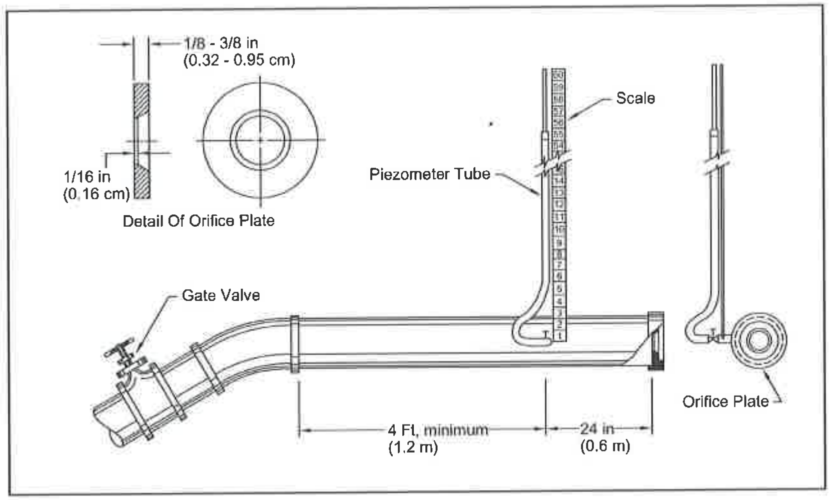

Figure 6.4. Diagram of a circular orifice weir.

A piezometer (manometer) tube or transducer is fitted to this small hole to measure the water head (pressure) in the discharge pipe. The piezometer consists of a clear plastic tube 4 ft (1.2 m) to 5 ft (1.5 m) long. One end is connected to pipe fittings that are tapped into the hole in the discharge pipe. The nipple, which is screwed into the tapped hole, must not protrude inside the pipe. A scale is fastened to a support so that the vertical distance from the center of the discharge pipe up to the water level in the piezometer tube can be measured. The water level in the piezometer tube indicates the pressure head in the approach pipe when water is being pumped through the orifice. A sensitive pressure transducer also can be used in place of the piezometer. For any given size of orifice discharge pipe, the rate of flow through the orifice varies with the pressure head as measured in this manner. Appendix 6.B (DVD) provides more information on orifice weirs.

Weirs and Flumes

Another method used to measure flow from a well is the placing of a constriction in a discharge channel originating at the well head. In most cases the flow from the pumping well can be channelized. A calibrated constriction placed in the channel changes the level of the water in or near the constriction. The rate of flow through or over the constriction is a function of the water level behind the constriction. The water-level measurement near the constriction is used to calculate the discharge, and it is important to know the dimensions of the constriction.

Two types of constricting structures—weirs and flumes—can be used to measure flows (Figure 6.5). Each type has its advantages and can provide discharge measurements to a sufficient accuracy. Selecting which type to use depends on criteria such as cost, sediment retention potential, ease of installation, site configuration, flow rates, and accuracy requirements.

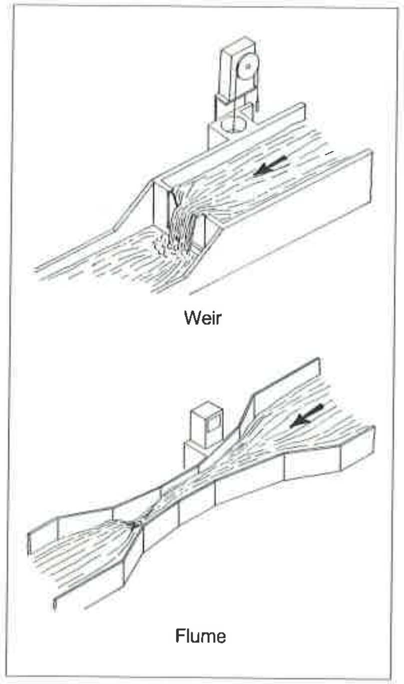

Figure 6.5. Weirs and flumes are the two primary types of constricting structures (Instrumentation Specialties Co. 1979).

A weir is a vertical baffle that restricts the total flow of water in an open or closed channel, and is a simple device to use to measure flow. The weir crest is either the bottom of the notch or the level to which water must rise before it can spill over the weir. Weir crests are constructed either including a notch (weir with end contractions) or without a notch (weir without end contractions). If the weir is contracted at the ends and bottom, then the ends of the weir should be

no closer to the sides of the channel than two times the head of the weir. Additional discussion and diagrams of flow measurements using weirs are contained in Appendix 6.C (DVD).

A flume is a device used to measure flow in open channels. It is constructed so that a restriction in the channel causes the water to accelerate, producing a corresponding change (drop) in the water level (Figure 6.5). The head then can be related to discharge. Smaller flumes are ideal for measuring discharges from wells. All flumes used for measuring discharges from wells should have the following characteristics (Grant 1979).

- The flume should be located in a straight stretch of the channel (no bends immediately upstream).
- High approach velocities should be avoided; water should be free of turbulence and waves.
- Flow is restricted in a flume, therefore the channel upstream should have banks high enough to contain the flow.
- The channel approaching the flume should be regularly shaped so that flow is well distributed in the approach channel.
- Excessive submergence of the flume throat caused by backwater downstream should be avoided because it reduces the accuracy of discharge measurements.

Several types of flumes have been developed. A common flume for measuring flows is the Parshall flume (Parshall 1950). More information regarding the Parshall flume is contained in Appendix 6.D (DVD).

Another type of flume has come into use because its construction is simpler than that of the Parshall flume, and it can operate satisfactorily under both free-flow and submerged conditions. This type of flume is called a cutthroat flume because it does not have a throat with parallel walls (*see* Figure 6.6). The cutthroat flume has a flat floor, so it can be placed on a channel bed or inside a concrete-lined channel (Skogerboe et al. 1973). This type of flume is an ideal discharge-measuring device for use by water-well professionals because of its low cost and high accuracy. Appendix 6.E (DVD) contains additional information on cutthroat flumes that are practical for measuring well yields.

Flumes offer several advantages as compared to weirs. The most important of these is the self-cleaning capacity of flumes compared with that of sharp-edged weirs. Head losses through a flume also are much less than through a weir, therefore if the available head is limited then flumes are more desirable.

Flumes can function over a wide range of discharges and still require only a single upstream head measurement. Collapsible cutthroat flumes provide a rapid and reliable means to measure flow rates and also require minimal setup time. Figure 6.6 shows a collapsible cutthroat flume installed in a stream.

Figure 6.6. A collapsible cutthroat flume (Baski, Inc.).

Measuring Drawdown in Wells

Observation Wells and Piezometers

Drawdown measurements should be made in both the pumping well and appropriately placed observation wells or piezometers (if the budget allows for the installation of observation wells). The accuracy of water-level data taken from the pumping well usually is less reliable due to turbulence created by the pump and also because of well losses. The resulting water-level drawdowns usually are not representative of drawdown in the aquifer. As such, at least one observation well should be used when practicable. Observation wells should be of sufficient diameter to allow accurate and rapid measurement of water levels. Examples include 1-in (25-mm) to 2-in (50-mm) diameter wells. Many transducers can be placed in 1-in (25-mm) casings. Small-diameter wells are best, because the volume of water contained in a large-diameter observation well can cause a time lag in drawdown changes. Using large-diameter production wells

as observation wells should be avoided, and the data collected from such wells should be evaluated with care.

The length of the screens recommended to be installed in observation wells depends on the objectives and budget of the aquifer test. Generally, fully penetrating observation wells are preferred. Partially penetrating observation wells produce measurements that are skewed by the effects of partial penetration and the vertical anisotropy of the aquifer (the ratio of vertical to horizontal hydraulic conductivity). In a stratified formation, the cone of depression becomes distorted by the spatially varying hydraulic conductivity. Distortion of the cone of depression and the effects of partial penetration extend from the pumped well a distance equal to 1 or 2 aquifer thicknesses divided by the square root of the vertical anisotropy ratio. Using fully penetrating observation wells and a pumped well that penetrates most or all of the aquifer thickness eliminates the complicating effects of partial penetration.

An exception to this is the situation where the aquifer test is intended to help quantify the vertical anisotropy ratio of the aquifer. In this case, a partially penetrating pumped well is required, along with partially penetrating observation wells—some completed in the upper portion of the aquifer and some completed in the lower portion (Schafer 1998). Schafer's 1998 paper is contained in Appendix 6.Q (DVD).

As a practical matter, in very thick aquifers fully penetrating wells might be too costly to install or could create vertical conduits that enable unwanted vertical groundwater or contaminant migration. In these cases it might be necessary to perform the investigation using partially penetrating wells. Under these circumstances, the analyst must both recognize the limits imposed on the well design and account for the effects of partial penetration.

For unconfined aquifers, observation wells and piezometers should be placed no farther than 100 ft (30.5 m) to 300 ft (91 m) from the pumped well. For thick confined aquifers that are stratified, observation wells should be placed within 300 ft (91 m) to 700 ft (214 m) of the pumped well. If the well is screened across the entire thickness of the aquifer then observation wells can be placed closer to the pumping well. Locating the wells too far away is not good practice because the aquifer test must be continued for a longer time to produce sufficient drawdowns at the most distant points, and small measurement errors could be a significant percentage of the total drawdown in the observation well. Boundary effects also might not be noticed. Boundary effects are discussed in subsequent sections of this chapter.

Screens for observation wells should be installed at about the same depth as the central portion of the screen in the production well. This is especially important if a short screen is used in the pumping well (resulting in partial penetration), because the distribution of drawdown is more distorted. If the recommended procedure is followed, then the reduction in pressure or water level at the observation well usually occurs within moments of this occurrence in the pumping well (assuming that the observation well is the proper distance from the pumped well). Observation wells occasionally are terminated in strata above or below that tapped by the pumped well, to assess the hydraulic interconnection between the geologic units. Naturally, the response of these observation wells to pumping can be delayed significantly, depending on the degree of hydraulic connection. Additional discussion on this topic can be found in Kruseman and de Ridder (1992).

The appropriate number of observation wells to be used depends upon the amount of information desired and upon the funds available for the test program. The data obtained by measuring the drawdown at a single observation well enables calculation of the average hydraulic conductivity, transmissivity, and storage coefficient of the aquifer. If two or more observation wells are placed at different distances, then the test data can be analyzed using both the time-drawdown and the distance-drawdown relationships. If possible, observation wells should be located at logarithmically spaced distances from the pumping well. This spacing arrangement is based upon analytical procedures discussed in this chapter. Use of as many observation wells as conditions and budget allow is recommended because the hydraulic conductivity can vary in one or more directions away from the pumping well. Observation wells placed radially from the pumping well should indicate directional differences in drawdown that results from differences in transmissivity.

Before beginning the aquifer test, a complete program for depth-to-water measurements should be established. It is not necessary to make the measurements in all the wells simultaneously. Watches and transducers or dataloggers that are used for timing the measurements, however, should be synchronized so that the time of each reading can be referenced to the time that pumping is started.

Previously, water levels generally were measured using a chalked tape. Although this methodology is acceptable there are other, more efficient, techniques. One of the most common tools used to measure water levels is an electric water-level sounder (or meter) (Figure 6.7). The water-level meter uses

a probe attached to a permanently marked polyethylene tape fitted on a reel, and is powered by a battery. The probe at the bottom of the tape incorporates an insulating gap between electrodes (Figure 6.8). When contact is made with water the circuit is completed, activating a buzzer or light. The water level is determined by taking a reading directly from the tape at the top of the well casing or borehole.

Figure 6.7. An electric sounder consists of an electrode, a two-wire cable, and a light which indicates a closed circuit when the electrode touches water (Solinst®).

An air line also can be used to measure water levels in a well. An air line works on the principle that the air pressure required to push all the water out of the submerged portion of the tube equals the water pressure of a water column of the same height. Additional discussion regarding air lines is contained in Appendix 6.F (DVD).

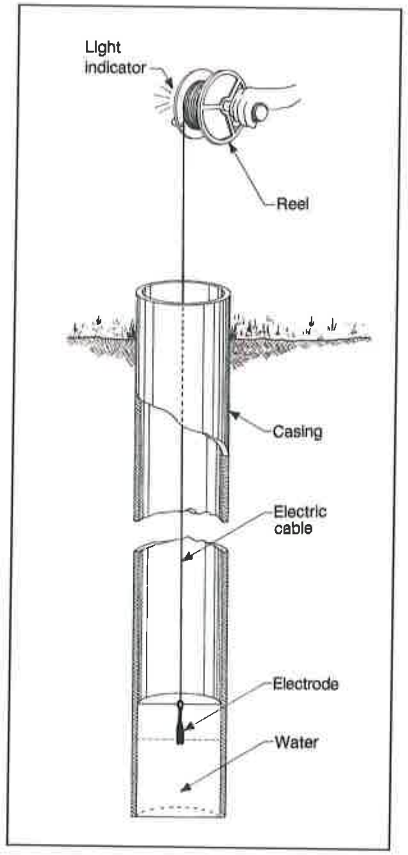

Figure 6.8. Diagram of a water-level measurement device.

Advances in microchip technology have enabled the development of pressure transducers combined with data loggers that can be placed in the pumping or observation wells. These automatic data loggers can be programmed to record water levels over a wide variety of time intervals. The water-level data then can be downloaded from the datalogger to a computer for analysis. Figure 6.9 shows a transducer, and additional information is contained in Appendix 6.G (DVD).

Figure 6.9. A transducer for measuring water levels (Solinst®).

Prior to conducting an aquifer test, background water-level trends should be recorded, at least for several days and preferably for a week. Background water-level trends then can be used to adjust the measured water levels taken during the test so that water-level changes induced by the pumping can be analyzed. It is also important to have a "background" well in which water levels can be measured before, during, and after the test to assess any trends in water levels that are unrelated to the aquifer test. The background well should be located in the same hydrostratigraphic unit as the pumping well, but be sufficiently distant so that water levels are not impacted by pumping. Water-level corrections are discussed in more detail elsewhere in this chapter.

During an aquifer test, water levels in the pumping well and observation wells must be measured multiple times and with an accuracy of at least 0.01 ft (0.3 cm). Water-level measurements should be made within brief intervals during the first few hours of the beginning of the test because water levels decline rapidly. As the test continues, the time intervals between measurements can be lengthened. Table 6.1 and Table 6.2 provide a recommended range of intervals for the pumped well and observation wells. The actual number of measurements that can be taken depends upon the number of available personnel and measuring devices. If electronic data loggers are used then readings can be made more frequently because data storage generally is not an issue. Water levels in wells that are equipped with transducers periodically should be checked manually (e.g., with a water-level meter) in case a failure of the transducer has occurred.

Table 6.1. Recommended Time Intervals for Measuring Drawdown in the Pumped Well During a Aquifer Test

Time Since Pumping Started (or Stopped)	Time Intervals Between Measurement
0–5 min	0.5–1 min
5–15 min	1 min
15–60 min	5 min
60–120 min	20 min
120 to termination of test	60 min

Table 6.2. Recommended Time Intervals for Measuring Drawdown in the Observation Wells During an Aquifer Test

Time Since Pumping Started (or Stopped)	Time Intervals Between Measurement
0–2 min	Approx. 10 sec
2–5 min	30 sec
5–15 min	1 min
15–50 min	5 min
50–100 min	10 min
100 min to 5 hr	30 min
5 hr to 48 hr	60 min
48 hr to 6 days	Every 8 hr
6 days to shutdown of pumping	Once per day
(adapted from Kruseman and de Ridder 1992)	

Where turbulence is a problem in a pumping well, a transducer provides a better means to measure drawdown, especially if the transducer is located in an access pipe that is inserted into the well.

Early test data are extremely important, and as much information as possible should be obtained in the first 10 min of pumping for every observation well that is located near the pumping well, because as the cone of depression moves outward from the well it might encounter heterogeneities which cause either acceleration or deceleration of drawdown with increasing time. Any unusual event (i.e., the pump stopping, the weather changing, a train passing) should be noted on a water-level measurement form, along with the time the event occurred. An example of an aquifer-test form is contained in Appendix 6.H (DVD).

Ideally, aquifer tests should be continued until equilibrium is reached; that is, until the cone of depression stabilizes. In practice, however, this rarely is possible. In confined aquifers, the cone of depression spreads rapidly because dewatering (drainage) does not take place, only a pressure reduction is occurring outward from the well. Thus, a maximum of 24 hr usually is sufficient to run an aquifer test in a confined aquifer. To obtain sufficient information for unconfined aquifers, 72 hr usually is required to dewater the materials within the cone of depression because of the slow downward percolation of water in many stratified deposits. This process is termed "delayed yield." The duration of the test can be reduced if equilibrium conditions are established before 24 hr (for a confined aquifer) or 72 hr (for an unconfined aquifer) have elapsed.

It is recommended that preliminary drawdown data be plotted during the course of the aquifer test. Anomalies in the data should become apparent and necessary adjustments can be made so that the subsequent data are more useful. Plotting the data also indicates when equilibrium conditions have been reached. In this case, the pumping portion of the test can be shortened without losing necessary data.

Barometric change, tidal change, and surface loadings can influence drawdown data, especially in a confined aquifer. A barometric pressure change of 1 in (25.4 mm) of mercury, for example, can result in a rise or fall of up to 1 ft (0.3 m) in the potentiometric surface for confined aquifers that have high barometric efficiency. Barometric efficiency refers to the difficulty with which changes in atmospheric pressure are delivered through the overburden and aquifer materials to the pore space in the aquifer. For example, 100% barometric efficiency means that changes in atmospheric pressure cause no change in the pore-water pressure, and a barometric efficiency of 0 means that any change in the barometric pressure causes an equivalent change in the pore-water pressure within the aquifer. Given this potential influence, it is recommended that the nature and time of any weather changes be recorded on the water-level measurement form. Unusually high or low oceanic tides also can affect drawdown data in wells located near coastlines. Appendix 6.I (DVD) provides more information regarding corrections to water-level data.

Recovery Data

Recovery water-level data should be collected to verify the accuracy of water levels collected during the extraction phase of the test. Often the recovery water-level data from a pumping well are more reliable than pumping data, because turbulence induced in the pumping well is eliminated. Recovery water-level measurements should be recorded with the same frequency as measurements taken during the pumping portion of the aquifer test.

AQUIFER-TEST ANALYSIS

Steady-State Flow in Confined and Unconfined Aquifers

Thiem Equation

In a confined aquifer, the rate of discharge by a pumping well under steady-state (time-invariant) conditions is predicted by the Thiem (1906) equation (equation 6.1). Note that the equations can be solved using consistent units for length (L), time (t), head (length (L)).

$$Q = \frac{2\pi T(h_2 - h_1)}{ln(r_2 / r_1)} = \frac{2\pi Kb(h_2 - h_1)}{ln(r_2 / r_1)} \quad \textbf{(6.1)}$$

Where:

- Q = pumping rate (L^3/t);
- T = transmissivity (L^2/t);
- K = hydraulic conductivity (L/t);
- b = aquifer thickness (L);
- r_1, r_2 = radial distances of two observation wells from the discharging well (L); and
- h_1, h_2 = steady-state water-level elevations (heads) in the two observation wells (L).

As noted on Figure 6.10, the heads are measured from the base of the aquifer in this example. The function *ln* in equation 6.1 is the natural logarithm. Figure 6.10 shows the various parameters listed in equation 6.1. Note that the pumping well can be used as an observation well; however, due to well losses, the drawdown probably is greater than that in the adjacent aquifer. Figure 6.10 shows drawdown in a well that is 100% efficient.

The Thiem equation is based on the following important assumptions.

- The aquifer is confined with infinite areal extent.
- The aquifer is homogeneous and isotropic, with uniform thickness.
- The potentiometric surface is horizontal prior to pumping.
- The pumped well is fully penetrating (screened or open over the full thickness of the aquifer) and discharges at a constant rate.
- Flow in the aquifer is horizontal and laminar.
- Drawdown in the aquifer has reached a steady state (is constant over time).

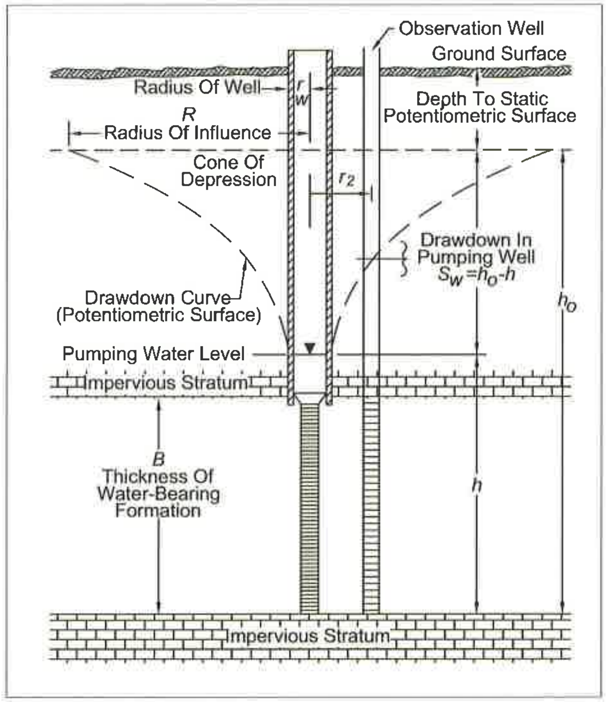

Figure 6.10. Diagram showing parameters for the Thiem equation.

The Thiem equation shows that the yield of a well is directly proportional to the transmissivity of an aquifer. An increase in T by a factor of 2 results in a doubling of the discharge rate.

The Thiem equation can be rewritten in terms of drawdown as follows.

$$Q = \frac{2\pi T(s_1 - s_2)}{\ln(r_2 / r_1)} \tag{6.2}$$

Where s_1 and s_2 are steady-state drawdowns (L) measured in two observation wells.

Drawdown is given by the following.

$$s = h_0 - h \tag{6.3}$$

Where:

h_0 = head prior to pumping (L); and

h = steady-state head (L).

If drawdown data only are available from an observation well and the pumped well, then the Thiem equation is expressed as shown in equation 6.4.

$$Q = \frac{2\pi T(s_w - s_2)}{\ln(r_2 / r_w)} \tag{6.4}$$

Where:

s_w = drawdown in the pumped well (L); and

r_w = radius of the pumped well (L).

Equation 6.4 assumes that the pumped well is 100% efficient; therefore well losses might limit the use of this equation.

In an unconfined aquifer, a modified version of the Thiem equation can be used by assuming Dupuit (1863) conditions.

$$Q = \frac{2\pi T(s'_1 - s'_2)}{\ln(r_2 / r_1)} \tag{6.5}$$

The Dupuit (or Dupuit-Forchheimer) theory assumes that groundwater flow is horizontal with no vertical flow resistance, and that the flow velocity is proportional to the slope of the water table. The theory further assumes that the slope of the water table is very small and therefore drawdown due to pumping also is very small relative to aquifer thickness.

Rewritten in terms of drawdown in an unconfined aquifer, equation 6.5 is rendered as shown below.

$$Q = \frac{2\pi T(s'_1 - s'_2)}{\ln(r_2 / r_1)} \tag{6.6}$$

Where the corrected drawdown is given by the following.

$$s' = s - s^2 / 2b \tag{6.7}$$

The foregoing equations can be used to compute the discharge rate of a well when the hydraulic conductivity (K) and thickness (b) of an aquifer are known in conjunction with steady-state water levels in two observation wells. Alternatively, by pumping a well at a constant discharge rate (Q), K of a confined aquifer can be determined by rearranging equation 6.1.

$$K = \frac{Q \ln(r_2 / r_1)}{2\pi b(h_2 - h_1)} \tag{6.8}$$

Rearranging equation 6.2 enables the determination of K from drawdown measurements.

$$K = \frac{Q \ln(r_2 / r_1)}{2\pi b(s_1 - s_2)} \tag{6.9}$$

Similarly, rearranging equation 6.5 enables the determination of K in an unconfined aquifer as follows.

$$K = \frac{Q \ln(r_2 / r_1)}{\pi (h_2^2 - h_1^2)} \tag{6.10}$$

The steady-state assumption appears to be a severe restriction on the use of the Thiem equation; however Butler (1988) has shown that the Thiem equation can be applicable to late-time data from an aquifer test even when heads in individual observation wells are declining with time. After a sufficient period of pumping has elapsed, the shape of a pumping cone of depression reaches a pseudo-steady state; consequently, the Thiem equation still can be used to determine the hydraulic conductivity of an aquifer from water levels measured in observation wells.

The example below, expressed in U.S. customary units, demonstrates how equation 6.9 can be used. The example is based on data for a confined aquifer 150-ft thick, pumped at a constant rate of 200 gpm (38,500 ft^3/day) for a sufficiently long period such that drawdown is not changing appreciably with time. The drawdown in the pumped well (r = 0.5 ft) is 25 ft; in an observation well located 200 ft away, the drawdown is 2.5 ft. Equation 6.9 can be used to compute the hydraulic conductivity of the aquifer as shown below.

$$K = \frac{Q\ln(r_2/r_1)}{2\pi b(s_1 - s_2)} = \frac{(38{,}500)\ln(200/0.5)}{2\pi(150)(25-2.5)} = 10.9 \text{ ft/day}$$

In the United States, values of hydraulic conductivity are expressed as feet per day (ft/day) or gallons per day per foot squared (gpd/ft^2). To convert ft^2/day to gpd/ft, multiply the value in ft^2/day by 7.48 gal/ft^3. Likewise, transmissivity can be expressed as ft^2/day or gpd/ft. In this chapter transmissivity is expressed using both units.

Transient Flow in Confined Aquifers

Theis Method

Applying principles from the study of heat conduction, Theis (1935) was the first to derive equations that account for unsteady (transient) drawdown around a pumping well. The introduction of the Theis equation was a major advance in the study of well hydraulics. Countless studies have relied on this method for the evaluation of groundwater resources. The original work by Theis also spawned subsequent research into more-complex well configurations and aquifer geometries. An excellent reference that discusses these more-complex situations is Kruseman and De Ridder (1992).

The following assumptions apply to the Theis equation.

- The aquifer is confined with infinite areal extent and no vertical leakage or recharge.
- The aquifer is homogeneous and isotropic with uniform thickness.
- The potentiometric surface prior to pumping is horizontal.
- The pumped well has an insignificantly small radius, fully penetrates the aquifer (screened or open over the full thickness of the aquifer), and discharges at a constant rate.
- Flow in the aquifer is horizontal and laminar.
- Drawdown changes with time.

The Theis equation for flow to a well in a confined aquifer is as follows.

$$s = \frac{Q}{4\pi T}\int_u^{\infty}\frac{e^{-x}}{x}dx \tag{6.11}$$

Where:

$u = r^2S/4Tt$;
x = a dummy variable of integration;
Q = pumping rate (L^3/t);
T = transmissivity (L^2/t);
S = storativity (- (dimensionless));
s = drawdown at observation well (L);
r = radial distance between pumping and observation wells (L); and
t = time.

Note that any set of consistent units can be used with equation 6.11. The integral in equation 6.11 commonly is known as the Theis well function, and is designated by $W(u)$. The following infinite series can be used to compute $W(u)$.

$$W(u) = \int_u^{\infty}\frac{e^{-x}}{x}dx = -0.5772 - \ln u + u - \frac{u^2}{2\cdot 2!} + \frac{u^3}{3\cdot 3!} - \frac{u^4}{4\cdot 4!} + \ldots \tag{6.12}$$

Therefore equation 6.11 can be written in the following compact form.

$$s = \frac{Q}{4\pi T}W(u) \tag{6.13}$$

An extensive table of values for the Theis well function for selected values of u is provided in Appendix 6.J. (DVD). It is recommended that water-well professionals have a copy of this table available for use in the field.

When the aquifer coefficients T and S are known, the Theis equation can be used to compute drawdown in a confined aquifer for a given Q, r, and t.

Example

A confined aquifer has a transmissivity of 50,000 ft^2/day (or 374,000 gpd/ft) (in U.S. customary units) and storativity of $5 \cdot 10^{-4}$. A pumping well that fully penetrates the aquifer discharges at a constant rate of 100 gpm (545 m^3/day). To compute the theoretical drawdown in an observation well located 100 ft

(33 m) from the pumped well after 1 day of steady pumping, first compute the value of u as follows.

$$u = \frac{r^2 S}{4Tt} = \frac{(100)^2(0.0005)}{4(50{,}000)(1)} = 2.5x10^{-5}$$

The table for the Theis well function provided in Appendix 6.J (DVD), shows that, for this value of u, the corresponding value of $W(u)$ is 10.0194. Before computing drawdown using equation 6.13, the pumping rate first must be converted to consistent units. In this case, consistent units are feet and days.

$$(100 \text{ gpm})(1440 \text{ min/day})(\text{ft}^3/7.481 \text{ gal}) \approx 19{,}250 \text{ ft}^3/\text{day}$$

The drawdown in the observation well then can be computed as follows.

$$s = \frac{Q}{4\pi T}W(u) = \frac{(19{,}250)}{4\pi(50{,}000)}(10.0194) = 0.31\,ft$$

Thus, for the given aquifer properties, drawdown in the observation well is predicted to be 0.31 ft after one day of pumping.

When the aquifer coefficients, T and S, are known, equation 6.13 can be used to solve for drawdown directly. When the aquifer properties are not known it is possible to determine them by conducting an aquifer test. To estimate unknown T and S values from measured values of s and Q, the Theis equation can be used to find a solution to what is known as the "inverse problem." Theis devised a graphical method based on type-curve matching to determine T and S from field data. More recently, computer software based on optimization techniques (sometimes referred to as automatic curve matching) can be employed to estimate T and S. Programs such as AQTESOLV™ (software created by HydroSOLVE, Inc. © 2006) combine traditional graphical curve matching with automatic curve matching to estimate aquifer properties from aquifer-test data.

To apply the graphical method of determining T and S devised by Theis, perform the following steps using drawdown data from an aquifer test measured in an observation well located 400 ft (121.9 m) from the pumped well (Table 6.3). The pumped well discharged at a constant rate of 500 gpm (2,725 m³/day).

Table 6.3. Drawdown Measurements in an Observation Well 400 ft (121.9 m) from a Pumped Well

Time Since Pump Started	Drawdown (s)	
min	ft	m
1	0.16	0.05
1.5	0.27	0.08
2	0.38	0.12
2.5	0.46	0.14
3	0.53	0.16
4	0.67	0.20
5	0.77	0.23
6	0.87	0.27
8	0.99	0.30
10	1.12	0.34
12	1.21	0.37
14	1.30	0.40
18	1.43	0.44

Time Since Pump Started	Drawdown (s)	
min	ft	m
24	1.58	0.48
30	1.70	0.52
40	1.88	0.57
50	2.00	0.61
60	2.11	0.64
80	2.24	0.68
100	2.38	0.73
120	2.49	0.76
150	2.62	0.80
180	2.72	0.83
210	2.81	0.86
240	2.88	0.88

A "type curve" with values of $W(u)$ plotted as a function of $1/u$ on double logarithmic (log-log) graph paper (Figure 6.11) is used in this analysis.

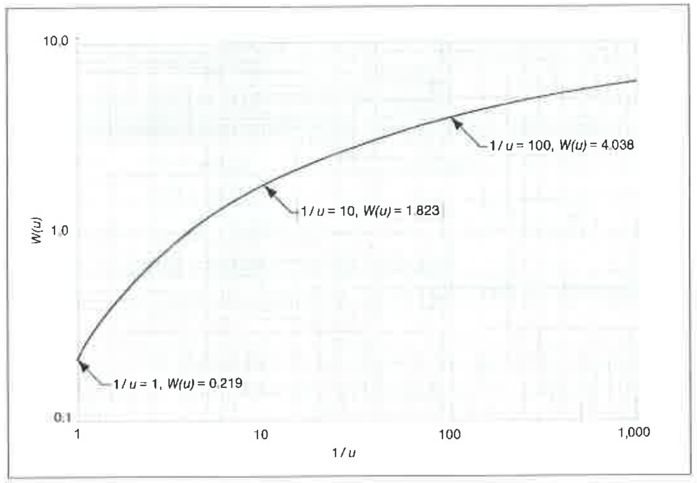

Figure 6.11. Type curve for graphic solution of Theis nonequilibrium equation shows values of *W(u)* well function of *u*, corresponding to values of 1/*u*. Curve is plotted on logarithmic graph.

Using log-log graph paper of the same scale as the type curve, prepare a plot of drawdown measured in an observation well as a function of elapsed time since the start of pumping (Figure 6.12).

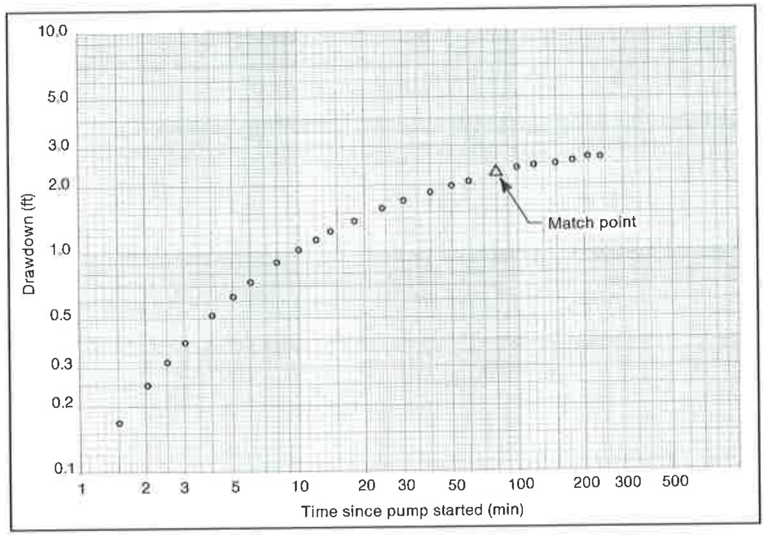

Figure 6.12. Plot of data provided in Table 6.3, on logarithmic graph.

With the graph of the data placed over the type curve, adjust the position of the data to match the type curve while simultaneously ensuring that the axes of both plots remain parallel (Figure 6.13). After achieving a satisfactory match between the data and the type curve, select an arbitrary match point where the two plots overlap (Figure 6.13). The match point can be any convenient point on the graph (that is, where u equals 100 or s equals a whole number). Often, the point is selected in the center of the area of best overlap, as shown in Figure 6.13.

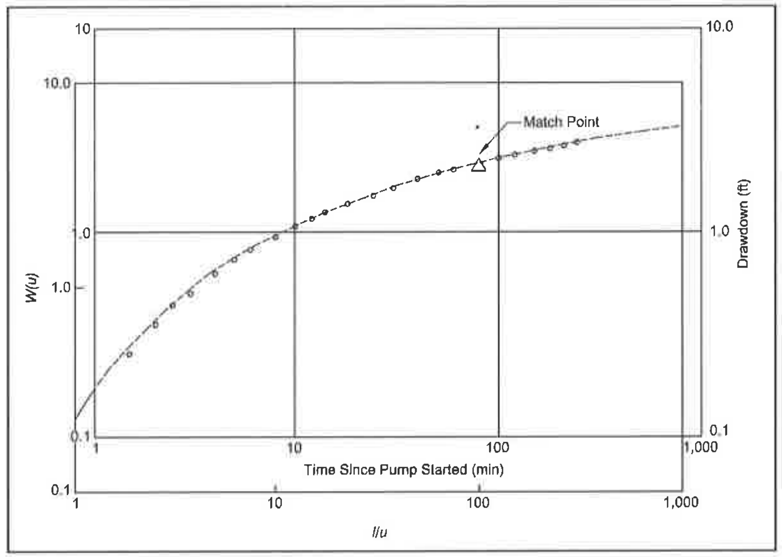

Figure 6.13. Diagram of plotted point representing aquifer-test data superimposed on the type curve. Match point chosen for 1/*u* = 100.

The match point values (the asterisk (*) is the value at the match point) from the field data plot are $t^* = 83$ min and $s^* = 2.3$ ft. On the type curve plot the corresponding match point values are $W(u)^* = 4.038$ and $1/u^* = 100$ (Figure 6.13). Use the match point values (t^*, s^*, $W(u)^*$, and $1/u^*$) to solve for T and S. The flow rate, Q, is 500 gpm or 96,250 ft^3/day.

$$S = \frac{4Tt^*u^*}{r^2} = \frac{4(13{,}450)(0.0576)(0.01)}{(400)^2} = 1.94 \cdot 10^{-4}\,\text{lay}$$

Note that these equations for determining T and S assume consistent units for all values.

It is evident from equation 6.13 that s is directly proportional to Q for constant t, r, T, and S. Thus, when s is known for a given Q, drawdown at any Q can be calculated. If Q is doubled, for example, then drawdown also is doubled.

Cooper-Jacob Method

Cooper and Jacob (1946) noted that when the value of u in the Theis well function is small, equation 6.12 can be approximated as follows.

$$W(u) \approx -0.5772 - \log u \tag{6.14}$$

Using this approximation for $W(u)$, equation 6.11 can be reformulated as follows.

$$s = \frac{2.303Q}{4\pi T} log \frac{2.25Tt}{r^2 S} \tag{6.15}$$

The symbols represent the same quantities as given in equation 6.11. Note that the log function in the above equation is the common or decimal logarithm.

Equation 6.15 becomes valid when time (t) is sufficiently large and distance to the observation well (or radius of pumping well) (r) is sufficiently small. Different criteria have been proposed for what constitutes a sufficiently small value of u. According to Kruseman and De Ridder (1992), the error resulting from equation 6.14 is less than 2% when values of u are less than 0.05.

From equation 6.15, it is apparent that drawdown varies linearly as a function of log t or log t/r^2 (Figure 6.14). Therefore a plot of s versus t, or s versus t/r^2, on semilogarithmic (semilog) graph paper forms a straight line when the value of u is small. In most instances, it far easier to match a straight line to field data than to match a type curve; consequently, fitting a straight line with the Cooper-Jacob method generally is preferable to matching the Theis-type curve.

To use the Cooper-Jacob method, first use semilog graph paper to plot values of time horizontally on the logarithmic axis and values of drawdown vertically on the linear (arithmetic) axis; then identify late-time data, forming a straight line on the plot, and draw a line over it.

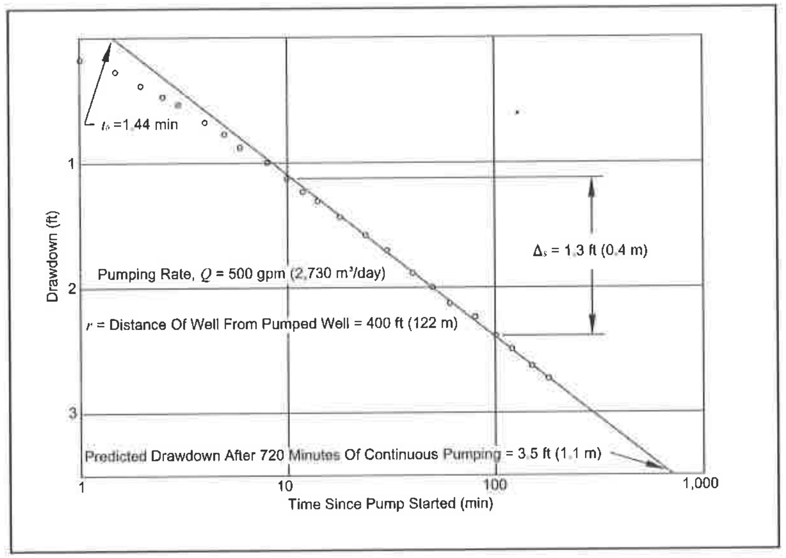

Figure 6.14. Plot of data from Table 6.3 on semilogarithmic graph.

Except for the data for the first 10 min of the test, most of the data fall on a straight line. The transmissivity of the aquifer is determined using the following equation.

$$T = \frac{2.303Q}{4\pi \, \Delta s} \tag{6.16}$$

Where:

T = transmissivity (L^2/t);

Q = pumping rate (L^3/t); and

Δs = the change in drawdown given by the straight line over one log-cycle time (L).

In the example, Δs is 1.3 ft between 10 min and 100 min. Given a pumping rate of 500 gpm (96,244 ft³/day), transmissivity is computed as follows.

$$T = \frac{2.303(96{,}244)}{4\pi(1.3)} = 13{,}570 \, ft^2 / day$$

The result compares favorably with the earlier estimate of T from the Theis type-curve analysis.

Compute the storativity of the aquifer using the following formula.

$$S = \frac{2.25Tt_0}{r^2} \tag{6.17}$$

Where:

T = transmissivity (L^2/t);

t_0 = the intercept of the straight line on the x-axis (time); and

r = radial distance from pumped well to observation well (L).

In the example, t_0 is 1.44 min or 0.001 day. Given $r = 400$ ft, the storativity is computed as follows.

$$S = \frac{2.25(13{,}570)(0.001)}{(400)^2} = 1.91 \cdot 10^{-4}$$

The above estimate of S agrees with the Theis curve matching result.

The Cooper-Jacob equation shows that s is directly proportional to Q assuming constant values of t, r, T, and S. Once s is known for a given Q, the calculation to predict drawdowns for different pumping rates is straightforward.

The Cooper-Jacob equation also can be used to make quick predictions of drawdown in an aquifer, given estimates of T and S. To do so, first extend the straight line on a semilog plot to extrapolate drawdowns beyond the time period of a aquifer test (Figure 6.14). Next, use equation 6.15 to compute s for any value of t/r^2 given values of T, S, and Q. Using the previous example, the drawdown at 5 days is predicted as indicated below.

$$s = \frac{2.303(96{,}244)}{4\pi(13{,}570)} \log \frac{2.25(13{,}570)(5)}{(400)^2(1.91 \cdot 10^{-4})} = 4.81 \text{ ft}$$

Predictions of this sort assume no recharge to—or discharge from—the aquifer (e.g., precipitation, boundaries, wells, or vertical leakage).

Distance-Drawdown Analysis

The Cooper-Jacob (1946) relationship shows that drawdown (s) varies with the log t/r^2 or with the log of distance. Using this relationship, a semilog distance-drawdown graph can be constructed. Simultaneous drawdown measurements in at least 3 observation wells, each at a different distance from the pumped well, are needed to construct a distance-drawdown graph (Figure 6.15). If drawdowns in the same 3 wells are plotted on a semilog diagram, in theory the drawdown curve becomes a straight line, as shown in Figure 6.16. Points representing drawdown in other observation wells farther away from the pumped well fall a little below the straight line in Figure 6.16, because at some distance from the pumped well u is greater than 0.05. If u is greater than 0.05, then the straight-line relationship of s to log r no longer holds.

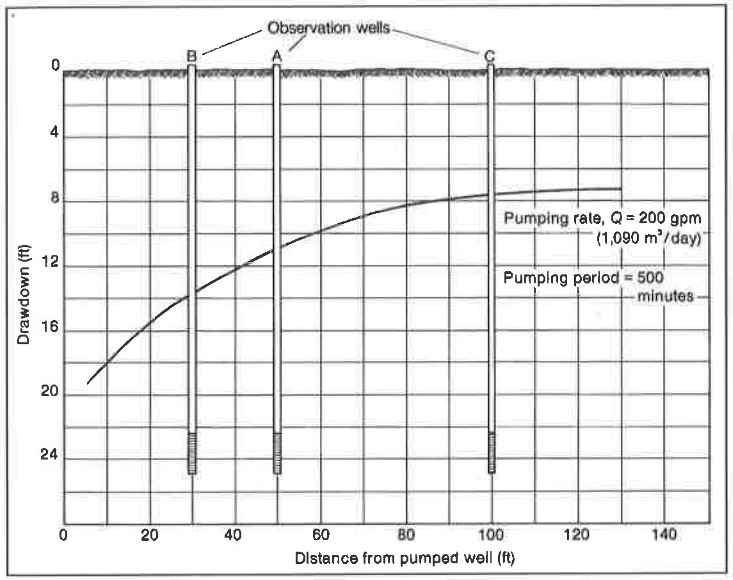

Figure 6.15. Plotting drawdowns for 3 observation wells defines part of the cone of depression.

The semilog plot of the cone of depression (distance-drawdown diagram) simplifies application of the distance-drawdown relationship. The straight line can be extended to the left, to the right, or to intermediate points between the observation wells to determine the effect of pumping at any distance from the pumped well. As noted, however, the u value must remain less than 0.05. At sufficiently great distances from the pumped well, u is greater than 0.05, and the extrapolated drawdown values are invalid when using the semilog approach. For a more general extrapolation, it is necessary to plot drawdown versus the reciprocal of the distance squared on a log-log graph and perform Theis curve matching. Drawdown at any distance from the pumped well then can be extrapolated along the fitted Theis type curve.

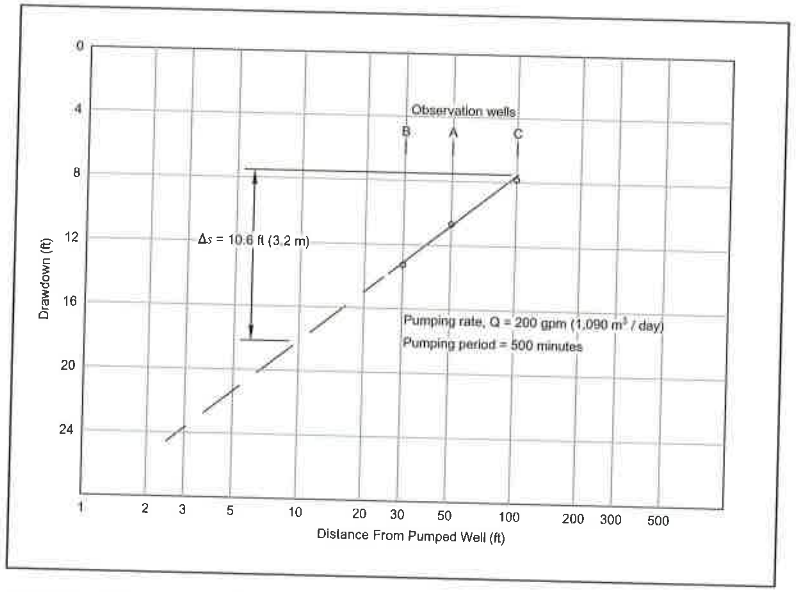

Figure 6.16. Trace of the cone of depression plotted on semilogarithmic graph paper becomes a straight line. Drawdown in each observation well was measured 500 minutes after start of the aquifer test.

Transmissivity

An arrangement of equation 6.16 enables calculation of transmissivity from the distance-drawdown diagram. The slope of the straight line is used in a manner similar to the procedure for use of the time-drawdown diagram. The equation for this method is expressed in U.S. customary units in equation 6.18, and in SI units in equation 6.19. Note that in U.S. customary units transmissivity can be reported in ft^2/day or in gallons per day per foot (gpd/ft).

$$T = \frac{528Q}{\Delta s} \qquad \textbf{(6.18)}$$

Where:

T = transmissivity (gpd/ft);

Q = pumping rate (gpm); and

Δs = slope of distance-drawdown graph expressed as the change in drawdown in feet, between any two values of distance on the log scale with a ratio of 10.

In SI units the equation is as follows.

$$T = \frac{0.366Q}{\Delta s} \qquad \textbf{(6.19)}$$

Where:

T = transmissivity (m^2/day);

Q = pumping rate (m^3/day); and

s = slope of distance drawdown graph expressed as the change in drawdown in meters, between any two values of distance on the log scale with a ratio of 10.

For the example shown in Figure 6.16 the results are as follows.

$$T = \frac{528 \cdot 200}{10.6} = 9{,}960 gpd / ft \qquad T = \frac{0.366 \cdot 1090}{3.2} = 125 m^2 / d$$

Storage Coefficient

The storage coefficient can be obtained from the distance-drawdown diagram by using the following equations. In U.S. customary units the equation is rendered as follows.

$$S = \frac{0.3Tt}{r_o^2} \qquad \textbf{(6.20)}$$

Where:

S = coefficient of storage;
T = transmissivity (gpd/ft);
t = time elapsed since pumping started (days); and
r_0 = intercept of extended straight line at zero drawdown (ft).

In SI units, the equation is expressed as follows.

$$S = \frac{2.25Tt}{r_o^2} \tag{6.21}$$

Where:

S = coefficient of storage;
T = transmissivity (m²/day);
t = time elapsed since pumping started (days); and
r_0 = intercept of extended straight line at zero drawdown (m).

From Figure 6.16, the value of r_0 is 500 ft (152 m), T is 9,960 gpd/ft (124 m^2/day), and t is 500 min (0.347 day). Results are shown below, in both U.S. customary and SI units.

$$S = \frac{0.3 \cdot 9{,}960 \cdot 0.347}{(500)^2} = 4.1 \cdot 10^{-3}$$ U.S. customary units

$$S = \frac{2.25 \cdot 124 \cdot 0.347}{(152)^2} = 4.2 \cdot 10^{-3}$$ SI units

Thus the aquifer storage coefficients can be calculated using data from the following two relationships obtained via an aquifer test.

- Rate of lowering of the water level at any place within the cone of depression on the time-drawdown diagram
- Shape and position of the cone of depression at any given time on the distance-drawdown diagram

These calculations are independent of each other, so the result from one can be used to check the result of the other.

Hydrogeologic Boundaries and Images

The discussion of the Theis equation and its Jacob modification is based upon the assumptions that the aquifer is uniform in thickness, isotropic, homogeneous, and of infinite areal extent, and that no recharge occurs. A basic understanding of geology indicates that these assumptions seldom are applicable in the field; however it is up to the analyst to determine when the assumptions are valid in a particular case. This section presents basic understandings of boundaries and image theory. Bear (1979) and Kresic (1997) provide a more in-depth discussion on this topic.

Recharge

The assumption that an aquifer receives no recharge during the pumping period is one of the fundamental conditions upon which the nonequilibrium formulas are based. Therefore, all water discharged from a well is assumed to be taken from storage within the aquifer. This situation must occur because, as pumping continues, drawdown increases and the cone of depression expands. This fundamental concept makes it possible to calculate the transmissivity from the time-drawdown data, employing the Cooper-Jacob modification of the Theis equation. Assuming no recharge during pumping also permits extension of the time-drawdown curve at its initial slope to predict drawdowns at future times. The time-drawdown curves presented in this section represent well performance during periods of no recharge. If an aquifer test is performed while recharge reaches the aquifer, the time-drawdown graph from the aquifer test reflects the recharge.

Figure 6.17 shows a time-drawdown graph for a pumped well operating under conditions of no recharge. The well was pumped at a constant rate of 350 gpm (1,910 m^3/day) and the drawdown measurements were obtained at various intervals during 360 min of pumping. The points plotted on semi-logarithmic paper define a straight line with a slope, or Δs value, of 9.3 ft (2.8 m) per log cycle of time. As shown previously, the future drawdown in this well for any period of continuous pumping at 350 gpm can be estimated by extending the straight line. The drawdown corresponding to 5,000 min of continuous pumping is 73.3 ft (22.3 m) in this case.

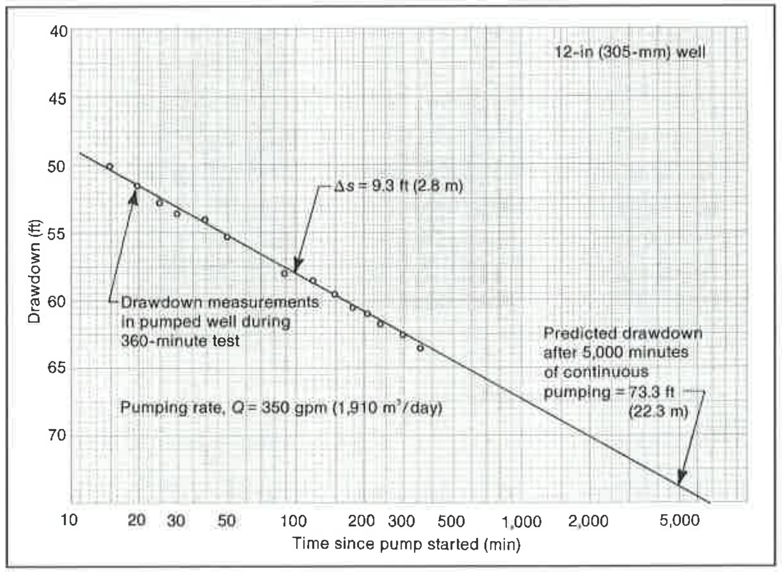

Figure 6.17. Time-drawdown graph for a pumped well (no recharge to aquifer) can be extended to predict drawdown for a period of continuous pumping longer than the test itself.

This method enables easy determination of the anticipated pumping level and the pump setting required to provide adequate submergence for the pump bowls. A safety factor can be applied to the calculated pumping level to offset changes in well performance resulting from incrustation or from interference effects if other wells later are constructed nearby.

Continuous pumping means operating 24 hr per day, with no opportunity for recovery of the water level. A well that is pumped for only part of each day does not show the same cumulative drawdown after 7, 30, or 90 days as would be predicted by a time-drawdown graph such as Figure 6.17. A well that is pumped on a cycle of 12 hr on and 12 hr off benefits from recovery of the water level during the 12-hr idle period.

If insufficient recharge takes place while the pump is off, then the water level does not fully recover to the original static level. Each time pumping is resumed drawdown starts from a new level, which is slightly below the level at the start of the previous pumping period.

Drawdown stabilizes when recharge within the zone of influence of the pumping well equals the rate of discharge of the well. No further lowering of

the water levels will occur as pumping continues at a constant rate. The time-drawdown curve then becomes horizontal, as shown in Figure 6.18.

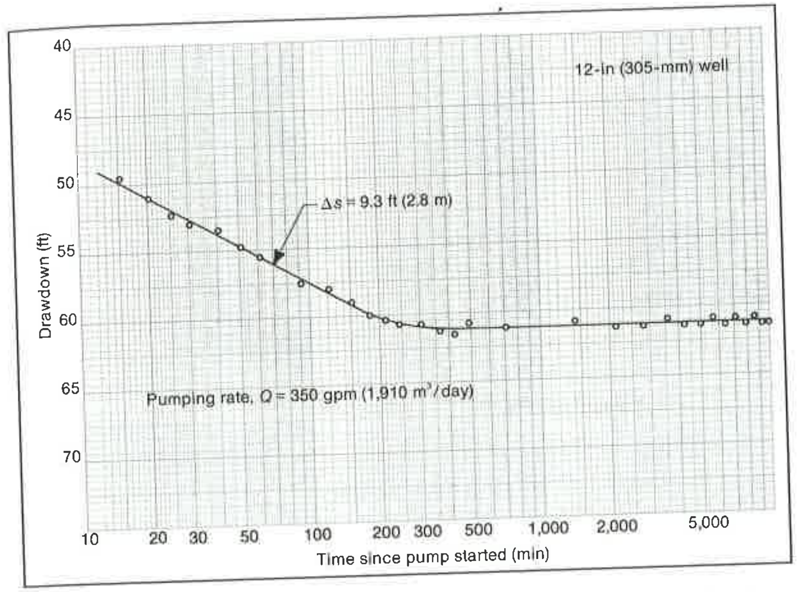

Figure 6.18. When recharge to the aquifer occurs within the zone of influence of the well, the slope of the time-drawdown curve flattens. The horizontal leg indicates that recharge equals well discharge after 240 min of pumping.

The first part of the curve illustrated in Figure 6.18 shows that the cone of depression was enlarging during the first 240 min of pumping. After 240 min, the cone of depression (or area of influence of the well) encountered a source of recharge. In the second part of the curve, the rate of recharge within the area of influence was sufficient to equal the rate of pumping, resulting in stabilized water levels throughout the area of influence. Recharge generally occurs gradually; it does not occur instantaneously. Recharge could be from a lake or river located in only a part of the cone of depression. After the recharge boundary is encountered, the drawdown increases slowly in areas away from the recharge source until equilibrium is established.

Sometimes the recharge rate within the cone of depression is lower than the pumping rate of the well. Although this changes the slope of time-drawdown curve, the second part might not become horizontal. Thus, the slope becomes flatter than the initial slope and indicates that the cone of depression is enlarging more slowly than during the first part of the pumping period (Figure 6.19). Future drawdown in a well can be predicted by extending the straight line of the second leg of the curve and reading the drawdown indicated for a particular time in the future.

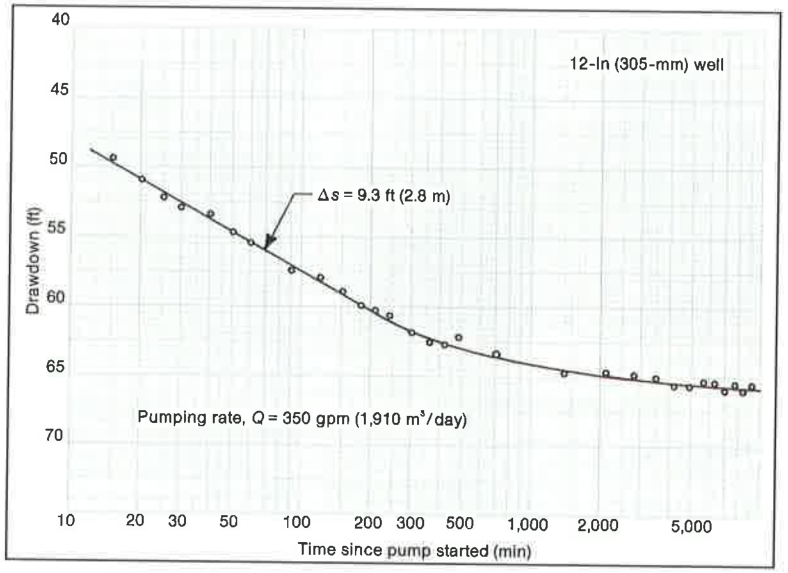

Figure 6.19. Recharge rate is somewhat less than the pump discharge; thus, the second part of the drawdown curve does not become horizontal.

The calculation of transmissivity (T) of a water-bearing formation should be made from Δs that is based on the first part of the time-drawdown curve. A value for transmissivity based upon the second slope of the graph is not used in analyzing the aqufier test data because recharge is occurring. If recharge is not occurring then the change in slope could indicate a physical change in the aquifer. The aquifer might thicken in one or more directions away from the well, for example, causing reduction in the slope of the time-drawdown curve. The hydraulic conductivity of the aquifer has not changed, only the volume of water available for withdrawal. Thus, the second Δs does not represent the true hydraulic conductivity of the primary aquifer sediments; it represents a geologic boundary within the radius of influence. Similarly, if a zone of higher hydraulic conductivity is encountered within the cone of depression, the time-drawdown curve flattens somewhat. Considerable variation in aquifer thickness and hydraulic conductivity

violates two of the assumptions inherent in the Theis methodology. The time-drawdown curve reflects this departure from idealized conditions.

Recharge from a River

Equilibrium conditions that stabilize the cone of depression around a pumping well could develop in several general situations. One of these is when an aquifer is recharged from a river or lake. Figure 6.20 illustrates this situation after equilibrium has been reached.

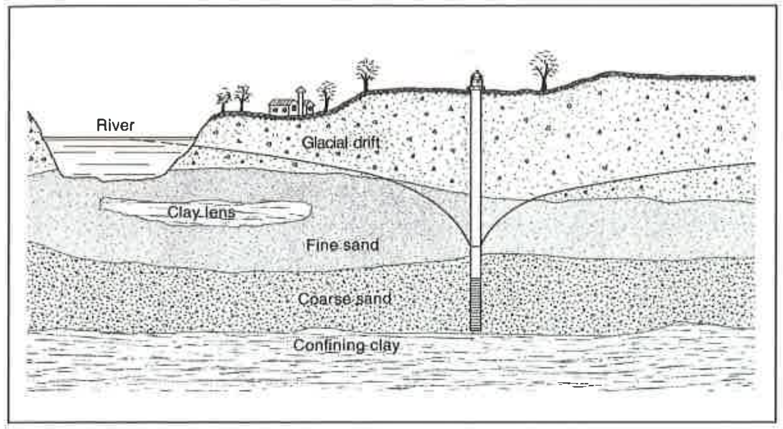

Figure 6.20. Cone of depression expanding beneath a riverbed creates a hydraulic gradient between the aquifer and river, and can result in induced recharge to the aquifer from the river.

During the early part of the pumping period, the cone of depression does not extend to the river and no recharge is evident. The pumping level in the well goes down as pumping continues, which is shown by the first part of the drawdown curve in Figure 6.18. When the cone of depression intersects a river channel, a hydraulic gradient develops between the groundwater in the aquifer and the water in the river. If the streambed is hydraulically connected with the aquifer, then river water percolates downward through the previous streambed under the influence of the hydraulic gradient. Thus, the river recharges the aquifer at an increasing rate as the cone of depression enlarges. When the rate of recharge to the aquifer equals the rate of discharge from the well, the cone of depression and the pumping level become stable. This condition corresponds to the horizontal part for the drawdown curve in Figure 6.18, and the situation shown in Figure 6.20.

Extension of the drawdown curve in Figure 6.18 shows that the predicted drawdown after 5,000 min of continuous pumping is 61.2 ft (18.7 m). Recharge to the aquifer in this case reduces the drawdown 12.1 ft (3.7 m) from the values shown in Figure 6.17 after the same period of continuous pumping.

Image Wells

When an aquifer has finite dimensions, analysis of the data by the Theis or Cooper-Jacob methodologies is not possible; however, this difficulty can be addressed through images. Imaginary wells or streams sometimes can be used at various locations to duplicate the hydraulic effects on the flow system caused by the physical boundary. Use of the image essentially is equivalent to substituting the physical boundary with a hydraulic one. The following descriptions provide an insight into how images are used.

An aquifer boundary formed by a low-permeability barrier such as a fault or a low-permeability geologic material (e.g., unfractured granite) that prevents the flow of water is termed a low-flow or no-flow boundary. Conversely, a geologic or hydrologic feature that allows for recharge to the aquifer (in the case of a surface-water body) or a highly transmissive zone is a recharge boundary or a line source. Although most geologic boundaries do not occur as abrupt discontinuities, it is possible to treat them as such (Ferris et al. 1962). When possible, it is convenient for the purpose of analysis to substitute a hypothetical image system for the boundary conditions of the real system.

If during an aquifer test the water levels in the aquifer are controlled by a perennial stream, then the physical situation can be represented as shown in Figure 6.21(A) If the stream stage is not lowered by the flow to the extraction well, then there is established a boundary condition and no drawdown along the stream. For most field situations it can be assumed that the stream is fully penetrating the aquifer and is equivalent to a line source at a constant head.

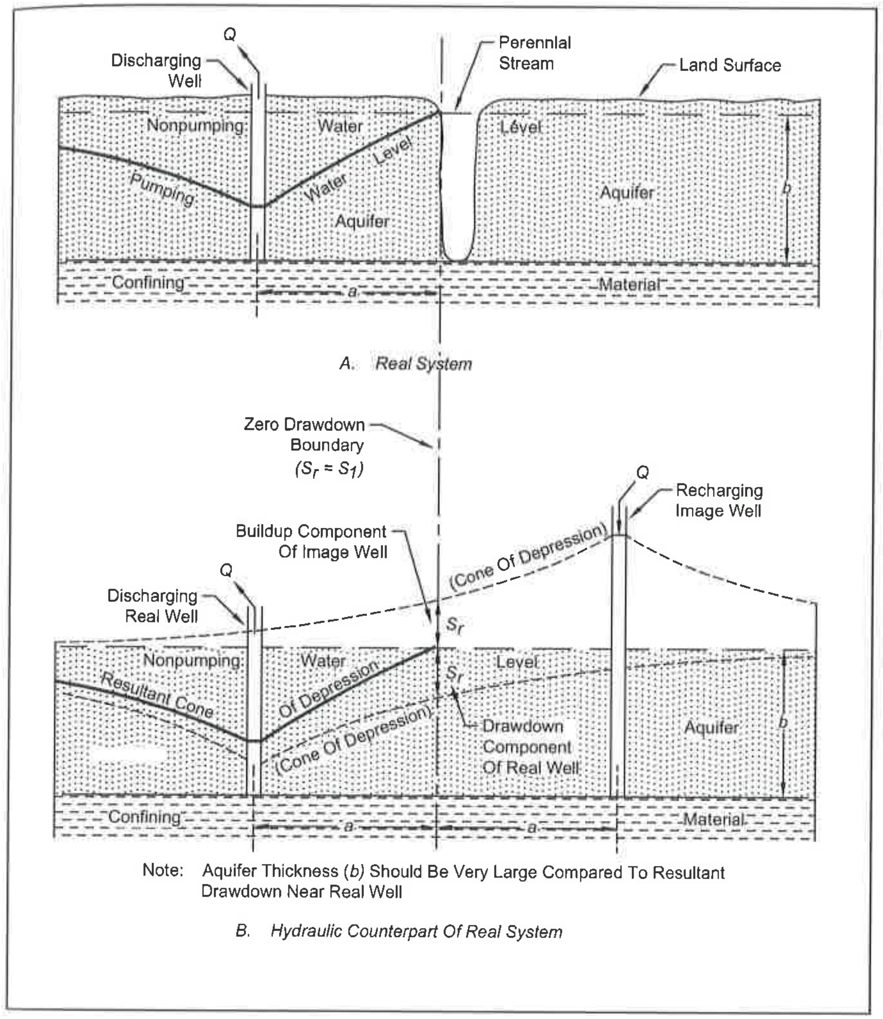

Figure 6.21. Idealized section views of a discharge in a well in (A) a semi-infinite aquifer bounded by a perennial stream, and the equivalent hydraulic system in (B) an infinite aquifer.

An image system that satisfies the above-described boundary condition (Figure 6.21(B)) allows a solution of the real problem through use of the Theis non-equilibrium methodology. Note in Figure 6.21(B) that an imaginary recharging well has been placed at the same distance as the real well from the line source, but on the opposite side. Both wells are situated on a common line perpendicular to the line source. The imaginary recharge well operates in the same manner as the real well, and returns water to the aquifer at the same rate that it is withdrawn by the real well. The resulting drawdown at any point on the cone of depression the real region is the algebraic sum of the drawdown caused by the real well and the buildup produced by its image.

An idealized section through a discharging well in an aquifer bounded on one side by an impermeable barrier is shown in Figure 6.22. It is assumed that the irregularly sloping boundary can be replaced by a vertical boundary, occupying the position shown by the vertical dashed line in Figure 6.22(A). The hydraulic condition imposed by the vertical boundary is that there can be no groundwater flow across it. No water is contributed to the discharging well. The image system that satisfies this condition and permits a solution of the real problem by the Theis equation is shown in Figure 6.22(B). An imaginary discharging well has been placed at the same distance as the real well from the boundary but on the opposite side, and both wells are on a common line perpendicular to the boundary. At the boundary the drawdown produced by the image well is equal to the drawdown caused by the real well. The resultant drawdown at any point on the cone of depression in the real region is the algebraic sum of the drawdowns produced at that point by the real well and its image.

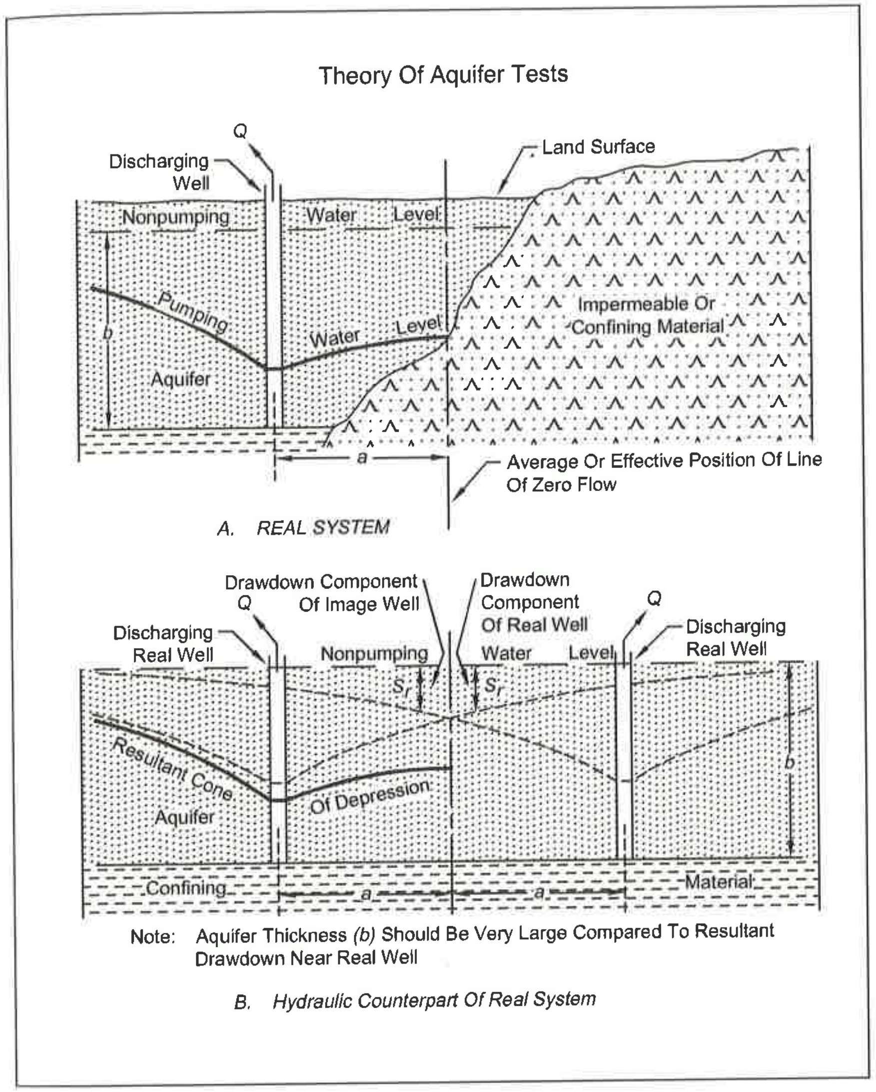

Figure 6.22. Idealized section view of a discharging well in a semi-infinite aquifer bounded by an impermeable formation, and the equivalent hydraulic system.

Vertical Infiltration (Vertical Recharge)

Another situation leading to equilibrium conditions is when vertical recharge occurs throughout the area of influence around a pumping well. An example is a well completed in an unconfined aquifer where all the material in the vadose zone from the ground surface to the water table is permeable sand. Assume that infiltration is from rain falling within the cone of depression of the well. When the quantity of water percolating to the water table within that circle equals the discharge from the well, the cone of depression stops spreading and the pumping levels within the well and the aquifer become stable.

Slow Drainage

The more common type of recharge producing equilibrium conditions is vertical leakage from saturated layers above an aquifer tapped by a production well. The upper strata of saturated material might have considerably lower hydraulic conductivity than the deeper material in which the well terminates. The difference in hydraulic conductivity between the upper and lower strata could be so great that the upper material is not considered to be part of the aquifer. When the area of the cone of depression covers a large area, the total vertical seepage from the upper material—even though it has a relatively low hydraulic conductivity—can equal the well discharge and thus bring about equilibrium in the pumping level. This situation also could develop in lenticular formations in which only the lower part of the formation is screened.

In unconfined conditions, slow drainage within the cone of depression can affect the validity of early time-drawdown data. Figure 6.23 shows the effect on the drawdown curve. Typically, the effect of slow drainage lasts only a matter of hours before the slope of the drawdown line reflects true aquifer characteristics.

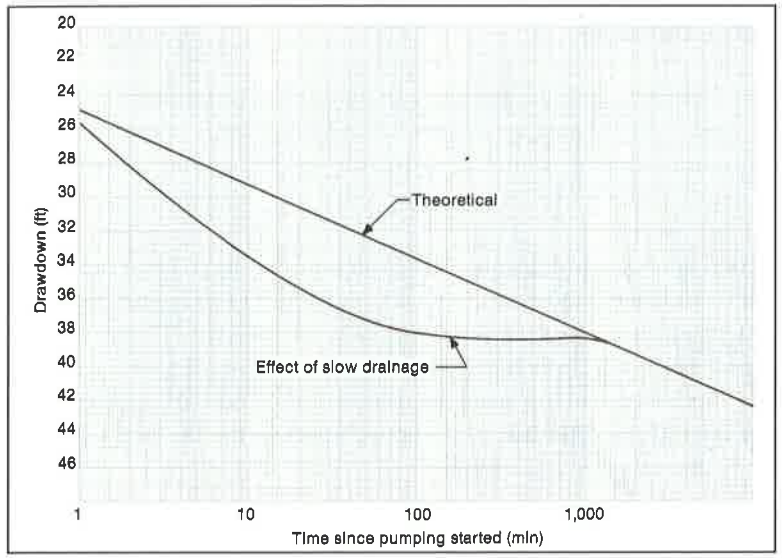

Figure 6.23. Time-drawdown curve showing the effect of slow drainage on the early part of the curve.

This phenomenon is caused by the great difference between the horizontal and vertical hydraulic conductivity in some sediments, and the severely limited hydraulic conductivity of sediments overlying many aquifers. In glacial drift, for example, layers of rather coarse sand or gravel commonly lie between thin layers of silt or clay. Water can flow freely in a horizontal direction, but vertical flow is greatly retarded. When pumping begins, the amount of vertical water movement toward the screen is relatively small but, as time passes and the cone of depression widens, a larger percentage of the water moves toward the well vertically, thereby reducing the slope of the time-drawdown.

It is important to recognize the effects of slow drainage because the curve might otherwise suggest that a recharge boundary has been encountered after a few minutes of pumping. Actually, the true transmissivity can be calculated only after several hours of pumping. Earlier transmissivity values might seem appropriate for a particular aquifer, but the storage values will be much lower than expected. Often the storage coefficients are similar to those encountered in a confined aquifer. Abnormally low values are clear indications of slow drainage. Thus, early data in slow drainage situations do not indicate true aquifer conditions.

Vertical Leakage

Like slow drainage, vertical leakage can distort the time-drawdown curve, but it occurs later. Vertical leakage occurs when an aquifer is confined geologically by two aquitards. During pumping, reduction in head within the cone of depression can cause leakage from the aquitards that are located above and below the aquifer.

Detailed analyses of leakage effects on aquifers have been developed by a variety of researchers, and Kruseman and deRidder (1992) provide a variety of analytical procedures to assess the impacts of leakage. Figure 6.24. shows distortion of the Jacob curve by vertical leakage. In complex geologic situations, computer modeling can be used to assess the effects of leakage and other boundary conditions. The use of computer models for the analysis of aquifer tests also is discussed in this chapter.

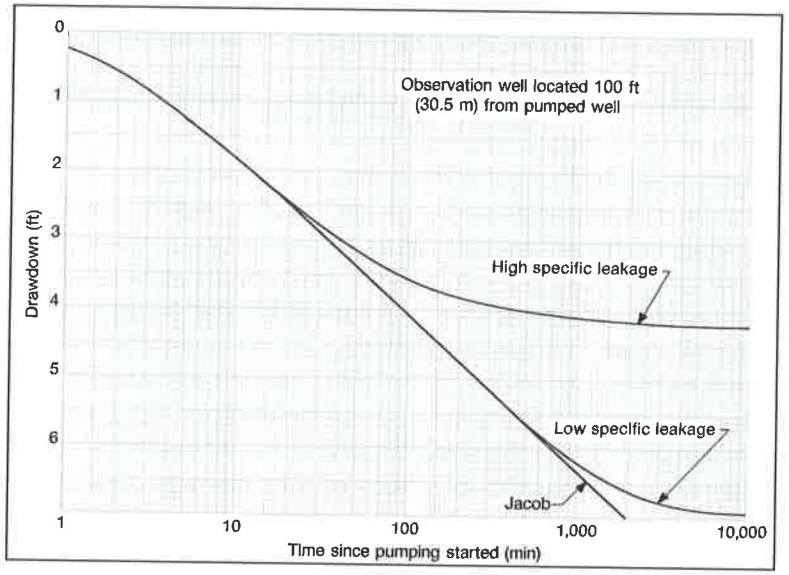

Figure 6.24. Drawdown graphs showing both low and high specific leakage.

Wellbore Storage

The mathematics of the Theis equation represents the pumped well by a line source which implies an insignificantly small well radius. As a consequence of this assumption, the Theis equation ignores water stored within the casing of the pumped well as a source of water during pumping. Casing (or wellbore) storage has its greatest influence at the outset of pumping. As the time of pumping progresses, however, this source of water is depleted and the well derives increasingly more water from the aquifer.

Papadopoulos and Cooper (1967) developed an equation that accounts for wellbore storage, and reported the time required to deplete wellbore storage as shown in equation 6.22.

$$t > \frac{250 r_c^2}{T} \tag{6.22}$$

Where:

t = time since start of pumping;

r_c = casing radius (L); and

T = transmissivity (L^2/t).

The criterion given in equation 6.22 indicate that the period affected by wellbore storage is shorter in small-diameter wells and in high-transmissivity aquifers. Equation 6.22 is applicable only to wells that are 100% efficient and fully penetrating.

Graphical methods also are useful for determining the period influenced by wellbore storage. On a log-log plot, the wellbore storage effect results in early-time drawdown data plotting with a unit (1:1) slope. A more useful approach to assessing wellbore is found in Schafer (1978) (a copy of this paper is contained in Appendix 6.Q (DVD)).

Schafer (1978) suggested that, in many instances, early aquifer test data might not fit Jacob's modification of the nonequilibrium theory, and that calculations based on this early Δs value have erroneous results. These early data reflect the removal of water stored in the casing because, when pumping begins, the water in the casing is removed first. As the water level in the casing falls, water from the surrounding formation begins to enter the well. Gradually, a greater percentage of the well's yield comes from the aquifer. The Δs value is higher during the time required to exhaust the casing storage, which gives an erroneously low transmissivity value in the early stages of the aquifer test. Figure 6.25 shows data from a typical aquifer test in which casing storage has distorted the early part of the time-drawdown curve.

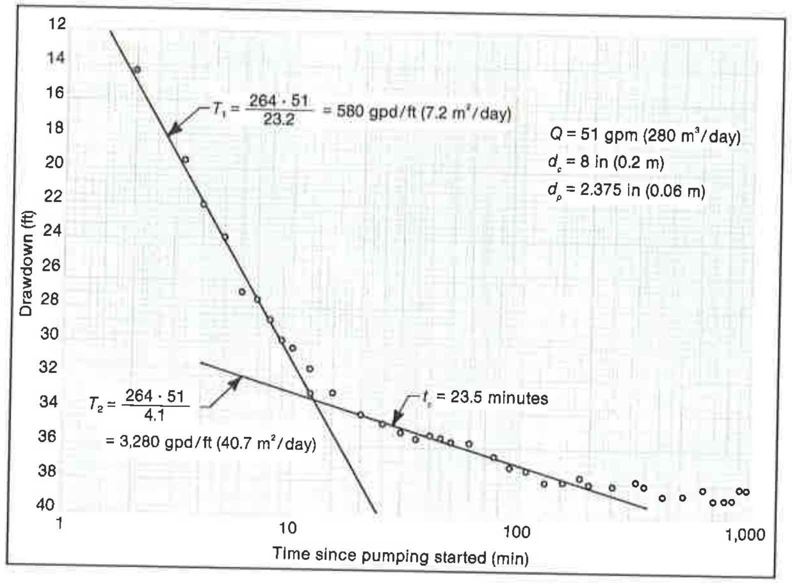

Figure 6.25. Pumping test data in which casing storage has altered the early part of the time-drawdown plot.

Before the effect of casing storage on aquifer test data was recognized, the flattened portion (or second part) of the drawdown curve might have been mistaken as an indication of aquifer recharge. The duration of the effect of casing storage varies greatly from well to well, depending on the casing diameter and specific capacity. In general, the storage effect lasts longer for wells that have large diameters and low specific capacities.

Papadopulos and Cooper (1967) and Ramey et al. (1973) present equations that modify the early part of the Jacob and Theis curves by taking into account casing storage (Papadopulos-Cooper equation for determining t_c is shown in Figure 6.26). These equations indicate the critical time after which casing storage no longer contributes to the yield of a well. Presumably, drawdown data collected after this time represents the true physical conditions within an aquifer. Unfortunately, these equations only can be used if the transmissivity and well efficiency are known in advance of the aquifer test.

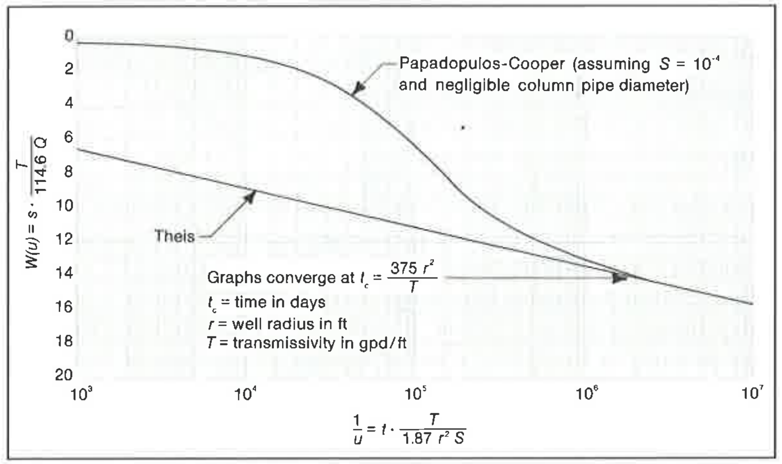

Figure 6.26. Graphic representation of the Papadopulos-Cooper equation which takes into account casing storage.

Schafer suggested that the critical time can be calculated using the following equations (6.23 as U.S. customary units; 6.24 as SI units).

$$t_c = \frac{0.6(d_c^2 - d_p^2)}{Q/s} \quad \textbf{(6.23)}$$

Where:

t_c = time (min) when casing storage effect becomes negligible;
d_c = inside diameter of well casing (in);
d_p = outside diameter of pump pipe (in); and
Q/s = specific capacity of the well in gpm/ft of drawdown at time t_c.

$$t_c = \frac{0.017(d_c^2 - d_p^2)}{Q/s} \quad \textbf{(6.24)}$$

Where:

t_c = time (min) when casing storage effect becomes negligible;
d_c = inside diameter of well casing (mm);
d_p = outside diameter of pump pipe (mm); and
Q/s = specific capacity of the well in m³/day/m of drawdown at time t_c.

Estimating a representative transmissivity value depends on being able to identify the difference between a casing storage effect and a recharge boundary encountered early in the aquifer test.

Example

Assume that it is unknown whether the aquifer-test data presented in Figure 6.26 represents boundary conditions or casing storage. If an iteration using equation 6.23 is performed (throughout, equation 6.24 also can be used), it is possible to estimate whether the change in the slope of the curve is caused by a casing storage effect or if it instead represents a recharge boundary. If the change in slope is caused by a casing storage effect, then the time of the change can be predicted using equation 6.23.

Equation 6.23 requires that the drawdown at time t_c be known; thus, there appear to be two unknowns, s and t_c. Initially, however, any drawdown value s_1 can be chosen and a trial t_c value can be calculated. Using the trial value of t_c and the time-drawdown graph, a new drawdown value (s_2) is obtained and can be used in equation 6.23 to calculate a second trial value of t_c. This procedure is repeated two or three times until the calculated value for t_c does not change significantly between iterations. Using this equation does not require knowledge of either the transmissivity or well efficiency.

To use equation 6.23, an initial drawdown value is selected and a t_c value is calculated. Assume, for example, that the initial s is 20 ft and values for the remaining parameters are listed in Figure 6.25 in U.S. customary units; the estimated t_c then is calculated as follows.

$$t_c = \frac{0.6((8)^2 - (2.375)^2)}{51/20} = 13.7 \text{ min} \qquad \textbf{(6.25)}$$

At 13.7 min, the drawdown is 33 ft (read from time-drawdown graph, Figure 6.25). Another calculation is made using 33 ft instead of 20 ft, resulting in a t_c value of approximately 22.7 min. The drawdown at 22.7 min is 34.2 ft. Substituting this value in equation 6.23 leads to a t_c of 23.5 min. Note that the change in the t_c value between the last two iterations is only 0.8 min—a small value. The three iterations using equation 6.23 suggest that the casing storage effect becomes negligible at approximately 23 min. Thus, the initial slope (Figure 6.25) provides an erroneous T value. Any predictions of the well's performance instead should be based on the T value calculated using the latter part of the curve. If the change in slope was caused by a boundary effect, then the time of the change in slope could not be predicted by applying equation 6.23, and instead the initial T value must be used to predict well performance.

Casing and Filter-Pack Storage

In some filter-packed wells, the static water level intersects the filter pack and can be close enough to the well screen to enable rapid drainage of the filter pack when the pump is started. In such cases, the storage effect is increased by the volume of water that can drain from the filter-pack storage.

$$t_c = \frac{0.6\left[(d_c^2 - d_p^2) + S_y(D_W^2 - d_o^2)\right]}{Q/s} \qquad \textbf{(6.26)}$$

Where:

- d_c = inside diameter of well casing (in);
- d_p = outside diameter of pump pipe (in);
- S_y = short-term specific yield of filter-pack material (typically 20%);
- D_w = diameter of the borehole (in); and
- d_o = outside diameter of well casing (in).

If water draining from the filter pack contributes to the storage effect, this equation should be used in lieu of equation 6.23 for computing t_c.

Partial Penetration

To understand partial penetration, consider a well with a screen or open interval of length L in an aquifer of thickness b. A well is "fully penetrating" when its dimensionless screen length L/b is equal to unity. When L/b is less than 1, the well is "partially penetrating."

In an ideal confined aquifer, flow to a fully penetrating pumping well is horizontal throughout the aquifer and no vertical flow occurs. When a pumping well partially penetrates an aquifer, however, convergence of flow toward the well screen results in vertical flow gradients that increase in magnitude as distance from the well decreases (Figure 6.27). Thus, the drawdown in a pumped well that partially penetrates an aquifer is greater than that of a fully penetrating well, due to the head loss associated with the convergence of flow. Vertical flow gradients produced by partial penetration extend beyond the pumped well and can affect the drawdown response in nearby observation wells.

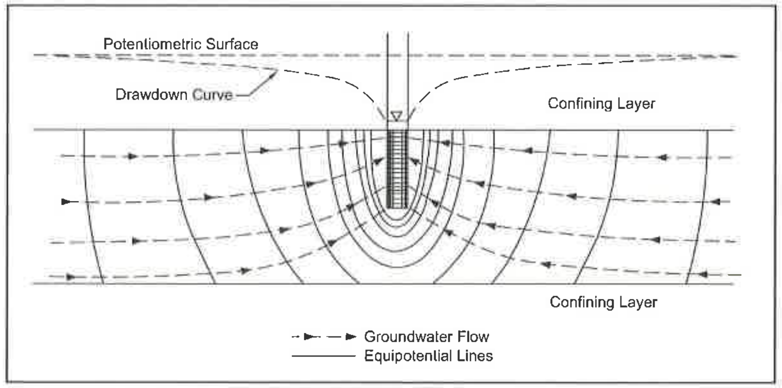

Figure 6.27. When the intake section of a well partially penetrates a confined aquifer, flow lines deviate somewhat from the radial flow pattern associated with a fully penetrating well.

The Theis and Cooper-Jacob equations considered thus far both assume that the wells are fully penetrating. Hantush (1961; 1964) considered the effects of partial penetration on wells in confined and leaky confined aquifers. Hantush showed that the partial penetration effect becomes negligible when observation wells are sufficiently distant from the pumped well.

$$r > 1.5b / \sqrt{K_z / K_r} \qquad \textbf{(6.27)}$$

Where:

r = radial distance from the pumped well (L);

b = aquifer thickness (L);

K_z = vertical hydraulic conductivity in the aquifer (L/t); and

K_r = radial hydraulic conductivity in the aquifer (L/t).

According to Hantush, the Theis or Cooper-Jacob equations can be applied to partially penetrating wells to determine the transmissivity and storativity of a confined aquifer when the criterion in equation 6.27 is satisfied. An example (using U.S. customary units), is a 50-ft confined aquifer having an estimated hydraulic conductivity anisotropy ratio $K_z/K_r = 0.5$. Using equation 6.27, the predicted effect of partial penetration becomes negligible when r is greater than 150 ft.

It also is true that the partial penetration effect reaches a constant maximum after sufficient time has elapsed. Hantush (1961) showed that the correct semilog slope occurs after the following time criterion is met.

$$t > bS / 2K_z \qquad \textbf{(6.28)}$$

Where:

t = time elapsed since pumping began;

b = aquifer thickness (L);

S = storage coefficient; and

K_z = vertical hydraulic conductivity in the aquifer (L/t).

According to Hantush, the Cooper-Jacob equation can be applied to partially penetrating wells to determine the transmissivity of a confined aquifer when the criterion in equation 6.28 is satisfied. For a 50-ft confined aquifer with an estimated storativity of 0.0005 and vertical hydraulic conductivity of 0.1 ft/day, for example, equation 6.28 enables a prediction that drawdown data in partially penetrating wells can be analyzed using the Cooper-Jacob equation when the time since pumping began exceeds 0.125 days or 180 min.

The criteria given in equations 6.27 and 6.28 enable the equations for fully penetrating wells to be used to analyze aquifer-test data. For data not meeting these criteria, the Hantush equation (1961; 1964)—which explicitly accounts for partially penetrating wells in confined aquifers—can be used. Type curves for the Hantush equation are a function of the penetration depths of the pumping and observation wells, in addition to the transmissivity and storativity of the aquifer. Tabulated values for selected pumping and observation well depths are available in Reed (1980), and Kruseman and De Ridder (1992). For other cases not available in tabulated form, computer programs are available for determining the well function for the Hantush equation (e.g., Reed 1980) as well as commercially available software packages (e.g., AQTESOLV™). Additional discussion on partial penetration is contained in Appendix 6.K (DVD).

Well Efficiency and Well Loss

Well efficiency is defined as the theoretical drawdown divided by the actual drawdown. Appendix 6.L (DVD) provides additional discussion regarding well efficiency. The following example (in U.S. customary units) defines and calculates the efficiency of a well.

A 12-in pumping well discharges at a constant rate of 200 gpm (38,500 ft^3/day) from a confined aquifer that has a transmissivity of 10,000 ft^2/day and storativity of 0.0005. Using the Theis formula supplied in equation 6.11, the theoretical drawdown in the pumped well after one day is computed as follows.

$$u = \frac{r^2 S}{4Tt} = \frac{(0.5)^2(0.0005)}{4(10{,}000)(1)} = 3.125 \cdot 10^{-9}$$

$$s = \frac{Q}{4\pi T} W(u) = \frac{(38{,}500)}{4\pi(10{,}000)}(19.0) = 5.82\,ft$$

This calculation assumes that the well is fully penetrating and 100% efficient (i.e., no additional drawdown occurs in the well due to sources of well loss such as friction, turbulence, or reduced permeability in the vicinity of the wellbore). If the actual drawdown measured in the pumped well during the same period is 7.5 ft, then the efficiency of the well is computed as follows.

$$efficiency = \frac{theoretical\, drawdown}{actual\, drawdown} \cdot 100 = \frac{5.82\,\text{ft}}{7.5\,\text{ft}} \cdot 100 = 78\%$$

The factors contributing to excess drawdown in wells (inefficiency) can be grouped into two classes. One class is comprised of factors primarily related to choices made in the design of wells; the other class includes factors related to construction. The following is a summary of the factors in the two classes.

Design Factors

- Using a well screen that has insufficient open area creates entrance velocities that are too high, resulting in greater-than-normal entrance (head) losses.
- Choosing a well screen that has poor distribution of screen openings causes excessive convergence flow near the individual openings, and this can produce more drawdown than necessary (*see* Figure 6.28).
- Using a screen that results in partial penetration of the aquifer distorts the flow pattern for some distance around the well (*see* Figure 6.27). In such cases, flow to the well screen includes major

vertical and the main horizontal components. Vertical hydraulic conductivity generally is lower than horizontal hydraulic conductivity, therefore considerable head losses result from the vertical flow. Although extra drawdown might result, in many instances a shorter well screen can be used due to other design considerations. The effect of screen length is discussed in Chapter 9.

- Selecting improperly sized filter packs or those made from angular or plate-like materials can restrict flow into a well screen. Particle shape and size, and grain-size distribution all affect the hydraulic conductivity of the pack.

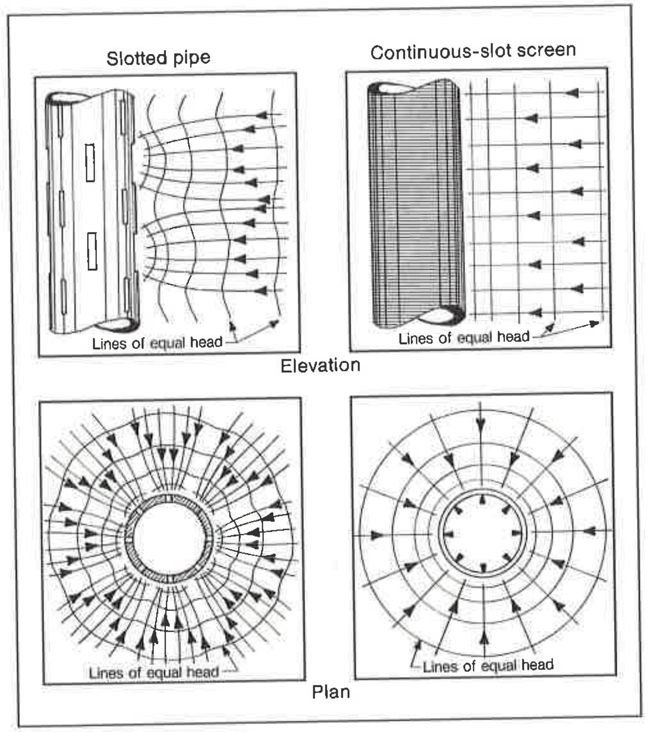

Figure 6.28. Flow nets around screen devices. Water approaches openings along the lines indicated by arrows. Flow lines for slotted pipe converge to individual slots; flow lines for continuous-slot well screens are less distorted.

Construction Factors

- Inadequate development of a well can leave drilling fluid and small particles in the aquifer around the screen, reducing the original hydraulic conductivity. Developing wells can be very difficult when screens with insufficient open area or poor distribution of slot openings are installed. Additional discussion regarding well development is contained in Chapter 11.
- Placement of the well screen at a depth that does not correspond to the best water-yielding stratum can lead to excess drawdown.

Analysis of Recovery Data

Analyzing water-level recovery data provides a means of checking results obtained with time-drawdown analyses. Values obtained from analysis of the recovery record can be used to validate the calculations based on the pumping record.

During an aquifer test, measurement of the recovery of water levels in wells monitored should continue after pumping stops. When pumping stops, well and aquifer water levels rise toward their pre-pumping levels, and recording the rate of recovery enables calculation of values for transmissivity and storage. The time-recovery record therefore is an important part of an aquifer test. The time-drawdown measurements taken during the pumping period and the time-recovery measurements taken during the recovery period provide two different sets of information from a single aquifer test. In all cases, water levels should be measured in the pumped well and in each observation well during the recovery phase of the test.

Recovery data can be analyzed as described below only when the pumping portion of the aquifer test is conducted at a constant rate. Recovery measurements following a variable-rate test, such as a step-drawdown test, are difficult to analyze. The exact time of starting and stopping the pump must be recorded, in addition to any changes in pumping rate and the time that each incident occurs.

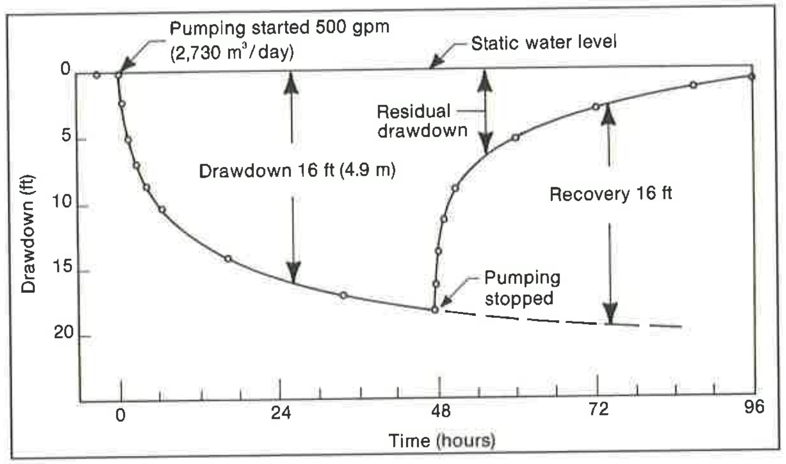

Figure 6.29. Typical drawdown and recovery plots for a well pumped for 48 hr at a constant rate of 500 gpm (2,730 m^3/day), followed by a 48-hr water-level recovery period.

Figure 6.29 shows how water levels in a well change over time. The left half of the diagram corresponds to the pumping period and the right half to the recovery period. The recovery curve nearly is an inverted image of the drawdown curve. The shape of each curve is determined by the physical characteristics of the aquifer.

The points plotted for the recovery curve (right half of Figure 6.29) represent the residual drawdown in the well during the recovery period. Each point represents the difference between the original static water level and the depth to water at a given instant during the recovery period.

The hydraulic theory of well and aquifer performance assumes that water-level changes during the recovery period are the result of input from a theoretical recharge well. If such a well injects water into the aquifer at the same rate the real well pumps it out—with both wells working simultaneously after a given instant—then the recovery curve is similar to that depicted in Figure 6.29. The rise in water level (because of the imaginary recharge well) is the vertical distance between an extension of the time-drawdown curve and the actual time-recovery curve.

Recovery therefore is the difference between the measured water level in an observation well at a given time after pumping stops and the level to which the water would have dropped if pumping had continued until that instant. When defined in this way, the degree of water-level recovery at any time after the end of the pumping period theoretically is equal to—but opposite of—the drawdown during the same pumping period.

Example

A 6-in (152 mm) test well and an observation well 50 ft (15.2 m) away are used for an aquifer test. After pumping the well at 200 gpm (1,090 m^3/day) for 500 min, the pump is stopped and water-level measurements are made during the first 390 min of the recovery period.

Table 6.4 shows the depth-to-water measurements in the observation well and the residual drawdown for time intervals measured from both the beginning of the aquifer test and the beginning of the recovery period. These intervals are defined as t and t', respectively. The ratios of the two time periods (t/t') also are shown in the table.

Figure 6.30 shows the recovery curve plotted for the observation well. Extension of the preceding drawdown curve indicates the drawdown that would have occurred if pumping had been continued beyond the pumping period. The water-level recovery for various time intervals is the vertical difference between the curves in this diagram. Values are provided in Table 6.4.

Table 6.4. Residual Drawdown and Calculated Recovery in the Observation Well

Time Since Pump Started (t)	Time Since Pump Stopped (t')	Ratio	Depth to Water		Residual Drawdown (s')		Drawdown (s) from Pumping Curve		Calculated Recovery (s - s')	
min	min	(t/t')	ft	m	ft	m	ft	m	ft	m
500	0	—	18.60	5.67	10.60	3.23	10.60	3.23	0.00	0.00
501	1	501.00	18.55	5.66	10.55	3.22	10.60	3.23	0.05	0.01
502	2	251.00	18.50	5.64	10.50	3.20	10.60	3.23	0.10	0.03
503	3	168.00	18.40	5.61	10.40	3.17	10.61	3.23	0.21	0.06
504	4	126.00	18.09	5.52	10.09	3.08	10.61	3.23	0.52	0.15
506	6	84.00	17.72	5.40	9.72	2.96	10.62	3.24	0.90	0.28
508	8	64.00	17.22	5.25	9.22	2.81	10.63	3.24	1.41	0.43
510	10	51.00	16.64	5.07	8.64	2.63	10.64	3.24	2.00	0.61

Time Since Pump Started (t)	Time Since Pump Stopped (t')	Ratio	Depth to Water		Residual Drawdown (s')		Drawdown (s) from Pumping Curve		Calculated Recovery (s - s')	
min	min	(t/t')	ft	m	ft	m	ft	m	ft	m
520	20	26.00	15.27	4.66	7.27	2.22	10.67	3.25	3.40	1.03
530	30	17.70	14.50	4.42	6.50	1.98	10.70	3.26	4.20	1.28
540	40	13.50	13.63	4.16	5.63	1.72	10.73	3.27	5.10	1.55
560	60	9.35	12.95	3.95	4.95	1.51	10.80	3.29	5.85	1.78
590	90	6.55	12.01	3.66	4.01	1.22	10.96	3.34	6.95	2.12
650	150	4.33	10.80	3.29	2.80	0.85	11.15	3.40	8.35	2.55
710	210	3.38	10.70	3.26	2.70	0.82	11.35	3.46	8.65	2.64
770	270	2.85	10.06	3.07	2.06	0.63	11.56	3.52	9.50	2.89
830	330	2.51	9.96	3.04	1.96	0.60	11.76	3.59	9.80	2.99
890	390	2.28	9.60	2.93	1.60	0.49	11.95	3.64	10.35	3.15

Note: Static water level is 8 ft (2.44 m).

Average pumping rate during preceding pumping period was 200 gpm (1,090 m^3/day).

The recovery curve plotted in Figure 6.30 can be analyzed using either the Theis (1935) corollary to the non-equilibrium equation (presented below) or Jacob's (1946) modification of the non-equilibrium equation. As shown, the time-drawdown curve for the pumping period becomes a straight line on a semi-logarithmic diagram. The same simplification can be used for the time-recovery plot, where the horizontal scale represents the logarithm of time during the recovery period and the vertical scale represents water-level recovery ($s - s'$).

Data from Table 6.4 plotted in this way are shown in Figure 6.31. The result is similar to a time-drawdown plot for the pumping phase of the same aquifer test. Theoretically, the drawdown and recovery plots should be identical if the aquifer conditions conform to the basic assumptions of the Theis equation.

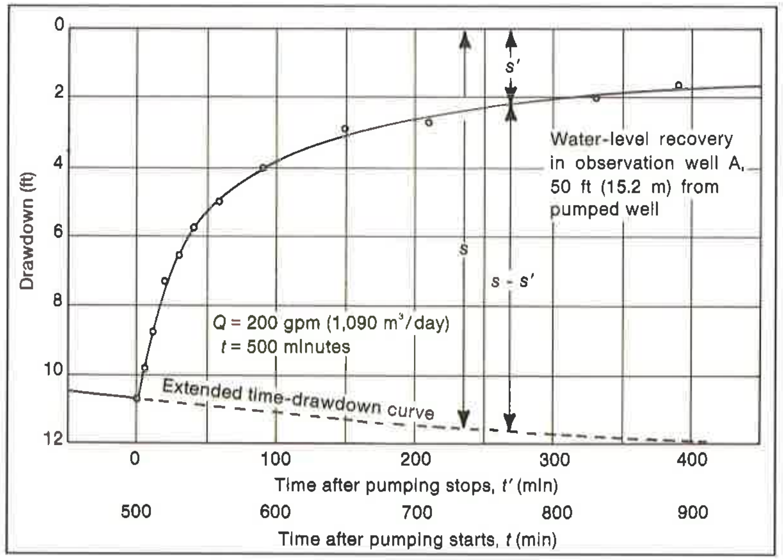

Figure 6.30. Residual-drawdown curve from piezometer, with extended time-drawdown curve (on arithmetic scales) showing how calculated recovery is determined at any instant during the recovery period. Producing well pumped 200 gpm (1,090 m³/day) for 500 min.

The time-recovery plot for the pumped well is more accurate than its time-drawdown plot because the residual-drawdown measurements are more accurate. During the recovery period, water-level measurements taken within the pumping well can be made without the effects of turbulence, vibrations, or well losses within the pumping well. The time-recovery data measured in a piezometer can be analyzed in the same manner as that used for the pumping-well data.

In an analysis of the time-recovery plot, the slope is of primary interest. Two factors determine the slope of the straight line in Figure 6.31. One is the average pumping rate during the preceding pumping period, the other is the aquifer transmissivity.

In Figure 6.31, the slope of the straight line is expressed numerically as the change in the water-level recovery per logarithmic cycle. It is designated using $\Delta(s - s')$. Its value in Figure 6.31 is 5.2 ft (1.6 m), which is the recovery during the period from 10 min to 100 min after pumping stopped.

The next step is to calculate the transmissivity of the aquifer from the following equation (given in both U.S. customary and SI units).

$$T = \frac{264Q}{\Delta(s - s')} \quad \text{in gpd/ft} \qquad \text{U.S. customary} \qquad \textbf{(6.29)}$$

$$T = \frac{0.183Q}{\Delta(s - s')} \quad \text{in m}^2\text{/day} \qquad \text{SI}$$

Figure 6.31 shows the value of T to be about 10,200 gpd/ft (127 m²/day). The values for T calculated from drawdown and recovery data should agree.

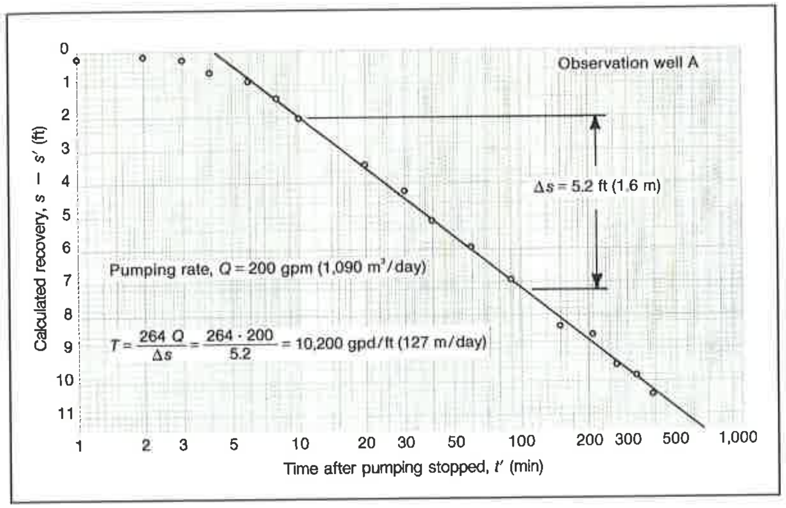

Figure 6.31. Time-recovery plot for observation well becomes a straight line when plotted on a semilog diagram (similar to the time-drawdown diagram for the preceding pumping period).

The commonly used method of plotting the data permits direct use of the residual drawdown without calculating the recovery from an extension of the time-drawdown plot. It can be shown that the residual drawdown is related to the logarithm of the ratio t/t' as follows (mathematical development of this relationship is given in Appendix 6.M).

$$s' = \frac{264Q}{T}\log\frac{t}{t'} \qquad \text{U.S. customary} \qquad \textbf{(6.30)}$$

Where T is in gpd/ft and Q is gpm.

$$s' = \frac{0.183Q}{T} \log \frac{t}{t'} \qquad \text{SI}$$

Where T is in m^2/day and Q is in m^3/day.

This equation shows that when values of s' are plotted against corresponding values of t/t' on semilogarithmic graph paper; a straight line can be drawn through the plotted points. Figure 6.32 shows the data from Table 6.4 plotted on a semilog diagram, with s' indicated on the vertical arithmetic scale and t/t' on the horizontal logarithmic scale. The transmissivity then is calculated using the following equation.

$$T = \frac{264Q}{\Delta s'} \qquad \text{U.S. customary} \qquad \textbf{(6.31)}$$

$$T = \frac{0.183Q}{\Delta s'} \qquad \text{SI}$$

Figure 6.32 (below) shows that during the recovery period time increases toward the left in this method of plotting, as compared with the time-drawdown and time-recovery plots in which time increases toward the right.

Use of the residual-drawdown plot (Figure 6.32)—instead of the recovery plot (Figure 6.31)—might be preferable for calculating transmissivity. The method used for a residual-drawdown plot provides an independent check on the results calculated from the pumping period. The method used for a recovery plot depends upon extension of the time-drawdown plot through the recovery period; thus the drawdown plot itself determines the values used in the recovery plot, and any inaccuracies in the drawdown plot are projected into the recovery plot.

If no observation well is available for measurement, then the recovery data from the pumped well provides an acceptable basis for calculating the transmissivity of the aquifer. In this case, the residual-drawdown plot (Figure 6.32) should be used.

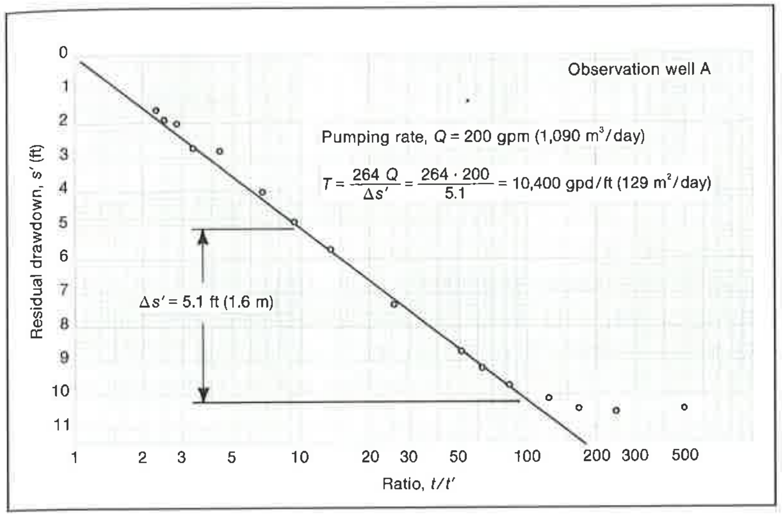

Figure 6.32. Residual-drawdown plotted against the ratio $^t/_{t'}$ becomes a straight line on semilog graph and enables calculation of transmissivity as shown. Time during recovery period increases toward the left in this diagram.

Variable-Rate Tests

Constant-head, variable-rate tests are used when a flowing well in a confined aquifer is shut-in long enough for the head to recover, and then the well is opened and allowed to flow for some period. A similar situation occurs in tunneling or mining projects when the tunnel is below the water table. An exploratory corehole is advanced away from the tunnel and a packer-piping assembly is inserted into the corehole and the piping, equipped with a valve, is shut-in. When the static pressures are achieved the valve is opened and, in many cases, the flows decline with time.

Straight-line solutions to address constant-head, variable-flow rates were developed by Jacob and Lohman (1952). For large values of time (t) and sufficiently small values of u, $W(u)$ again can be closely approximated by 2.30 $\log_{10} 2.25Tt/r^2S$. The following equation (rendered in both SI and U.S. customary units) was developed by Jacob and Lohman (1952).

$$T = \frac{2.30}{4\pi\Delta(s_w / Q)/\Delta \log_{10} t / r_w^{\ 2}} \qquad \text{U.S. customary} \qquad \textbf{(6.32)}$$

$$T = \frac{2.30}{4\pi s_w (\Delta 1/Q)/\Delta \log_{10} t} \qquad \text{SI}$$

The following example was taken from Lohman (1979). A well was shut-in for several days; the static head just prior to the test was 92.33 ft above the measuring point. The data from the test are shown in Figure 6.33. The s_w/Q data were derived by dividing 92.33 ft by the flow rate at a particular time.

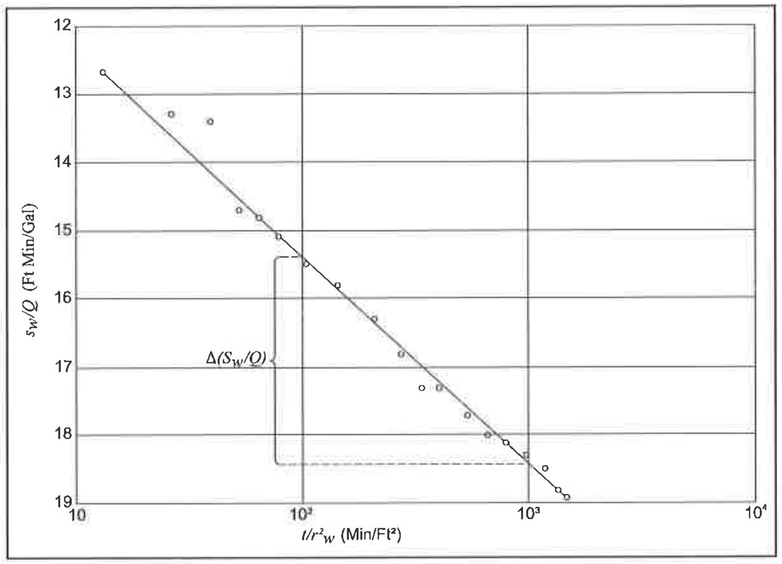

Figure 6.33. Semilogarithmic plot of *sw/Q* versus *t/r*$_w^2$ for a constant-head, variable-rate test.

From the values of $\Delta(s_w/Q)$ per log cycle of $t/r_w^{\ 2}$ obtained from Figure 6.33, transmissivity (T) can be calculated using equation 6.32.

$$T = \frac{(2.30)(1.44\times10^3 \text{ min } day^{-1})}{(4\pi)[(18.4-15.38) ft \cdot gal^{-1} \text{ min}](7.48 gal / ft^3)}$$

$T = 11.7$ ft^2 day^{-1}

Diagnostic Flow Plots

Diagnostic flow plots (e.g., radial flow, linear flow, bilinear flow, and spherical flow plots) commonly are used in the analysis of well tests in the petroleum industry. These plots also can be used to improve the interpretation of aquifer-test data.

Table 6.5. Useful Diagnostic Plots for Analyzing Aquifer Tests

Plot Type	Axes
Radial	s vs. log t and log s vs. log t
Linear	log s vs. log $t^{1/2}$

Diagnostic flow plots can help identify the following conditions.

- **Wellbore storage.** A radial flow plot with log-log axes can help identify wellbore storage effects in a pumped well. Early-time data exhibiting a unit slope on this plot type are indicative of wellbore storage.
- **Infinite-acting aquifer.** A radial flow plot with log-linear axes can help identify radial flow in an infinite-acting aquifer (i.e., late-time data plotting as a straight line). Late-time behavior on a semi-log plot is the basis for the Cooper-Jacob straight-line method of analysis.
- **Linear flow to a fracture.** A linear flow plot with log-log axes can help identify linear flow conditions to a fracture. Early-time data exhibiting a unit slope on this plot type are indicative of linear flow to a single fracture with infinite conductivity or uniform flux along the fracture.

For additional information on diagnostic flow plots (e.g., flow to a fracture, other flow conditions), see Horne (1995).

STEP-DRAWDOWN TESTS

The equations developed to calculate aquifer parameters are based on the assumption that laminar flow conditions exist in the aquifer during pumping. If the flow is laminar, then drawdown is directly proportional to the pumping rate. Turbulent flow occurs in some wells—especially those producing from fractured rock—when they are pumped at a sufficiently high rate. Under turbulent conditions, the linear relationship between drawdown and pumping rate no longer holds. Figure 6.34 shows the total drawdown (s) in a well. Well loss is the difference between the theoretical drawdown and the actual drawdown in the well. The losses include linear and nonlinear components. In Figure 6.34 total drawdown (s) in a well has four components.

$$s = s_l + s_n + s_e + s_w$$

Where:

- s = total drawdown in a well (L);
- s_l = head loss due to viscous drag as water moves through the aquifer with low velocity in the laminar flow region (L);
- s_n = head loss due to a non-laminar flow in the aquifer in the high-velocity region in the in the immediate vicinity of the well (L);
- s_e = head loss as the water exits the aquifer into the wellbore (L); and
- s_w = head loss as the water flows within the wellbore to the pump intake (L).

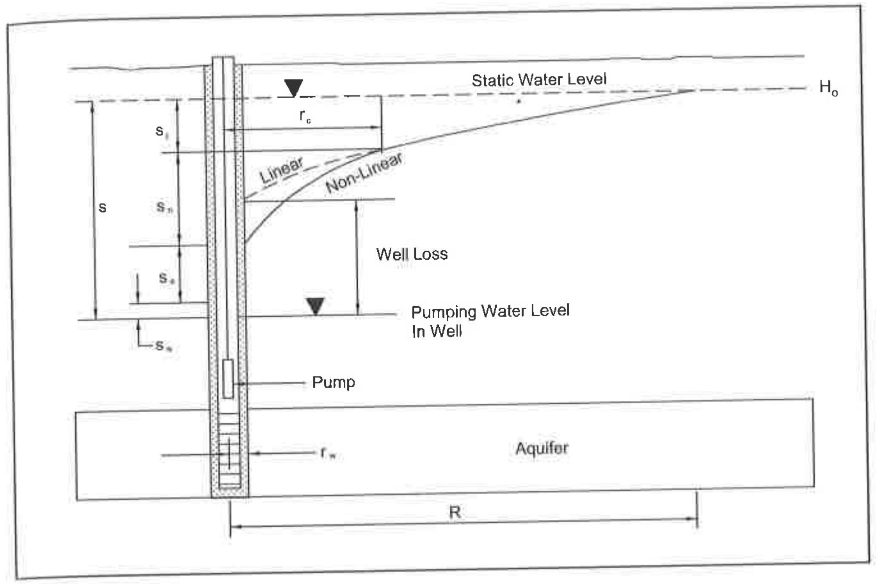

Figure 6.34. Components of drawdown in a well (HCItasca, Inc.).

When turbulent flow occurs, the specific capacity declines—often dramatically—as the discharge rate is increased. When this happens, it is useful to have a means of computing the turbulent and laminar drawdown components to be able to make proper judgments concerning the optimum pumping rate and pump-setting depth.

The step-drawdown test was developed to examine the performance of wells having turbulent flow (Jacob 1946). In a step-drawdown test, the well is pumped at several successively higher pumping rates and the drawdown for each rate—or step—is recorded. The entire test usually is conducted in one day, and calculations are simplified if all the pumping times are the same for each discharge rate. Usually 4 to 6 pumping steps are used, each lasting approximately 1 hr. The data from a step test can be used to calculate the relative proportion of laminar and turbulent flow occurring at selected pumping rates. A discussion of the approach to analyze turbulent versus laminar components of drawdown is contained in Appendix 6.N (DVD). Appendix 6.N also provides a discussion on a method to obtain an estimate of transmissivity from a measure of specific capacity.

In sum, the step-drawdown test can be used to calculate the following.

- Specific capacity of the well at various discharge rates. This information can be used to select optimum discharge rates. The length of time the test is run, however, might not be sufficient to encounter boundary effects. Therefore, the actual long-term specific capacity at a certain discharge can be different than originally calculated from the step-drawdown data. Slow drainage also can distort the discharge data in highly stratified aquifers.
- Percentage of total head loss attributable to laminar flow (however this percentage must not be confused with the true hydraulic efficiency of the well).
- Transmissivity and storage-coefficient values for the aquifer from time-drawdown and distance-drawdown graphs plotted from data for one of the constant-rate steps of the step test.

Analysis of Aquifer Tests Using Numerical Codes

As discussed in this chapter, the standard method of analyzing aquifer-test data is using the type-curve or straight-line methods. Type-curve and straight-line methods of analysis both have a number of drawbacks, including the fact that it generally is not possible to use a consistent approach to evaluate all of the data collected during the aquifer test. Aquifer geometries and boundary conditions also in many cases cannot be represented accurately (Spiliotopulos & Andrews 2006).

Complex aquifer test conditions can be analyzed using numerical models. In the past, the synthesis and calibration of a numerical model often was very labor intensive. Graphical user interfaces (GUIs) for numerical models, such as MODFLOW (Harbaugh et al. 2000) and PEST (Doherty 2005), have made it possible to rapidly analyze aquifer-test data without the limitations of the type-curve methods (Spiliotopulos & Andrews 2006). Additional discussion on the use of numerical codes to analyze aquifer tests is provided in Appendix 6.O (DVD).

Summary

Constant-rate, variable-rate, and step-drawdown aquifer tests offer methods for obtaining aquifer parameters such as transmissivity and storage coefficient. The hydraulic conductivity values determined from aquifer-test data are more accurate than hydraulic conductivities calculated on the basis of field samples tested in a laboratory, because samples rarely are representative of the undisturbed formation. Additionally, aquifer tests utilizing observation wells provide more-representative values for aquifer parameters than do single-well tests (e.g., slug tests), because a larger volume of aquifer is stressed. The aquifer test measures the performance of the aquifer over a larger volume of aquifer.

In addition to estimating hydraulic conductivity, transmissivity, and coefficient of storage, aquifer-test data can be used to estimate interferences between wells at various spacings and rates of pumping other than those employed in the test itself. Under some conditions, aquifer-test data also can indicate what drawdown might be expected from long-term pumping at different discharge rates, the presence of boundaries that limit the extent of the aquifer, and the existence of recharge sources—which might not be otherwise apparent based on observations of land features.

In many cases, aquifer-test data can be interpreted in more than one way. Even in the best-controlled of tests, data could be confusing unless all hydraulic and geologic factors are taken into account. A goal of aquifer-test design should be that the test be as simple as possible, to include the fewest deviations from the theoretical basis of the analysis. Wells should be fully penetrating, for example, and pumping rates should be constant. Analysts must be able to identify unreliable data so that the calculated values for transmissivity and storage coefficient correctly predict aquifer performance. An analyst also should review the test data in terms of a conceptual hydrogeologic model for the site. Figure 6.35 shows the relative components of an aquifer test and the potential impact on analysis. As shown in Figure 6.35, early-time data can be impacted by wellbore storage effects, whereas late-time data can be impacted by hydraulic boundaries.

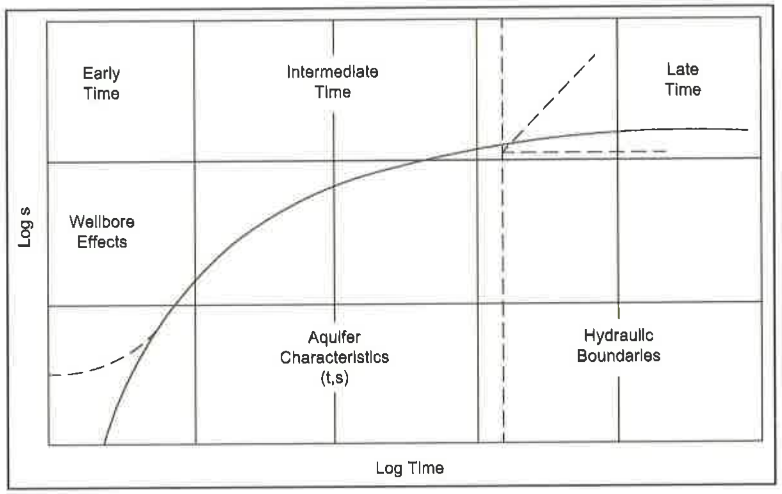

Figure 6.35. Components of a constant-rate test (HCItasca, Inc.).

Appendix 6.P (DVD) provides a number of examples of aquifer tests and interpretations of the data. Analysis of aquifer-test data requires an appreciation of all the factors that can affect the drawdown data. Such consideration chiefly comes from understanding the hydrogeology of the site, the assumptions inherent to the analytical methods, and the practical aspects of conducting the test. The accuracy of the data-collection methodologies also must be recognized when analyzing the data. The person performing the analysis must be able to visualize the physical nature of the aquifer and how it deviates from the basic assumptions upon which well-hydraulic equations are based. The limitations of hydraulic equations and the hydrogeologic nature of the aquifer system always must be kept in mind while analyzing aquifer-test data. Importantly, water-well professionals also must determine what data are valid for inclusion in the analysis to meet the objectives of each test.

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