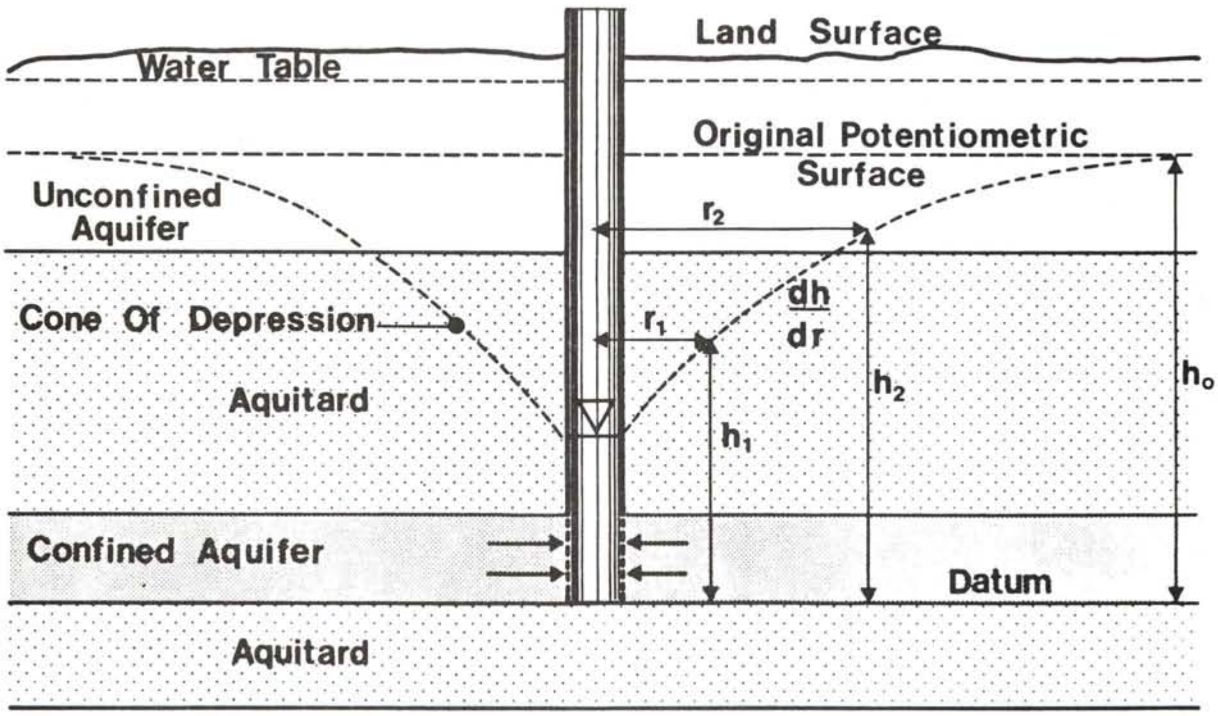
Well Hydraulics and Aquifer Testing



Idaho Water Resources Research Institute
University of Idaho
Moscow, Idaho

in cooperation with the

Department of Geology and Geological Engineering

WELL HYDRAULICS AND AQUIFER TESTING

April 16-17, 1990
Idaho Falls Center
1776 Science Center Drive
Idaho Falls, Idaho

April 18-19, 1990
Morrison-Knudsen Central Plaza
720 Park Blvd
Boise, Idaho

PURPOSE

Well Hydraulics and Aquifer Testing will be a two-day program that presents an overview of the concepts, theories, and procedures that have been developed to evaluate important aquifer coefficients. The course is designed for geoscientists, engineers, and other professionals who want to increase their knowledge of current aquifer testing methods and their applications.

Discussions will focus on the meaning of important aquifer coefficients in confined and unconfined aquifers and aquitards. Topics included are porosity, Darcy's Law, hydraulic head, hydraulic gradient, hydraulic conductivity, transmissivity, specific yield, storativity, specific storage and aquifer and aquitard compressibility.

Accepted theories of aquifer testing will be presented. Discussions will include the use and limitations of the Thiem and Theis equations, the Jacob Straight Line Method, the Hantush-Jacob Method, the Hantush Modified Method, the Ratio Method, and unconfined aquifer methods.

PARTICIPANTS

This course will cover important topics in aquifer test design and analysis. The discussions are designed to increase your understanding of aquifer testing methods and their limitations. Practicing geoscientists, engineers, and other professionals, and graduate students will benefit from this course. The course is designed for professionals with a basic understanding of ground water flow.

INSTRUCTORS

Dr. James Osiensky is an Associate Professor of Hydrogeology at the University of Idaho and is stationed in the Department of Geology and Geophysics at Boise State University.

Mr. Thomas Brooks is an Associate Professor of Hydrogeology in the Department of Geology and Geological Engineering at the University of Idaho.

COURSE OUTLINE

Day 1

Session Begins: 8:30 a.m.

I Introduction

II Physical Properties and Equations of Ground Water Flow

III Aquifer Testing Considerations

Lunch

IV Thiem and Theis Equations

V Jacob Straight Line Method

Day 2

Session Begins: 8:30 a.m.

VI Hantush-Jacob Method

VII Hantush Modified Method

Lunch

VIII Ratio Method

IX Unconfined Methods

Breaks will be mid-morning and mid-afternoon of Day 1 with lunch scheduled for 12:00 p.m.
Break will be mid-morning of Day 2 with lunch scheduled for 12:00 p.m.
Day 1 will conclude at 4:30 p.m.
Day 2 will conclude at 3:30 p.m.

TABLE OF CONTENTS

Corrections for:
Well Hydraulics and Aquifer Testing

Page 14 1) Add subscript "i" to S so that S_i extends from "observed depth to water" to base of $|h_o - h_i|$.
2) Add **S** which spans from "observed depth to water" to top of $|h_o - h_i|$.

Page 23 Omit B.1.a. and B.2.a.

Page 44 III.B.--equation should include **S** in numerator.

Page 46 Add IV.A.2.c.--**"Aquifer thickness = 824 ft."**

Page 49 Rewrite B.5.b.1) as **"Little** drawdown beyond this distance."

Page 51 Label horizontal axis, **"Distance from pumping well (ft)."**

Page 56 IV.B.3.--Replace 100 with **10.**

Page 58 E.3., last line--Replace 100 with **10**, replace both 7.5's with .75.

Page 74 D.3.b.--Add capital gamma (Γ) to numerator

Page 80 2.b. Replace U_B with $\mathbf{U_A}$.

Page 81 3.b. Add capital gamma (Γ) to numerator

SESSION I

PHYSICAL PROPERTIES AND EQUATIONS OF GROUND WATER FLOW

Specific yield (S_y) is defined as the volume of water, expressed as a percentage of the total volume of a saturated aquifer, that can be drained by gravity drainage.

$$S_y = \frac{100 W_d}{V}$$

where: S_y is the specific yield (%)
W_d is the volume of water drained
V is the total volume of the aquifer

Specific retention (S_r) is defined as the volume of water, expressed as a percentage of the total volume of a saturated aquifer, that is retained by molecular and surface tension forces, against the force of gravity.

$$S_r = \frac{100 W_r}{V}$$

where: S_r is the specific retention (%)
W_r is the volume of water retained
V is the total volume of the aquifer

$$Porosity(n) = S_y + S_r$$

Specific yield and specific retention of unconsolidated aquifer materials depend on the following factors:

1) grain size
2) shape of grains
3) distribution of pores
4) compaction of the medium
5) amount of time allowed for gravity drainage

Example

A 100 ft. thick sand aquifer occurs under a one square mile area. The original water table is 25 feet below land surface.

Ground water is pumped from the aquifer at a rate of 1000 gallons per minute for 29 days.

The pumping results in a one foot decline of the water table to 26 feet below land surface.

What is the specific yield of the aquifer?

The volume of aquifer drained =

$$(5280 ft. \times 5280 ft.) \times 1 ft. = 2.8x10^7 ft.^3$$

The volume of water drained from the aquifer =

$$\frac{1000 gal./\min. \times 1440 \min./day \times 29 days}{7.48 gal./ft.^3}$$

$$= 5.6 \times 10^6 ft.^3$$

$$S_y = \frac{100 W_d}{V} = \frac{100(5.6 \times 10^6 ft.^3)}{2.8 \times 10^7 ft.^3} = 20\%$$

DARCY'S LAW

Darcy's Law can be written in its one dimensional form as

$$Q = -KA\frac{dh}{dl}$$

or

$$q = \frac{Q}{A} = -K\frac{dh}{dl}$$

where:
Q is the discharge rate $[L^3/T]$
A is the cross-sectional area $[L^2]$
K is the hydraulic conductivity [L/T]
dh/dl is the hydraulic gradient [dimensionless]
q is the specific discharge [L/T]

Experiments have been conducted with ideal porous media of uniform glass beads of diameter d.

When various fluids of density ρ and dynamic viscosity μ are run through the porous media under a constant hydraulic gradient dh/dl the following proportionality relationships were observed:

$$q \alpha d^2$$

$$q \alpha \rho g$$

$$q \alpha \frac{1}{\mu}$$

where: d is the mean grain diameter
ρ is the fluid density
μ is the dynamic viscosity
g is the acceleration due to gravity

Knowing that q is proportional to d^2, ρg, and $1/\mu$ we can write

$$K = \frac{C\rho g d^2}{\mu}$$

where: C is a new constant of proportionality

so:

$$q = -\frac{C\rho g d^2}{\mu}\frac{dh}{dl}$$

where: Cd^2 characterizes the properties of the medium

C represents unknowns such as:

1) the distribution of grain sizes
2) sphericity and roundness of grains
3) packing of grains

$\rho g/\mu$ characterizes the properties of the fluid

If we define $k = Cd^2$ as a function of the medium alone then

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

where: k is known as the intrinsic permeability of the medium $[L^2]$

HETEROGENEITY AND ANISOTROPY OF HYDRAULIC CONDUCTIVITY

A heterogeneous formation is one in which hydraulic conductivity values vary through space such that K is dependent upon position within the formation.

A homogeneous formation is one in which hydraulic conductivity values are independent of position within a formation.

An anisotropic formation is one in which K values vary with direction of measurement at any given point in a formation.

An <u>isotropic formation</u> is one in which K values are independent of the direction of measurement at a point in a formation.

In an xyz coordinate system with coordinate directions corresponding to principal directions of anisotropy (K_x, K_y, K_z):

At any point in an isotropic formation

$$K_x=K_y=K_z$$

In an anisotropic formation

$$K_x \neq K_y \neq K_z$$

Figure 1.1 illustrates four possible combinations of heterogeneity and anisotropy.

TRANSMISSIVITY AND STORATIVITY OF CONFINED AQUIFERS

<u>Storativity</u> of a confined aquifer of thickness b is defined as the volume of water that an aquifer releases from storage per unit surface area of aquifer per unit of decline in the component of hydraulic head normal to that surface.

For confined aquifers, storativity (S) ranges from about 0.005 to 0.00005 (dimensionless).

<u>Transmissivity</u> of a confined aquifer is equal to the product of hydraulic conductivity (K) times the aquifer thickness (b).

TRANSMISSIVITY AND STORATIVITY OF UNCONFINED AQUIFERS

<u>Storativity</u> of an unconfined aquifer is the specific yield (S_y). <u>Specific yield</u> is defined as the volume of water that an unconfined aquifer releases from storage per unit area of aquifer per unit decline in the water table.

The usual range for S_y is 0.01 to 0.30.

<u>Transmissivity</u> for unconfined aquifers is not as well defined as for confined aquifers.

b for unconfined aquifers equals the saturated thickness which changes as the position of the water table changes $T = Kb$.

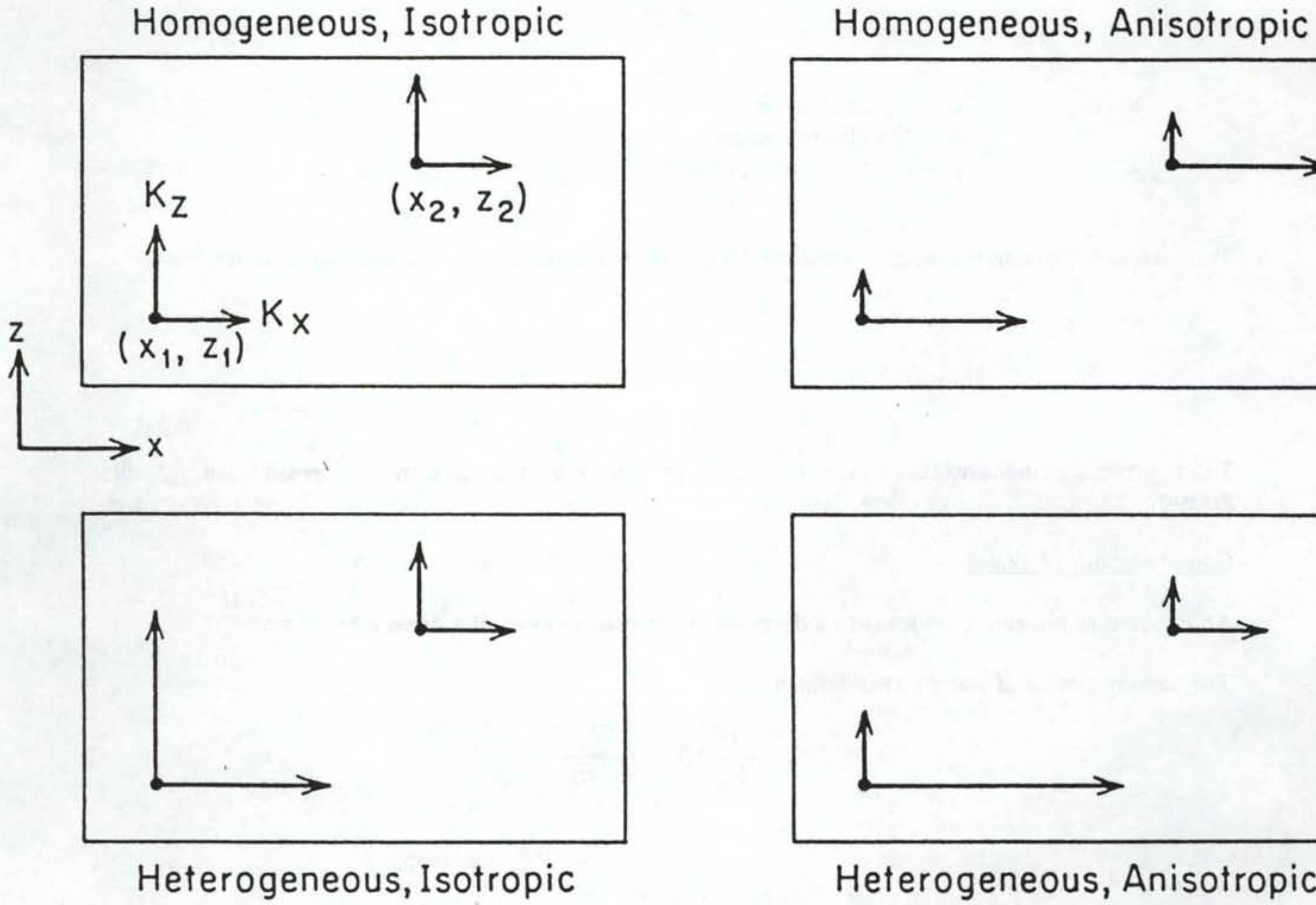


FIGURE 1.1 FOUR POSSIBLE COMBINATIONS OF HETEROGENEITY AND ANISOTROPY (AFTER FREEZE AND CHERRY, 1979)

COMPRESSIBILITY AND EFFECTIVE STRESS

The total vertical stress acting on a horizontal plane at any depth below the land surface equals

$$\sigma = \bar{\sigma} + p$$

where:
σ = the total stress
$\bar{\sigma}$ = the effective stress
p = the pore water pressure

Total stress (σ) due to the weight of the overlying rock and water usually is essentially constant.

so

$$d\bar{\sigma} = -dp$$

The negative sign indicates that a decrease in fluid pressure is accompanied by an increase in intergranular pressure.

Compressibility of Water

An increase in pressure (dp) leads to a decrease in the volume (V_w) of a given mass of water.

The compressibility of water (β) is defined as

$$\frac{1}{E_w} = \beta = -\frac{dV_w/V_w}{dp}$$

or for a given mass of water

$$\beta = -\frac{d\rho/\rho}{dp}$$

where:
E_w is the bulk modulus of compression of the fluid
dp is the change in pore water pressure
ρ is the fluid density
The negative sign is necessary for β to be positive.

Compressibility of a Porous Medium

$$\alpha = -\frac{dV_t/V_t}{d\bar{\sigma}} = \frac{1}{E_s}$$

where: E_s is the bulk modulus of compression of the aquifer skeleton
V_t is the total volume of the porous medium
$d\bar{\sigma}$ is the change in effective stress

$$V_t = V_s + V_v$$

where: V_s is the volume of the solids
V_v is the volume of the voids

If we assume the change in the volume of the solids (dV_s) = 0, so

$$dV_t = dV_s + dV_v$$

then,

$$dV_t = dV_v$$

A decrease in hydraulic head (h) where:

$$h = z + \psi$$

infers a decrease in fluid pressure (p) and an increase in effective stress ($\bar{\sigma}$).

Water is produced from storage in a <u>confined aquifer</u> under the conditions of decreasing (h) by two mechanisms:

1) Compaction of the aquifer caused by increasing $\bar{\sigma}$.
2) The expansion of water caused by decreasing p.

The first mechanism is controlled by the aquifer compressibility (α)

The second mechanism is controlled by the fluid compressibility (β).

By the First Mechanism

$$dV_w = -dV_t = \alpha V_t d\bar{\sigma}$$

amount of water produced	=	volumetric reduction of the aquifer	=	compress-ibility of the aquifer	x	volume of the aquifer	x	change in effectiveness

For a unit volume V_t = 1

$$d\sigma = -\rho g dh$$

For a unit decline in head dh = -1

$$dV_w = \alpha \rho g$$

By the Second Mechanism

$$dV_w = -\beta V_w dp$$

volume of water produced	=	compressibility of water	x	volume of water	x	change in pressure

The volume of water V_w in the total unit volume V_t is nV_t where n is the porosity.

with $V_t = 1$, and

$$dp = \rho g[d\psi] = \rho g[d(h-z)] = \rho g[dh]$$

For a unit decline in head dh = -1

$$dV_w = \beta n \rho g$$

Specific Storage (S_s) is the sum of

$$S_s = \alpha \rho g + \beta n \rho g$$

$$S = S_s b = \rho g b(\alpha + n\beta)$$

Specific Storage (S_s) of the saturated aquifer is the volume of water that a unit volume of aquifer releases from storage under a unit decline in hydraulic head.

Example
A 20-foot thick sandstone aquifer extends over an area of 2×10^7 ft.^{2} Assume a 100 foot thick clay bed overlies the sandstone. The clay has a saturated weight density (ρg) or specific weight of 130 lbs/ft.^{3}

The original head in the aquifer stands 50 feet above land surface. The specific weight of water equals 62.4 lbs/ft.^{3}

The total vertical stress acting on a unit area at the bottom of the confining layer is

Total stress (σ) = 130 lbs/ft^3 (100 ft.) = 13000 lbs/ft.^{2}

The fluid pressure (pore water pressure) at the same point is

$p = 62.4$ lbs/ft.^{3} $(H + h) = 62.4$ lbs/ft.^{3} (100 ft. + 50 ft.)

where: H is the thickness of the clay
h is the height of the water level above land surface

$$p = 9360\,lbs/ft.^2$$

The effective stress at the same point is

$$\sigma - p = 13000\,lbs/ft.^2 - 9360\,lbs/ft.^2$$

or

$$\overline{\sigma} = 3640\,lbs/ft.^2$$

Therefore, 9360 lbs/ft.^{2} of the total stress exerted by the confining layer (claybed) is supported by the water in the aquifer and only 3640 lbs/ft.^{2} is supported by the aquifer skeleton.

If, after a long period of pumping, the artesian head is lowered uniformly about 40 ft., the fluid pressure (p) in the aquifer will decrease as follows:

$$\rho g \Delta h = 62.4\,lbs/ft.^3 (40\,ft.) = 2496\,lbs/ft.^2$$

Because

$$\sigma = \overline{\sigma} + p$$

The new effective stress ($\overline{\sigma}$) must increase by the same amount.

So at the end of pumping, 6864 lbs/ft.^{2} of the total stress exerted by the confining layer is supported by the water in the aquifer, and 6136 lbs/ft.^{2} is supported by the aquifer skeleton.

Terzaghi (1925) found that the porosity of sandstone decreases 0.013 percent/1 percent load increase.

The percentage load increase for the example is the change in effective stress ($\Delta\overline{\sigma}$) to the original effective stress ($\overline{\sigma}$).

or

$$\frac{2496\,lbs/ft.^2}{3640\,lbs/ft.^2} = 68.6\%$$

So the percent porosity decrease = 68.6 x 0.013 = 0.89%.

Therefore, if the original porosity is 10%, the final porosity is about 9%.

A 10% porosity value is equivalent to 2 feet of void space for each 20 feet vertical column of aquifer. A 1% decrease in porosity per column of aquifer is about 0.2 feet.

$$E_s = \frac{d\bar{\sigma}}{dz/z} = \frac{2496\,lbs/ft.^2}{0.2ft./20ft.}$$

$$E_s = 250{,}000\,lbs/ft.^2$$

where: E_s is the bulk modulus of compression for the aquifer
z is the aquifer thickness
$\bar{\sigma}$ is the effective stress

The vertical compressibility of the aquifer is:

$$\alpha = \frac{1}{E_s} = \frac{1}{250{,}000\,lbs/ft.^2} = 4x10^{-6}\,ft.^2/lb$$

The component of storativity (S_a) attributed to aquifer compressibility is

$$S_a = b\rho g\alpha = (20ft.)(62.4lb/ft.^3)(4\times10^{-6}ft.^2/lb)$$

$$S_a = 5\times10^{-3}$$

With the use of the original porosity of 10% and a constant coefficient of fluid compressibility (2.3×10^{-8}ft.^{2}/lb), the component of storativity (S_f) attributed to expansion of the fluid is

$$S_f = b\rho g n\beta = (20ft.)(62.4lbs/ft.^3)(0.10)(2.3x10^{-8}ft.)$$

$$S_f = 2.96x10^{-6}$$

Storativity (S) = $S_a + S_f$

$$S = 5.00296x10^{-3}$$

GROUND WATER FLOW EQUATIONS

Steady-State Conditions

Remember that for an isotropic medium

$$K_x = K_y = K_z$$

if the medium also is homogeneous

$$K(x,y,z) = constant$$

The equation for steady-state ground water flow in a homogeneous, isotropic porous medium is:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

This equation is the Laplace Equation. The solution of the equation is a function h(x, y, z) that describes the value of the hydraulic head (h) at any point in a three-dimensional flow field.

Transient (non-steady state) Conditions

The equation for non-steady state ground water flow in a homogenous, isotropic porous medium can be written as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_s}{K} \frac{\partial h}{\partial t}$$

If we expand S_s, we get

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{\rho g(\alpha + n\beta)}{K} \frac{\partial h}{\partial t}$$

This equation is known as the Diffusion Equation.

The solution to the diffusion equation is a function, h (x, y, z, t), that describes the value of hydraulic head at any point in the flow field at any time.

Solution requires knowledge of K, α, and n of the porous medium

and

and β of the fluid.

For the special case of a homogeneous, isotropic confined aquifer of thickness b,

$$S = S_s b \text{ and } T = Kb$$

The two dimensional form of the diffusion equation is

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S}{T} \frac{\partial h}{\partial t}$$

Solution to the two dimensional form of the diffusion equation requires knowledge of S and T for the aquifer.

For homogeneous, isotropic unconfined aquifers, the equation will have the following form for two dimensional flow:

$$\frac{\partial^2 h}{\partial x^2}\,\frac{\partial^2 h}{\partial y^2}=\frac{S_y}{Km}\,\frac{\partial h}{\partial t}$$

where: S_y is the specific yield
K is the hydraulic conductivity
m is the average saturated thickness of the aquifer

SESSION II

AQUIFER TEST CONSIDERATIONS

I. Antecedent Trends *[fig. 2.1]*

A. Water levels measured during an aquifer test include:
   1. Water levels changes that occur regardless of an aquifer test.
   2. Water level lowering caused by pumping a well.

B. Water level trend can be extrapolated.
   1. Recharge--rising water levels
      a. Precipitation
      b. Rising stream stage
   2. Discharge--lowering water levels
      a. Draining down gradient during dry periods.
      b. Water pumped from the aquifer.

C. Separate antecedent trend from test drawdowns:
   1. Measure the water levels in test wells and piezometers:
      a. Days before an aquifer test.
      b. Days after an aquifer test.
   2. Draw a corrected aquifer-test hydrograph.
      a. Plot the water levels before and after an aquifer test.
      b. Extrapolate water levels during the test that would have occurred without the test.
   3. Remove antecedent trend from each aquifer test water-level measurement.
      a. Decreasing antecedent water level trend

$$s=s_1-|h_o-h_1|$$

      b. Rising antecedent water level trend

$$s=s_1+|h_o-h_1|$$

      c. Variables

         s = corrected drawdown
         s_1 = measured drawdown
         h_o = water level at test start
         h_1 = extrapolated water level without pumping

D. Readings
   1. Fetter, p. 206-207
   2. Heath, p. 34
   3. Kruseman and De Ridder, p. 179-180
   4. Stallman, p. 17
   5. U.S. Bureau of Reclamation, p. 230

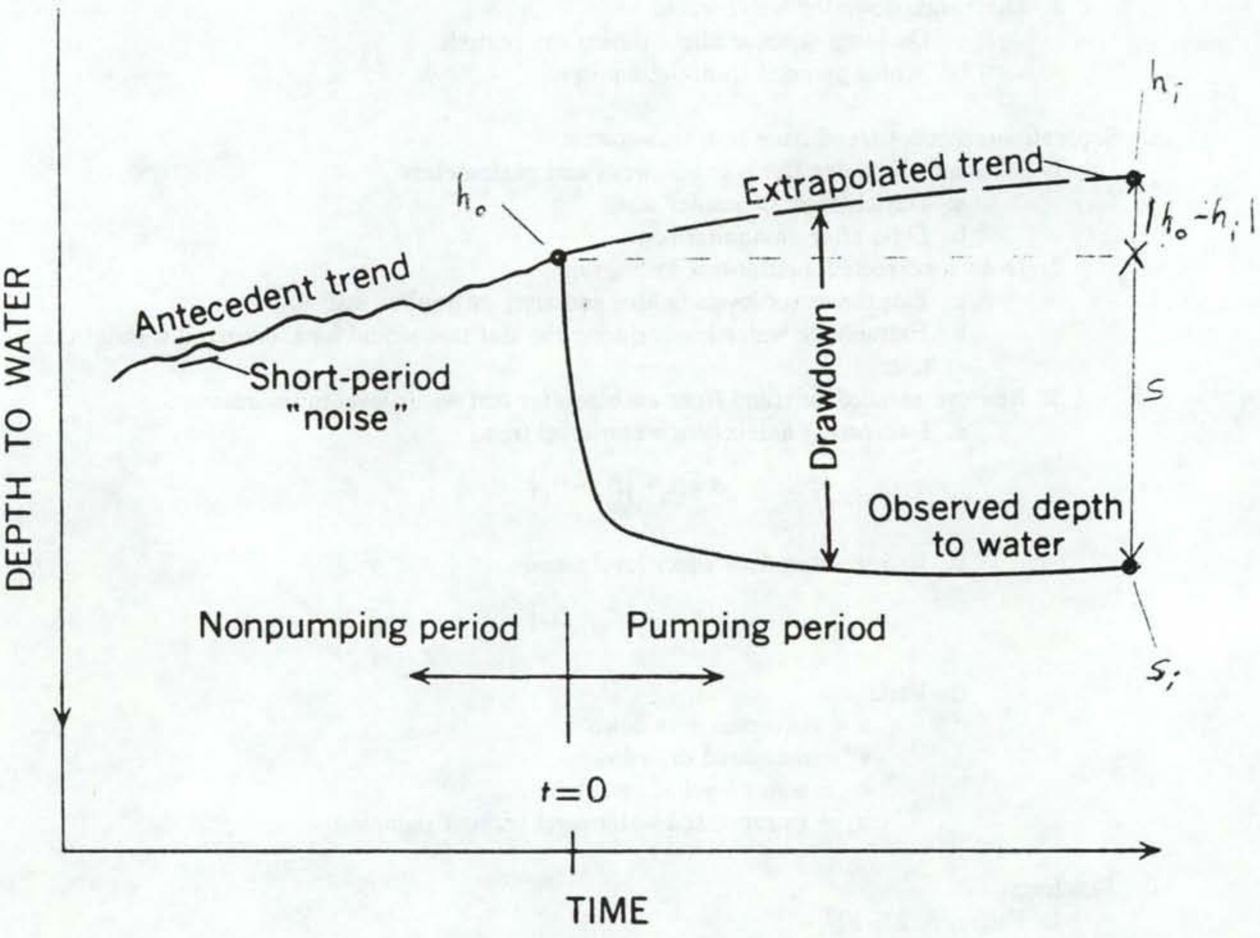


Figure 2.1 **Hydrograph for observation well showing antecedent trend before pumping begins and extrapolated during pumping (Stallman, fig. 4).**

II. Barometric Effects *[fig. 2.2]*

A. Water level decreases in a confined aquifer as atmospheric pressure increases.

B. Cause *[fig. 2.3]*

C. Barometric efficiency

$$BE = \frac{S_w}{S_b} \times 100$$

where:

S_w = water level change
S_b = barometric pressure change

D. Determine relation for water level and atmospheric pressure in the weeks before test. *[fig. 2.4]*

1. Plot on arithmetic graph paper:
    a. Water level change, S_w
    b. Atmospheric pressure change, S_b
2. Draw best-fit straight line through data.
3. Calculate BE from any point on line.

E. Correct drawdown data from test.

1. Example for 50% barometric efficiency.
    a. 0.1-inch atmospheric pressure drop corresponds to a 0.05-inch water-level rise.
    b. Subtract 0.05 inch from test data.

F. Readings

1. U.S. Bureau of Reclamation, p. 154-157
2. Fetter, p. 206-207
3. Freeze and Cherry, p. 234
4. Price, p. 72-73
5. Stallman, p. 15
6. Todd, p. 235-238

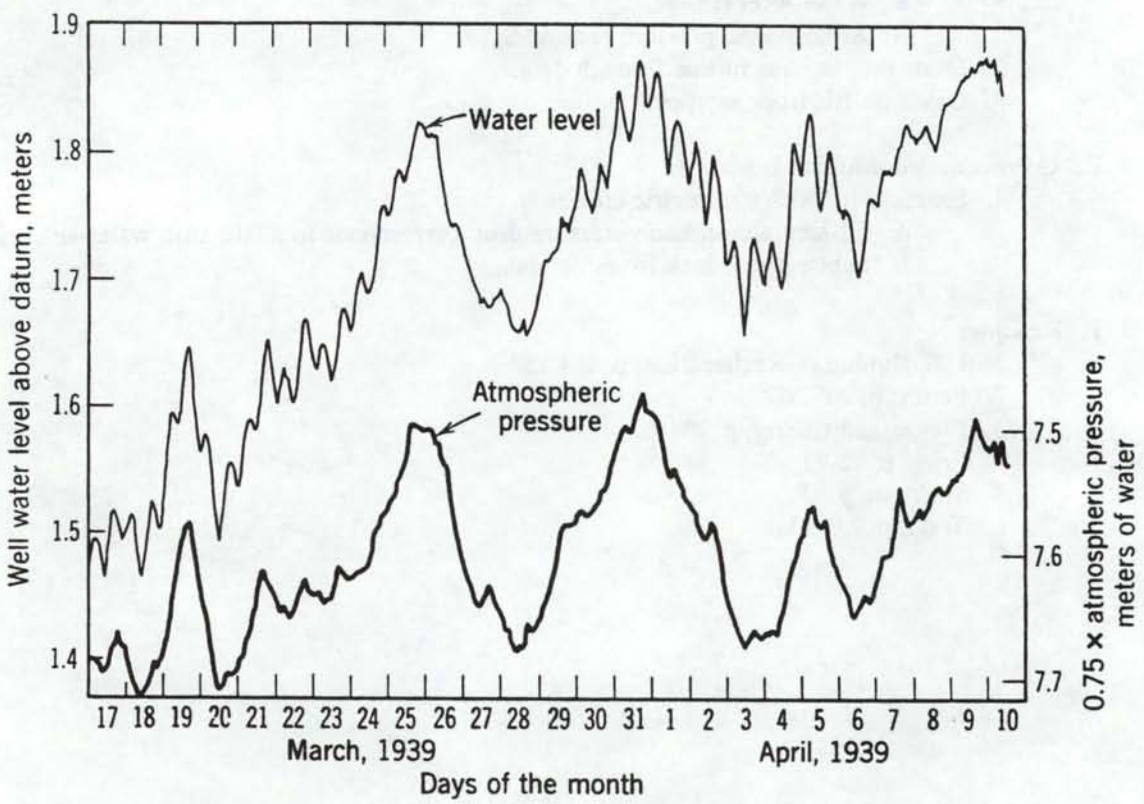


Figure 2.2 Water-level response in a well completed in a confined aquifer to atmospheric pressure changes, showing a 75 percent barometric efficiency (Todd, fig. 6.17).

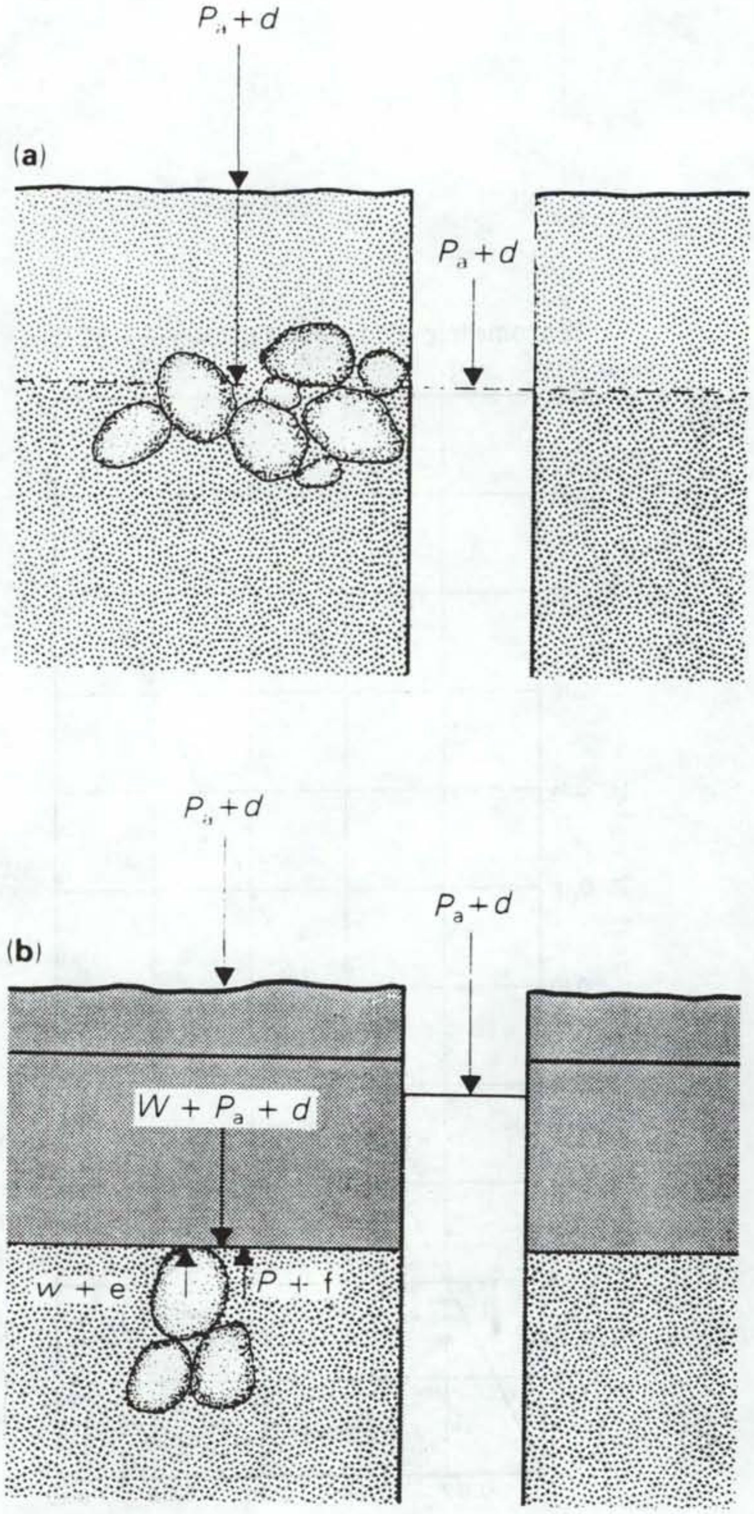


Figure 2.3 **Barometric effects** (a) In an unconfined aquifer any increase *d* in atmosphere pressure P_a is transmited equally to water in the aquifer and to water in well. (b) In a confined aquifer, some of the increase *e* is taken by the aquifer grains, and the remainder *f* is taken by the pore water. In the well however the full increase *d* is transmitted to the water surface, forcing some water from the well into the aquifer (Price, fig. 7.4).

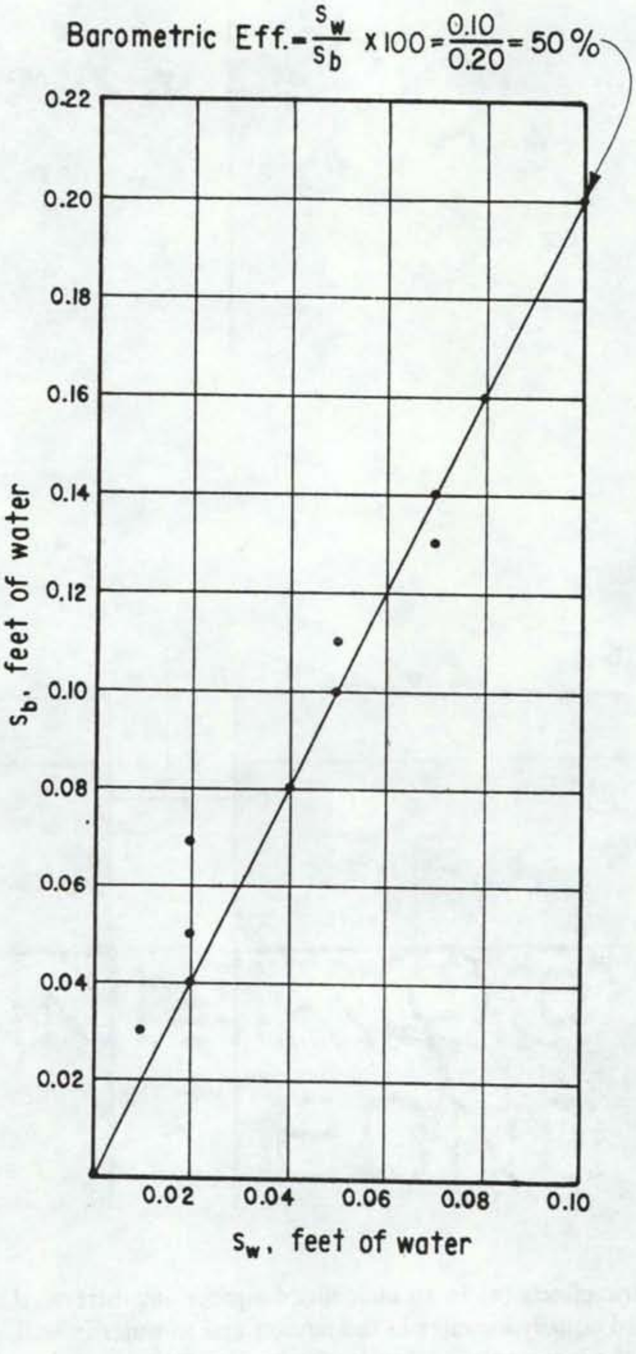


Figure 2.4 Relation of water levels S_w and barometric presure S_b (U.S. Bureau of Reclamation, fig. 5-24).

III. Earth Tide Effects *[fig. 2.5]*

A. Affects confined aquifers and caused by the attraction between the earth's crust and the moon.

B. Correction
   1. Record and plot ground-water levels before an aquifer test.
   2. Determine if water levels are cyclic after separating out:
      a. Antecedent trends
      b. Barometric effects
   3. Correct using isolated tide cycle. *[fig. 2.6]*
      a. Subtract spring tide effects.
      b. Add ebb tide effects.

C. Reading
   1. Fetter, p. 156-157
   2. Kruseman and De Ridder, p. 180-181
   3. Todd, p. 247

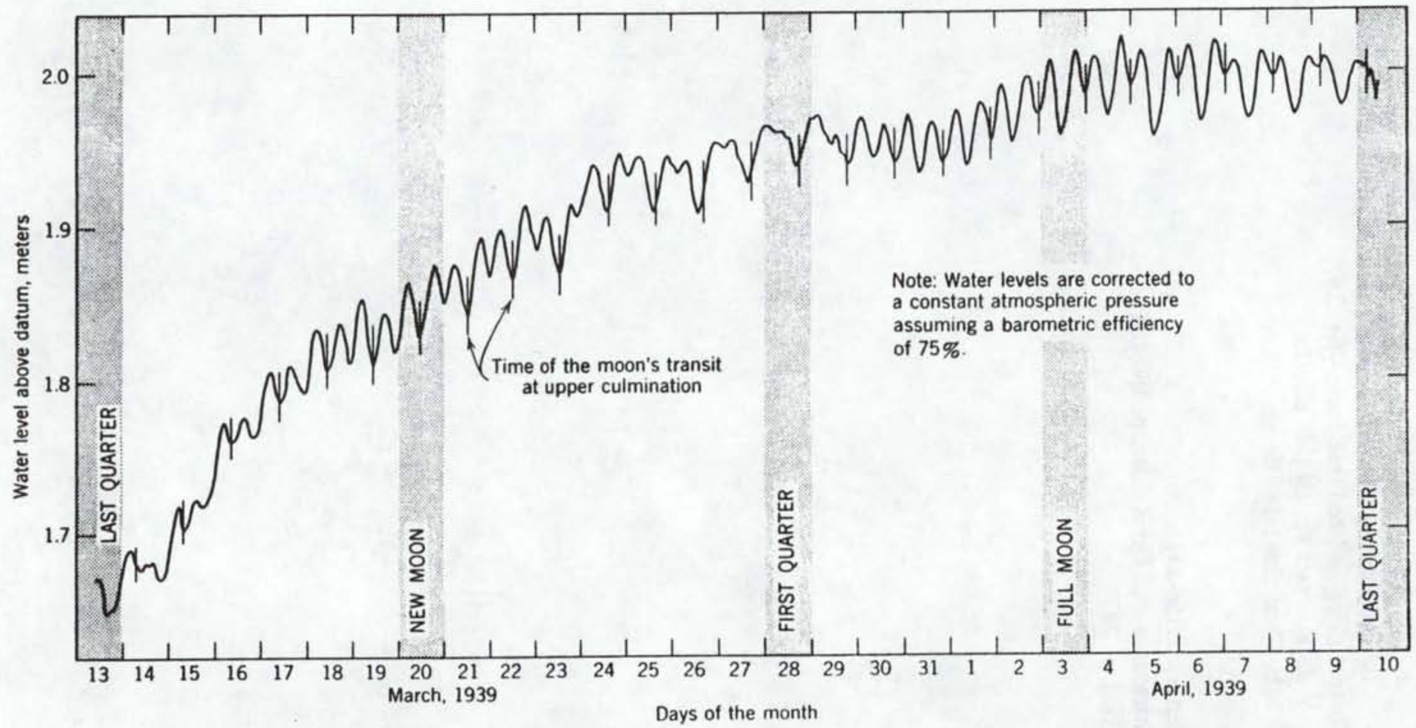


Figure 2.5 **Water level fluctuations in a confined aquifer produced by earth tides (Todd, fig. 6.28).**

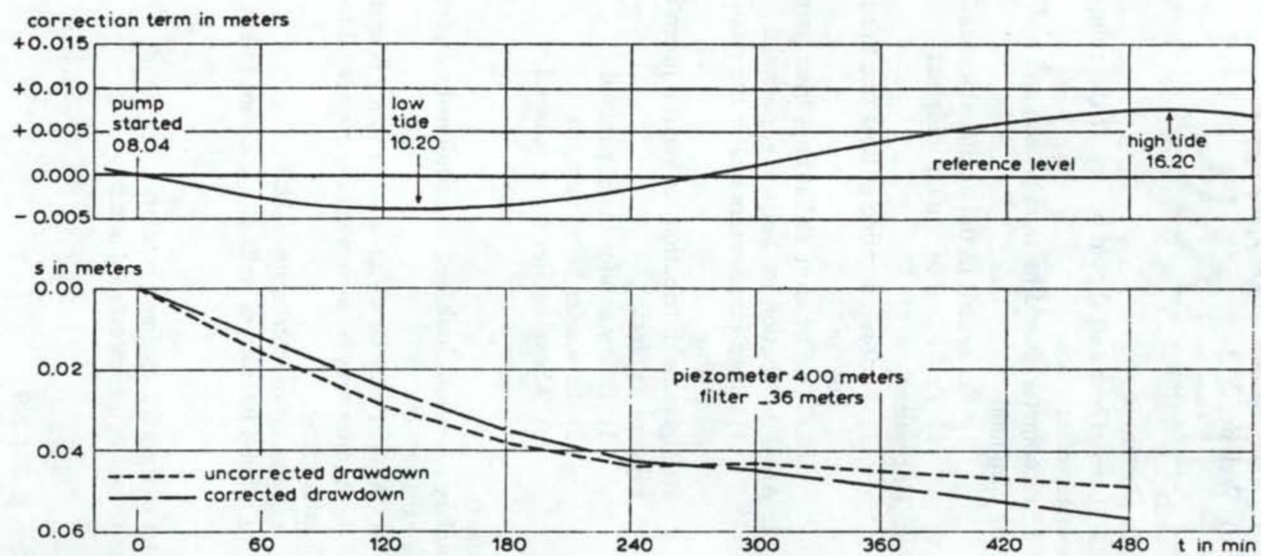


Figure 2.6 Correction of piezometer data for tidal influence. The upper part of the figure shows the curve of the ground-water tide under non-pumping conditions. Vertical scale differs in upper and lower graphs (Kruseman and DeRidder, fig. 61).

IV. Test Design

A. Pumped well
   1. Reliable pump and power supply.
   2. Access needed to measure water level.
   3. Identify well completion.
      a) Open-section's length
      b) Open-section's diameter
      c) Open-section's stratigraphic position
      d) Well depth

B. Observation wells
   1. Surface placement
      a. Unconfined well should be nearer the pumping well than a confined well.
   2. Piezometer completion
      a. Piezometer should be sand packed and hydrologically connected with aquifer.
      b. Confined
         1) Perforated to full aquifer thickness, or
         2) Perforated to aquifer midpoint
      c. Unconfined
         1) Perforated casing at least one third from static water table to aquifer bottom.
         2) Two or more radial lines from pumping well.
      d. Areal distribution for horizontal potential
         1) Place piezometers nearer the pumping well for unconfined aquifer testing.
      e. Stratigraphic distribution for vertical potential; indicate any leakage from adjacent aquifers.
         1) Below aquifer being pumped
         2) In aquifer being pumped
         3) Above aquifer being pumped

C. Other conditions
   1. Boundary conditions limit test assumption--be aware that examples such as the following may exist:
      a. Aquifer may be within a small valley bounded by bedrock mountains.
      b. Aquifer may be hydrologically connected to a stream or lake.
   2. Well interference
      a. Shut down interfering wells.
      b. Pump interfering wells at a constant rate.

D. Duration
   1. Hours to days for confined aquifer.
   2. Days to weeks for unconfined aquifer.

E. Readings
   1. Fetter, p. 204-209
   2. Heath, p. 34-35
   3. Kruseman and De Ridder, p. 26-41
   4. Stallman, p. 7-8
   5. U.S. Bureau of Reclamation, p. 228-245V.

V. Response Prediction

A. Purpose
   1. Determines if wells and piezometers are to be analyzed as:
      a. Confined
      b. Semi-confined
         1) With no storage in semi-confining layer
         2) With storage in the semi-confining layer
      c. Unconfined
   2. Determines how test design, such as pumping rate and piezometer locations, will produce a complete type curve response in water-level data.

B. Aquifer tests are done to determine:
   1. Transmissivity
      a. Determined using late, flattening drawdown data.
   2. Storativity
      a. Determined using early, steep drawdown data.

C. Method
   1. Determine aquifer-test controls.
      a. Estimate hydraulic conductivity.
      b. Estimate storativity.
      c. Propose pumping rate.
      d. Determine aquifer thickness.
      e. Measure distances from pumping well to observation well.
   2. Produce a plot of water-level response from, estimated, proposed, and known values.
      a. Substitute estimate and proposed values into appropriate analysis-method equation, such as the Theis equation.
      b. Plot resulting drawdown at various times.
   3. Analyze results
      a. Where plot includes incomplete range, steep to flattening, drawdown curve.
         1) Revise
            a) Pumping rate
            b) Observation well distances
      b. Where plot includes complete range, steep to flattening, drawdown curve.
         1) Use the proposed test controls for aquifer test.

D. Readings
   1. Stallman, p. 8-11

VI. Field Observation

A. Draft a plan-view map of wells, piezometers, streams, bedrock, and landmarks.
   1. Record distance from pumping well to observation wells to within 0.5%.

B. Record during test: *[fig. 2.7]*
   1. Water-level measurements
      a. Water level
         1) Steel tape
         2) Electric tape
         3) Transducer(s) and data logger
         4) Water-level recorder
      b. Accuracy
         1) Within 0.01 ft
      c. Measurement point
         1) Clearly label the measuring point.
         2) Reference all measurements to this point.
         3) Survey measuring point elevations if aquifer analysis includes distance drawdown plots.
   2. Time
      a. Synchronize digital watches before test to nearest second.
   3. Discharge
      a. Monitor most frequently at test start.
      b. Do not let pumping rate vary more than 10%.

D. Measurement frequency
   1. 10 measurements, evenly spaced, per log cycle

E. Readings
   1. Fetter, p. 204-209
   2. Stallman, p. 11-17

AQUIFER-TEST DESIGN, OBSERVATION, AND DATA ANALYSIS

County :______________________ Observation well no.________

Location:_____________________ Pumped well no. ____________

Average Q__________ gpm r =________ft. r^2 =________

Date	Hour	t (min)	t′ (min)	t/t′	Depth to water	s (unad-justed)	Adjust-ment Δs	s′ (ad-justed)	Q (gpm)	Remarks

Figure 2.7 **Sample data record form (Stallman, fig. 3).**

VII. Discharge

A. Pipe pumped water away from well field.
   1. Especially for aquifer testing shallow, unconfined aquifers.

B. Monitor discharge.
   1. Methods
      a. Circular orifice with manometer
      b. Constant head valve
      c. Weirs and flumes
   2. Frequency
      a. 5, 10, 20, 30, 60, 120, 240, 480, 720, 1440 minutes.
      b. Once a day after first day

C. Maintain pumping at constant rate.
   1. Do not vary pumping rate more than 10%.

D. Readings
   1. U.S. Bureau of Reclamation, p. 229-230 and 233-242
   2. Stallman, p. 16-17

VIII. Drawdown Corrections

A. Recorded data must be corrected as final data that can be used for analysis.

B. Plot:
   1. Time/drawdown for each observation well.
   2. Time/discharge for pumped well.

C. Revise data.
   1. Separate out antecedent water-level trend.
   2. Correct for identifiable anomalies.
      a. Barometric trends
      b. Tide cycles
   3. Remove unidentifiable anomalies.
      a. Measurement error
      b. Train
      c. Blasts

D. Plot final hydrograph for analysis.

E. Readings
   1. Kruseman and De Ridder, p. 179-180
   2. Stallman, p. 17

IX. Recovery *[fig. 2.8]*

A. Recovery is an aquifer test that starts when the pump is shut down and water levels rebound upwards.

B. Purpose
   1. Independently checks the transmissivity and storativity values determined during pumping period.
   2. "Discharge", the average pumping rate during the pumping period, is constant during recovery.
      a. A gift when the pumping rate has been erratic.

C. Record
   1. Water level recovery.
   2. For same length of test or less.
      a. extended water levels of ground-water trends
   3. Time

D. Readings
   1. U.S. Bureau of Reclamation, p. 245
   2. Freeze and Cherry, p. 329
   3. Todd p. 131-134

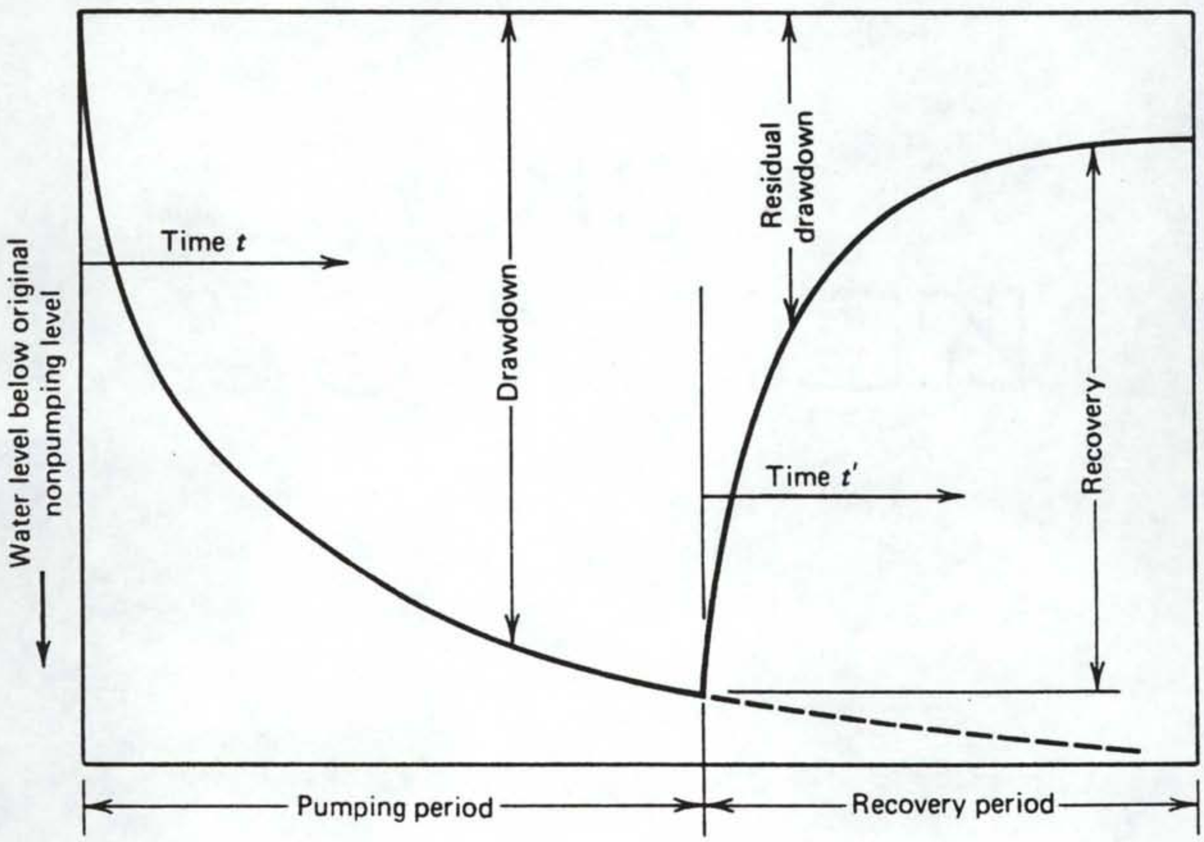


Figure 2.8 Drawdown and recovery curves in an observation well near a pumping well (Todd, fig. 4.13).

SESSION III

THIEM AND THEIS EQUATIONS

The two dimensional form of the diffusion equation for a homogeneous isotropic confined aquifer of thickness b is:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=\frac{S}{T}\frac{\partial h}{\partial t}$$

The equivalent equation for radial flow to well in a homogeneous, isotropic confined aquifer of thickness b is:

$$\frac{1}{r}\frac{\partial h}{\partial r}+\frac{\partial^2 h}{\partial r^2}=\frac{S}{T}\frac{\partial h}{\partial t}\ [L^{-1}]$$

where:

$$r=\sqrt{x^2+y^2}$$

For steady radial flow $dh/dt = 0$ and the equation becomes

$$\frac{1}{r}\frac{\partial h}{\partial r}+\frac{\partial^2 h}{\partial r^2}=0\ [L^{-1}]$$

In dimensionless form the equation is written

$$\frac{\partial h}{\partial r}+\frac{\partial^2 h}{\partial r^2}r=0$$

Figure 3.1 shows that the cross sectional area available for flow decreases toward the pumping well (i.e., at decreasing values of r). Therefore, the hydraulic gradient must increase near the well.

<u>Thiem Equation</u>

The shape of the cone of depression formed by the pumping well can be determined by Darcy's Law.

To do this, we must express Darcy's Law in polar coordinates with the pumping well at the origin.

From Darcy's Law, the quantity of water (Q) moving toward the pumping well at any distance r from the well is:

$$Q=KA\frac{dh}{dl}=2\pi rbK\frac{dh}{dr}$$

for flow to a cylindrical well

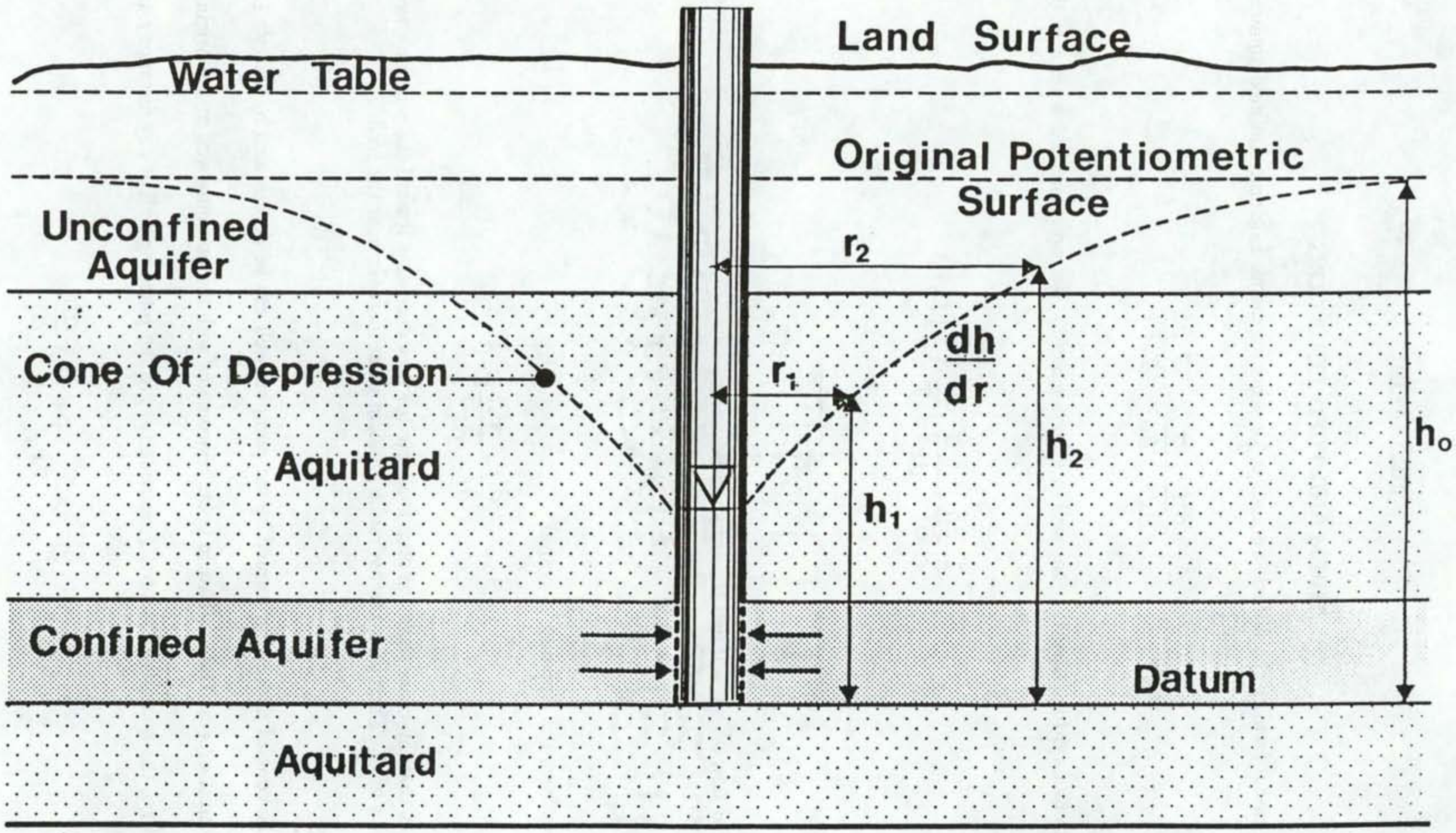


FIGURE 3.1 EXAMPLE OF A CONE OF DEPRESSION FORMED BY PUMPING A CONFINED AQUIFER

where: r is the radial distance from the well
b is the aquifer thickness
$2\pi rb$ is the surface area of a cylinder of radius r and height b

Rearranging the equation in order to integrate we can write

$$dh = \frac{Q}{2\pi}\frac{dr}{rbK}$$

If we integrate between any two points on the cone of depression, located at distances r_1 and r_2 from the pumping well, where the heads are h_1 and h_2, respectively, we obtain

$$\int_{h_1}^{h_2} dh = \frac{Q}{2\pi bK}\int_{r_1}^{r_2}\frac{dr}{r}$$

Remember from your calculus that

$$\int \frac{dr}{r} = ln.r$$

So we get

$$h_2 - h_1 = \frac{Q}{2\pi bK}(ln.r_2 - ln.r_1)$$

or

$$h_2 - h_1 = \frac{Q}{2\pi bK} ln(\frac{r_2}{r_1})$$

and

$$Q = \frac{2\pi bK(h_2 - h_1)}{ln(\frac{r_2}{r_1})}$$

This equation shows that the head in the vicinity of a pumping well varies linearly with the log of distance from the well.

This equation was developed by Thiem (1906) from Darcy's Law and is known as the Thiem Equation.

The Thiem Equation can be used to estimate the transmissivity of an aquifer after steady-shape or steady-state conditions have been reached.

Thiem Equation in a confined aquifer

If we have two observation wells at different distances from a pumping well we can estimate T.

EXAMPLE

Suppose we have two observation wells located 100 ft. (r_1) and 300 ft. (r_2), respectively, from a pumping well in a confined aquifer.

The pumping well has been pumping at a constant rate of 500 gallons per minute for several days (i.e., long enough to establish steady-shape conditions).

The measured drawdown in observation well 1 is 3.6 ft.; measured drawdown in observation well 2 is 1.7 ft.

So: Q = 500 gpm
r_1 = 100 ft.
r_2 = 300 ft.
s_1 = 3.6 ft.
s_2 = 1.7 ft.

Because we are only interested in the amount of drawdown from the original head at each point (observation well) we can rewrite the equation in terms of drawdown rather than head as follows:

From

$$Q=\frac{2\pi bK(h_2-h_1)}{ln(\frac{r_2}{r_1})}$$

realizing that $s_1 = h_0 - h_1$ and $s_2 = h_0 - h_2$

where: h_0 is the original head prior to pumping

we can write

$$Q=\frac{2\pi bK(s_1-s_2)}{ln(\frac{r_2}{r_1})}$$

where: $s_1 > s_2$

or

so:

$$T=\frac{Q[ln(\frac{r_2}{r_1})]}{2\pi(s_1-s_2)}$$

$$T=\frac{(500gpm/7.48gal/ft.^3)(1440min/day)[ln(300ft./100ft.)]}{2\pi(3.6ft.-1.7ft.)}$$

$$T=8860ft.^2/day$$

Thiem Equation in an unconfined aquifer

The cylindrical area of flow to a well in an unconfined aquifer is $2\pi r(b - s)$, where s is the dewatered drawdown and (b - s) equals h (saturated thickness of the aquifer).

The area of flow equals

$$A=2\pi rh$$

Applying Darcy's Law

$$Q=-K\frac{dh}{dr}2\pi rh$$

rearranging and integrating

$$\int_{h_1}^{h_2}hdh=\frac{Q}{2\pi K}\int_{r_1}^{r_2}\frac{dr}{r}$$

This gives

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

or

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

EXAMPLE

A 30 cm diameter well fully penetrates a 50 meter thick unconfined aquifer. After a long period of pumping at a rate of 0.03 m^3/s, the drawdowns in observation wells located 15 m and 45 m from the pumping well were 1.7 m and 0.8 m, respectively.

Determine the transmissivity.

Remember that the area of flow for an unconfined aquifer is A = 2πrh, where: h = b - s.

Solution:

$$Q=\frac{\pi K[(49.2m)^2-(48.3m)^2]}{ln(\frac{45m}{15m})}$$

where:

$$Q=0.03m^3/s$$

so

$$K=1.2\times10^{-4}m/s$$

and

$$T=Kb=(1.2x10^{-4}m/s)(60s/\text{min})(1440\text{min}/d)(50m)$$

$$T=520m^2/d$$

Use of the Thiem Equation requires satisfying the following assumptions:

1) Stabilized drawdown--steady state flow or at least steady shape

2) Constant aquifer thickness

3) Homogeneous, isotropic aquifer of infinite areal extent

4) Complete aquifer penetration by the pumping well and a 100% efficient well if using the pumping well to determine transmissivity.

5) Radial horizontal flow into the well

6) For unconfined flow, the hydraulic gradient must be constant with depth and equal to the slope of the drawdown curve at any point

Theis Equation

In 1935, Theis introduced an equation that describes nonsteady state flow to a well in a confined aquifer.

The equation, known as the Theis Equation is a solution to the polar-coordinate form of the non-steady state ground water flow equation.

$$\frac{\partial^2 h}{\partial r^2}+\frac{1}{r}\frac{\partial h}{\partial r}=\frac{S}{T}\frac{\partial h}{\partial t}$$

The Theis Equation can be written as:

$$h_0-h=s=\frac{Q}{4\pi T}\int_u^\infty \frac{e^{-u}}{u}du \qquad [\textit{known as the exponential integral}]$$

$$u=\frac{r^2S}{4Tt}$$

where: r = radial distance from the pumping well
S = storativity of the aquifer
s = drawdown in the aquifer

The solution to the integral is an infinite alternating series as follows:

$$h_0-h=s=\frac{Q}{4\pi T}[-0.5772-ln\;u+u-\frac{u^2}{2\times2!}+\frac{u^3}{3\times3!}-\frac{u^4}{4\times4!}+...]$$

The initial and boundary conditions for the Theis Equation can be written as follows:

Initial condition [h(r,t)] h(r,o) = h_0 for all r where h_0 is the constant initial hydraulic head.

Boundary conditions assume no drawdown in hydraulic head at the infinite boundary

$$h(\infty,t)=h_0 \qquad (\textit{for all } t)$$

and a constant pumping rate $Q[L^3/T]$ at the well (i.e., constant withdrawal rate at a well with an infinitesimally small diameter).

$$\lim_{r\to0}(r\frac{\partial h}{\partial r})=\frac{Q}{2\pi T} \qquad (\textit{for } t>0)$$

The exponential integral is a function only of the lower limit of integration so we can write the equation as follows:

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

where:

$$u=\frac{r^2S}{4Tt}$$

W(u) is termed the well function of u.

Values for W(u) versus u have been tabulated.

A plot of values of W(u) versus u or 1/u on log-log graph paper commonly is called the Theis type curve.

Figure 3.2 illustrates the Theis type curve.

A plot of drawdown (s) versus time (t) on log-log graph paper reveals the exact shape of the drawdown curve as a function of time for any fixed distance (r).

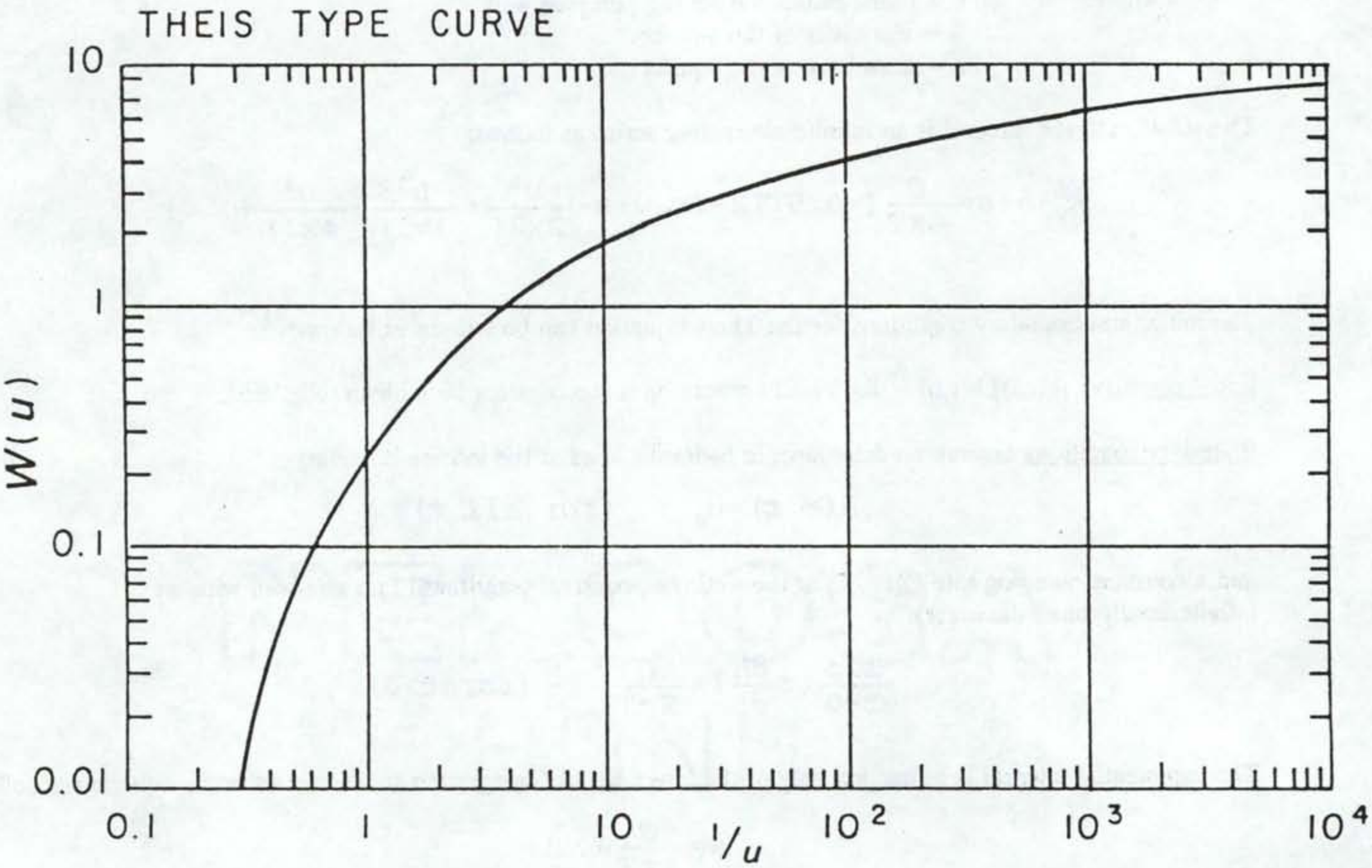


FIGURE 3.2 EXAMPLE OF THE THEIS TYPE CURVE (AFTER HEATH, 1983)

Values of s are dependent on the distance from the pumping well, the pumping rate, and the hydraulic properties of the aquifer.

A plot of drawdown (s) versus t/r^2 on log-log paper reveals the exact profile of a cone of depression as a function of distance (r^2) from a pumping well.

Values of s are dependent on the pumping rate, the hydraulic properties of the aquifer, and the time of observation.

The value of u depends on time, distance, transmissivity and storativity, and u determines the radius of the cone of depression.

Therefore, the radius of the cone of depression increases with time, but for a given time, is larger for smaller values of S and larger values of T.

Figure 3.3 compares drawdown cones at a given time for different values for T and S.
By examining the equations,

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

and

$$u=\frac{r^2S}{4Tt}$$

We can see that drawdown at any point for a given time is proportional to discharge and inversely proportional to T.

The lateral extent of a cone of depression at any given time, and its rate of growth are controlled by (u) and are independent of the pumping rate.

If drawdown (s) can be measured at a given distance (r) from a pumping well for several different values of time, and if the discharge is constant and known, T and S can be determined by graphical superposition.

Field data composed of drawdown (s) versus time (t) collected at a nonpumping observation well at a known distance (r) from a pumping well are plotted on log-log graph paper of the same scale as the type curve.

Figure 3.4 illustrates the Theis type curve matching process.

The data curve is superimposed on the type curve with the coordinate axes of the two curves kept parallel while matching the field data to the type curve.

Any point (i.e., your match point) on the overlapping sheets is selected arbitrarily (the point need not be on the matched curves).

The variable W(u) is related to 1/u in the same manner as the variable s, is to t.

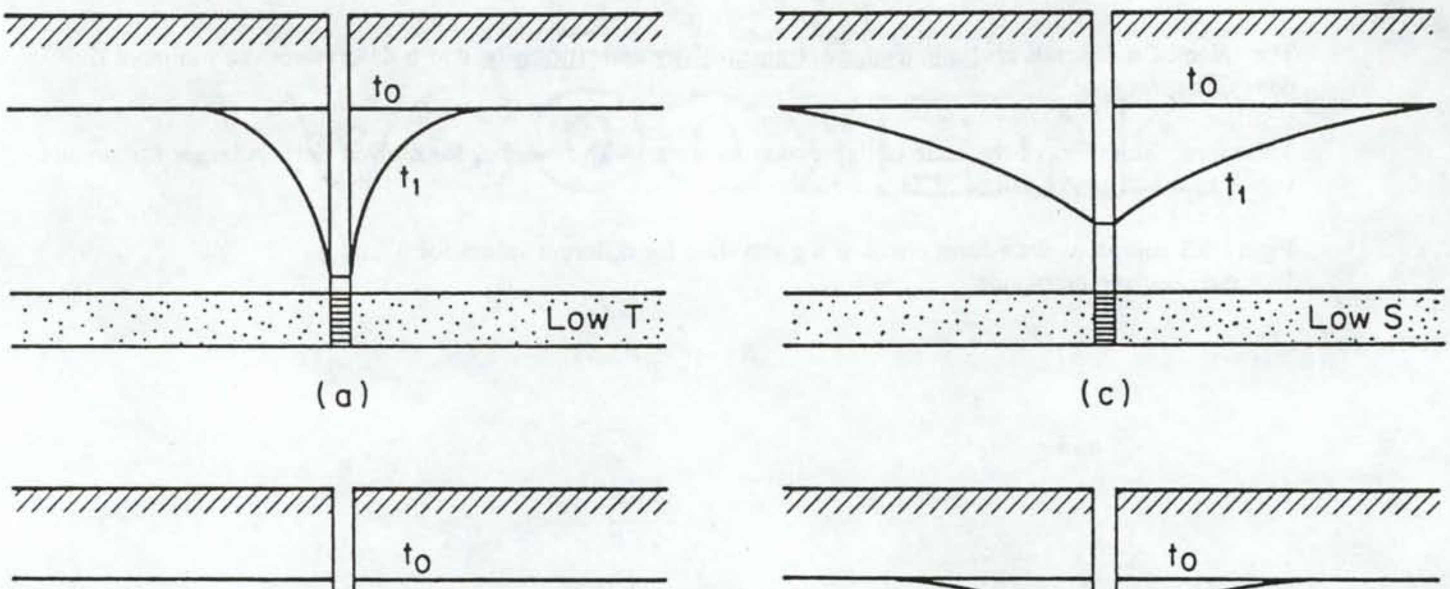


FIGURE 3.3 COMPARISON OF CONES OF DEPRESSION FOR DIFFERENT VALUES OF T AND S (AFTER FREEZE AND CHERRY, 1979)

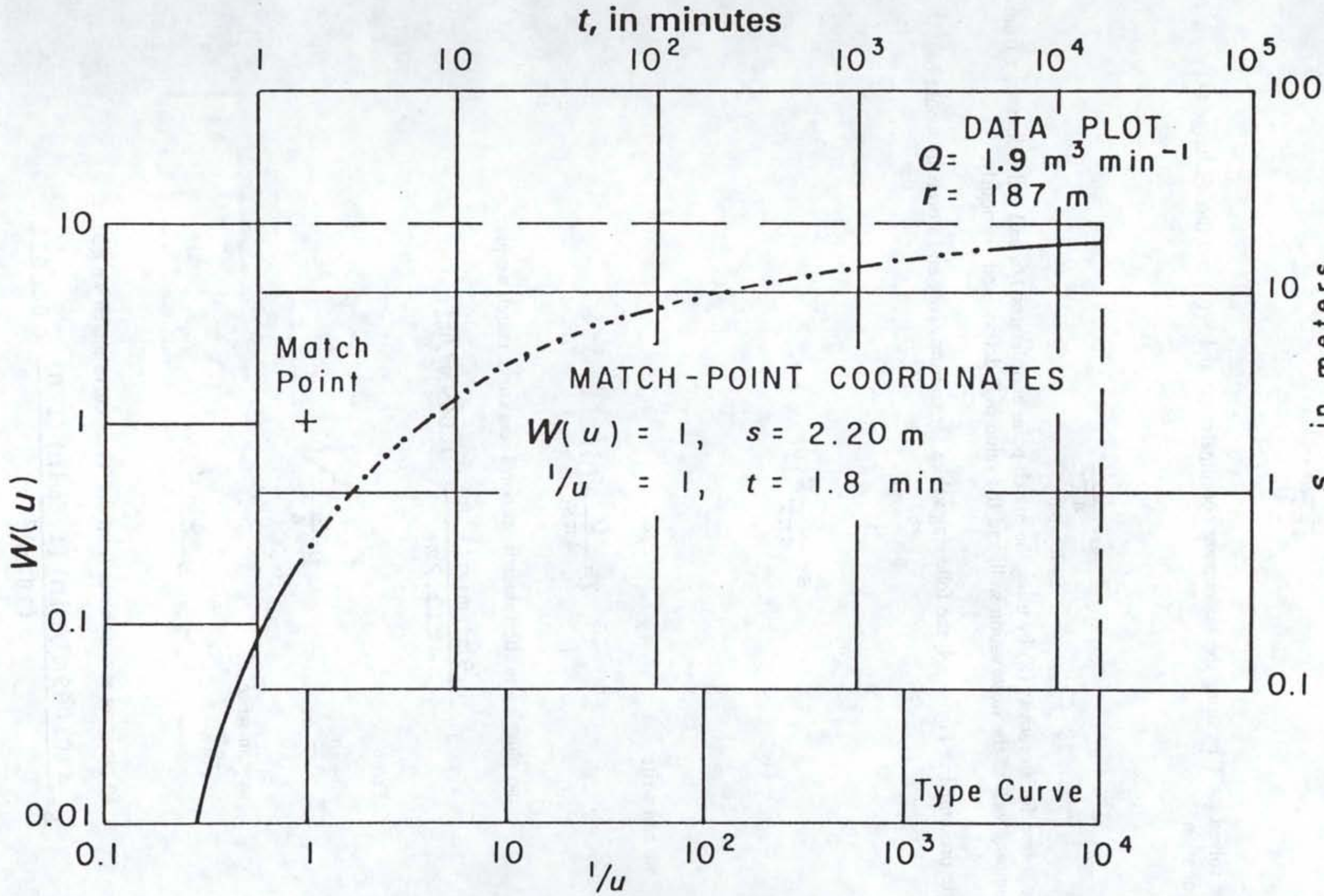


FIGURE 3.4 EXAMPLE OF THE TYPE CURVE MATCHING PROCESS (AFTER HEATH, 1983).

The equation

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

can be solved for T by using the matchpoint coordinates (s) and W(u) and the discharge Q. The equation

$$u=\frac{r^2S}{4Tt}$$

can be solved for storativity (S) by using the match-point coordinates 1/u and t, the distance r from the pumping well to the observation well, and the value of T determined previously.

For the example presented in Figure 3.4, the following values for transmissivity and storativity would be calculated:

From

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

we can write

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

so plugging in the values from our Theis curve match we get:

$$T=\frac{1.9m^3/\text{min}(1.0)}{4\pi(2.2m)}=0.069m^3/\text{min}$$

From

$$u=\frac{r^2S}{4Tt}$$

we can write

$$S=\frac{4Ttu}{r^2}$$

so plugging in the values from our Theis curve match we get:

$$S=\frac{4(0.069m^3/\text{min})(1.8\text{min})(1.0)}{(187m)^2}=.000014$$

The Theis Equation is based on the following assumptions:

1) The aquifer is homogeneous and isotropic.
2) The aquifer is of infinite areal extent.
3) Transmissivity is constant at all times and at all places.
4) Water is removed from storage instantaneously with decline in head.
5) The well penetrates, and receives water from, the entire thickness of the aquifer.
6) The well has an infinitesimally small diameter.
7) The aquitards confining the aquifer are impermeable.

SESSION IV

JACOB METHOD

I. Theory

A. The nonequilibrium formula describes the expansion of the cone of depression as:

$$h_o - h = \frac{Q}{4\pi T}\left[-0.5772 - \ln u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} \cdots\right]$$

1. The well function part, W(u), is:

$$W(u) = \left[-0.5772 - \ln u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} \cdots\right]$$

B. Jacob approximates nonequilibrium formula, using 1st two terms (largest part) of W(u) series

$$W(u) \approx -0.5772 - \ln u$$

C. Equations for time-drawdown plot.

1. Transmissivity

$$T = \frac{2.3Q}{4\pi\Delta(h_o - h)}$$

2. Storativity

$$S = \frac{2.25Tt_o}{r^2}$$

3. Hydraulic conductivity

$$K = \frac{T}{b}$$

4. Variables

Q = constant pumping rate
$\Delta(h_o\text{-}h)$ = drawdown over one log cycle
t_o = line intercept on 0 drawdown axis
r = distance from pumping well
T = transmissivity
S = storativity
b = aquifer thickness

II. Assumptions

A. Common assumptions

1. Aquifer has infinite areal extent.
2. Aquifer is isotropic and homogeneous.
3. Potentiometric surface is nearly horizontal before pumping.
4. Discharge is constant.
5. Well or piezometer fully penetrates the aquifer.

B. Specific to unsteady-state methods

1. Storage in well is negligible.
2. Water removed from aquifer storage is instantaneously discharged with head decline.

III. Limitations *[fig. 4.1]*

A. Ground-water conditions

1. Unsteady radial flow
2. Confined aquifer

B. Valid where:

$$u = \frac{r^2}{4Tt} < 0.01$$

1. Criteria met
   a. Later time; after about 10 minutes.
   b. Monitor wells nearer the pumping well will meet criteria before more distant well.

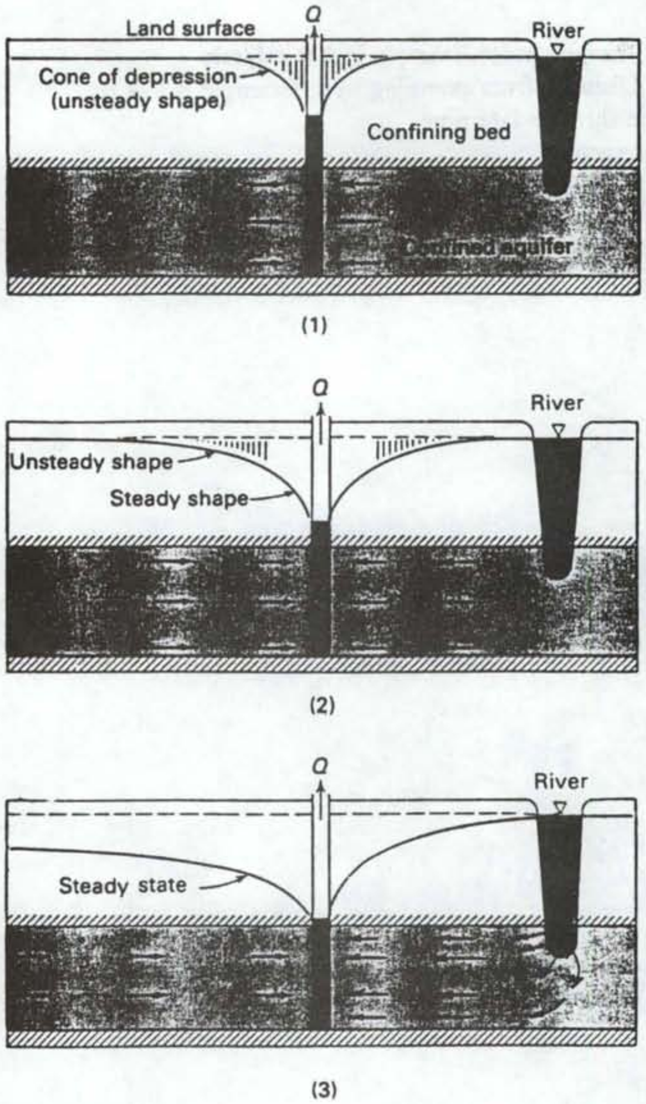


Figure 4.1 **Development of the cone of depression** (1) Unsteady shape occurs when the pump is turned on--entire cone shape is unsteady as it quickly expands. (2) In time, the cone shape has constant shape near the pumping well but continues to expand further from the pumping well. (3) Steady shape throughout the cone after pumping continues for a long time--recharge to the cone area about equals discharge from the well (Heath, p. 38).

IV. Example from Fetter, p. 171 *[fig. 4.2]*

A. Time-drawdown

1. Table and plot data.
    a. Drawdown on arithmetic vertical axis
        1) Increasing downward
    b. Time in minutes on logarithmic horizontal axis
        1) Increasing to the right
2. Record
    a. Pumping rate; **Example is 220 gal/min**
    b. Distance from pumping well; **Example is 824 ft**
3. Draw line through late time
4. Note and record:
    a. $\Delta(h_o - h)$, drawdown over one log cycle; **Example is 5.5 ft**
    b. t_o, where line intercepts 0-drawdown axis; **Example is 5.2 min**

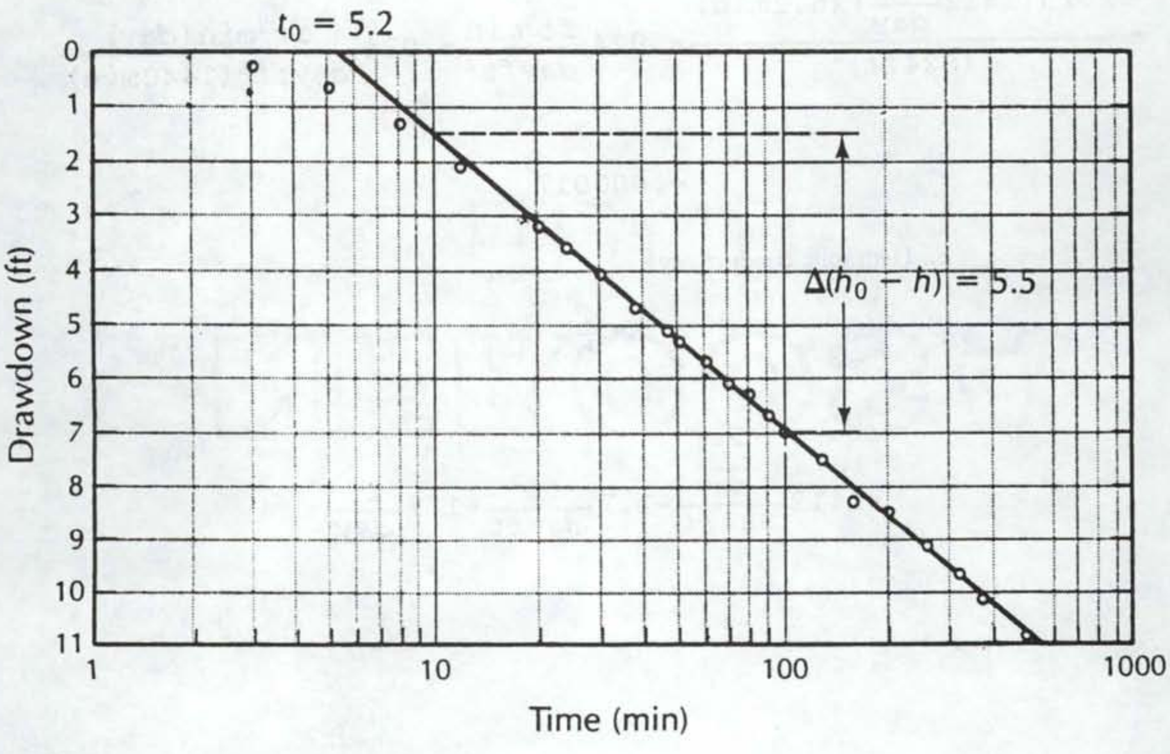


Figure 4.2 Jacob method solution for aquifer test of a fully confined aquifer using time/drawdown data (Fetter, fig. 6.6).

5. Calculations

a. Transmissivity

$$T = \frac{2.3Q}{4\pi\Delta(h_o - h)}$$

$$= \frac{2.3(220\frac{gal}{\min})}{4\pi 5.5ft} = 7.3\frac{gal}{\min ft} = 7.3\frac{gal(1440\min)(ft^3)}{\min(day)(7.48gal)ft}$$

$$= 1412\frac{ft^2}{day}$$

b. Storativity

$$S = \frac{2.25Tt_o}{r^2}$$

$$= \frac{2.25(1412\frac{ft^2}{day})(5.2\min)}{(824ft)^2} = .024\frac{ft^2\min}{dayft^2} = .024\frac{ft^2\min(day)}{dayft^2(1440\min)}$$

$$= .000017$$

c. Hydraulic conductivity

$$K = \frac{T}{b}$$

$$= 1412\frac{\frac{ft^2}{day}}{824ft} = 1.7\frac{ft^2}{dayft} = 1.7\frac{ft}{day}$$

B. Distance-drawdown *[fig. 4.3]*

1. At standard times, measure drawdown in each monitor well.
2. Record
   a. Pumping rate; **Example is 220 gal/min**
   b. Distance from pumping well; **Example is 824 ft**
3. Table and plot data.
   a. Plot drawdown on arithmetic vertical axis.
      1) Increasing upwards
   b. Plot distance in feet on logarithmic horizontal axis.
      1) Increasing to right
4. Draw line through data points of closest wells.
5. Note and record
   a. $\Delta(h_o\text{-}h)$ is drawdown over one log cycle; **Example is 8.8 ft.**
   b. r_o, where line intercepts 0-drawdown axis; **Example is 460 ft.**
      1) No drawdown beyond this distance
6. Equations
   a. Transmissivity

$$T=\frac{2.3Q}{4\pi\Delta(h_o-h)}$$

   b. Storativity

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

   c. Hydraulic conductivity

$$K=\frac{T}{b}$$

V. Readings:

A. Fetter, p. 170-173

B. Freeze & Cherry, p. 347-349

C. Heath, p. 38-41

D. Johnson, p. 138-142

E. Kruseman and Kidder, p. 59-65

F. Lohman, p. 19-27

G. Todd, p. 129-130

H. U.S. Bureau of Reclamation, p. 112-114

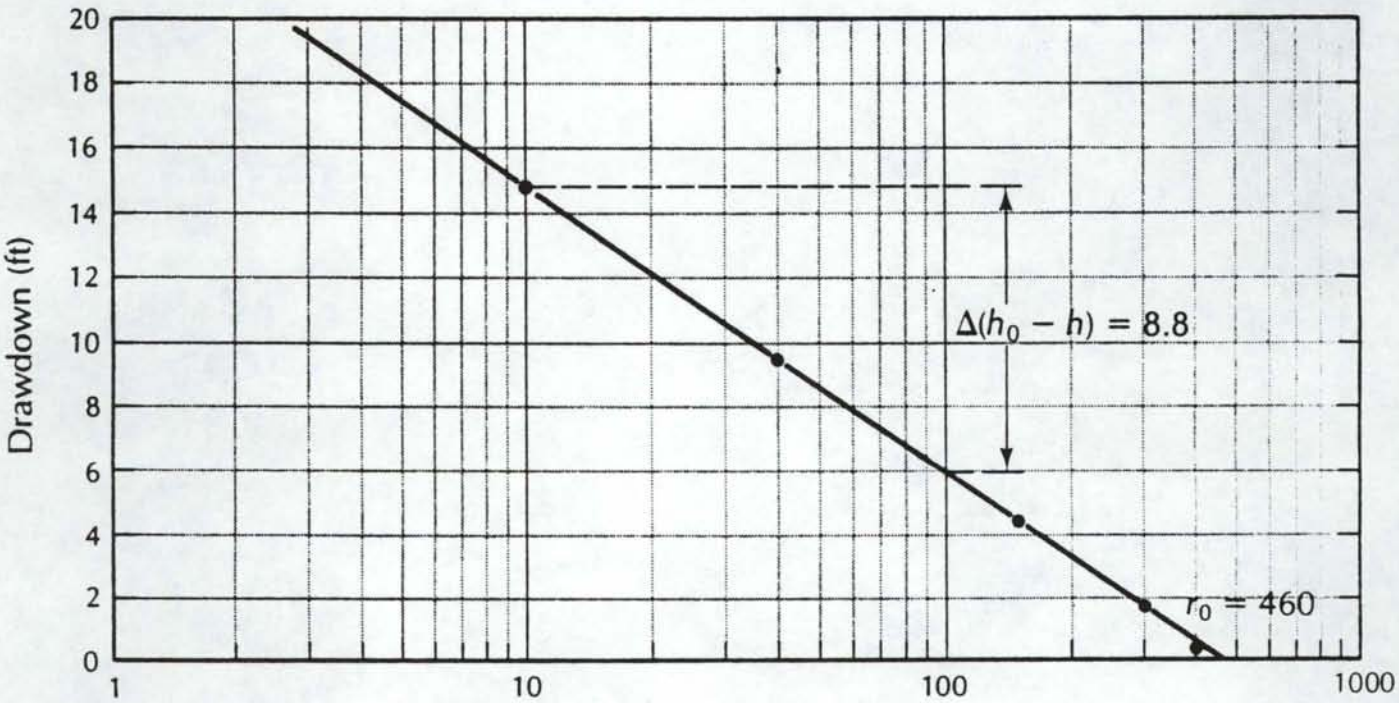


Figure 4.3 **Jacob method solution for aquifer test of a fully confined aquifer using distance/drawdown data (Fetter, fig. 6.7).**

SESSION V

HANTUSH-JACOB METHOD

I. Theory *[fig. 5.1]*

A. Describes the cone of depression as Theis solution; but with interpolation between:
1. Early time
2. Later time

B. Introduces a leakage term.
1. Ground water moves through a semi-confining layer to recharge the semi-confined aquifer during pumping the semi-confined aquifer.
2. Dimensionless r/B
   a. r is distance of monitor well from pumping well.
   b. B is leakage factor.
   c. Approaches Theis as r/B goes to 0.

C. Equations
1. Transmissivity

$$T=\frac{Q}{4\pi (h_o-h)} W(u,\frac{r}{B})$$

2. Storativity

$$S=\frac{4Ttu}{r^2}$$

3. Vertical hydraulic conductivity of semi-confining layer

$$K'=\frac{Tb'(\frac{r}{B})^2}{r^2}$$

4. Variables
   Q = discharge, the pumping rate
   $(h_o$-h) = drawdown
   W(u,r/B) = leaky well function
   B = leakage factor
   r/B = read from match curve
   r = distance from pumping well
   T = transmissivity
   t = time into the pumping test
   u = part of the well function term
   b' = thickness of semi-confining layer

D. Type curve *[fig. 5.2]*
1. Vertical axis is W(u,r/B)
2. Horizontal axis is 1/u
3. Increasing r/B describes increasing leakage; Theis for no leakage

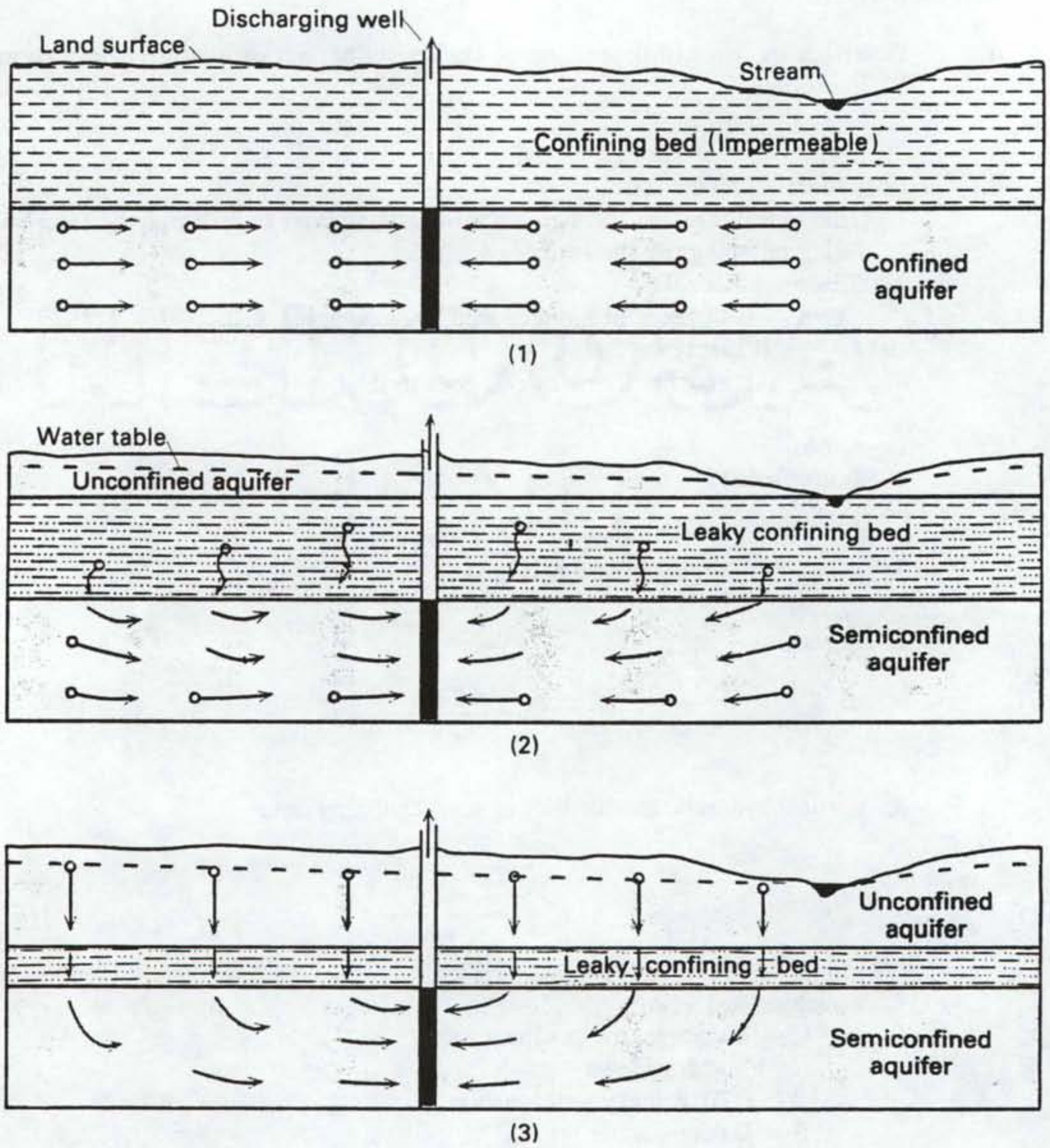


Figure 5.1 **Ground water conditions supplying water to a well** (1) Confined aquifer is overlain by an impermeable confining bed so that water only comes from storage in the confined aquifer. Theis method may be used. (2) Semiconfined aquifer overlain by a leaky confining bed--the confining bed yields significant water from its storage during an aquifer test. (3) Semiconfined aquifer overlain by a leaky confining bed--water moves from the unconfined aquifer through the semiconfining bed, without any water originating from the semiconfining bed itself (Heath, p. 50).

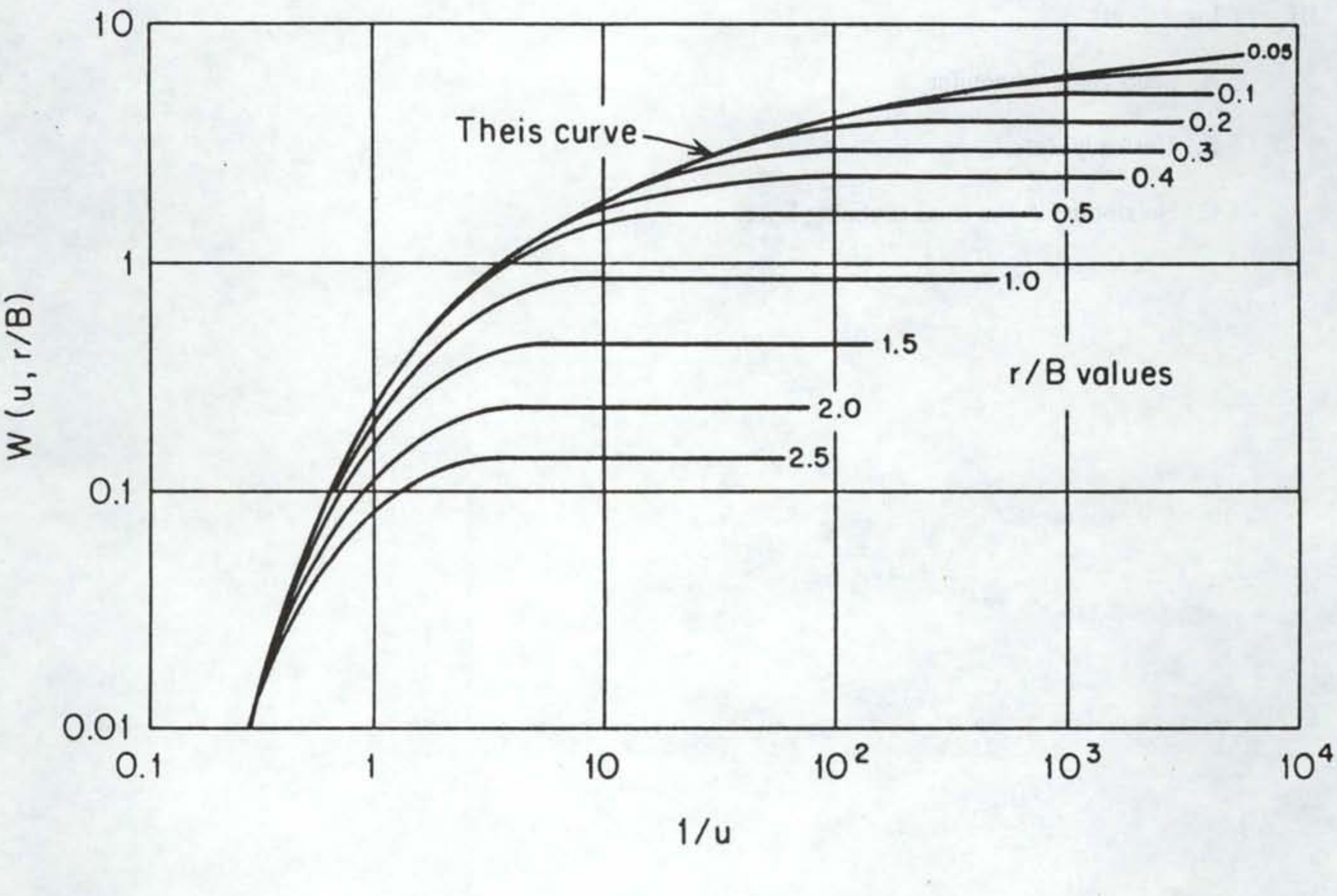


Figure 5.2 Theoretical curves for W(u,r/B) versus 1/u for a semiconfined aquifer (Freeze and Cherry, fig. 8.8).

II. Assumptions

A. General

1. Aquifer has infinite areal extent.
2. Aquifer is isotropic and homogeneous.
3. Potentiometric surface is nearly horizontal before pumping.
4. Discharge is constant.
5. Well or piezometer fully penetrates the aquifer.
6. Storage in well is neglected

B. Specific to the Hantush-Jacob method.

1. The hydraulic head of the unpumped aquifer does not change during pumping of pumped aquifer.
   a. Unpumped aquifer can supply unlimited water through semi-confining layer to pumped aquifer.
   b. Water level does not drop in unpumped aquifer.
2. The leakage rate into the pumped aquifer is proportional to drawdown.
   a. More pumping causes more drawdown with proportional leakage.
   b. The semi-confining layer has negligible storage.

III. Limitations

A. Semi-confined aquifer

B. Unsteady state

C. No storage in the semi-confining layer.

IV. Example from Freeze and Cherry p. 324 *[fig. 5.3]*

A. Table and plot data
   1. Time on logarithmic horizontal axis
      a. Increases to the right
   2. Drawdown on logarithmic vertical axis
      a. Increases upwards
   3. Same scale as type curve

B. Record
   1. Pumping rate; **Example is 63 gal/min**
   2. Distance from pumping well; **Example is 180 ft**
   3. Thickness of semi-confining layer; **Example is 100 ft**

C. Type curve fitting
   1. General curve fitting
      a. Move data plot vertically.
      b. Move data plot horizontally.
      c. Keep axes parallel.

D. Select arbitrary match point that relates the data and type curve solution.
   1. This relation is the unique graphical solution.
      a. For W(u,r/B) = 1, there is only one value for (h_o-h).
      b. For 1/u = 1, there is only one value for t.
   2. Simplify calculations by picking convenient values for:
      a. W(u,r/B), such as 1; **Example is 1**
      b. 1/u, such as 1; **Example is 1**
   3. Record corresponding values for:
      a. Drawdown, ho - h; **Example is .4 ft**
      b. Time, t; **Example is 3.3 min**
      c. Leakage curve r/B; **Example is 1.0**

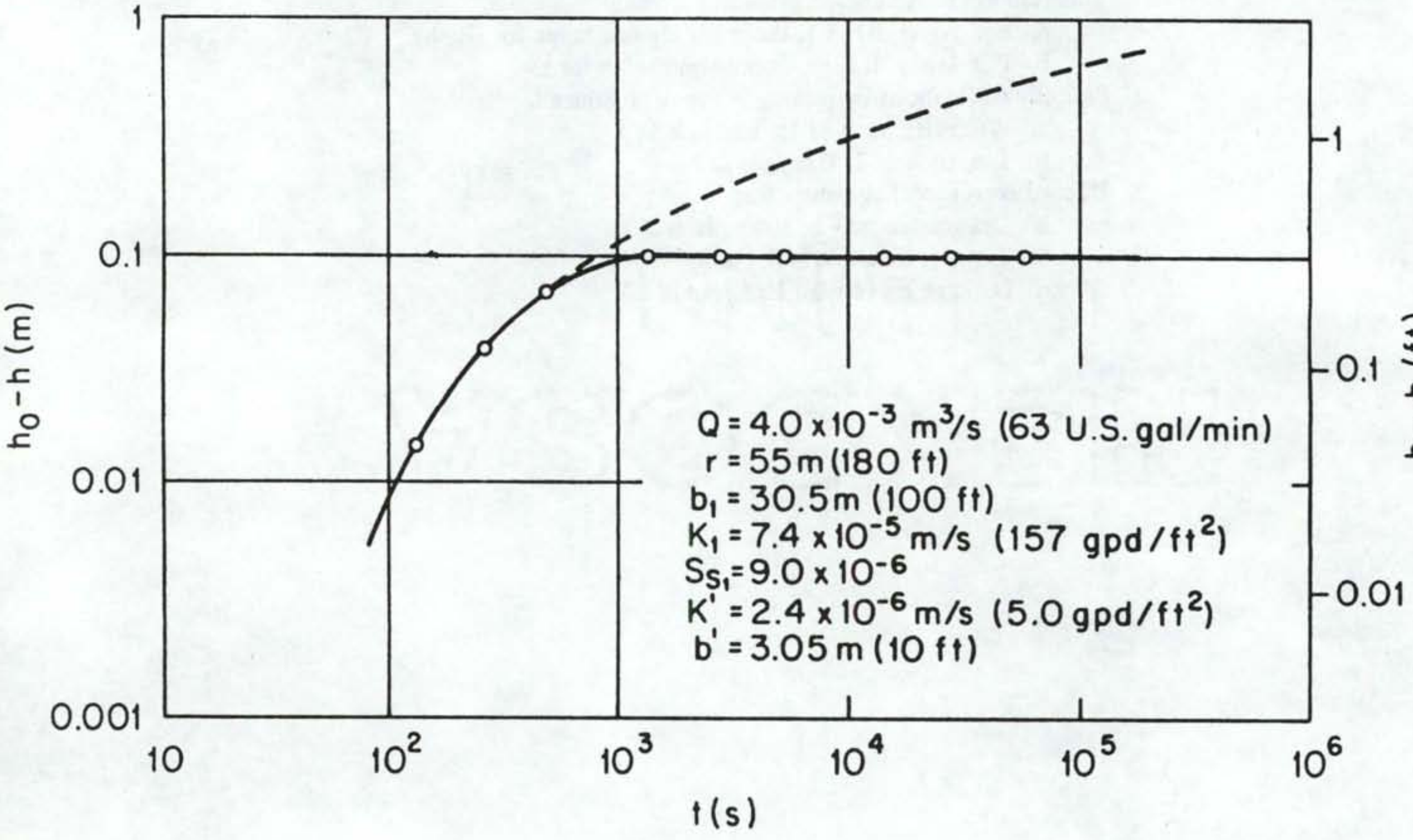


Figure 5.3 Hantush-Jacob method solution for drawdown and time data (Freeze and Cherry, fig. 8.10).

E. Calculations

1. Transmissivity

$$T=\frac{Q}{4\pi(h_o-h)}W(u,\frac{r}{B})$$

$$=\frac{63\frac{gal}{\min}}{4\pi(.4ft)}1=12.5\frac{gal}{minft}=12.5\frac{(1440\min)(ft^3)gal}{(day)(7.48gal)minft}$$

$$=2414\frac{ft^2}{day}$$

2. Storativity

$$S=\frac{4Ttu}{r^2}$$

$$=\frac{4(2414\frac{ft^2}{day})(3.3\min)(1)}{(180ft)^2}=.09\frac{ft^2\min}{dayft^2}=.09\frac{ft^2(day)\min}{day(1440\min)ft^2}$$

$$=.0007$$

3. Vertical hydraulic conductivity of leaky layer

$$K'=\frac{Tb'(\frac{r}{B})^2}{r^2}$$

$$=\frac{2414\frac{ft^2}{day}(100ft)(1.0)^2}{(180ft)^2}=7.5\frac{ft^2ft}{dayft^2}=7.5\frac{ft}{day}$$

V. Readings:

A. Fetter, p. 178-179, 182-185

B. Freeze & Cherry, p. 320-323

C. Heath, p. 50-51

D. Kruseman and De Ridder, p. 79-84

E. Lohman, p. 30-31

F. Todd, p. 136-139

SESSION VI

HANTUSH MODIFIED METHOD (1960)(β Method)

In 1960, Hantush modified his original work (Hantush and Jacob, 1955) to include the effects of water released from storage in the confining layers.

Unlike the Hantush-Jacob method, the Hantush modified method was developed for an aquifer system where the pumped aquifer is underlain by a leaky aquitard and overlain by a leaky aquitard.

The method accounts for water released from storage in the aquitards but not for water leaking through the aquitards from the overlying and underlying aquifers.

The main equations are

$$H(u,\beta)=\int_u^\infty \frac{e^{-y}}{y}\,erfc\left(\frac{\beta/\sqrt{u}}{\sqrt{y(y-u)}}\right)dy \qquad [dimensionless]$$

where:

$$u=\frac{r^2S}{4Tt} \qquad [dimensionless]$$

$$T=\frac{Q}{4\pi s}H(u,\beta) \qquad [L^2/T]$$

and

$$\beta=\frac{r}{4b}\left(\sqrt{\frac{K'S_s'}{KS_s}}+\sqrt{\frac{K''S_s''}{KS_s}}\right) \qquad [dimensionless]$$

where:

K = the hydraulic conductivity of the aquifer
K', K" = the hydraulic conductivities of the confining layers

$S = bS_s$
$S' = b'S_s'$ }storage coefficients of the
$S'' = b''S_s''$ aquifer and the confining layers, respectively

S_s, S_s', S_s'' = specific storage (storage coefficient per vertical unit thickness) of the aquifer and confining layers (b, b', and b"), respectively

β represents the water released from storage in the aquitards (not to be confused with the compressibility of water).

Figure 6.1 illustrates two-aquifer system.

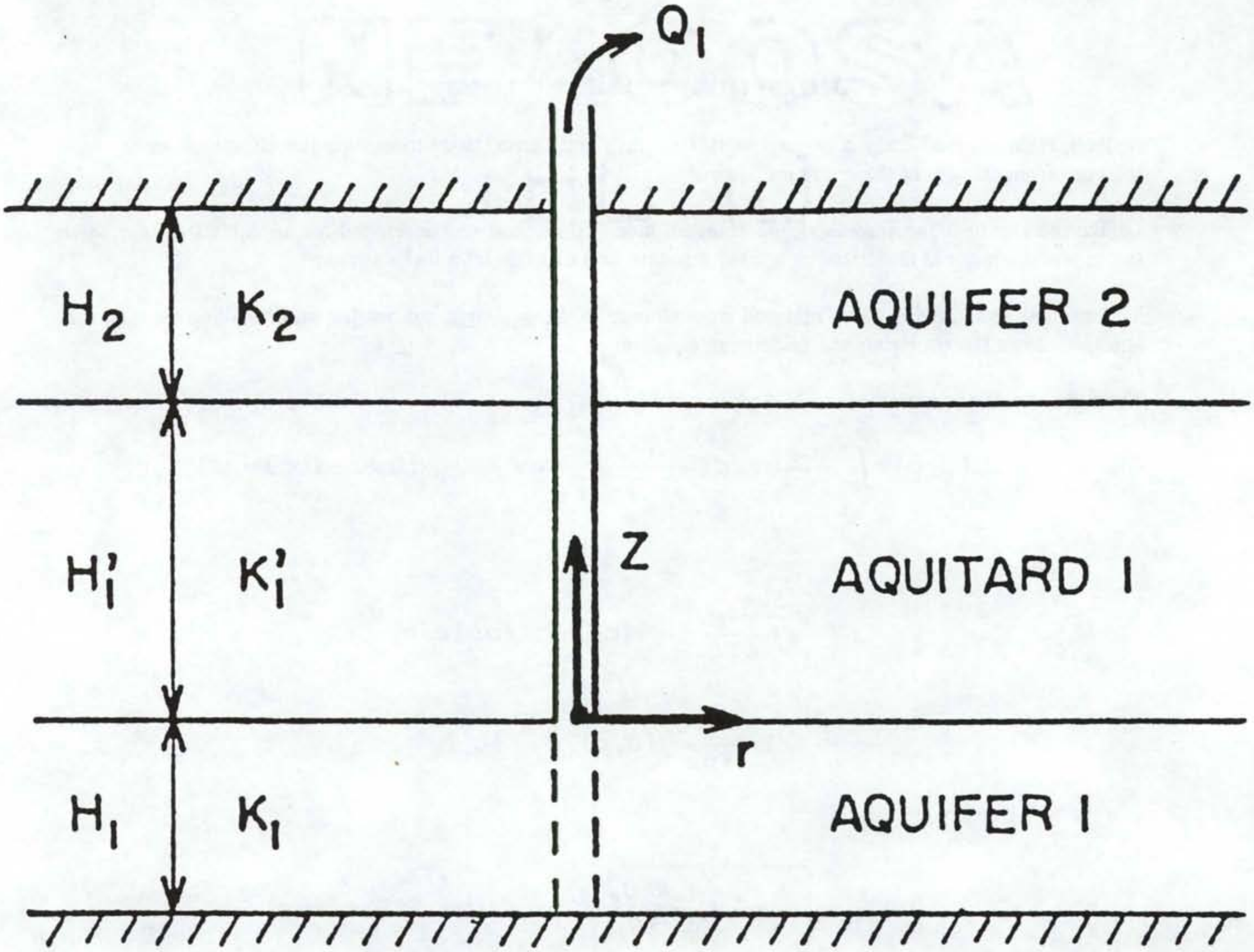


FIGURE 6.1 SCHEMATIC DIAGRAM OF A TWO-AQUIFER SYSTEM (AFTER NEUMAN AND WITHERSPOON, 1972)

Figure 6.2 illustrates three-aquifer system.

The Hantush (1960) modified method is not commonly used because the vertical hydraulic conductivity of the aquitards cannot be estimated directly.

Only the product of the vertical hydraulic conductivity and the specific storage of the aquitards can be determined from

$$\beta=\frac{r}{4b}\left(\sqrt{\frac{K'S'_s}{KS_s}}+\sqrt{\frac{K''S''_s}{KS_s}}\right)$$

Figure 6.3 shows the type curves for the Hantush modified method.

Steps for Using the Hantush Modified Method

1) Plot drawdown versus time [drawdown versus (time/distance^2) for multiple observation wells] on log-log graph paper.

2) Match your data curve to one of the β type curves on Plate 4 of Lohman. The top curve (i.e., $\beta = 0$) is the Theis curve which indicates no leakage.

3) Determine the coordinates s, t, 1/u, H(u, β) for your curve match.

4) Note the β curve being matched

Determine T and S for the aquifer as follows:

$$T=\frac{Q}{4\pi s}H(u,\beta)$$

$$S=\frac{4Ttu}{r^2}$$

The value of β is a function of the water released from storage in the aquitards and of the radial distance from the pumping well.

The closer the observation wells are located to pumping well, the smaller are the deviations from the Theis curve.

Drawdown data for observation wells located too close to the pumping well may mistakenly be matched to the Theis curve indicating no leakage. However, the greatest amount of leakage occurs close to the pumping well.

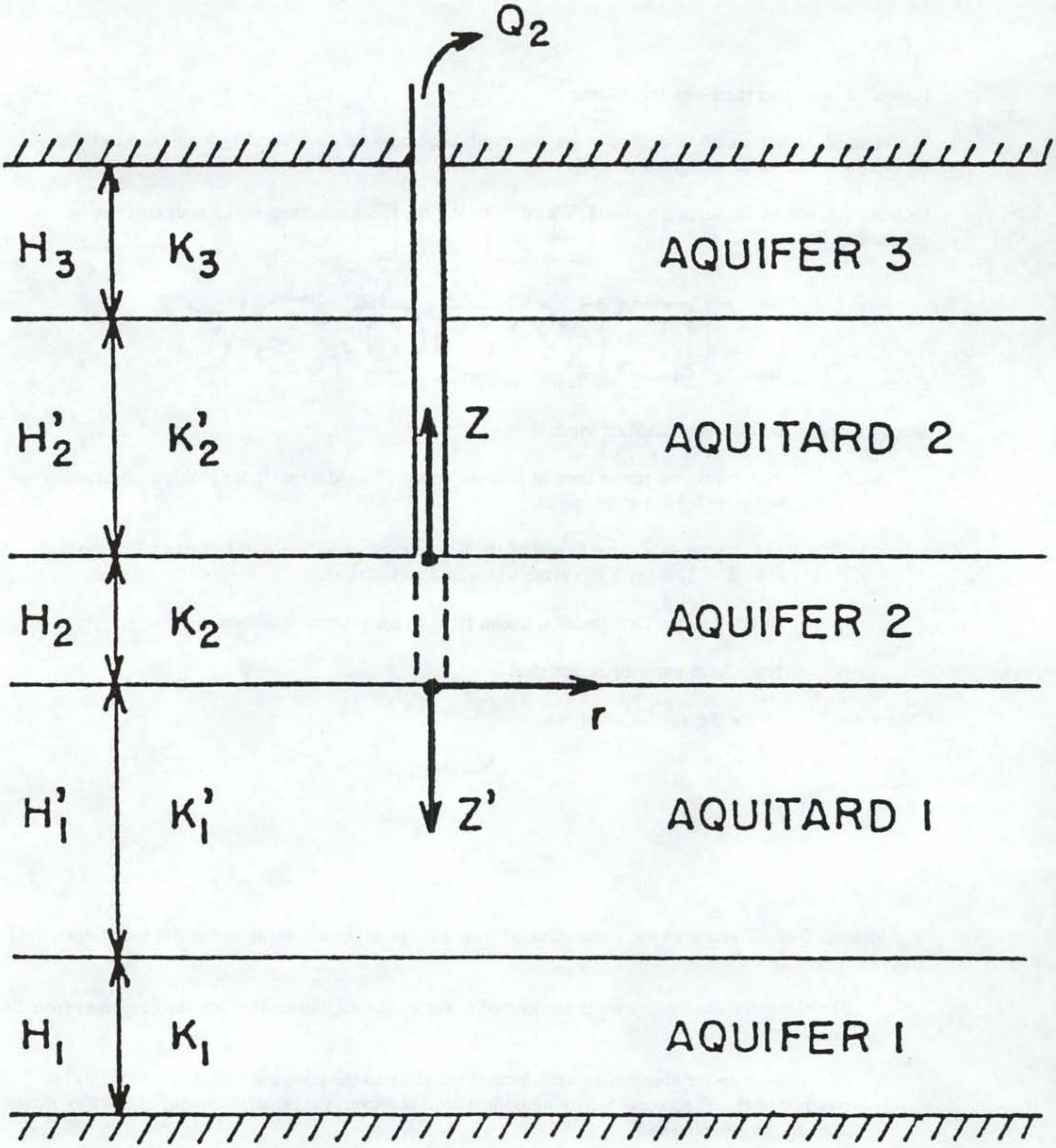


FIGURE 6.2 SCHEMATIC DIAGRAM OF A THREE-AQUIFER SYSTEM (AFTER NEUMAN AND WITHERSPOON, 1972)

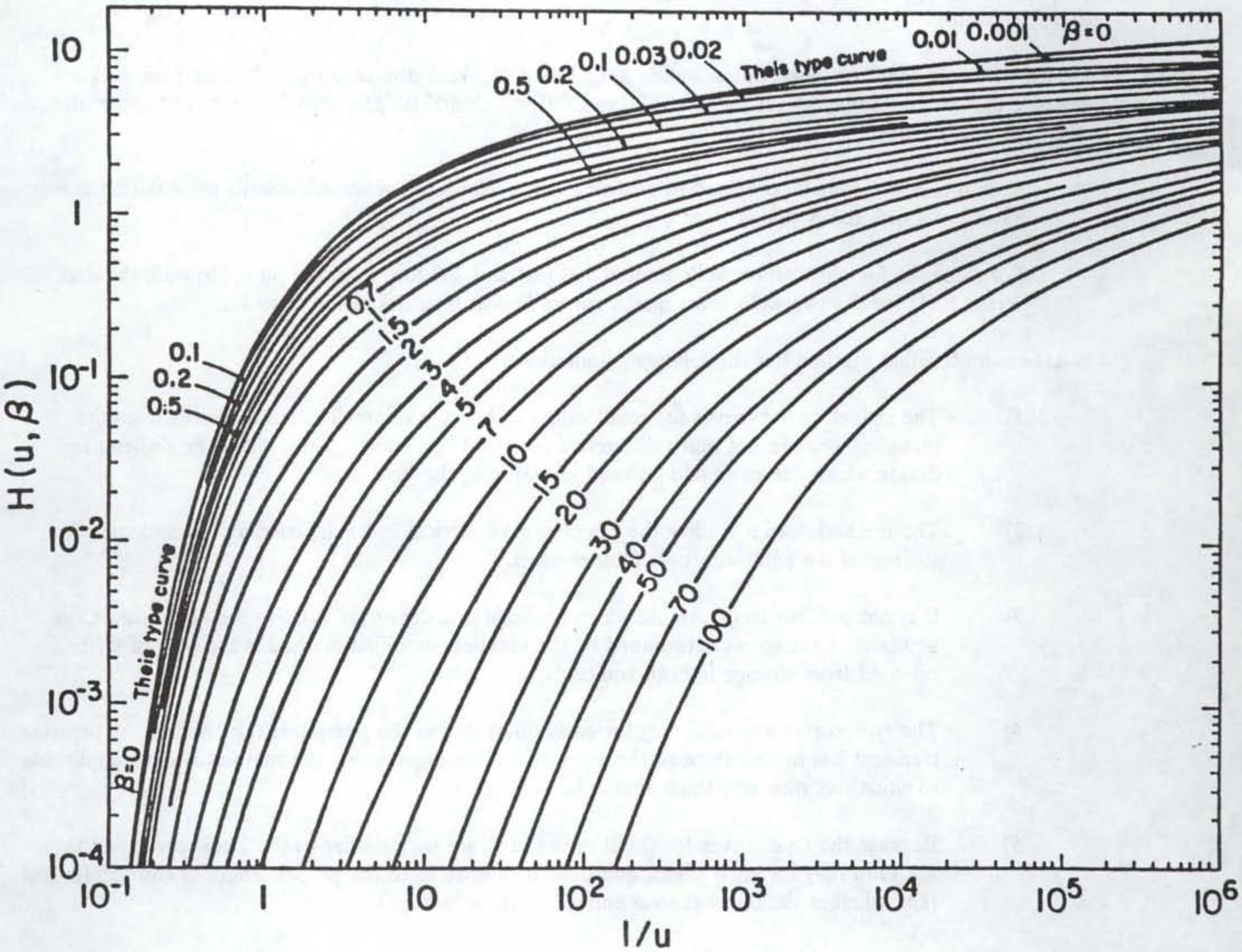


FIGURE 6.3 TYPE CURVES FOR THE HANTUSH MODIFIED METHOD (AFTER JAVANDEL, 1979)

Because there is so little difference in the shape of the curves for values of β between zero (i.e., the Theis curve) and about 0.7, it is not possible to obtain unique values of T, S, and β from data for a single observation well.

Therefore, if more than one observation well is available at different distances from the pumping well, a composite plot of drawdown (s) versus (time/distance^2)(t/r^2) should be prepared for each well and matched to the family type curves.

A unique match may be obtained by realizing that r values for observation wells must fall on curves having proportional β values.

For example, for observation wells located 100 feet and 200 feet from the pumping well, the data (s versus t/r^2) for the two wells must match curves having β values of the ratio 1:2.

The Hantush Modified Method has the following limitations:

1) The shapes of the curves for small values of leakage or small radial distances from the pumping well are not much different from the Theis curve. Thus, it may be difficult to decide which curves should be used in matching the field data.

2) The method does not allow the calculation of vertical hydraulic conductivity and specific storage of the aquitards, only their product.

3) It is not possible to determine which aquitard is leaky or the relative amounts from each aquitard. Leakage as determined by the Hantush modified method is the sum of water released from storage in both aquitards.

4) The type curves are valid only for early times during the pump test (i.e., before the pressure transient has moved through the aquitards. This implies that the method is most applicable to situations with very thick aquitards.

5) Because the type curves for small values of β are very similar to the Theis curve and they are valid only for early times, questions may arise as to the proper length of the aquifer test (i.e., whether the test was long enough to show leakage).

SESSION VII

RATIO METHOD (Neuman and Witherspoon, 1968, 1971)

An aquifer testing method that provides a much more complete analysis of a leaky aquifer system than the Hantush-Jacob and Hantush modified methods is the Ratio method developed by Neuman and Witherspoon (1968).

Witherspoon and Neuman (1967) derived the following expression which gives drawdown in the aquitard as a function of time (t) and elevation z above the top of the aquifer.

$$s' = \frac{Q}{2\pi^{3/2}Kb}\int_{\sqrt{1/4t'_D}}^{\infty} -Ei\left[-\frac{t'_D y^2}{t_D(4t'_D y^2-1)}\right] e^{-y^2} dy$$

where:

$$t'_D = \frac{K't}{S'_s z^2}$$

$$t_D = \frac{Kt}{S_s r^2}$$

$$-Ei(-x) = \int_x^{\infty} \frac{e^{-y}}{y} dy$$

z = vertical distance from the top of the aquifer

S_s, S_s' = specific storage of the aquifer and the aquitard, respectively

The ratio method at a minimum requires one observation well in the aquifer at a given radial distance (r) from the pumping well. Additional requirements include at least one observation well (i.e., piezometer) in each aquitard (confining layer) of interest (i.e., overlying and/or underlying) located at the same radial distance from the pumping well as the aquifer observation well.

Figure 7.1 presents a suggested arrangement for conducting a ratio-method test.

Features of the Ratio method

1) The method applies to leaky multiple aquifer systems with an arbitrary number of aquifers and aquitards. This is a distinct advantage over the other methods.

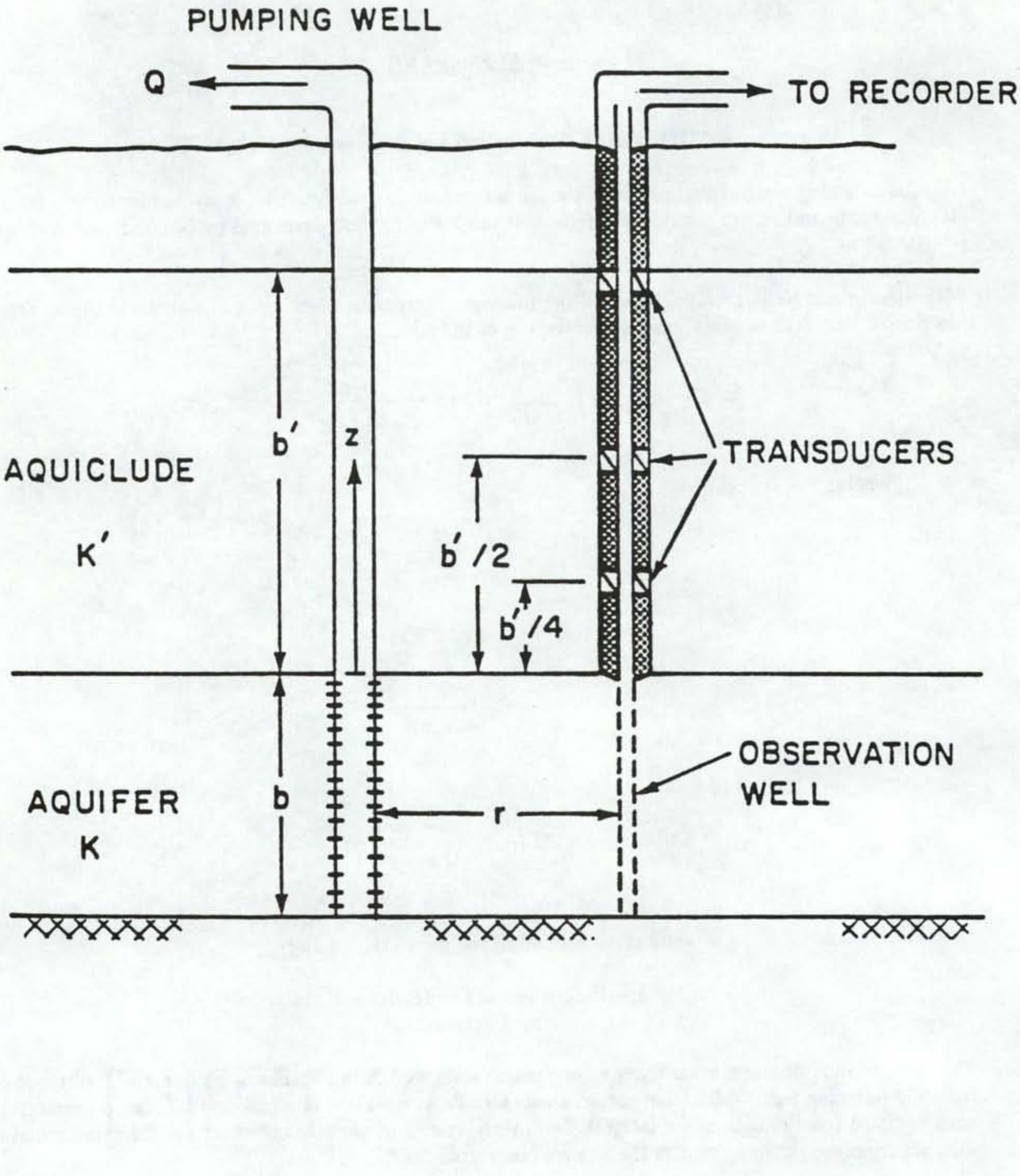


FIGURE 7.1 A SUGGESTED WELL ARRANGEMENT FOR A RATIO-METHOD TEST (AFTER JAVANDEL, 1979)

2) The pumped aquifer can be either confined or unconfined.

3) The confining layers can be heterogeneous and anisotropic. In this case, the ratio method gives the average vertical hydraulic conductivity over the thickness (z) of the aquitard being tested. All other methods require the assumption that the confining layers are homogeneous and isotropic.

4) The method relies only on early drawdown data and therefore the pumping test can be of relatively short duration. The other methods require longer pumping periods and many times there is question as to whether the test was long enough to show leakage.

5) Drawdown data collected in the unpumped aquifers or in the aquitards provide an in situ indication of the time limit at which the ratio method ceases to give reliable results.

6) The method is more sensitive to time lag than to the actual magnitude of the ratio of drawdown in the aquitard to drawdown in the aquifer (s'/s). Therefore, the accuracy with which drawdowns are measured in the aquitards is not overly critical.

7) The ratio method does not require prior knowledge of the aquitard thicknesses. Both the Hantush-Jacob and Hantush modified methods require knowledge of the aquitard thicknesses to plug into the equations.

8) The ratio method is simple to use and does not involve any subjective, graphical curve matching procedures.

   This is an advantage because:

   1) Curve matching often is prone to errors due to individual judgment.

   2) A more reliable result can be obtained by taking the arithmetic average of results obtained from several values of the ratio s'/s.

<u>Procedure for the Ratio Method</u>

1) Complete a pumping well through the total thickness of the aquifer.

2) Construct at least one observation well in the pumped aquifer at a distance r (<100 feet) from the pumping well.

3) Construct at least one observation well (piezometer) in each confining layer of interest (upper and lower) at the same distance r from the pumping well as the observation well in the aquifer. These piezometers must be completed within the aquitard(s) at a distance z from the top or bottom of the aquifer (different z for different piezometers in the same aquitard)(Figure 7.1).

   Ideally, the piezometers and observation wells should be arranged along a circular arc with its center at the pumping well.

   Water level responses are measured within the various hydrostratigraphic units at one unique radial distance from the pumping well.

   Multiple observation wells and piezometers should be used for all but the most simple and uniform hydrostratigraphic environments.

IT SHOULD BE NOTED THAT PROPERLY COMPLETED AND SEALED WELLS ARE NECESSARY FOR VALID RESULTS.

4) Record water levels (or pressures if measured with transducers) in the observation wells long before the start of the test. It is very important that the water levels in the confining layer(s) come to an equilibrium condition before beginning the test.

5) Start producing from the pumping well at a constant pumping rate Q. Pumping must continue long enough for sufficient drawdown data to be measured in the observation wells [in the aquifer and the aquitard(s)]. Early-time data are the most important.

6) Match a log-log plot of early-time drawdown data for the aquifer observation well to the Theis curve to estimate transmissivity (T) and storativity (S) for the aquifer.

EARLY-TIME DATA ARE THE MOST IMPORTANT!

7) Calculate the ratio of drawdown in the aquitard(s) to drawdown in the aquifer (s'/s) at a given distance from the pumping well (r) and at a given instant of time (t).

8) Determine the magnitude of dimensionless time (t_D) for the particular values of r and t at which s'/s have been measured, by the following equation:

$$t_D = \frac{Tt}{Sr^2}$$

9) Use the curves presented in Figure 7.2 to read off a value of t'_D corresponding to the computed ratio of s'/s.

Note that when $t_D < 100$ the curves in Figure 7.2 are sensitive to minor changes in the magnitude of this parameter and therefore a good estimate of t_D is desirable.

When $t_D > 100$ these curves are close to each other so that they can be assumed to be practically independent of t_D. In this case, even a crude estimate of t_D will be sufficient for the ratio method to yield satisfactory results.

10) Calculate the diffusivity of the aquitard(s) (α') of interest by the following equation:

$$\alpha' = \frac{z^2}{t} t'_D$$

NOTE: In Figure 7.2 that when s'/s is less than 0.l, the value of t'_D that is obtained by the ratio method is not very sensitive to the magnitude of s'/s.

As a result, the value of α' that one calculates depends very little on the actual magnitude of the drawdown in the aquitard.

The critical quantity that determines the value of α' at a given elevation z is the time lag, (t) between the start of the test and the time when the aquitard observation well(s) begin to respond.

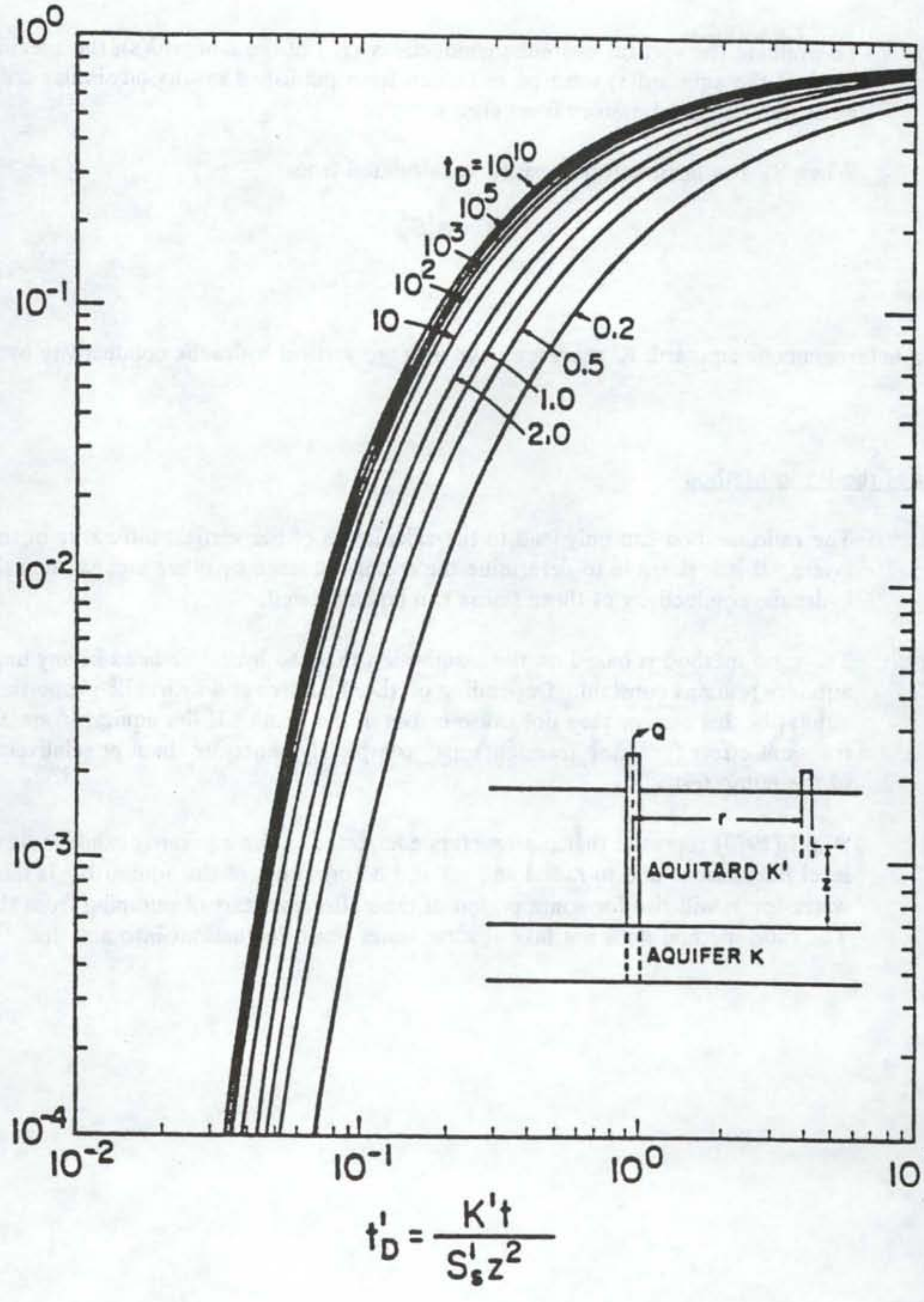


FIGURE 7.2 THE VARIATION OF s'/s VERSUS t'_D FOR A SEMI-INFINITE AQUITARD (AFTER NEUMAN AND WITHERSPOON, 1972)

11) To evaluate the vertical hydraulic conductivity (K') of the aquitard(s), the specific storage (S_s') of the aquitard(s) must be estimated from published results on similar sediments or measured in the laboratory from core samples.

When S_s' has been determined, K' is calculated from

$$K' = \alpha' S_s'$$

In case of a heterogeneous aquitard, K' represents the average vertical hydraulic conductivity over the thickness z.

Limitations of the Ratio Method

1) The ratio method can only lead to the calculation of the vertical diffusivity of the confining layers. If it is possible to determine the specific storage by other means, then the vertical hydraulic conductivity of those layers can be calculated.

2) The ratio method is based on the assumption that the hydraulic head in any unpumped aquifers remains constant. Depending on the thickness and hydraulic properties of the aquitards, this may or may not cause errors in the result. If the aquitards are thin, the transient effect (pressure transient) may completely penetrate them at relatively early stages of the pump test.

3) Wolff (1970) reported that piezometers completed in the aquitards exhibit reverse water-level fluctuations due to radial and vertical deformation of the aquitards. If this occurs, the water levels will rise for some period of time after the start of pumping from the aquifer. The ratio method does not take reverse water level fluctuations into account.

SESSION VIII

NEUMAN METHOD

I. Theory *[fig. 8.1]*

A. Describes the expanding cone of depression for an unconfined aquifer.
   1. Complicated by the saturated thickness varying during an aquifer test.
      a. T is harder to determine because T = Kb where b is saturated aquifer thickness.

B. Ground water removal affected by:
   1. Storativity (S or bS_s)
      a. Initial water released in response to a pressure drop.
   2. Specific yield (S_y)
      a. Later water released with gravity drainage.

C. Drawdown sequence *[fig. 8.1]*
   1. Early time (about 10 minutes) *[fig. 8.1b]*
      a. Pump turned on.
      b. Saturated, "pressure" cone of depression forms
         1) Water released from aquifer compression and water expansion.
      c. Theis predicts head change with time.
         1) Water removed from storage is proportional to head decline.
   2. Mid time (about 10 min. to 8 hours) *[fig. 8.1c]*
      a. Water begins to drain from cone of depression.
      b. Delayed yield.
         1) More common of fine-grained sediment.
      c. Vertical flow.
      d. Theis does not predict head change with time.
         1) More water drains from storage than Theis' head decline predicts.
   3. Late time (more than about 8 hours) *[fig. 8.1e]*
      a. Theis predicts head change with time.
         1) Water removed from storage is proportional to head decline.
      b. Horizontal flow

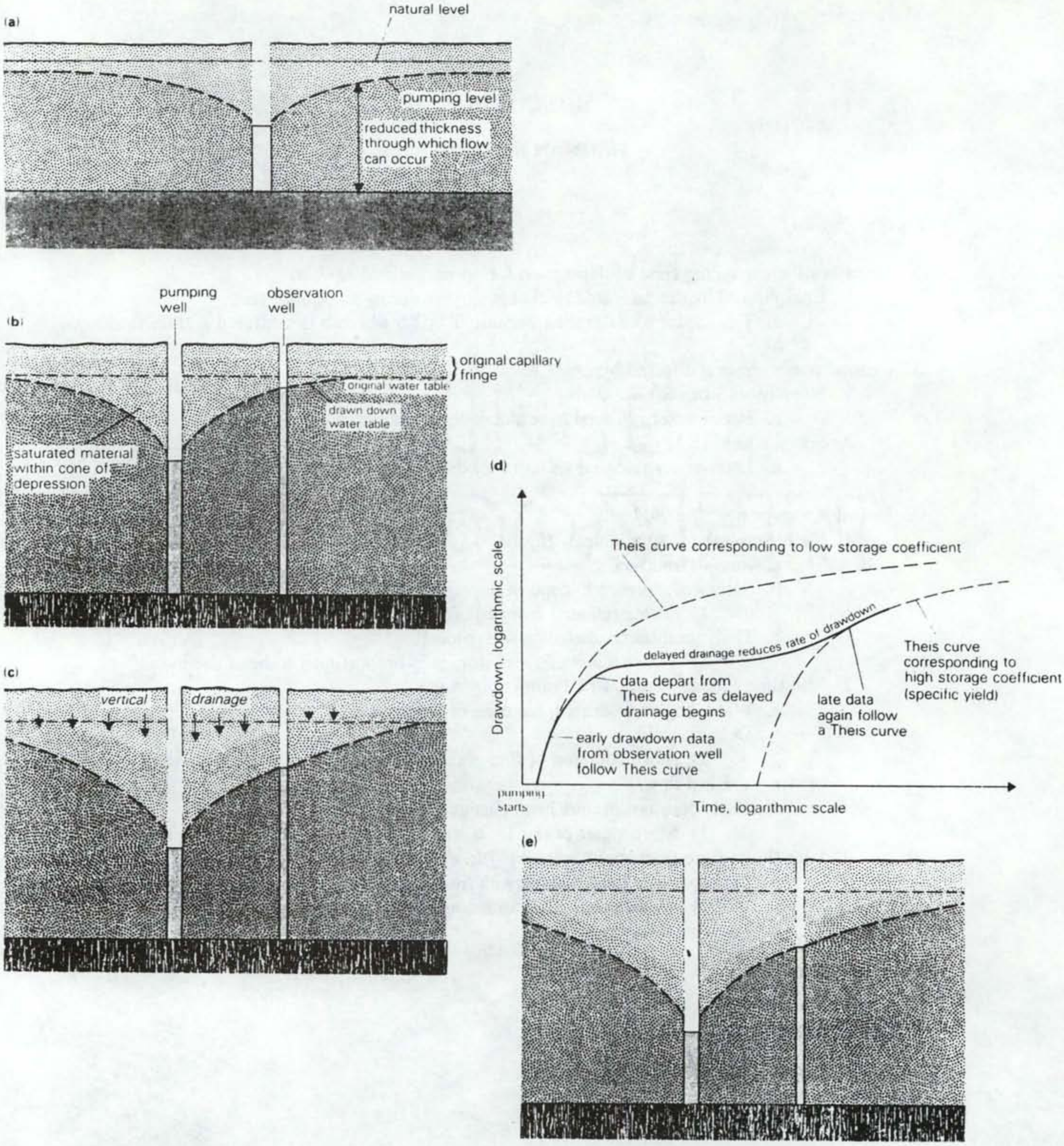


Figure 8.1 **Development of the cone of depression for an unconfined aquifer.** (a) Transmissivity calculation will be less than Theis calculates because saturated aquifer thickness decreases toward the pumped well. (b) First water released from storage is from aquifer compaction and water expansion--no water drains from pressure cone. (c) Water begins to drain from the pressure cone area. (e) All water comes from draining of cone of depression (Price, fig. 10.6).

D. Equations

1. Transmissivity

$$T=\frac{Q}{4\pi\,(h_o-h)}W(U_A,U_B,\Gamma)$$

2. Storativity

a. Early drawdown data

$$S=\frac{4TtU_A}{r^2}$$

b. Late drawdown data

$$S=\frac{4TtU_B}{r^2}$$

3. Hydraulic conductivity

a. Horizontal

$$K_h=\frac{T}{b}$$

b. Vertical

$$K_v=\frac{b^2K_h}{r^2}$$

E. Variables

Q = constant discharge from well
$W(U_A,\Gamma)$ = well function for unconfined aquifer; early time
$W(U_B,\Gamma)$ = well function for unconfined aquifer; late time
h_o-h = drawdown
T = transmissivity
t = time at match point
r = distance from pumping well to observation well
U_A = convenient type-curve match point; early time
U_B = convenient type-curve match point; late time
Γ = read from type curve (.001 - 7)
b = saturated aquifer thickness
K_h = horizontal hydraulic conductivity

F. Type curve *[Fetter fig. 8.2]*

1. Vertical axis; $W(U_A,U_B,\Gamma)$
    a. $W(U_A,\Gamma)$ for early time
    b. $W(U_B,\Gamma)$ for late time
    c. Γ is similar to r/B used in semiconfined conditions.
        1) Same value for late and early time.
2. Horizontal
    a. $1/U_A$ for early time
    b. $1/U_B$ for late time

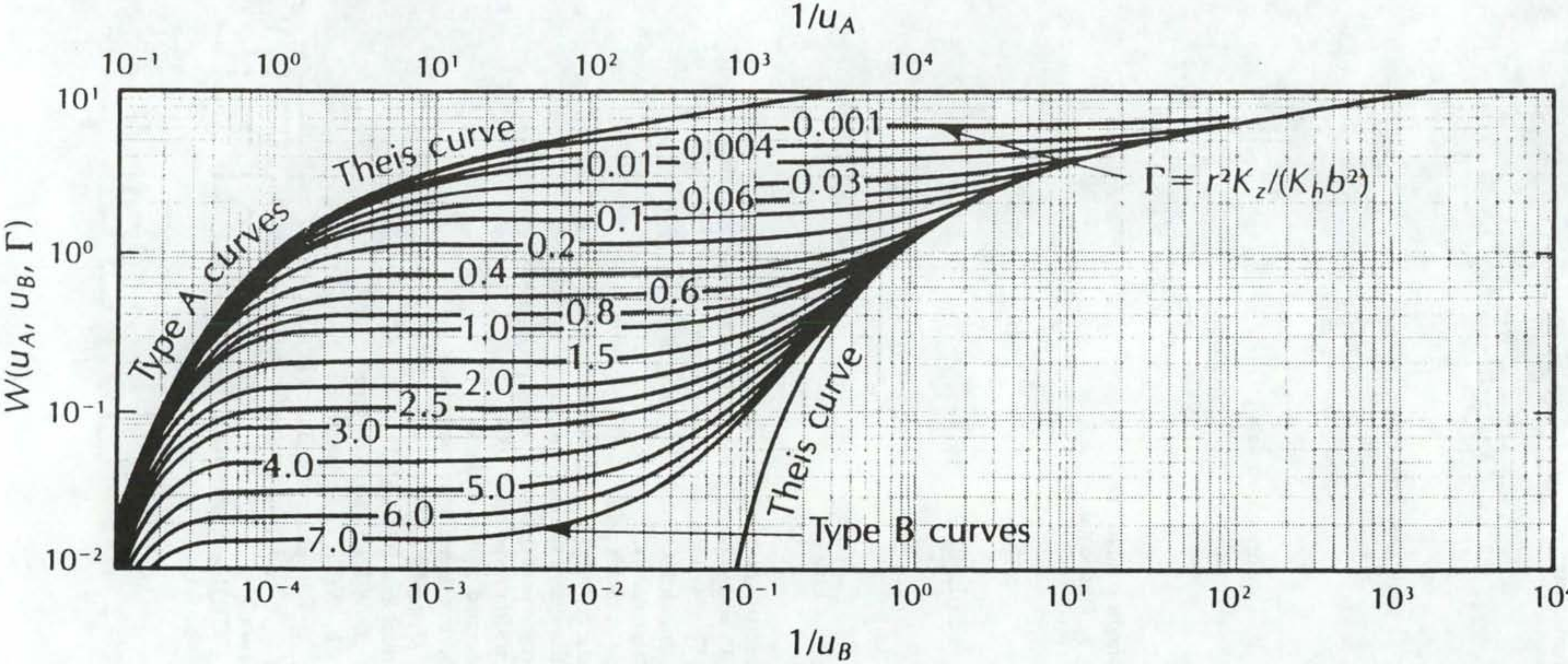


Figure 8.2 **Neuman type curves for drawdown data from fully penetrating wells in an unconfined aquifer (Fetter, fig. 6.18).**

II. Assumptions

A. Common assumptions
1. Aquifer has infinite areal extent.
2. Aquifer is isotropic and homogeneous.
3. Potentiometric surface is nearly horizontal before pumping.
4. Discharge is constant.
5. Well or piezometer fully penetrates the aquifer.

B. Specific to unsteady state conditions
1. Storage in well is negligible.

III. Limitations

A. Unsteady radial flow

B. Unconfined aquifer

IV. Example from Fetter, page 194-195 *[fig. 8.3]*

A. Table and plot data.
   1. Time is on logarithmic horizontal axis; increases to right.
   2. Drawdown is on logarithmic vertical axis; increases upwards.
   3. Same scale as type curve.

B. Record
   1. Pumping rate; **Example is 1000 gal/min**
   2. Saturated thickness; **Example is 100 ft**
   3. Distance from pumping well; **Example is 200 ft**

C. Type curve fitting
   1. General curve fitting.
      a. Move data plot vertically.
      b. Move data plot horizontally.
      c. Keep axes parallel.
   2. Late time fitting using type B curves.
      a. Select arbitrary match point that relates data and type curve solution.
         1) This relation is the unique graphical solution.
            a) For $W(U_B,\Gamma) = 1$, there is only one value for $(h_o\text{-}h)$.
      b. Simplify calculations by picking convenient values for:
         1) $W(U_B,\Gamma)$, such as 1; **Example is 0.1**
         2) $1/U_B$, such as 1; **Example is 10, therefore $U_B = .1$**
      c. Record corresponding data values for:
         1) Drawdown $(h_o\text{-}h)$; **Example is 0.043 ft**
         2) Time (t); **Example is 128 min**
      d. Record the Γ value; **Example is 0.1**
   3. Early time curve fitting using type A curves.
      a. Overlay type curve on data plot using same value as determined with late time.
      b. Select arbitrary match point that relates data and type curve solution.
         1) This relation is the unique graphical solution.
            a) For $W(U_A,\Gamma) = 1$, there is only one value for $(h_o\text{-}h)$.
      c. Simplify calculations by picking convenient values for:
         1) $W(U_A,\Gamma)$, such as 1; **Example is 0.1**
         2) $1/U_A$, such as 1; **Example is 1, therefore $U_A = 1$**
      d. Record corresponding data values for:
         1) Drawdown $(h_o\text{-}h)$; **Example is 0.041**
         2) Time (t); **Example is .9 min**

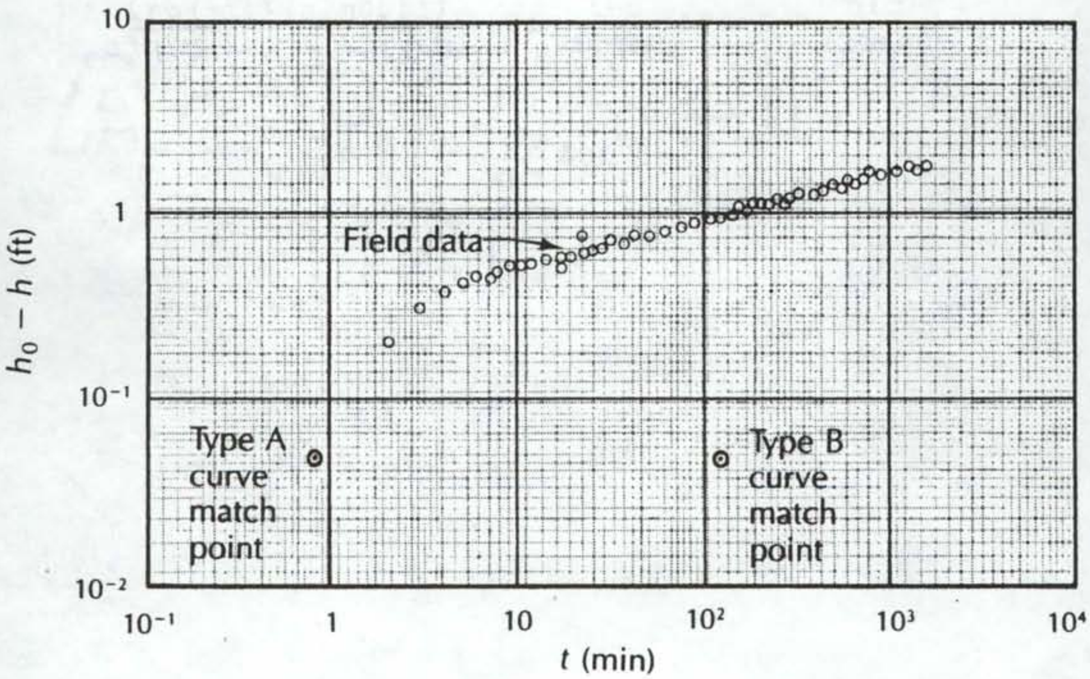


Figure 8.3 Neuman method solution for drawdown and time data for an unconfined aquifer (Fetter, fig. 6.19).

D. Calculations

1. Transmissivity

a. Late data

$$T=\frac{Q}{4\pi(h_o-h)}W(U_B,\Gamma)$$

$$=\frac{1000\frac{gal}{\min}}{4\pi(.043ft)}0.1=185\frac{gal}{minft}=185\frac{(1440\min)(ft^3)gal}{(day)(7.48gal)minft}$$

$$=35615\frac{ft^2}{day}$$

b. Early data

$$T=\frac{Q}{4\pi(h_o-h)}W(U_A,\Gamma)$$

$$=\frac{1000\frac{gal}{\min}}{4\pi(.041ft)}0.1=194\frac{gal}{minft}=194\frac{(1440\min)(ft^3)gal}{(day)(7.48gal)minft}$$

$$=37384\frac{ft^2}{day}$$

2. Storativity

a. Late data (S_y)

$$S=\frac{4TtU_B}{r^2}$$

$$=\frac{4(35615\frac{ft^2}{day})(128\text{min})(0.1)}{(200ft)^2}=45.6\frac{ft^2\text{min}}{dayft^2}=45.6\frac{ft^2\text{min}(day)}{dayft^2(1440\text{min})}$$

$$=0.032$$

b. Early data (bS_s)

$$S=\frac{4TtU_B}{r^2}$$

$$=\frac{4(37384\frac{ft^2}{day})(0.9\text{min})(1)}{(200ft)^2}=3.36\frac{ft^2\text{min}}{dayft^2}=3.36\frac{ft^2\text{min}(day)}{dayft^2(1440\text{min})}$$

$$=0.0023$$

3. Hydraulic conductivity

a. Horizontal

$$K_h = \frac{T}{b}$$

$$= \frac{36500\,\frac{ft^2}{day}}{100\,ft} = 365\,\frac{ft^2}{day\,ft} = 365\,\frac{ft}{day}$$

b. Vertical

$$K_v = \frac{b^2 K_h}{r^2}$$

$$= \frac{.1\,(100\,ft)^2\,(365\,\frac{ft}{day})}{(200\,ft)^2} = 9.1\,\frac{ft^3}{day\,ft^2}\,9.1\,\frac{ft}{day}$$

V. Reading

A. Fetter, p. 191-195

B. Freeze and Cherry, p. 324-327

C. Kruseman and De Ridder, p. 95-104

D. Lohman, p. 34-40

E. Price, p. 140-141

F. Todd, p. 134-136

G. U.S. Bureau of Reclamation, p. 132-140

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APPENDICES

WELL HYDRAULICS AND AQUIFER TESTING

AN AQUIFER TEST PROBLEM

To be completed in partial fulfillment of the
Requirements for one (1) Undergraduate or Graduate Credit

at

University of Idaho
Boise State University
or
Idaho State University

DUE DATE: June 1, 1990

Submit To:

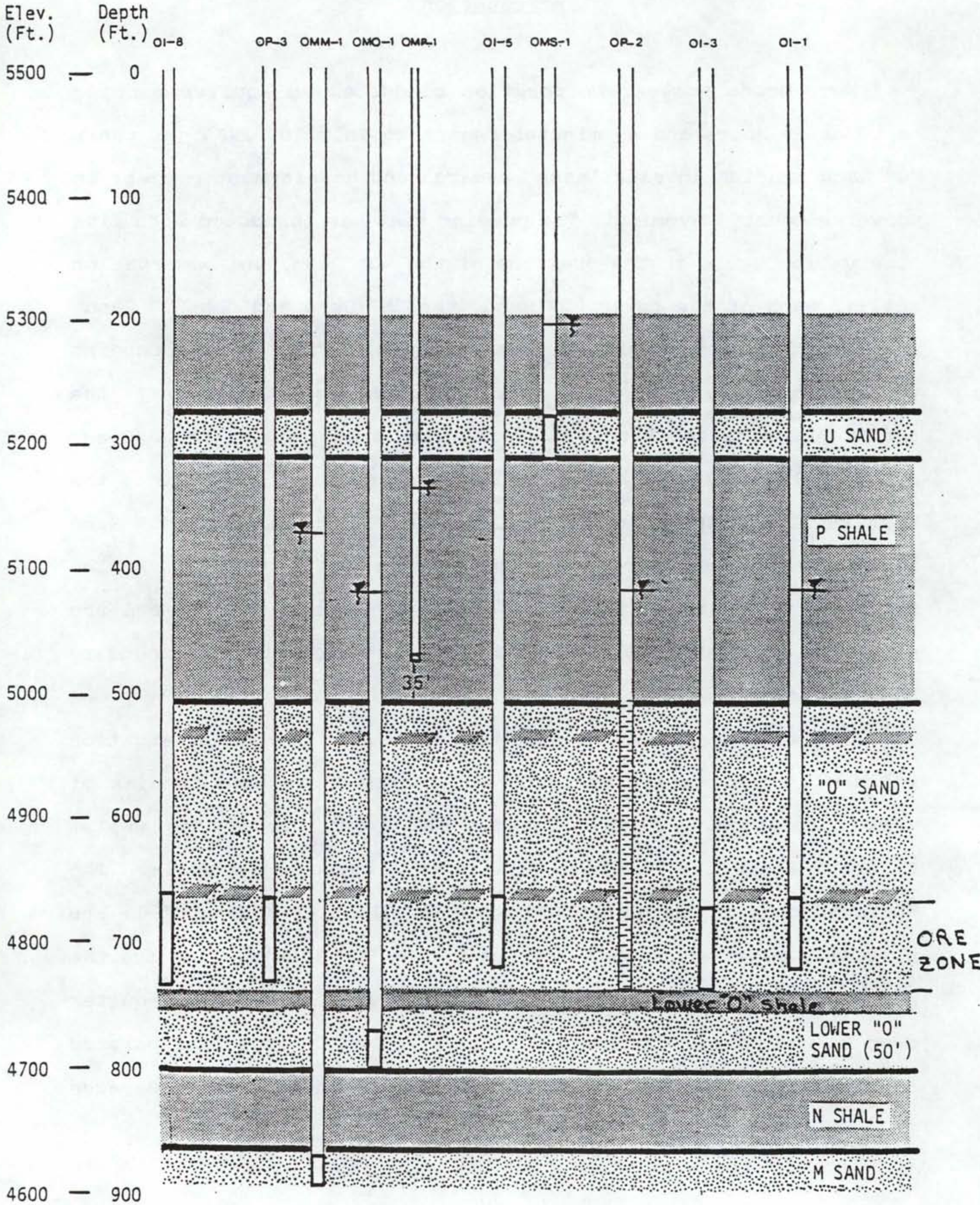
Dr. James L. Osiensky
Department of Geology and Geophysics
Boise State University
Boise, Idaho 83725

Introduction

Kerr-McGee Nuclear Corporation conducted an aquifer pumping test for 69 hours and 45 minutes beginning July 20, 1983, at their "O" Sand uranium in situ leach research and development project in Converse County, Wyoming. The pumping test was conducted with five observation wells in the ore zone of the "O" Sand, one observation well in each of the Lower "O" Sand, the "M" Sand and the "U" Sand, and one observation well in the "P" Shale (Figure 1). According to Kerr-MCGee, "the pump test was designed to collect the additional site specific hydrologic data requested by the NRC and DEQ hydrology staffs and to provide a more detailed analysis of the test results. Specific areas addressed ... include the "O" Sand aquifer characteristics, leakage properties of the "P" Shale, communication between the lower interval of the "O" Sand and the main body of the "O" Sand, directional transmissivity, and boundary conditions."

The well-field for the pump test consisted of nine observation wells plus the pumping well (OP-2). Table 1 presents details of well completion. Figure 2 shows the locations of the wells. Figure 1 shows the vertical relationship between the wells. The "O" Sand observation wells (OI-8, OP-3, OI-5, OI-3, and OI-1) are partially penetrating with respect to the "O" Sand aquifer and the pumping well. The pumping well penetrates the "O" Sand aquifer down to the top of the Lower "O" Shale. The "O" Sand is separated from the Lower "O" Sand by this Lower "O" Shale aquitard. Because

FIGURE 1 CROSS SECTION OF "O" SAND TEST PATTERN
(Horizontal not to Scale)



Potentiometric Levels 7/28/83

TABLE 1

WELL CONSTRUCTION AND COMPLETION TABLE
"O" SAND IN-SITU LEACH PROJECT
CONVERSE COUNTY, WYOMING
SECTION 26, T36N, R74W

Well No.	Total Depth (Ft.)	Drill Hole Size Depth (Ft.)	Drill Hole Size Dia. (In.)	Casing Depth (Ft.)	Casing Type	Open Interval (Depth-Ft.)
OI-1	730	676 730	7-7/8 3-7/8	671	4" Fiber-glass	671-730 59'
OI-5	730	675 730	7-1/2 3-7/8	672	4" Fiber-glass	672-730 (1) 58'
OI-8	736	670 736	7-7/8 3-7/8	662	4" Fiber-glass	662-715 (1) 53'
OP-3	736	675 736	9-7/8 5-7/8	670	6" Steel	670-713 43'
OMM-1	899	877 899	7-7/8 3-7/8	877	4" Steel	877-899 22'
OMS-1	320	290 320	7-7/8 3-7/8	285	4" Steel	285-320 35'
OP-2	745	510 745	8-3/4 5-7/8	510	6" Steel	510-745 235'
OI-3	740	675 740	6-3/4 3-7/8	675	4" Fiber-glass	675-740 65'
OMP-1	467	467	5-1/4	464	See Fig. 4	464-467 3'
OMO-1	805	775	6-3/4 3-7/8	775	4" Steel	775-805 30'

(1) - Fill in hole from bottom of open interval to total depth drilled.

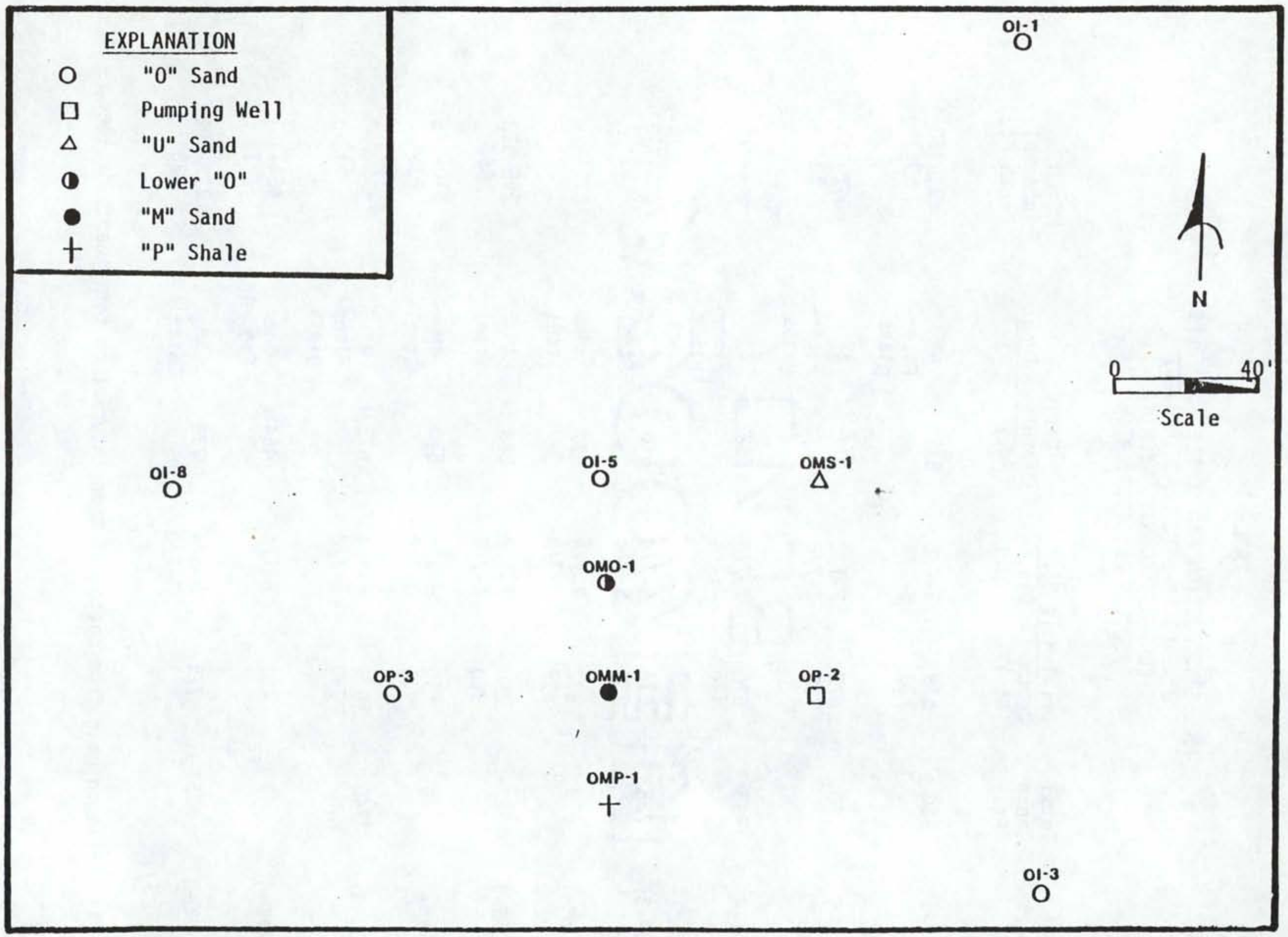


Figure 2. Plan view of "O" Sand test pattern.

the pumping well fully penetrates the "O" Sand aquifer above the Lower "O" Shale, Kerr-McGee considered that partial penetration by the observation wells should not influence the data significantly.

Aquifer Pump Testing Procedure

Kerr-McGee measured water levels in all observation wells "before, during, and after the pumping test" to analyze for antecedent trends. According to Kerr-McGee, changes in water levels were due to rising water levels at the Bill Smith Mine and errors in measurement due to differences between measuring instruments (electric tapes, transducer cables); water levels were influenced also by the variable rate discharge test conducted in well OP-2 on July 17, 1983. A definite rising water level trend (approximately 2.0×10^{-4} ft/min) was measured in the "O" Sand aquifer. Water levels were rising in the Lower "O" Sand aquifer at a rate of approximately 1.5×10^{-4} ft/min. Water levels were rising at a rate of about 8.7×10^{-5} ft/min in the underlying aquifer ("M" Sand). No apparent water level trend was detected in the overlying aquifer ("U" Sand). A downward water level trend was in progress in well OMP-1 in the "P" Shale prior to the test. Water level data for this well in the "P" Shale prior to the pump test were influenced by bailing to remove drilling fluid. But these interferences probably can be ignored for purposes of using the data for calculation of vertical hydraulic conductivity of the "P" Shale. The absence of a fully penetrating well in the aquifer at equivalent distance probably is more limiting.

Statement of the Problem

You are hired as a consultant to the U.S. Nuclear Regulatory Commission (NRC) to perform an independent analysis of aquifer test data provided by a uranium in situ mining company. As part of this task order, you must prepare a type written, professional report of your findings.

The aquifer test data for wells OI-1 and OI-8 (provided) are representative of the data for observation wells OP-3, OI-5, and OI-3 (i.e., the data plots are very similar). Data for these wells are not provided in order to limit the scope of the problem. Data for observation wells OMM-1, OMP-1, and OMO-1 also are not provided for the same reason.

Given the limited information provided above, your assigned task is to analyze the aquifer test data presented in Tables 2 and 3 as follows:

1. Plot the aquifer test data as drawdown (*s*) versus time/distance^2 (t/r^2) on 3x5 cycle log-log graph paper of the same scale as the type curves in Lohman (1972).

2. Match your data plots to the type curves that you believe are most appropriate to calculate the transmissivity and storativity of the aquifer (justify your selection).

3. Calculate the transmissivity of the aquifer in ft^2/d, and the storativity.

4. Calculate the vertical hydraulic conductivity of the aquitard(s) in feet/day. Discuss which aquitard(s) the value(s) represent (i.e., lower "O" shale, N shale, P shale). If the vertical hydraulic conductivity cannot be determined, present a detailed explanation of why.

5. If the data deviate from the type curve(s) significantly, present a detailed explanation for the cause(s) of the deviation(s) with respect to the limiting assumptions of the technique used to analyze the data.

6. Discuss the limitations of your analysis and list your assumptions and conclusions.

TABLE 2: Time-drawdown data for well OI-1 at Kerr-McGee

Time after pumping started (in minutes)	Drawdown (in feet)	Time after pumping started (in minutes)	Drawdown (in feet)
1.65	.04	86.0	3.15
2.70	.21	105.00	3.35
3.80	.35	125.00	3.60
4.50	.51	145.00	3.70
6.60	.66	165.00	3.85
7.60	.89	210.00	4.10
8.20	.99	230.00	4.20
8.60	1.15	270.00	4.40
9.20	1.25	330.00	4.60
10.00	1.35	450.00	5.00
13.50	1.45	520.00	5.10
14.50	1.50	580.00	5.25
16.50	1.65	650.00	5.50
18.00	1.75	730.00	5.52
20.00	1.80	800.00	5.60
22.00	1.90	850.00	5.65
24.00	2.00	930.00	5.75
26.00	2.10	1080.00	6.00
30.00	2.20	1250.00	6.20
35.00	2.30	1450.00	6.40
40.00	2.45	1800.00	6.50
45.00	2.55	2250.00	6.80
50.00	2.65	2950.00	7.00
60.00	2.85	4050.00	7.20

NOTE: Well OI-1 is located 191.5 ft from the pumping well (OP-2)

TABLE 3: Time-drawdown data for well OI-8 at Kerr-McGee

Time after pumping started (in minutes)	Drawdown (in feet)	Time after pumping started (in minutes)	Drawdown (in feet)
1.45	.02	46.00	2.50
1.70	.03	50.00	2.55
1.85	.04	55.00	2.65
2.00	.05	60.00	2.75
2.25	.09	64.00	2.85
3.00	.13	68.00	2.90
3.50	.19	72.00	2.95
4.00	.26	80.00	3.10
4.50	.33	94.00	3.30
5.00	.40	110.00	3.40
5.50	.49	150.00	3.70
6.00	.57	210.00	4.00
6.50	.66	250.00	4.20
7.00	.71	300.00	4.40
7.60	.78	365.00	4.60
8.00	.84	420.00	4.70
8.50	.90	480.00	4.80
9.00	.96	540.00	5.00
9.60	1.00	600.00	5.20
10.00	1.07	710.00	5.40
12.00	1.25	850.00	5.50
14.00	1.35	950.00	5.70
16.00	1.50	1100.00	5.90
18.00	1.60	1400.00	6.10
20.00	1.70	1600.00	6.20
22.00	1.80	1800.00	6.30
24.00	1.90	2100.00	6.60
26.00	1.95	2400.00	6.70
28.00	2.00	2800.00	6.90
30.00	2.10	3400.00	7.10
34.00	2.20	3700.00	7.20
38.00	2.30	4185.00	7.30
42.00	2.40		

NOTE: Well OI-8 is located 190.4 ft from the pumping well (OP-2)