

N-749

BIOGRAPHICAL RESUME

Name: Keros Cartwright Title: Geologist and Head
Hydrogeology and Geophysics Section
Home address: 310 West John Illinois State Geological Survey
Champaign, IL 61820
SS No.: 529-40-9288

Birth Date: July 25, 1934

Place of Birth: Los Angeles, CA Nationality: USA Sex: M

<u>Education - Institution and Location</u>	<u>Degree</u>	<u>Year</u>
University of California (Berkeley)	A.B. (Geology)	1959
University of Nevada (Reno)	M.S. (Geology)	1961
University of Illinois (Urbana)	Ph.D. (Geology)	1973

Professional Societies

Member of American Geophysical Union; Geological Society of America; National Water Well Association; AIME; American Society for Testing Materials; Association of Professional Geologists

Professional Experience

October 1961 - present: October 1961 to June 1963 - Research Assistant; July 1963 to August 1970 - Assistant Geologist; September 1970 to June 1975 - Associate Geologist; July 1975 to present - Geologist, Illinois State Geological Survey. In charge of the Hydrogeology and Geophysics Section since September 1974. Involved in ground-water studies and evaluation, application of geophysics to ground-water exploration, and evaluation of sanitary landfill sites for possible ground-water pollution problems. Research in geophysics (including geothermics, resistivity, and seismic refraction), ground-water flow systems and landfill studies.

September 1 - December 31, 1976: Visiting Associate Professor of Geology, the University of Waterloo, Waterloo, Ontario.

October 1960 - February 1961: Hydrologic Assistant, U. S. Geological Survey (temporary appointment). Involved with continued work on the Humboldt River Research Project.

June 1959 - September 1961: Geologist, State of Nevada (summers). Involved in the Humboldt River Research Project doing hydrogeologic mapping and geophysics.

Other Professional Activities

Consultant in hydrogeology with emphasis on problems associated with waste disposal; Secretary-Treasurer, Hydrogeology Division, Geological Society of America, 1973-75; Second Vice-Chairman, Hydrogeology Division, Geological Society of America, 1977-78; Associate Editor, Water Resources Research, American Geophysical Union, 1975-79; Member, City of Champaign Environmental Commission, 1976-78; Member, Technical Advisory Committee on "208" Planning for Illinois (Illinois Environmental Protection Agency), 1977; Chairman of Working Group on Solid Waste Siting, ASTM Subcommittee D18.14; Chairman, Hydrogeology Division, Geological Society of America, 1978-79.

Provided Hydrogeologic Consulting Services to the following:

Chem-Nuclear Services, Inc., Prosser, WA
Pullman, Inc., Chicago, IL
SCS Engineers, Long Beach, CA
Erdman, Anthony, Associates, Rochester, NY
James P. Collins and Associates, Boston, MA and Rochester, NY
Harza Engineering Co., Chicago, IL
U. S. Army Corps of Engineers
Law Engineering Testing Co., Marietta, GA
Canadian Environmental Law Association, Toronto, Ontario
U. S. Department of Energy
Lawrence Berkeley Laboratory, University of California, Berkeley, CA
Escambia County, Florida, Pensacola, FL
Ontario Ministry of the Environment, Toronto, Ontario
University of Arizona, Dept. Nuclear Engineering, Tucson, AZ
Argonne National Laboratory, Argonne, IL

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- Hawley, John P., William E. Wilson and Keros Cartwright, 1963, Geologic map in Bredehoeft, John D., 1963, Hydrology of the lower Humboldt River basin: Desert Research Institute, Tech. Report No. 3.
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- Bergstrom, R. E., Keros Cartwright, Kemal Piskin and M. R. McComas, 1968, Ground-water resources of the Quaternary deposits of Illinois: College of Agric., Special Publ. No. 14, p. 157-164.

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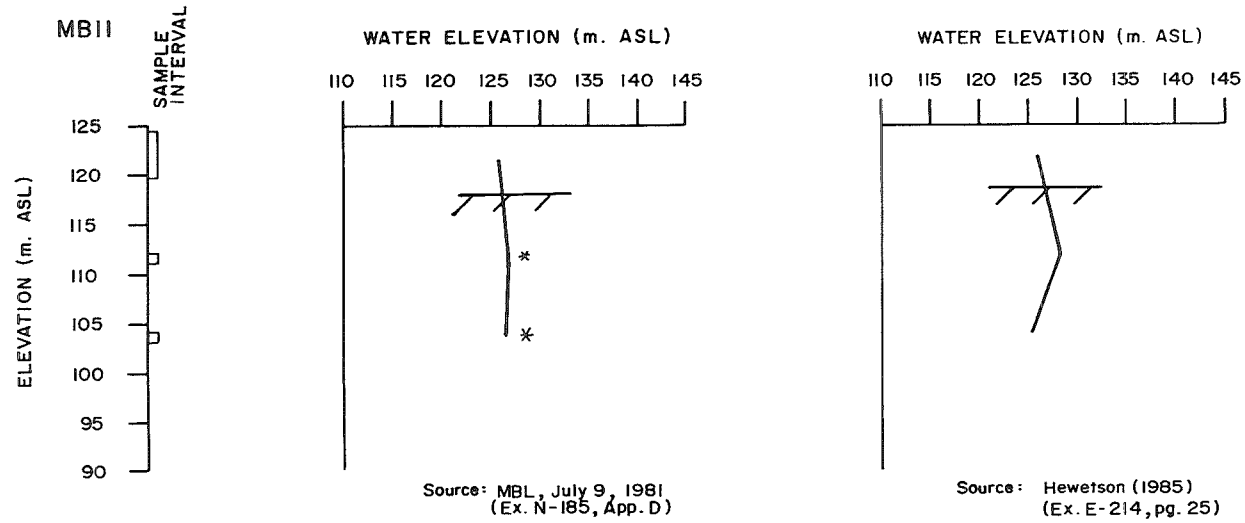
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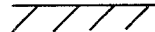
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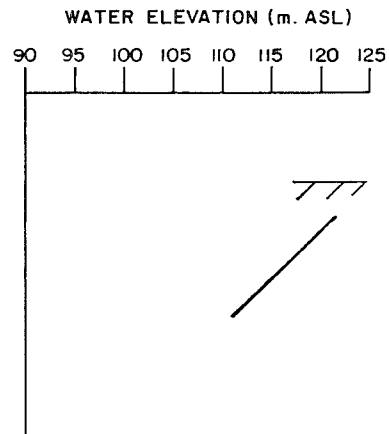
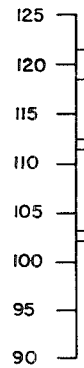


 Shale/Overburden Contact

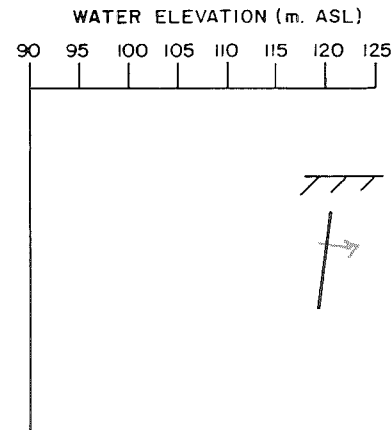
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WATER LEVEL IN MBII(3) IS 20cm ABOVE GROUND

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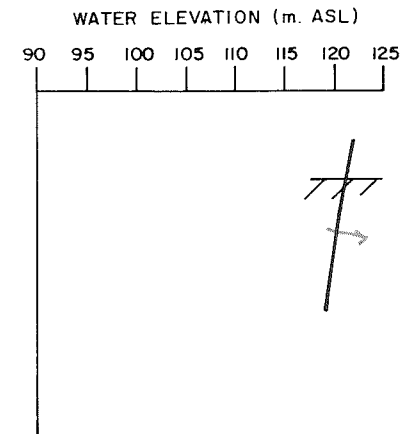
MB12



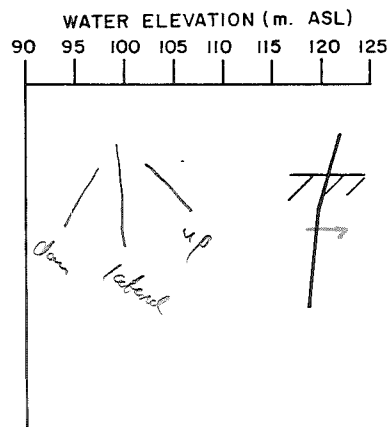
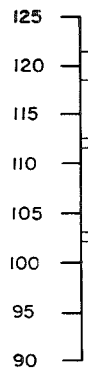
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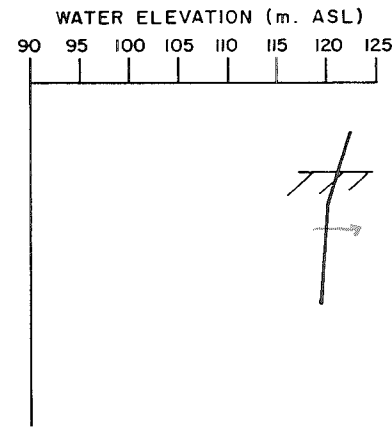
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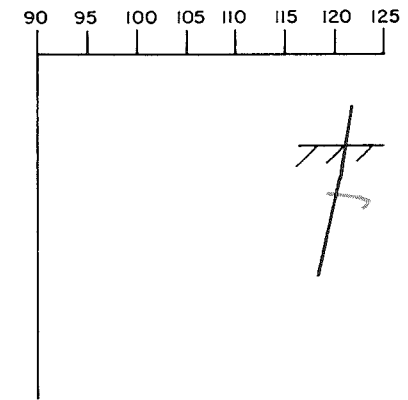
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(Ex. H-40A, App.8)



Source: Trow, February, 1986
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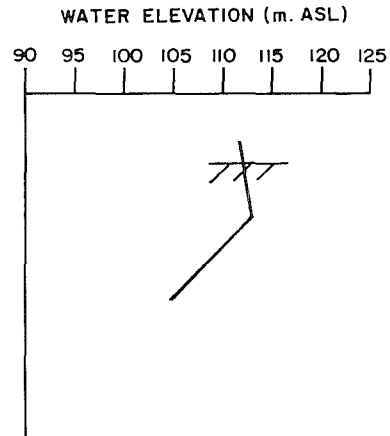
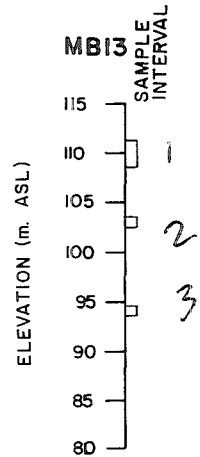
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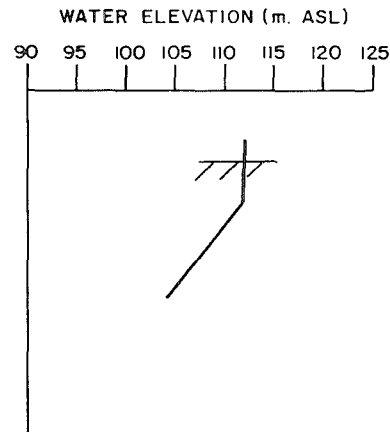
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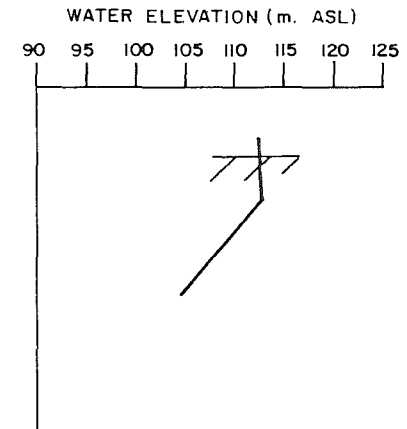
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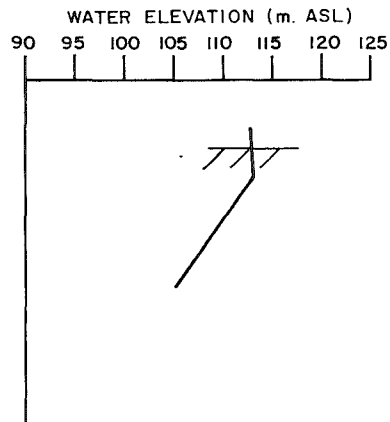
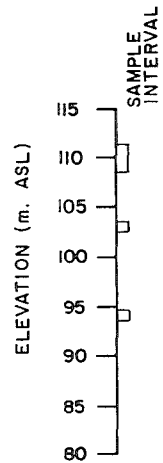
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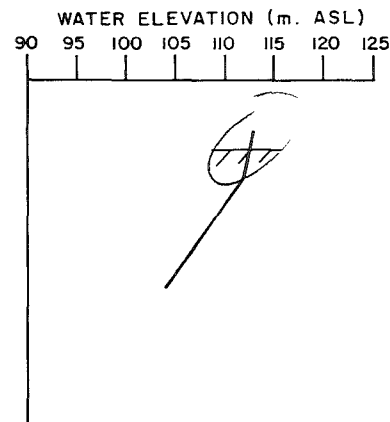
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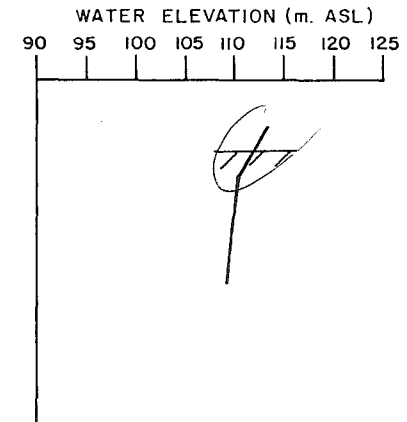
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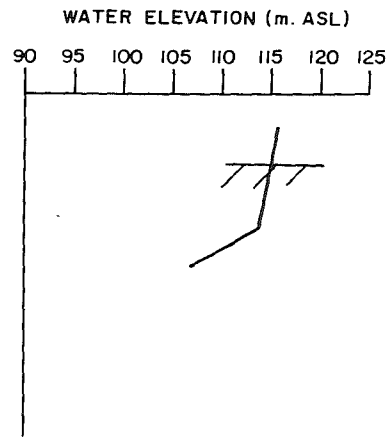
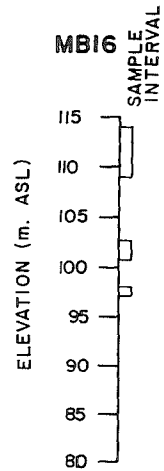


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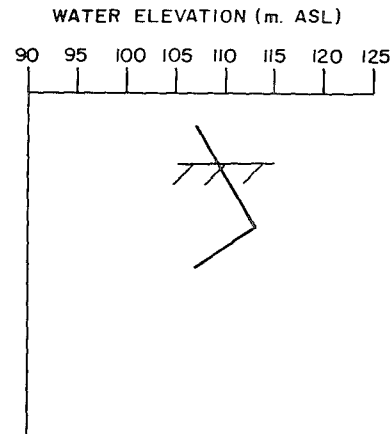


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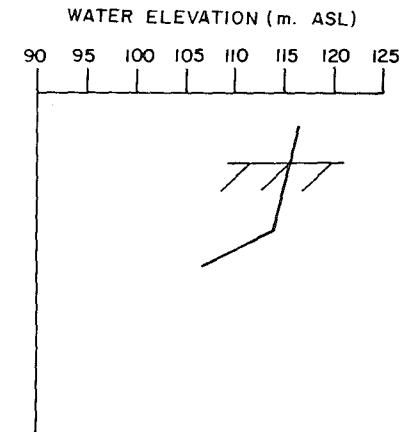
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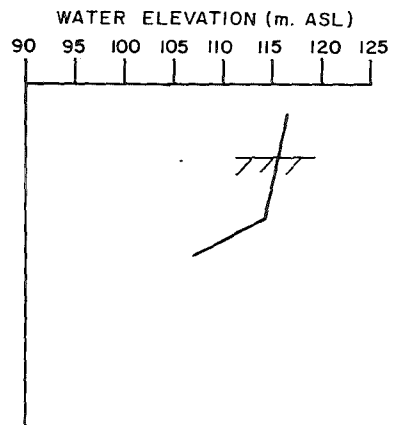
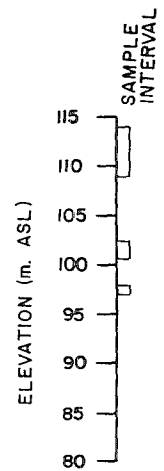
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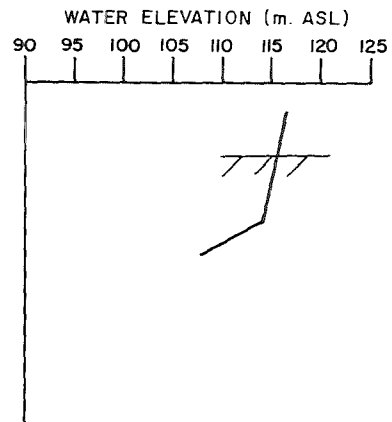
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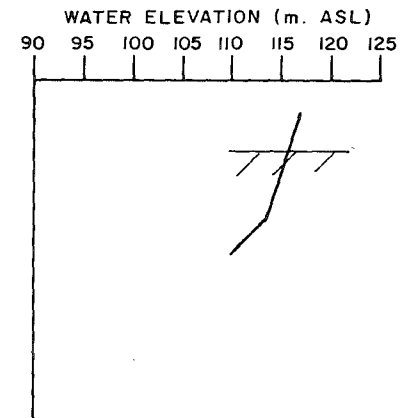
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Source: Trow, February, 1986
(Ex. H-40A, App. 8)



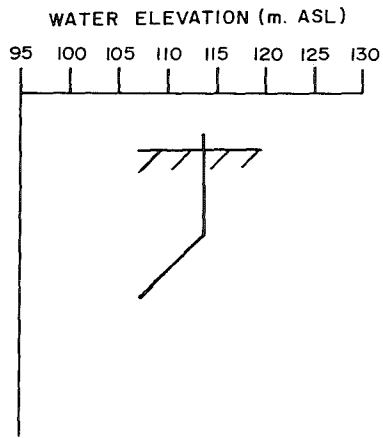
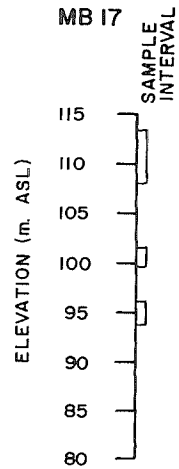
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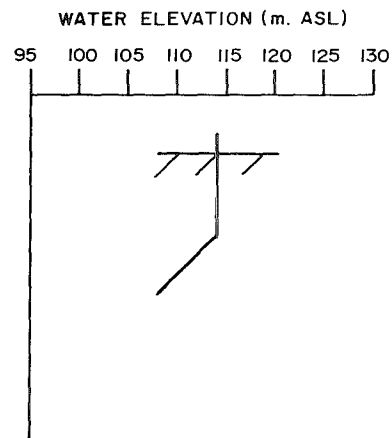
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(Ex. H-40A, App. 5)

/// Shale / Overburden Contact

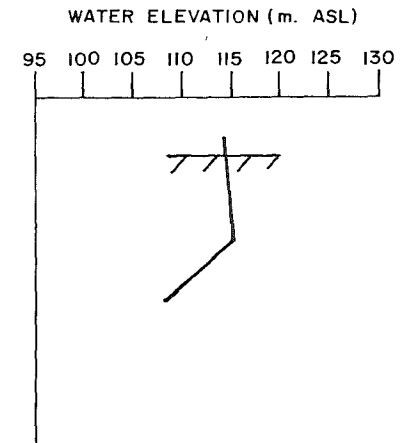
HYDRAULIC HEAD PROFILES - SITE F



Source: MBL, October 23, 1981
(Ex. N-185, App. D)



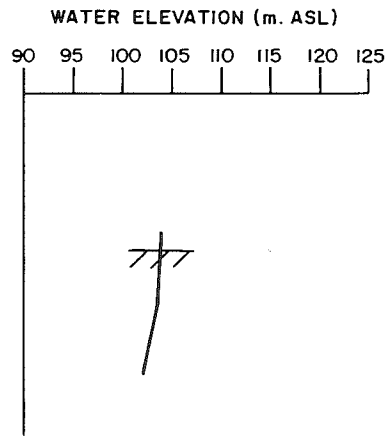
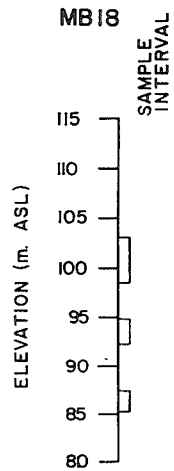
Source: Hewetson (1985)
(Ex. E-214, pg. 25)



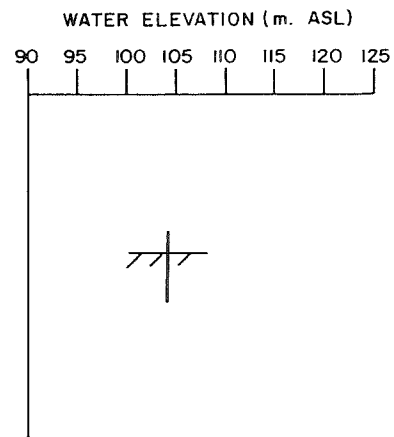
Source: Trow, March 12, 1986
(Ex. H-676)

Shale / Overburden Contact

HYDRAULIC HEAD PROFILES - SITE F



Source: MBL, October 23, 1981
(Ex. N-185, App. D)

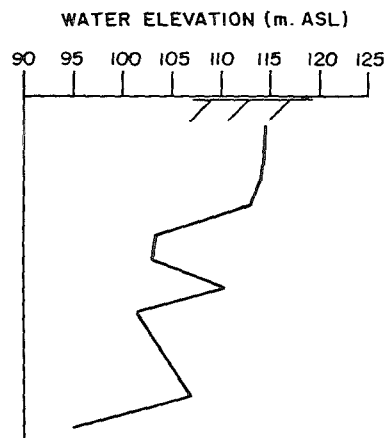
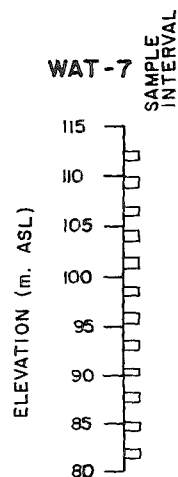


Source: Hewetson, 1985
(Ex. E-214, pg. 25)

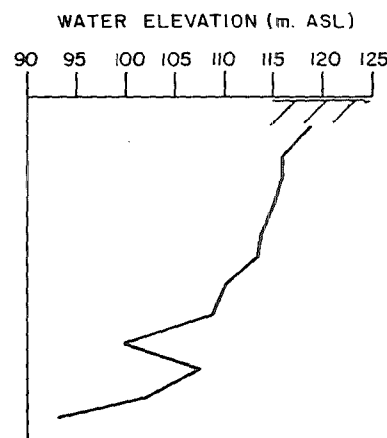
/// Shale /Overburden Contact

VERSION A

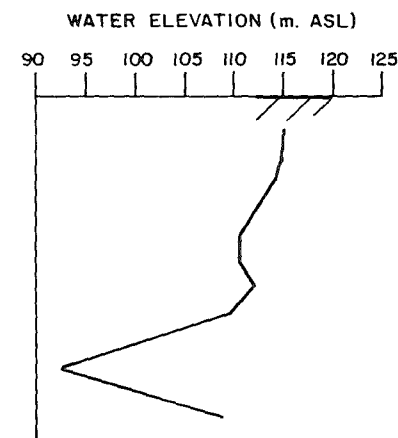
HYDRAULIC HEAD PROFILES - SITE F



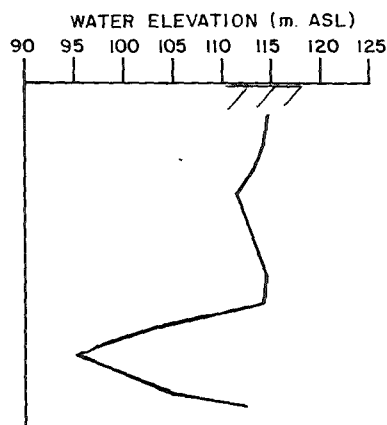
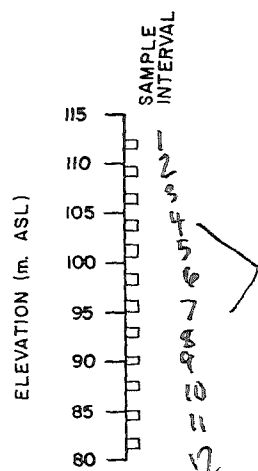
Source: Hewetson (1985)
(Ex. E-214, pg.25)



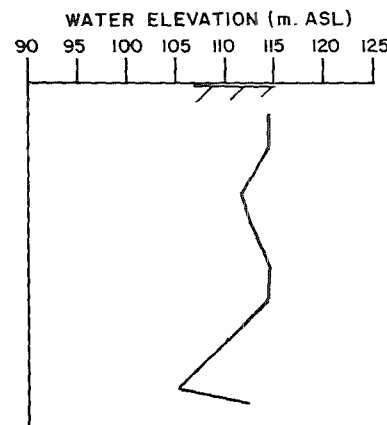
Source: Trow, December, 1985
(Ex. H-40A, App.5)



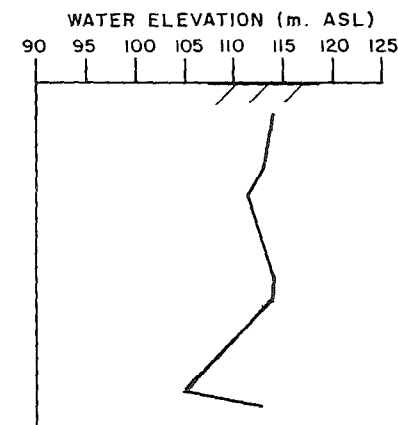
Source: Trow, April, 1986
(Ex. H-40A, App.5)



Source: Geocon, August 28, 1987
(Ex. M-437, App.A)



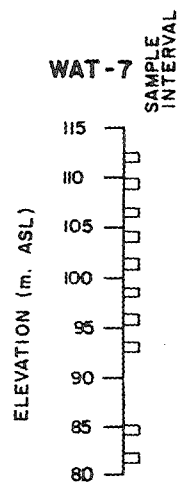
Source: Geocon, September 18, 1987
(Ex. M-437, App.A)



Source: Geocon, October 9, 1987
(Ex. M-437, App.A)

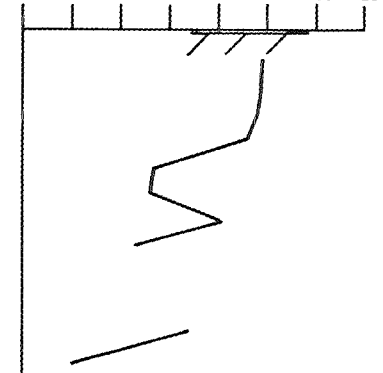
VERSION B

HYDRAULIC HEAD PROFILES - SITE F



WATER ELEVATION (m. ASL)

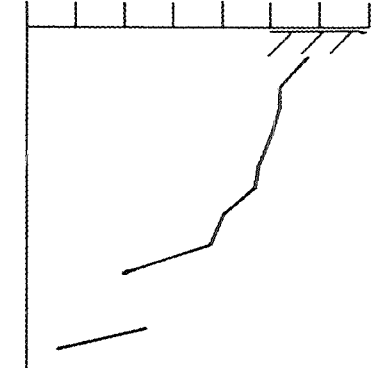
90 95 100 105 110 115 120 125



Source: Hewetson (1985)
(Ex. E-214, pg.25)

WATER ELEVATION (m. ASL)

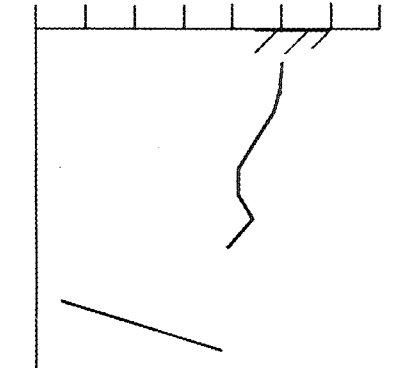
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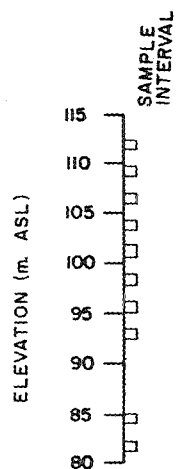
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WATER ELEVATION (m. ASL)

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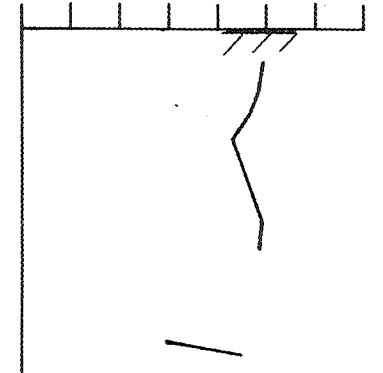


Source: Trow, April, 1986
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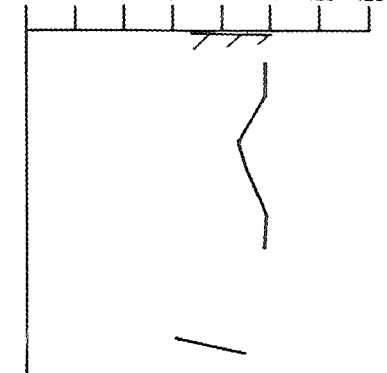
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Source: Geocon, August 28, 1987
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WATER ELEVATION (m. ASL)

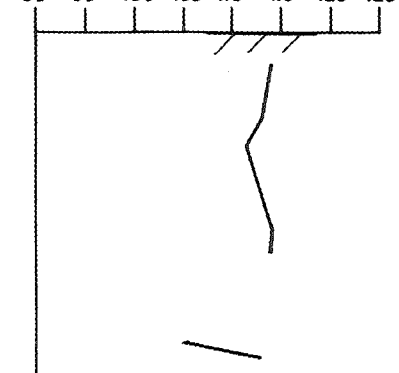
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Source: Geocon, September 18, 1987
(Ex. M-437, App.A)

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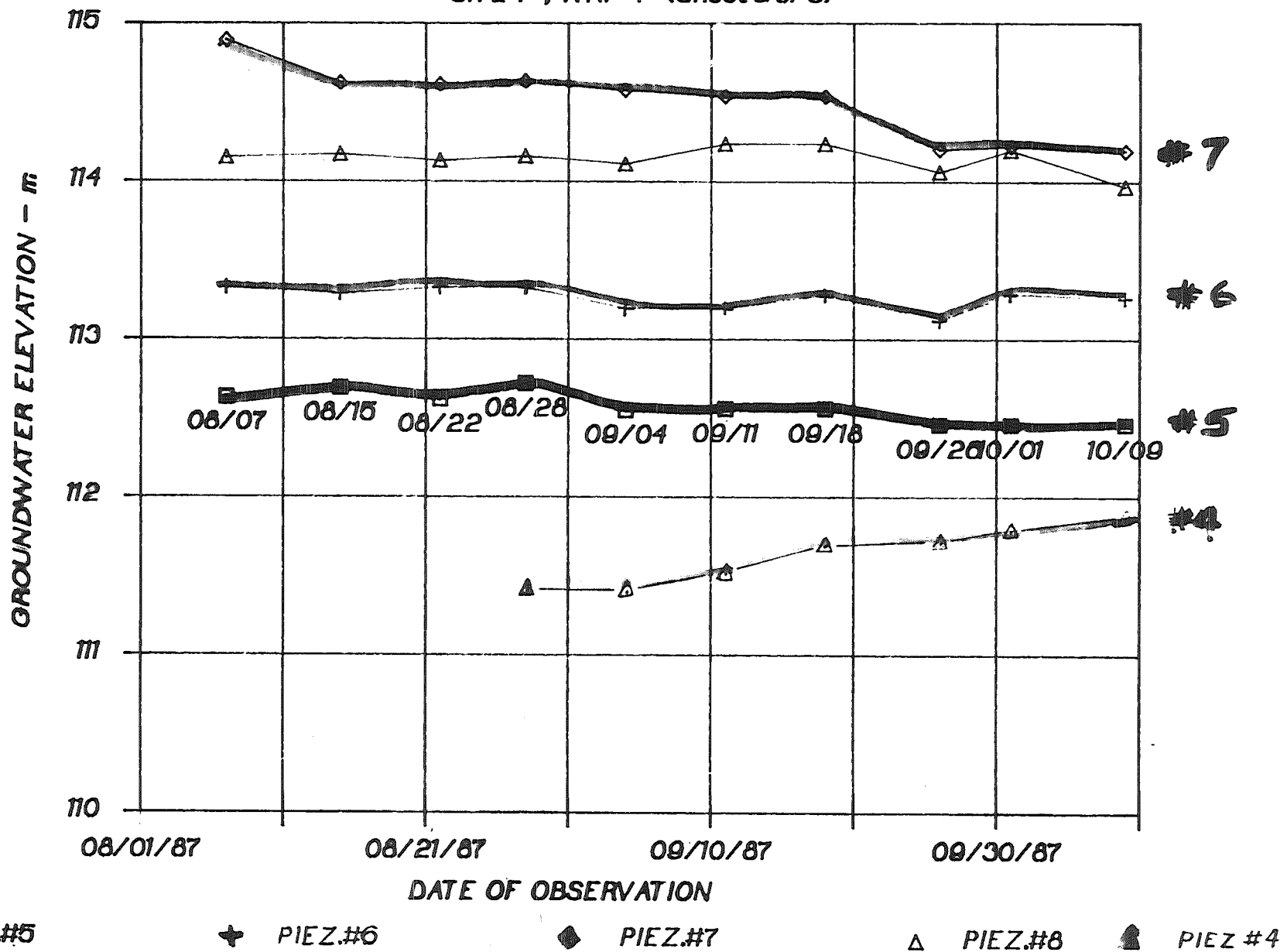
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Source: Geocon, October 9, 1987
(Ex. M-437, App.A)

N-751

SITE F, WAT-7 (Sheet 2 of 3)



HYDROGRAPHS FOR WAT-7
PIEZOMETERS (EX M-437, APP A)

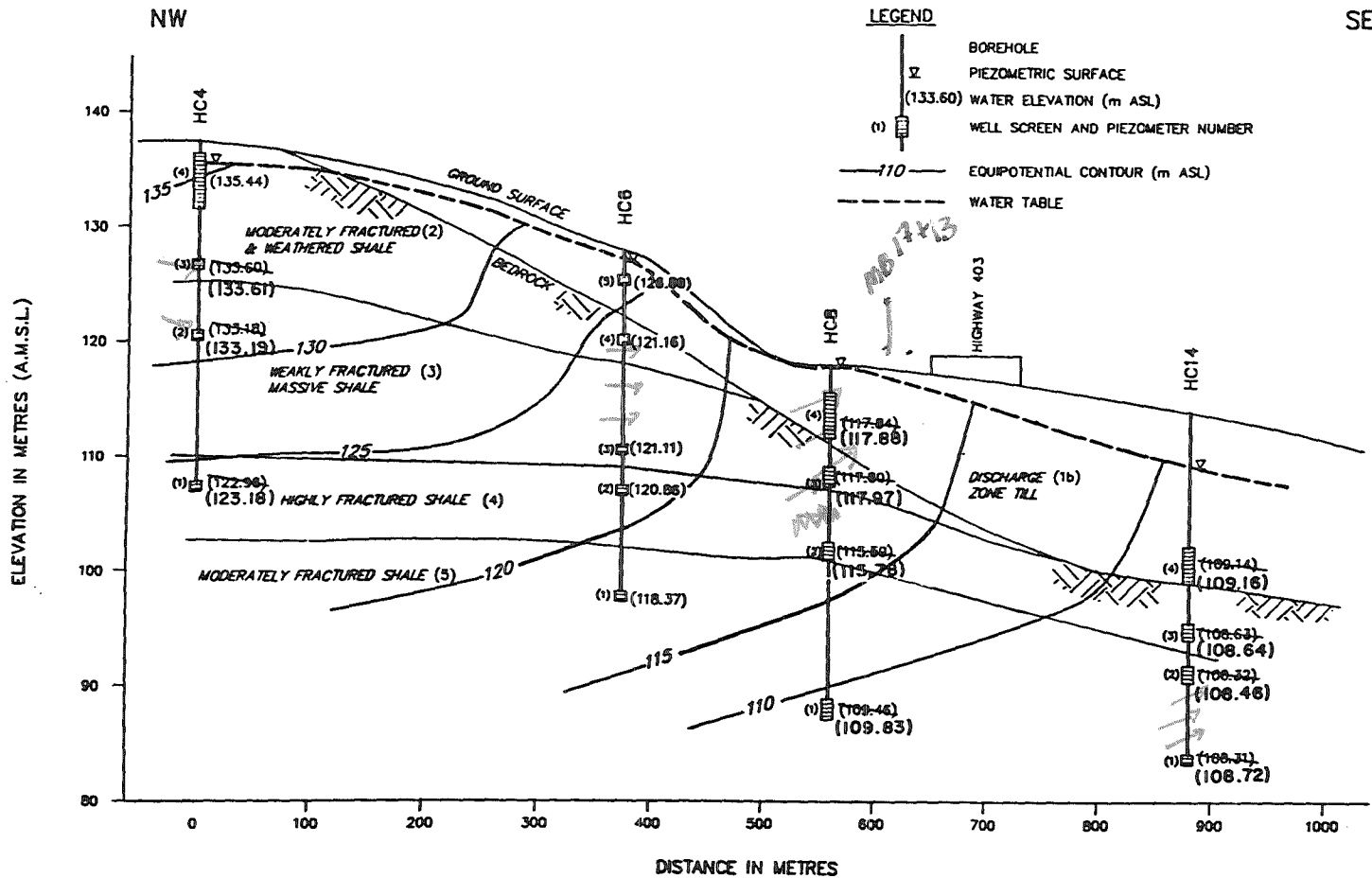
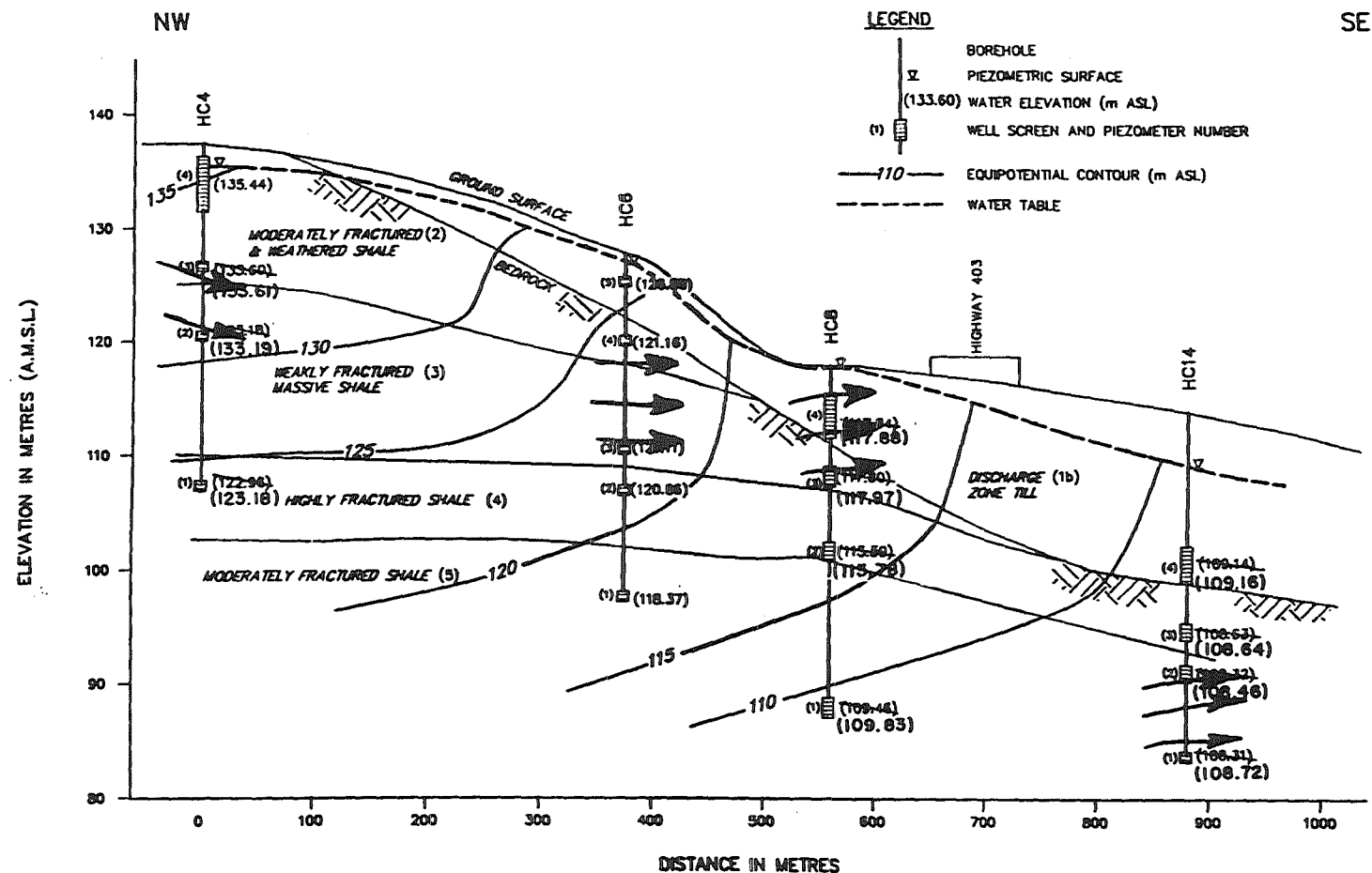


figure B.3.4
POTENTIOMETRIC DISTRIBUTION
Site 'F' - Hydrogeology - City of Burlington

1354-10-17/12/87-34-D-0

N-752



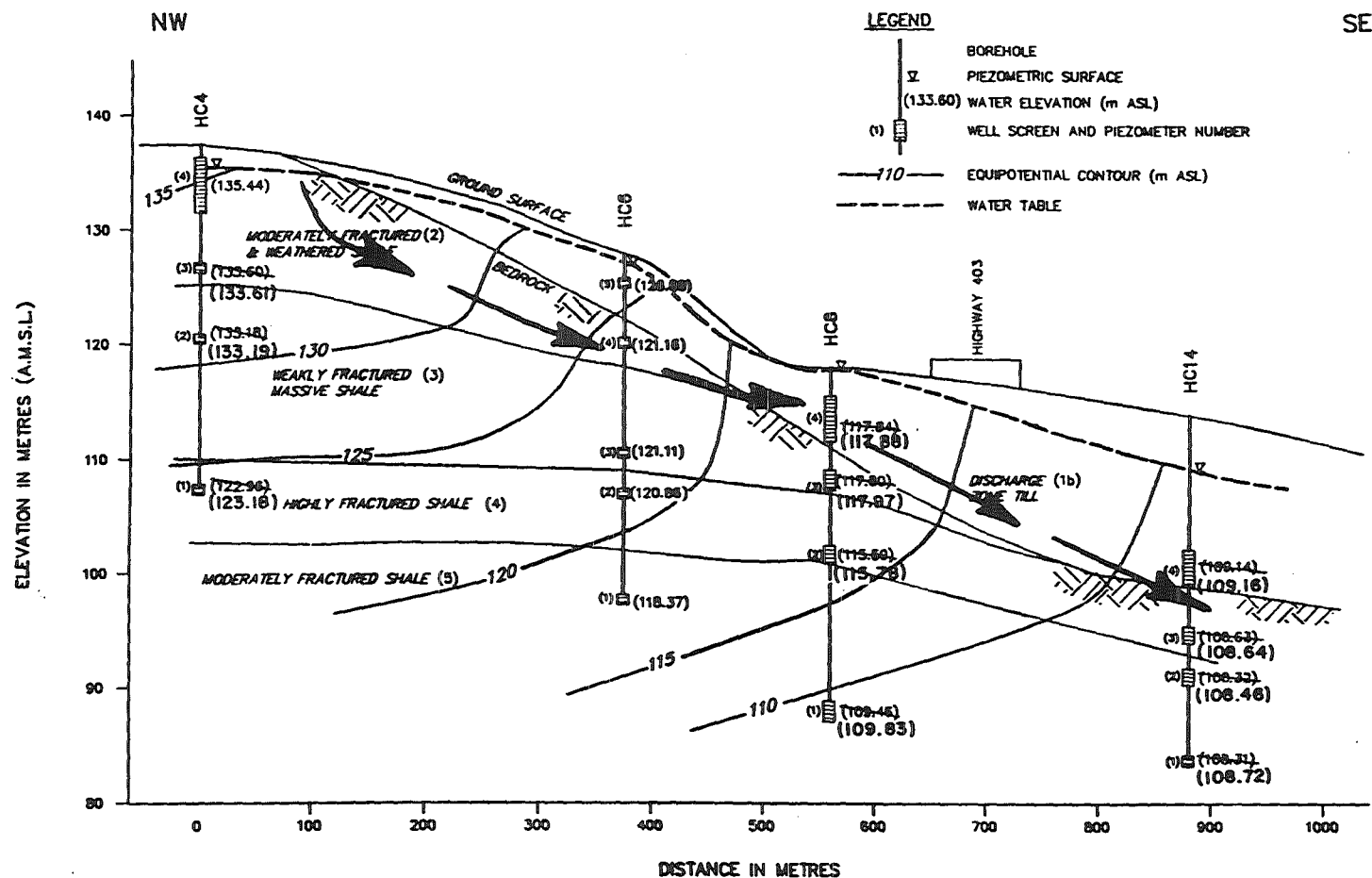
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figure B.3.4
 POTENTIOMETRIC DISTRIBUTION
 Site 'F' - Hydrogeology - City of Burlington

1354-10-17/12/87-34-D-0

FLOW PATHS AT SELECTED PIEZOMETERS

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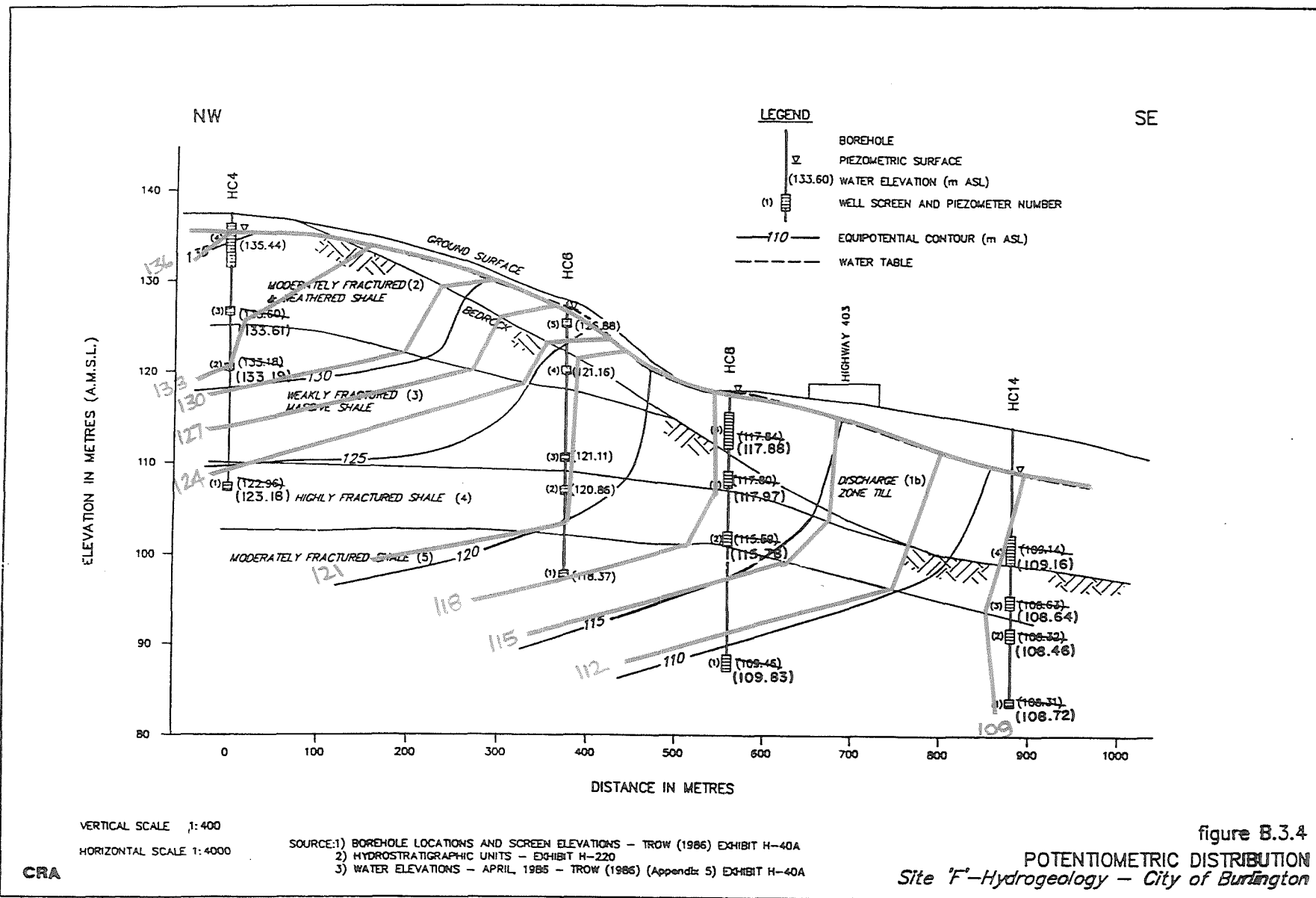
figure B.3.4
 POTENTIOMETRIC DISTRIBUTION
 Site 'F' - Hydrogeology - City of Burlington

1354-10-17/12/87-34-D-0

FLOW PATHS BASED ON CRA EQUIPOTENTIAL LINES

N 75 2 B

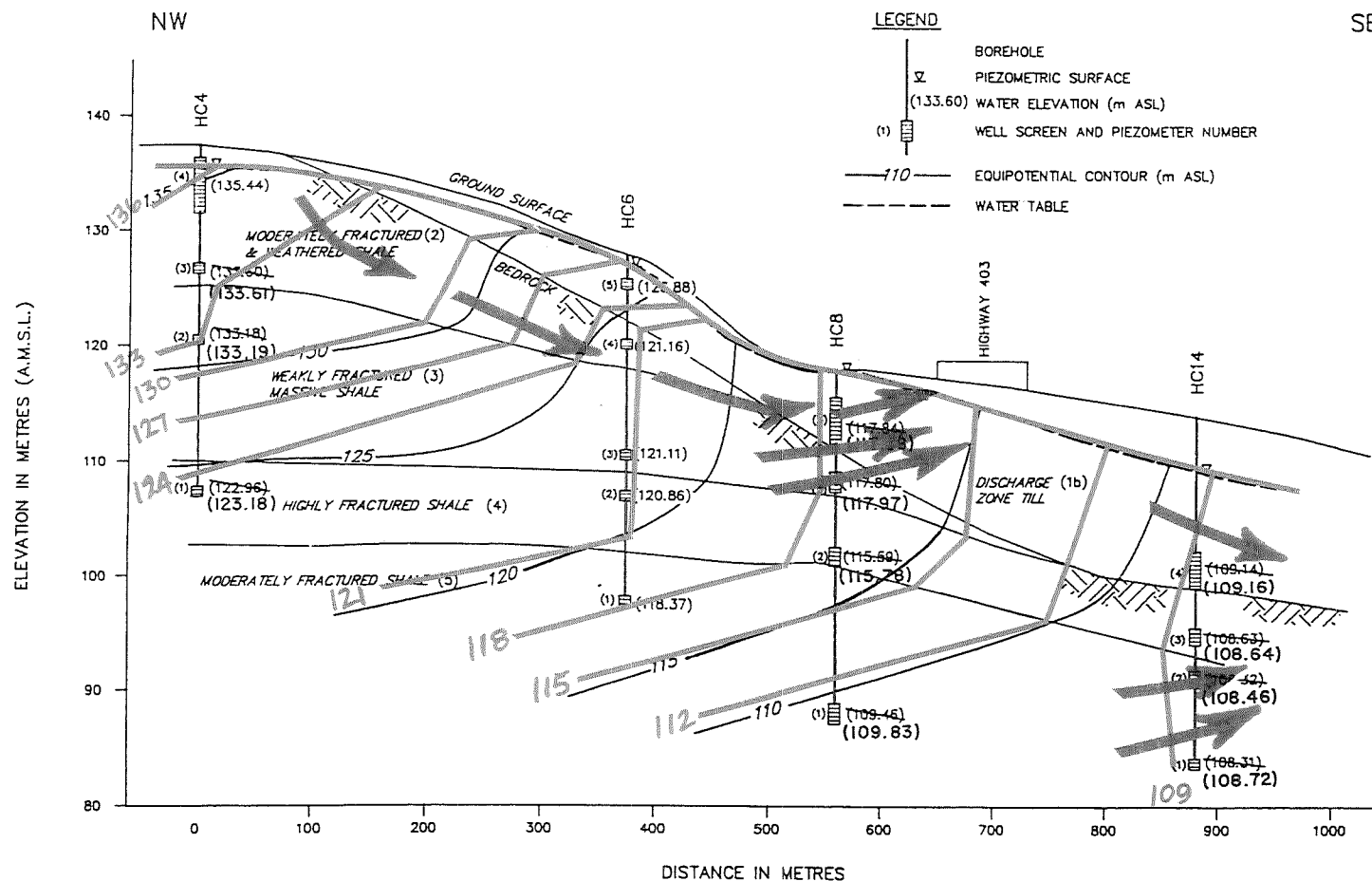
③



ALTERNATE INTERPRETATION OF EQUIPOTENTIAL LINES

N 752-C

①



CRA

SOURCE: 1) BOREHOLE LOCATIONS AND SCREEN ELEVATIONS - TROW (1986) EXHIBIT H-40A
 2) HYDROSTRATIGRAPHIC UNITS - EXHIBIT H-220
 3) WATER ELEVATIONS - APRIL 1986 - TROW (1986) (Appendix 5) EXHIBIT H-40A

figure B.3.4
 POTENTIOMETRIC DISTRIBUTION
 Site 'F'—Hydrogeology - City of Burlington

1354-10-17/12/87-34-0-0

ALTERNATE INTERPRETATION OF EQUIPOTENTIAL LINES
 FLOW PATHS BASED ON ALTERNATE EQUIPOTENTIAL LINES

N752-D. ⑤

①

N-753

PW 1. - site F.

~~SECRETARY ROOM To K~~

CRA

TROW

EX - B 508 G.M. $T = 2.2 \times 10^{-5} \text{ m}^2/\text{sec}$

~~SECRETARY ROOM To K~~

$K = 1.1 \times 10^{-4} \text{ cm/sec}$

b thickness 20.4

H40 p. 58. -

$T = 4.1 \times 10^{-5} \text{ (G.M.)}$

b = 83048

$K = 1.3 \times 10^{-2} \text{ cm/sec}$

EX - B 617 - $T = 1.3 \times 10^{-5} \text{ m}^2/\text{sec}$

b. 7m.

K. 2×10^{-4}

SUMMARY OF PUMPING TEST DATA - SITE "F"*

*Ex. H-676

**Ex. M-437, p. 4-6; Petrie Transcript
Volume 106, p. 25425

***Ex. H-84 (Addendum I)

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 1 (TW 1/86)		22.06		Date of Test: March 12/86
	TW 2/86 (OW 1/86)	4.58	16.4(N)	Pumping Time: 1530 min (25.5 hr)
	WAT 7(1)	0.20	±154(N)	
	7(2)	0 (0.04)	±154(N)	
	7(3)	0 (0.08)	±154(N)	
	7(4)	0.97	±154(N)	
	7(5)	2.33 —	±154(N)	
	7(6)	3.00	±154(N)	
	7(7)	0	±154(N)	
	7(8)	0 (+0.01)	±154(N)	
	7(9)	Plugged**		
	7(10)	Plugged**		
	7(11)	0		
	7(12)	0 (0.01)		
	MB 17(1)	0	±154(W)	
	17(2)	0	±154(W)	
	17(3)	0	±154(W)	
	MB 13(1)	0 (+0.07)	±19.0(W)	
	13(2)	6.37	±19.5(W)	
	13(3)	4.20	±19.5(W)	

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 1 (TW 1/86)	HC 8-85(1)	0	±479(W)***	
		8-85(2)	0	
		8-85(3)	0.03	
		8-85(4)	0	
	MB 16(1)	0 (+.13)	±156(E)	
		16(2)	3.40	
		16(3)	0.22	
	MB 12(1)	0 (0.05)	±234(N)	
		12(2)	0 (+0.01)	
		12(3)	0.10	
	HC 9-85(1)	0 (+0.01)	±312(S)	
		9-85(2)	0.01	
		9-85(3)	0.03	
		9-85(4)	0 (+0.10)	

- NOTES:
- Figure in brackets () indicates the actual drop or rise (+) in water level during pumping test.
 - A reduction in water level of less than 0.10 is taken as zero drawdown, except where an adjacent piezometer (above or below) shows a recovery or no change. Where water levels rise, drawdown is shown as 0.

N-755

Groundwater and Wells

Second Edition

Fletcher G. Driscoll, Ph.D.
Principal Author and Editor

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CHAPTER 9

Well Hydraulics

A well is a hydraulic structure which, when properly designed and constructed, permits the economic withdrawal of water from a water-bearing formation. Successful wells are designed and built by engineers and drilling contractors who:

1. Use materials that will provide an efficient well with a long service life.
2. Use techniques in drilling and well construction that take maximum advantage of the hydrogeologic conditions.
3. Apply the principles of hydraulics in a practical way to the analysis of wells and aquifer performance.

Individuals who design or construct wells must understand the fundamentals of well hydraulics. Some aspects of well hydraulics can be complicated, and few engineers have mastered all phases of the subject. In some cases, mathematicians have developed such complex solutions for specific well and aquifer conditions that practical application of the theory is nearly impossible in light of all the geologic and hydrologic uncertainties. Furthermore, certain geologic and aquifer environments are so complex that reliable analytical solutions for the flow patterns are almost unobtainable. Given the complexity of the underground environment, only the most fundamental hydraulic theories can be used successfully in everyday well design and construction. Remarkably, these basic methods regularly yield accurate results in most cases without laborious calculations. This chapter examines these fundamental and practical methods in detail.

The reader is cautioned, however, that extensive literature exists on hydraulics that is not treated in this chapter because of its complexity or narrow application. Good references particularly strong in hydraulic theory include works by Lohman (1979), McWhorter and Sunada (1977), Bennett (1976), Bear (1972, 1979), Deju (1971), and Walton (1970).

In Chapter 5, Darcy's law was introduced, the fundamental equations for groundwater movement were developed, and the storage and drainage characteristics of water in aquifers were examined. Application and extensions of these concepts can now be made to solve some typical problems of flow toward wells. Information in this chapter will enable practicing engineers and drilling contractors to make sound decisions on well design and construction for most situations.

Continuous-slot screens permit pumping tests to proceed. (Keck Geo-

ds. Not only can water quality for any test be somewhat higher,

DEFINITION OF TERMS

It is important to understand clearly the meaning of common terms related to pumping wells. Definitions are presented below, and several terms are defined diagrammatically in Figure 9.1.

Static Water Level (SWL) — This is the level at which water stands in a well or unconfined aquifer when no water is being removed from the aquifer either by pumping or free flow. It is generally expressed as the distance from the ground surface (or from a measuring point near the ground surface) to the water level in the well. For example, when the static water level in a well is 15 ft (4.6 m), it means that water stands 15 ft below the measuring point when there is no pumping. For an artesian well which flows at the ground surface, the static water level is expressed as a height above the ground surface. When artesian flow is stopped or contained at the ground surface, the pressure developed is referred to as the shut-in head. If the well has a shut-in head of 3 psi (20.7 kPa) at the surface, it means that the confining pressure will cause the water to

rise 7 ft (2.1 m) in a pipe extending above the ground surface.

Pumping Water Level (PWL) — This is the level at which water stands in a well when pumping is in progress. In the case of an artesian well, it is the aboveground level at which water is flowing from the well. The pumping water level is also called the dynamic water level as measured in the well.

Drawdown — Drawdown is the difference, measured in feet or meters, between the water table or potentiometric surface and the pumping water level. This difference represents the head of water (force) that causes water to flow through an aquifer toward a well at the rate that water is being withdrawn from the well. In the unconfined case, the head is represented graphically by the actual water level at a point along the drawdown curve. In confined conditions, the drawdown curve represents the pressure head at that point. To differentiate these two types of drawdown, all diagrams in this book show the water table for unconfined conditions as a solid line and the potentiometric surface for confined conditions as a dashed line.

Residual Drawdown — After pumping is stopped, the water level rises and approaches the static water level observed

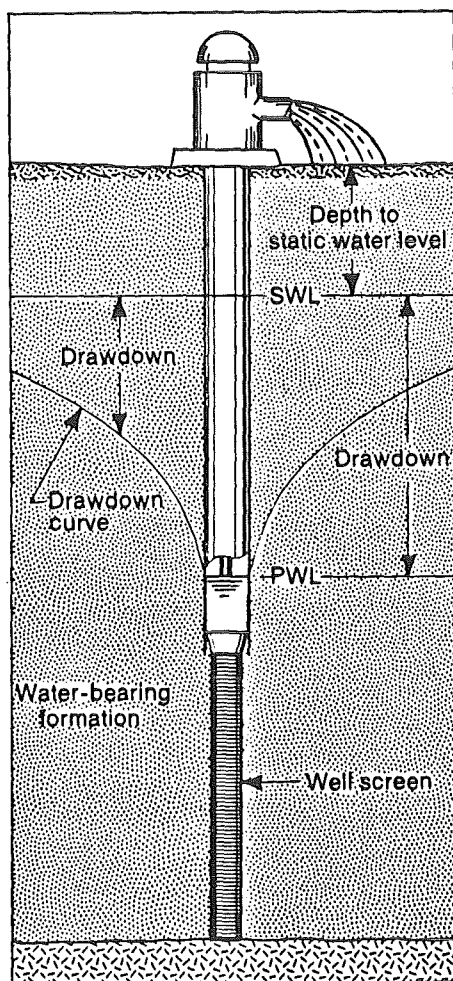


Figure 9.1. Terms relating to well performance.

before pumping began. During water-level recovery, the distance between the water level and the initial static water level is called residual drawdown.

Well Yield — Yield is the volume of water per unit of time discharged from a well, either by pumping or free flow. It is measured commonly as a pumping rate in gallons per minute or cubic meters per day.

Specific Capacity — Specific capacity of a well is its yield per unit of drawdown, usually expressed as gallons of water per minute per foot (gpm/ft) of drawdown or cubic meters per day per meter ($\text{m}^3/\text{day}/\text{m}$) of drawdown, after a given time has elapsed, usually 24 hours. Dividing the yield of a well by the drawdown, when each is measured at the same time, gives the specific capacity. For instance, if the pumping rate is 1,000 gpm ($5,450 \text{ m}^3/\text{day}$) and the drawdown is 30 ft (9.1 m), the specific capacity of the well is about 33.3 gpm per ft of drawdown ($599 \text{ m}^3/\text{day}/\text{m}$ of drawdown) at the time the measurements were taken. Specific capacity generally varies with duration of pumping — as pumping time increases, specific capacity decreases. Also, specific capacity decreases as discharge increases in the same well. The reasons for decreasing specific capacity are discussed later in this chapter.

Static water level, pumping water level, drawdown, and residual drawdown apply similarly to a pumped well or other nearby wells and observation wells. For example, if the water level in an observation well located 80 ft (24.4 m) from a pumping well dropped 3 ft (0.9 m) as a result of the pumping, this lowering in the observation well is called its drawdown.

NATURE OF CONVERGING FLOW

The water level in the vicinity of a pumped well under unconfined conditions is lowered when pumping begins, with the greatest drawdown occurring in the well. As the pump removes water, an area of low pressure develops near the well bore. Because the water level is lower in a pumped well than at any place in the water-bearing formation surrounding it, water moves from the formation into the well to replace water being withdrawn by the pump. The pressure (force) that drives the water toward the well is called the head, which is the difference between the water level inside the well and the water level at any place outside the well. At some distance from the well a point is reached where the water level is essentially unaffected. This distance varies for different wells. It also varies for the same well, depending on both the pumping rate and the length of time the well is pumped.

In confined formations, the saturated thickness of the aquifer is generally not reduced during pumping. Hydrostatic pressure, however, is reduced in the aquifer, and the pressure drop is greatest at the well bore. The pressure drop is directly analogous to the dewatering effect in unconfined aquifers.

During pumping, water flows toward the well from every direction. As the water moves closer to the well, it moves through imaginary cylindrical sections that are successively smaller in area. Thus, as the water approaches the well, its velocity increases. In Figure 9.2, A_1 represents the area of a cylindrical surface 100 ft (30.5 m) from the center of the well, and A_2 represents the area of a similar surface 50 ft (15.2 m) from the well. Because A_1 is twice A_2 , and the same quantity of water flows toward the pumped well through both cylinders, the velocity V_2 must be twice V_1 .*

*The equation for the surface area of a cylinder is $A = 2\pi rh$, where $\pi = 3.14$, r is the radius of the cylinder, and h is its height.

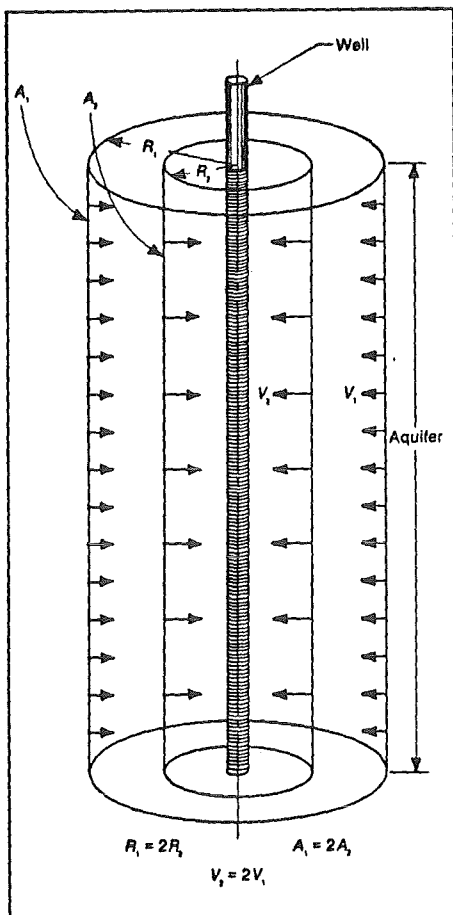


Figure 9.2. As flow converges toward a well, it passes through imaginary cylindrical surfaces that are successively smaller as the well is approached.

drilled at various distances from a pumped well. Only one side is shown; the other side is similar. This curve is called the drawdown curve and represents the lower limits of the cone of depression. In an unconfined aquifer, it represents the level to which the formation remains saturated. In a confined aquifer, it represents the hydrostatic pressure in the aquifer. Drawdown at any given point is the difference between the water level indicated by the curve and the static water level.

Head loss is a term used to describe the difference in head (pressure) that is required to cause flow from one point to another in an aquifer; it is a measure of the force required to overcome resistance to flow. The head losses from point to point along the pumping-water-level curve in Figure 9.4 are the differences in drawdown between these points.

Suppose, for example, that a well is pumped at a constant rate of 600 gpm (3,270 m³/day). At a distance of 28 ft (8.5 m) from the well, the drawdown is about 5 ft (1.5 m). This indicates that 5 ft of head are required to force 600 gpm of water through the formation from the outer limit of the cone of depression to within 28 ft of the well.

Darcy's law indicates that the velocity of flow through porous media varies directly with the hydraulic gradient. As the hydraulic gradient increases, velocity increases as flow converges toward a well. As a result, the lowered water surface develops a continually steeper slope toward the well. The form of this surface resembles a cone and is called the cone of depression. When pumped, all wells are surrounded by a cone of depression. Each cone differs in size and shape depending upon the pumping rate, pumping duration, aquifer characteristics, slope of the water table, and recharge within the cone of depression of the well.

Figure 9.3 shows two cones of depression around pumped wells that illustrate how transmissivity of an aquifer affects the shape of the cone. In a formation with low transmissivity, the cone is deep with steep sides and has a small radius. In a formation with high transmissivity, the cone is shallow with flat sides and has a large radius. The explanation for these different cone shapes is clear, for greater hydraulic head (feet of head) is required to move water through a less permeable formation than through a more permeable formation.

Figure 9.4 shows the levels at which water would be found in observation wells

Drawdown (ft)

Drawdown (ft)

Figure of de

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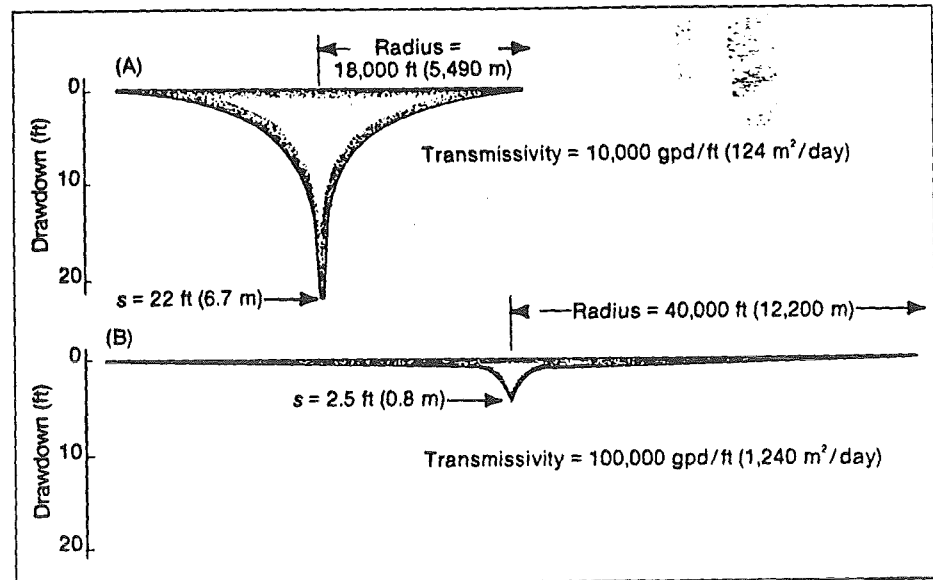


Figure 9.3. Effect of different coefficients of transmissivity on the shape, depth, and extent of the cone of depression. Pumping rate and other factors are constant.

Another 5 ft of head is required to move the same volume of water from 28 ft to within 14 ft (4.3 m) of the well. At this point, the drawdown is about 10 ft (3 m). The remainder of the total drawdown or head loss is used in pushing the water through the last 14 ft of the formation and through the well screen. The total drawdown of 20 ft (6.1 m) in the well is the head in feet required to move 600 gpm through the aquifer (within the cone of depression of the well) and into the well. This example shows that more head is expended for a given horizontal distance as the flow converges toward the well bore. The area through which the water moves decreases steadily while the velocity increases, resulting in increasing head loss along the flow path toward the well.

Before proceeding to the equations describing groundwater flow, three important terms must be defined. Each of these terms describes a characteristic of the aquifer that can be determined by pumping.

Radius of influence, R , is the horizontal distance from the center of a well to the limit of the cone of depression. It is larger for cones of depression in confined aquifers than for those in unconfined aquifers. The reason for this difference will become clear later in this chapter.

Coefficient of storage, S , of an aquifer represents the volume of water released

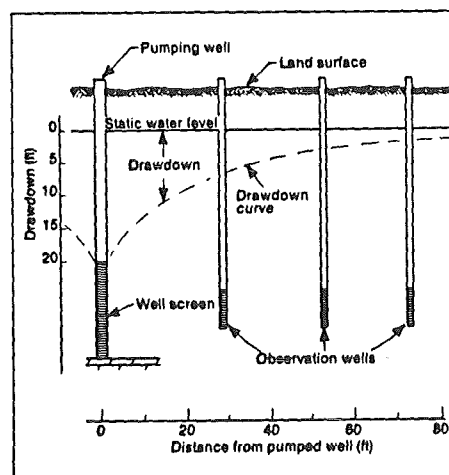


Figure 9.4. Trace of half a cone of depression showing variations in drawdown with distance from a pumped well.

from storage, or taken into storage, per unit of aquifer storage area per unit change in head. In unconfined aquifers, S is the same as the specific yield of the aquifer. In confined aquifers, S is the result of compression of the aquifer and expansion of the confined water when the head (pressure) is reduced during pumping. The coefficient of storage is dimensionless. Values of S for unconfined aquifers range from 0.01 to 0.3; values for confined aquifers range from 10^{-5} to 10^{-3} . Figure 5.6 (page 69) demonstrates how different volumes of water are released from storage in unconfined and confined aquifers per unit head loss.

Coefficient of transmissivity, T , of an aquifer, as defined in Chapter 5, is the rate at which water flows through a vertical strip of the aquifer 1 ft or 1 m wide and extending through the full saturated thickness, under a hydraulic gradient of 1 (100 percent). Figure 9.5 illustrates the concepts of hydraulic conductivity and transmissivity. Values of T range from less than 1,000 to more than 1 million gpd/ft (12.4 to over 12,400 m^2/day). If an aquifer has a transmissivity of less than 1,000 gpd/ft, it can supply only

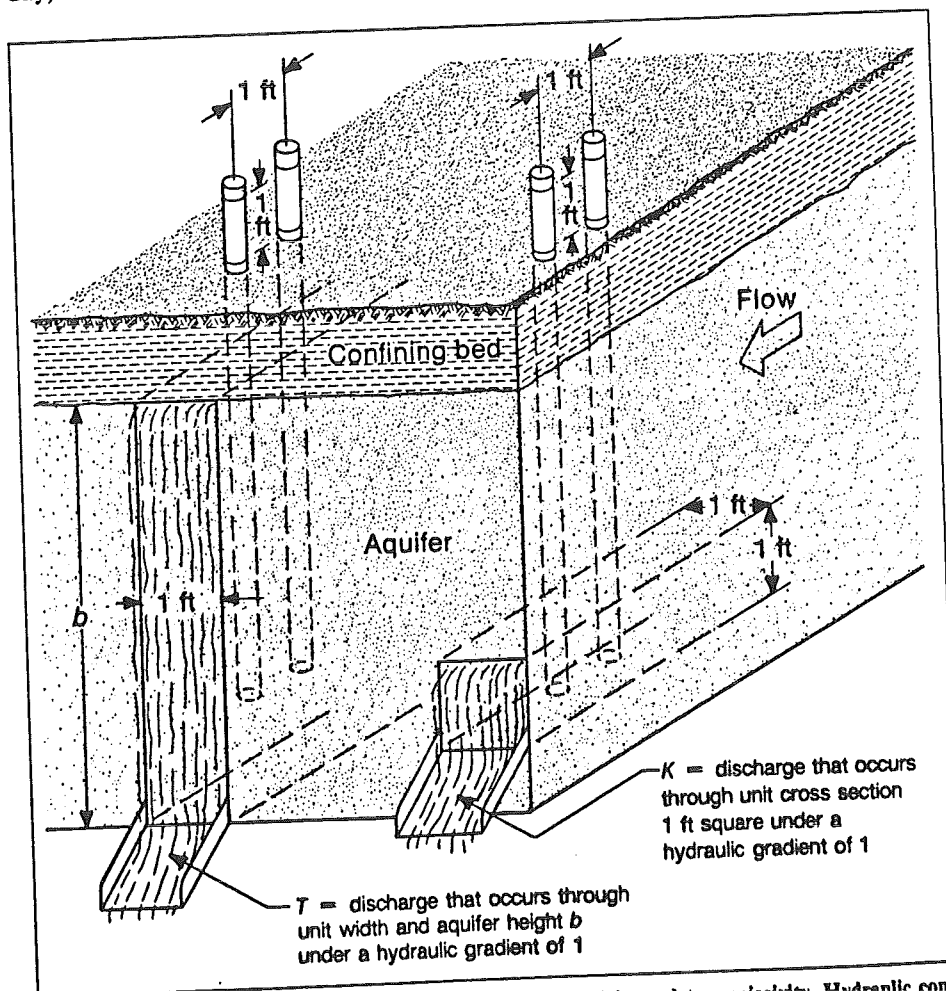


Figure 9.5. Illustration of the coefficients of hydraulic conductivity and transmissivity. Hydraulic conductivity multiplied by the aquifer thickness equals coefficient of transmissivity.

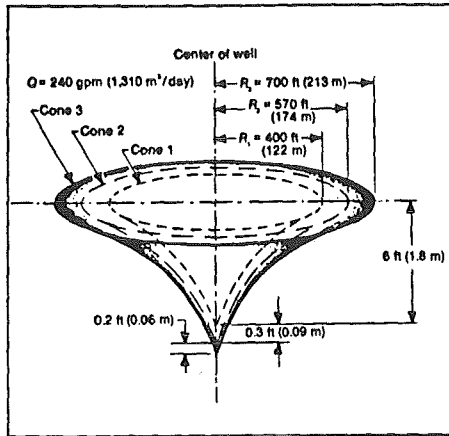


Figure 9.7. Changes in radius and depth of cone of depression after equal intervals of time, at constant pumping rate.

It is evident from this example that after some hours deepening or expansion of the cone during short intervals of pumping is barely discernible. This often leads observers to conclude that the cone has stabilized and will not expand or deepen as pumping continues. The cone of depression will continue to enlarge, however, until one or more of the following conditions is met:

1. It intercepts enough of the flow in the aquifer to equal the pumping rate.
2. It intercepts a body of surface water from which enough additional water will enter the aquifer to equal the pumping rate when combined with all the flow toward the well.
3. Enough vertical recharge from precipitation occurs within the radius of influence to equal the pumping rate.
4. Sufficient leakage occurs through overlying or underlying formations to equal the pumping rate.

When the cone has stopped expanding because of one or more of the above conditions, equilibrium exists. There is no further drawdown with continued pumping. In some wells, equilibrium occurs within a few hours after pumping begins; in others, it never occurs even though the pumping period may be extended for years.

EQUILIBRIUM WELL EQUATIONS

More than a hundred years ago, engineers began work on adapting Darcy's basic flow equation to groundwater flow toward a pumping well. The objective was to derive simple mathematical expressions for describing the flow regime of water in the ground. Because direct observation of groundwater movement is impossible, mathematical analysis offers a convenient and reliable way to predict what happens to water in the ground.

Well discharge equations for equilibrium conditions were derived by various investigators (Slichter, 1899; Turneaure and Russell, 1901; Thiem, 1906). These equations relating well discharge to drawdown assumed two-dimensional radial flow toward a well (the vertical component of flow is ignored). There are two basic equations; one for unconfined conditions and the other for confined conditions. For both equa-

has only extended outward an additional 170 ft (51.8 m) and deepened by an additional 0.3 ft (0.09 m). An additional radial expansion of only 130 ft (39.6 m) and an increase in depth of only 0.2 ft (0.06 m) occurs in the next 10 hours. Calculations of the volume of each of the cones would show that cone 2 has twice the volume of cone 1, and cone 3 has three times the volume of cone 1. This occurs because, at a constant pumping rate, the same volume of water is discharged from the well during each 10-hour interval. Thus, the increase in volume of the cone of depression is constant over time if the well is being pumped at a constant rate and the aquifer is homogeneous.

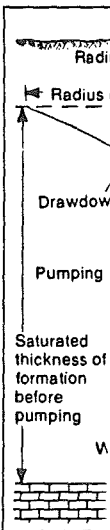


Figure 9.9. meaning of equation.

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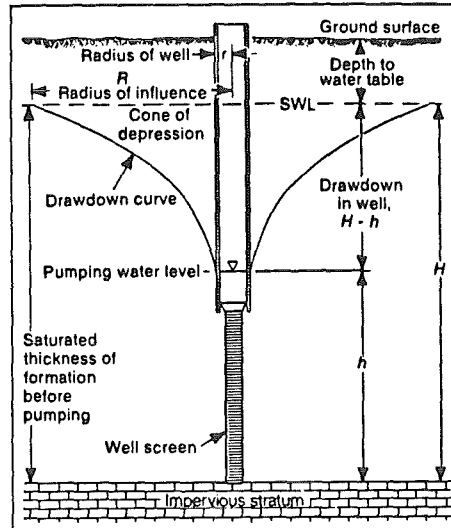


Figure 9.9. Well in a confined aquifer showing the meaning of various terms used in the equilibrium equation.

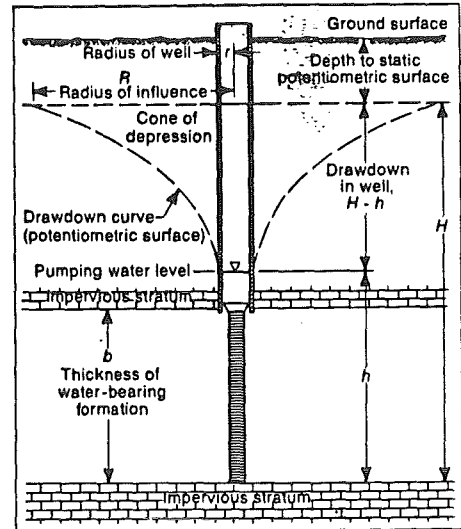


Figure 9.8. Well in an unconfined aquifer showing the meaning of the various terms used in the equilibrium equation.

tions, all dynamic conditions in the well and ground are assumed to be in equilibrium; that is, the discharge is constant, the drawdown and radius of influence have stabilized, and water enters the well in equal volumes from all directions. Both assume horizontal flow everywhere in the aquifer with recharge occurring at the periphery of the cone of depression. Figure 9.8 shows a vertical section of a well constructed in an *unconfined* aquifer. The equation for the well yield of an unconfined aquifer is:

English Engineering Units*

$$Q = \frac{K(H^2 - h^2)}{1,055 \log R/r}$$

where

Q = well yield or pumping rate, in gpm
 K = hydraulic conductivity of the water-bearing formation, in gpd/ft²

H = static head measured from bottom of aquifer, in ft
 h = depth of water in the well while pumping, in ft

International System of Units*

$$Q = \frac{1.366 K(H^2 - h^2)}{\log R/r} \quad (9.1)$$

where

Q = well yield or pumping rate, in m³/day
 K = hydraulic conductivity of the water-bearing formation, in m³/day/m² (m/day)

H = static head measured from bottom of aquifer, in m
 h = depth of water in the well while pumping, in m

*The United States is slowly converting to the International System of Units from the older English system. For years the scientific community has used the metric system [now System International (SI)], but only recently have other business and technological entities adopted it. In the future, many engineers and drillers will have to present information in the metric system. Therefore, equations in both the SI and English systems of measure are given throughout this text; the SI notation is on the right of each page and the English system is on the left. The complete International System of Units as it applies to water resources is given in Appendix 9.A. A convenient unit conversion table is also provided. Symbols used in the text are given in Appendix 9.B.

R = radius of the cone of depression, in ft R = radius of the cone of depression, in m

r = radius of the well, in ft

r = radius of the well, in m

Equation 9.1 is often called the equilibrium, or Thiem, equation.

Figure 9.9 is a vertical section of a well pumping from a *confined* aquifer. The equation for a well operating under confined conditions is:

$$Q = \frac{K b (H - h)}{528 \log R/r} \qquad Q = \frac{2.73 K b (H - h)}{\log R/r} \quad (9.2)$$

where

b = thickness of aquifer, in ft

All other terms are as defined for Equation 9.1

where

b = thickness of aquifer, in m

All other terms are as defined for Equation 9.1

Derivations of the foregoing equations are based on the following simplifying assumptions:

1. The water-bearing materials have a uniform hydraulic conductivity within the radius of influence of the well.
2. The aquifer is not stratified.
3. For an unconfined aquifer, the saturated thickness is constant before pumping starts; for a confined aquifer, the aquifer thickness is constant.
4. The pumping well is 100-percent efficient, that is, the drawdown levels inside and just outside the well bore are at the same elevation (see Chapter 16). (Head losses in the vicinity of the well are minimal.)
5. The intake portion of the well penetrates the entire aquifer.
6. The water table or potentiometric surface has no slope.
7. Laminar flow exists throughout the aquifer and within the radius of influence of the well.
8. The cone of depression has reached equilibrium so that both drawdown and radius of influence of the well do not change with continued pumping at a given rate.

These assumptions appear to limit severely the use of the two equations. In reality, however, they do not. For example, uniform hydraulic conductivity is rarely found in a real aquifer, but the average hydraulic conductivity as determined from pumping tests has proved to be reliable for predicting well performance. In confined aquifers where the well is fully penetrating and open to the formation, the assumption of no stratification is not an important limitation.

Assumption of constant thickness is not a serious limitation because variation in aquifer thickness within the cone of depression in most situations is relatively small, especially in sedimentary rocks. Where changes in thickness do occur, as in glacial sediments, for example, they can be taken into account. The assumption that a well is 100-percent efficient can cause the calculated well yield to be seriously in error if the real well is inefficient because of improper design or construction. Factors contributing to inefficiency are discussed in Chapter 16.

The assumption that the water table or potentiometric surface is horizontal before pumping begins is not correct. The slope or hydraulic gradient, however, is usually almost flat and the effect on calculation of well yield is negligible in most cases. Slope

of the water table or potentiometric surface does cause distortion of the cone of depression, making it more elliptical than circular.

Flow in all regions of an aquifer is considered to be laminar. Some investigators have theorized that turbulent flow near a well could result in relatively high head losses. Laboratory and field tests show, however, that some departure from laminar flow near a well causes only small additional head losses (Mogg, 1959).

Determining Aquifer Hydraulic Conductivity

Equations 9.1 and 9.2 can be modified to calculate hydraulic conductivity if Q , H , and R are determined from a pumping test, and b is known from the driller's log. For an unconfined aquifer, the equation for calculating K is:

$$K = \frac{1055 Q \log r_2/r_1}{(h_2^2 - h_1^2)}$$

$$K = \frac{Q \log r_2/r_1}{1.366 (h_2^2 - h_1^2)} \quad (9.3)$$

where

r_1 = distance to the nearest observation well, in ft

r_2 = distance to the farthest observation well, in ft

h_2 = saturated thickness, in ft, at the farthest observation well

h_1 = saturated thickness, in ft, at the nearest observation well

All other terms are as defined in Equation 9.1

where

r_1 = distance to the nearest observation well, in m

r_2 = distance to the farthest observation well, in m

h_2 = saturated thickness, in m, at the farthest observation well

h_1 = saturated thickness, in m, at the nearest observation well

All other terms are as defined in Equation 9.1

All the parameters on the right-hand side of Equation 9.3 can be determined from a pumping test. Two observation wells, located at distances r_1 and r_2 from the pumped well, are required to determine h_1 and h_2 .

Figure 9.10 shows a sectional view of a pumping test layout in an unconfined formation for determining the hydraulic conductivity of the formation. All pertinent factors are easily measured in this kind of test, and the hydraulic conductivity of the aquifer can be determined accurately.

For confined conditions, the equation for determining the hydraulic conductivity from a test installation similar to Figure 9.10 is:

$$K = \frac{528 Q \log r_2/r_1}{b (h_2 - h_1)}$$

$$K = \frac{Q \log r_2/r_1}{2.73 b (h_2 - h_1)} \quad (9.4)$$

where

all terms except the following are the same as for Equation 9.3

b = thickness of the aquifer, in ft

h_2 = head, in ft, at the farthest observation well, measured from the bottom of the aquifer

h_1 = head, in ft, at the nearest observation well

where

all terms except the following are the same as for Equation 9.3

b = thickness of the aquifer, in m

h_2 = head, in m, at the farthest observation well, measured from the bottom of the aquifer

h_1 = head, in m, at the nearest observation well

vation well, measured from the bottom of the aquifer

bottom of the aquifer

In addition to providing accurate means for calculating the average hydraulic conductivity of an aquifer, the equilibrium equations are useful for studying the relationship of various factors to each other and to well yield. They show, for example, that if all other parameters are equal, well yield is directly proportional to hydraulic conductivity. A formation with twice the hydraulic conductivity of another should provide twice the yield. For a confined aquifer, Equation 9.2 indicates that the yield is directly proportional to the formation thickness when all other parameters are equal.

Relationship of Drawdown to Yield

Equation 9.2 for a well operating under confined conditions shows that yield is directly proportional to drawdown, $H - h$, as long as the drawdown does not exceed the distance from the static potentiometric surface to the top of the aquifer. If the drawdown exceeds this amount, b will then be reduced and the proportionality no longer holds true. Theoretically, this means that if the drawdown is doubled, the yield is doubled. Stated another way, the specific capacity of a well is constant at any pumping rate as long as the aquifer is not dewatered.

For a well in an unconfined aquifer, the part of the formation within the cone of depression is actually dewatered during the pumping. This affects the ratio of drawdown to yield. When the drawdown is doubled, the well yield is less than doubled because the saturated thickness is reduced. The specific capacity decreases with increased drawdown; in fact it decreases directly in proportion to the drawdown.

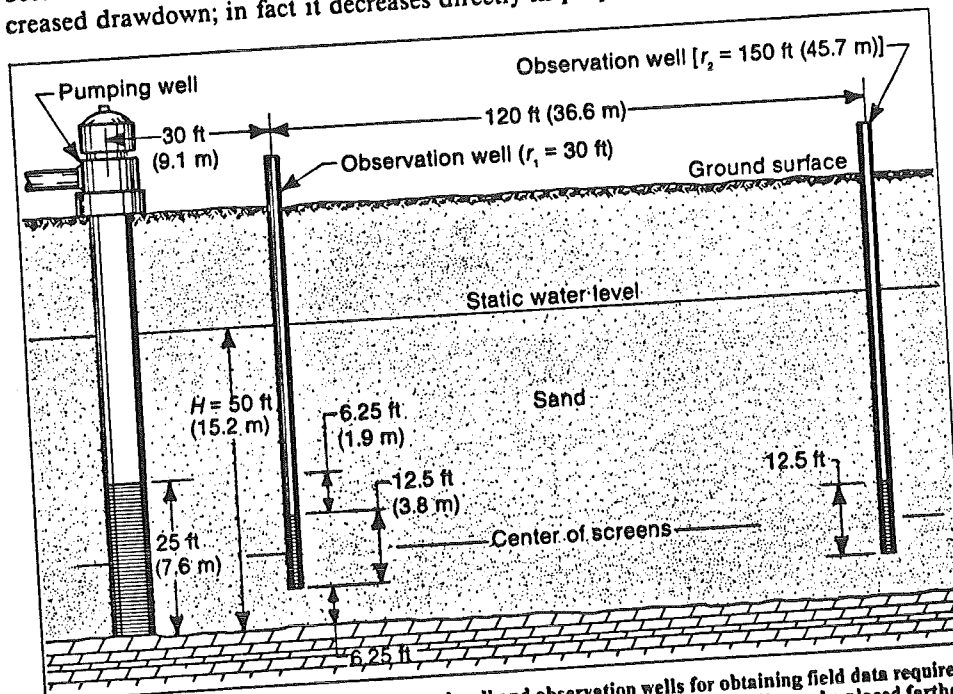


Figure 9.10. Typical arrangement of a pumped well and observation wells for obtaining field data required to calculate hydraulic conductivity from well-discharge equations. Observation wells can be placed farther away from a production well in confined conditions and still provide reliable data.

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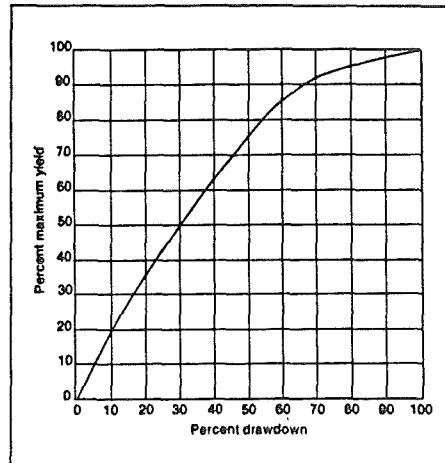


Figure 9.11. Comparison of yield with drawdown in an ideal unconfined aquifer that is fully penetrated and open to the well.

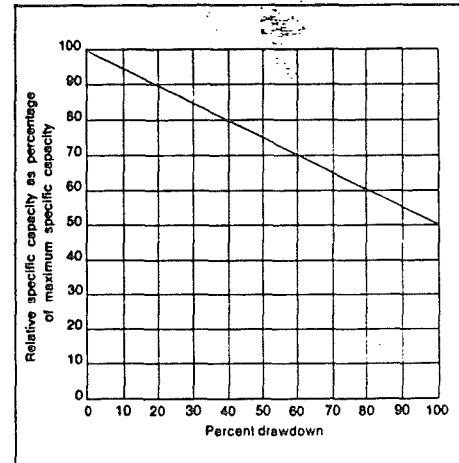


Figure 9.12. Relationship between specific capacity and drawdown in an unconfined aquifer that is fully penetrated and open to the formation.

Figure 9.11 shows the relationship between drawdown and yield for an unconfined aquifer. Maximum drawdown means lowering of the water level to the bottom of the well; 50-percent drawdown means lowering of the water level to a point halfway between the static water level and the bottom of the well. Maximum yield is the quantity a well will produce at maximum or 100-percent drawdown. For example, suppose that a well 40 ft (12.2 m) deep has a static water level of 5 ft (1.5 m) and the saturated thickness of the formation is 35 ft (10.7 m). During a test, the water was pumped at 16 gpm (87 m³/day) and the pumping level stabilized at 15 ft (4.6 m) below the ground surface, or at a drawdown of 10 ft (3 m). How much will the yield be with 20 ft (6.1 m) of drawdown and the pumping level at 25 ft (7.6 m)?

In this case, 100-percent drawdown is 35 ft. The 10-ft drawdown during the test is thus 29 percent of the total possible drawdown. The curve in Figure 9.11 shows that at 29-percent drawdown, the yield is 50 percent of the obtainable maximum; thus 16 gpm is 50 percent of the maximum yield of the well. A drawdown of 20 ft is 57 percent of the total possible. The curve shows that this drawdown would give 82 percent of the maximum yield. If 16 gpm is 50 percent of the maximum, then 82 percent of the maximum would be $82/50 \times 16 = 26$ gpm (142 m³/day). The well can be expected to yield 26 gpm at 20 ft of drawdown.

Figure 9.12 indicates how specific capacity varies with drawdown. Theoretically, maximum specific capacity corresponds to zero drawdown because there is no reduction in the saturated thickness; the minimum occurs when drawdown and yield are at the maximum. Note that the minimum specific capacity is 50 percent of the maximum. In the previous example, 85 percent of the maximum specific capacity would be obtained with 10 ft of drawdown and 71 percent with 20 ft of drawdown.

Figure 9.11 shows why it is uneconomical to operate a well with a drawdown greater than 67 percent of the maximum. At 67 percent of maximum drawdown, 90 percent of the maximum yield is obtained. To obtain the remaining 10 percent requires an additional 33-percent drawdown. Obviously, the extra pumping costs would be out of proportion to the increase in yield.

NONEQUILIBRIUM WELL EQUATION

Theis developed the nonequilibrium well equation in 1935. The Theis equation was the first to take into account the effect of pumping time on well yield. Its derivation was a major advance in groundwater hydraulics. By use of this equation, the drawdown can be predicted at any time after pumping begins. Transmissivity and average hydraulic conductivity can be determined during the early stages of a pumping test rather than after water levels in observation wells have virtually stabilized. Aquifer coefficients can be determined from the time-drawdown measurements in a single observation well rather than from two observation wells as required in Equations 9.3 and 9.4.

noneq-
well.

Derivation of the Theis equation is based on the following assumptions:

1. The water-bearing formation is uniform in character and the hydraulic conductivity is the same in all directions.
2. The formation is uniform in thickness and infinite in areal extent.
3. The formation receives no recharge from any source.
4. The pumped well penetrates, and receives water from, the full thickness of the water-bearing formation.
5. The water removed from storage is discharged instantaneously when the head is lowered.
6. The pumping well is 100-percent efficient.
7. All water removed from the well comes from aquifer storage.
8. Laminar flow exists throughout the well and aquifer.
9. The water table or potentiometric surface has no slope.

These assumptions are essentially the same as those for the equilibrium equation except that the water levels within the cone of depression need not have stabilized or reached equilibrium.

In its simplest form, the Theis equation is:

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

$$s = \frac{1}{4\pi} \frac{Q}{T} W(u) \quad (9.5)$$

where

s = drawdown, in ft, at any point in the vicinity of a well discharging at a constant rate

Q = pumping rate, in gpm

T = coefficient of transmissivity of the aquifer, in gpd/ft

$W(u)$ = is read "well function of u " and represents an exponential integral

where

s = drawdown, in m, at any point in the vicinity of a well discharging at a constant rate

Q = pumping rate, in m^3/day

T = coefficient of transmissivity of the aquifer, in m^2/day

$W(u)$ = is read "well function of u " and represents an exponential integral

In the $W(u)$ function, u is equal to:

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

$$u = \frac{r^2 S}{4 T t} \quad (9.5a)$$

where

r = distance, in ft, from the center of a

where

r = distance, in m, from the center of a

pumped well to a point where the drawdown is measured

S = coefficient of storage (dimensionless)

T = coefficient of transmissivity, in gpd/ft

t = time since pumping started, in days

pumped well to a point where the drawdown is measured

S = coefficient of storage (dimensionless)

T = coefficient of transmissivity, in m²/day

t = time since pumping started, in days

The well function of u [$W(u)$] originated as a term to represent the heat distribution in a flat plate with a heating element at its center. Theis recognized that this same concept could be applied to the regular distribution of the groundwater head around a pumping well even though water flows toward the point source rather than away from it. The mathematical principles remain the same.

Analysis of pumping test data* using the Theis equation can yield transmissivity and storage coefficients for all nonequilibrium situations. In actual practice, however, the Theis method is often avoided because it requires curve-matching interpretation and is somewhat laborious. In fact, the work of applying the Theis method can be avoided in most cases. For example, if the pumping test is sufficiently long or the distance from the well to where the drawdown is measured is sufficiently small, the $W(u)$ function can be replaced by a simpler mathematical function which makes the analysis easier. The Theis method is developed at the end of this chapter, but at this point the simplified version is examined because it serves well in most cases.

MODIFIED NONEQUILIBRIUM EQUATION

In working with the Theis equation, Cooper and Jacob (1946) point out that when u is sufficiently small, the nonequilibrium equation can be modified to the following form without significant error:

$$s = \frac{264Q}{T} \log \frac{0.3 T t}{r^2 S} \quad (9.5)$$

$$s = \frac{0.183Q}{T} \log \frac{2.25 T t}{r^2 S} \quad (9.6)$$

where the symbols represent the same terms as in Equation 9.5 and 9.5a.

For values of u less than about 0.05, Equation 9.6 gives essentially the same results as Equation 9.5. The value of u becomes smaller as t increases and r decreases. Thus, Equation 9.6 is valid when t is sufficiently large and r is sufficiently small. Equation 9.6 is similar in form to the Theis equation except that the exponential integral function, $W(u)$, has been replaced by a logarithmic term which is easier to work with in practical applications of well hydraulics.

For a particular situation where the pumping rate is held constant, Q , T , and S are all constants. Equation 9.6 shows, therefore, that the drawdown, s , varies with $\log t/r^2$ when u is less than 0.05. From this relationship, two important relationships can be stated:

1. For a particular aquifer at any specific point (where r is constant), the terms s and t are the only variables in Equation 9.6. Thus, s varies as $\log C_1 t$, where C_1 represents all the constant terms in the equation.

2. For a particular formation and at a given value of t , the terms s and r are the

*The performance of newly completed wells is often checked by pumping tests. During the test, the drawdown in the pumping well and observation wells is measured at a constant discharge rate. When properly conducted, these tests yield information on transmissivity and storage capability. See Chapter 16 for a detailed analysis of pumping test procedures.

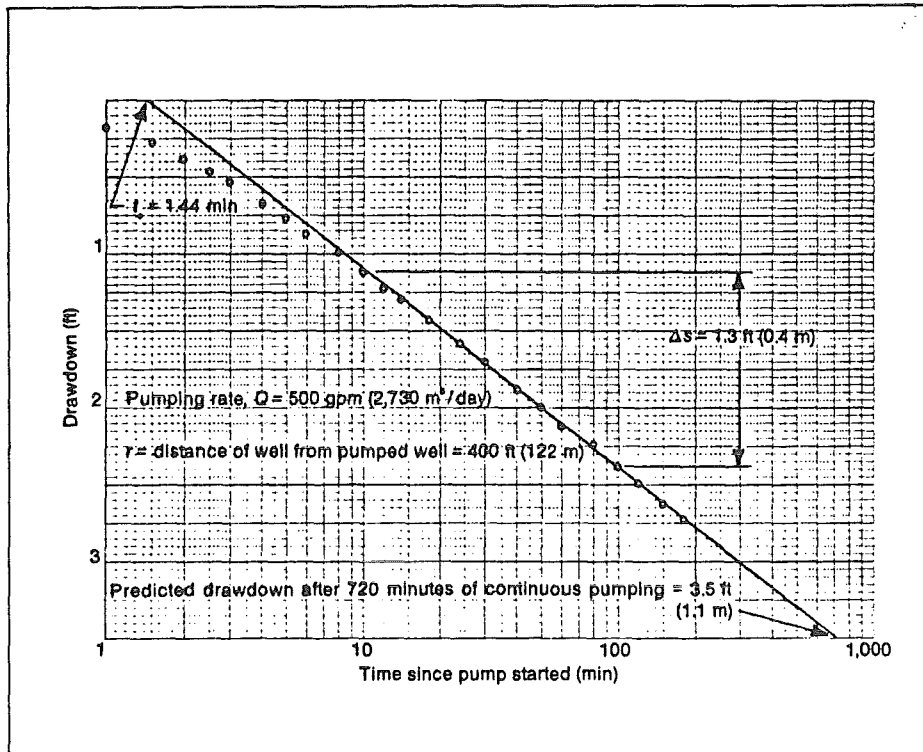


Figure 9.13. When data from Table 9.1 are plotted on semilogarithmic graph paper, most of the plotted points fall on a straight line. The reason for determining Δs and t_0 are explained in the text.

only variables in Equation 9.6. In this case, s varies as $\log C_2/r^2$, where C_2 represents all the constant terms in the equation, including the specific value of t .

By using these simplified relationships based on Equation 9.6, it is possible to derive information on the hydraulic characteristics of the aquifer by plotting drawdown and time data taken during a pumping test. The data are plotted on semilogarithmic paper* as shown in Figure 9.13. Applying the first of the relationships developed above, time, t , is plotted horizontally on the logarithmic scale; drawdown, s , is plotted vertically on the arithmetic scale. Figure 9.13 shows the data from Table 9.1 plotted as a semilog diagram, where most of the points fall on a straight line.

All the points except those representing measurements made during the first 10 minutes of pumping fit the line. During the first 10 minutes, the value of u is larger than 0.05 and so the modified nonequilibrium equation is not applicable within that phase of the test.

Transmissivity

The coefficient of transmissivity is calculated from the pumping rate and the slope of the time-drawdown graph by using the following relationship developed from Equation 9.6:

*Semilogarithmic graph paper is constructed so that one scale is arithmetic and the other is based on the logarithm of the number being plotted. Thus, a straight-line relationship can be shown to exist between two variables whose relationship is actually changing in time.

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

$$T = \frac{2.3 Q}{4\pi \Delta s} = \frac{0.183 Q}{\Delta s} \quad (9.7)$$

where

T = coefficient of transmissivity, in
gpd/ft

Q = pumping rate, in gpm

Δs = (read "delta s") slope of the time-drawdown graph expressed as the change in drawdown between any two times on the log scale whose ratio is 10 (one log cycle)

where

T = coefficient of transmissivity, in
 m^2/day

Q = pumping rate, in m^3/day

Δs = (read "delta s") slope of the time-drawdown graph expressed as the change in drawdown between any two times on the log scale whose ratio is 10 (one log cycle)

In the example, Δs is 1.3 ft (0.4 m), which is the change in drawdown between 10 minutes and 100 minutes after the start of the pumping test, and Q equals 500 gpm ($2,730 \text{ m}^3/\text{day}$); so:

$$T = \frac{264 \cdot 500}{1.3} = 102,000 \text{ gpd/ft}$$

$$T = \frac{0.183 \cdot 2,730}{0.4} = 1,250 \text{ m}^2/\text{day}$$

Table 9.1. Drawdown Measurements in an Observation Well
400 ft (122 m) from Pumped Well

Time since pump started, in min	Drawdown, s		Time since pump started, in min	Drawdown, s	
	ft	m		ft	m
1	0.16	0.05	24	1.58	0.48
1.5	0.27	0.08	30	1.70	0.52
2	0.38	0.12	40	1.88	0.57
2.5	0.46	0.14	50	2.00	0.61
3	0.53	0.16	60	2.11	0.64
4	0.67	0.20	80	2.24	0.68
5	0.77	0.23	100	2.38	0.73
6	0.87	0.27	120	2.49	0.76
8	0.99	0.30	150	2.62	0.80
10	1.12	0.34	180	2.72	0.83
12	1.21	0.37	210	2.81	0.86
14	1.30	0.40	240	2.88	0.88
18	1.43	0.44			

Coefficient of Storage

The coefficient of storage is also readily calculated from the time-drawdown graph by using the zero-drawdown intercept of the straight line as one of the terms in the equation. The following equation is derived from Equation 9.6:

$$S = \frac{0.3 T t_0}{r^2}$$

$$S = \frac{2.25 T t_0}{r^2} \quad (9.8)$$

where

S = storage coefficient

where

S = storage coefficient

T = coefficient of transmissivity, in gpd/ft	T = coefficient of transmissivity, in m^2/day
t_0 = intercept of the straight line at zero drawdown, in days	t_0 = intercept of the straight line at zero drawdown, in days
r = distance, in ft, from the pumped well to the observation well where the drawdown measurements were made	r = distance, in m, from the pumped well to the observation well where the drawdown measurements were made

In our example, $t_0 = 1.44$ minutes or 0.001 day, $T = 102,000$ gpd/ft ($1,270 \text{ m}^2/\text{day}$), and $r = 400$ ft (122 m). Therefore:

$$S = \frac{0.3 \cdot 102,000 \cdot 0.001}{(400)^2}$$

$$= 1.9 \times 10^{-4}$$

$$S = \frac{2.25 \cdot 1,250 \cdot 0.001}{(122)^2}$$

$$= 1.9 \times 10^{-4}$$

The data used in this example are from an observation well 400 ft from a production well which was pumped at a rate of 500 gpm ($2,730 \text{ m}^3/\text{day}$) for 240 minutes. If measurements had been taken in another observation well still farther from the pumped well, and if the data were plotted on Figure 9.13, the points would fall along a straight line parallel to but above the one shown. If measurements in an observation well closer to the pumped well, or in the pumped well itself, were plotted on Figure 9.13, they would fall on a straight line parallel to but below the one shown*. Values of T and S calculated from all three sets of data would be the same, however.

Predicting Drawdown from the Time-Drawdown Graph

In addition to its use for calculating the aquifer constants, the time-drawdown diagram provides a graphical means of predicting future drawdown. The straight line in Figure 9.13 may be extended to the right to indicate the drawdown that would occur in the observation well [400 ft (122 m) from the pumped well] after any period of continuous pumping at 500 gpm ($2,730 \text{ m}^3/\text{day}$).

In our example, the predicted drawdown after 12 hours (720 minutes) of continuous pumping is the point where the extended line crosses the vertical line representing 720 minutes. The drawdown graph shows that when $t = 720$ minutes, $s = 3.5$ ft (1.1 m).

Should the drawdown after 120 hours (7,200 minutes) be desired, the value of Δs (1.3) can be added to the drawdown at 12 hours (720 minutes). Note that 120 hours is ten times 12, or one log cycle of time beyond 12 hours. By definition, Δs represents the increase in drawdown over one log cycle. Thus, after 5 days of continuous pumping at 500 gpm, the drawdown (s) 400 ft from the pumped well would be $3.5 + 1.3 = 4.8$ ft (1.5 m).

The usefulness of this simple technique is readily seen. Once the slope of the time-drawdown graph has been established from a short-term pumping test, the straight line can be extrapolated to find the expected drawdown after longer periods of pumping at the same rate. Of course, at some point in the pumping test the hydraulic conditions in the aquifer may change, causing deflection of the straight line. Reasons for these changes are discussed later in this chapter.

*Storage coefficients calculated from drawdown data from the pumped well are generally not reliable.

It is good practice to pump a well in a confined aquifer for 24 hours to get data for the time-drawdown curve. A well in an unconfined aquifer should be pumped for 3 days. The 240-minute pumping period in our example is too short for most situations. A longer test provides data that define more reliably the slope and position of the line of best fit for the plotted points.

For a particular well, Equation 9.6 shows that when t is constant, the drawdown, s , is directly proportional to the pumping rate. Figure 9.13 shows that the drawdown in the observation well was 2.5 ft (0.8 m) after 120 minutes when the pumped well was discharging 500 gpm. If the pumping rate were 1,000 gpm (5,450 m³/day), the drawdown in the observation well after 120 minutes would be twice as much, or 5 ft (1.5 m). Likewise, at $Q = 2,000$ gpm (10,900 m³/day), s would be 10 ft (3 m).

For a given aquifer, Equation 9.7 shows that the slope, Δs , is directly proportional to the pumping rate. The test shows Δs to be 1.3 ft (0.4 m) at $Q = 500$ gpm. If Q were 1,000 gpm, Δs would be twice as great or 2.6 ft (0.8 m).

Good use can be made of these two relationships. Applying them to test data from one or more observation wells provides a means of predicting accurately the drawdown in the observation wells at rates of pumping different from the test rate. Use of these relationships also provides a way to make similar calculations for the performance of a pumped well if the well is efficient and the saturated thickness of the aquifer is not reduced significantly at potentially higher pumping rates.

HYDROGEOLOGIC CONDITIONS THAT AFFECT TIME-DRAWDOWN GRAPHS

This discussion of the Theis equation and the Jacob modification of it has followed the basic assumptions made earlier in this chapter in developing the equations for groundwater flow. At this point, it is helpful to review some of these assumptions to see the effects of some departures from them upon the time-drawdown relationship. Real aquifers do not conform fully to assumed geologic or hydrologic conditions. Thus, limits for the use of the Jacob equation must be set for those cases in which the differences are significant.

Recharge

The assumption that an aquifer receives no recharge during the pumping period is one of the six 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 Equation 9.7. Assuming no recharge during pumping also permits extension of the time-drawdown curve at its initial slope to predict drawdowns at future times.

It is known, however, that most formations receive recharge. Hydrographs from long-term observation wells monitored by the U.S. Geological Survey, various state agencies, and similar data-gathering agencies in other parts of the world show that most water-bearing formations receive continual or intermittent recharge.

When recharge is intermittent because of seasonal effects, the aquifer may perform without recharge for periods of 1 to 3 months or even longer. Ambroggi (1978) has shown that even though an aquifer has been drawn down continuously for 10 years or

more without recharge, water levels can recover completely within 1 year if the area receives normal precipitation. Thus, the time-drawdown curves presented here represent well performance during periods of no recharge. If a pumping test is performed carefully while abundant recharge is reaching the aquifer, the time-drawdown graph from the pumping test will reflect the recharge.

Figure 9.14 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 m³/day) and the drawdown measurements were obtained at various intervals during 360 minutes of pumping. The points plotted on semilogarithmic paper define a straight line with a slope, or Δs value, of 9.3 ft (2.8 m). As seen earlier, the future drawdown in this well for any period of continuous pumping at 350 gpm can be quickly estimated by extending the straight line. The drawdown corresponding to 5,000 minutes of continuous pumping is 73.3 ft (22.3 m) in this case.

This method determines easily the anticipated pumping level and the pump setting required to provide adequate submergence for the pump bowls. Any desired safety factor can be applied to the calculated pumping level to offset changes in well performance resulting from incrustation or from interference effects of other wells that might later be constructed nearby.

Continuous pumping means operating 24 hours a day with no opportunity for recovery of water level. A well that is pumped for only part of each day will not show the same cumulative drawdown after 7, 30, or 90 days as would be predicted by a time-

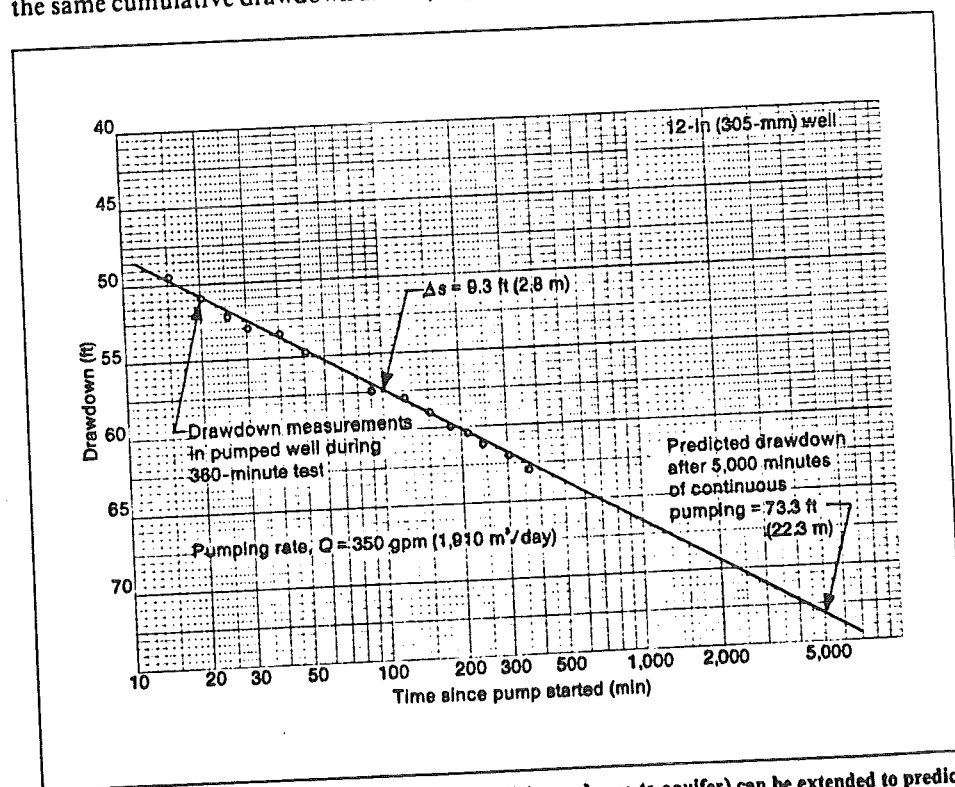


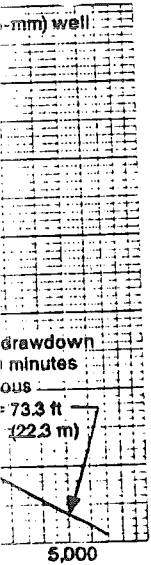
Figure 9.14. Time-drawdown graph for a pumped well (no recharge to aquifer) can be extended to predict drawdown for a period of continuous pumping longer than the test itself.

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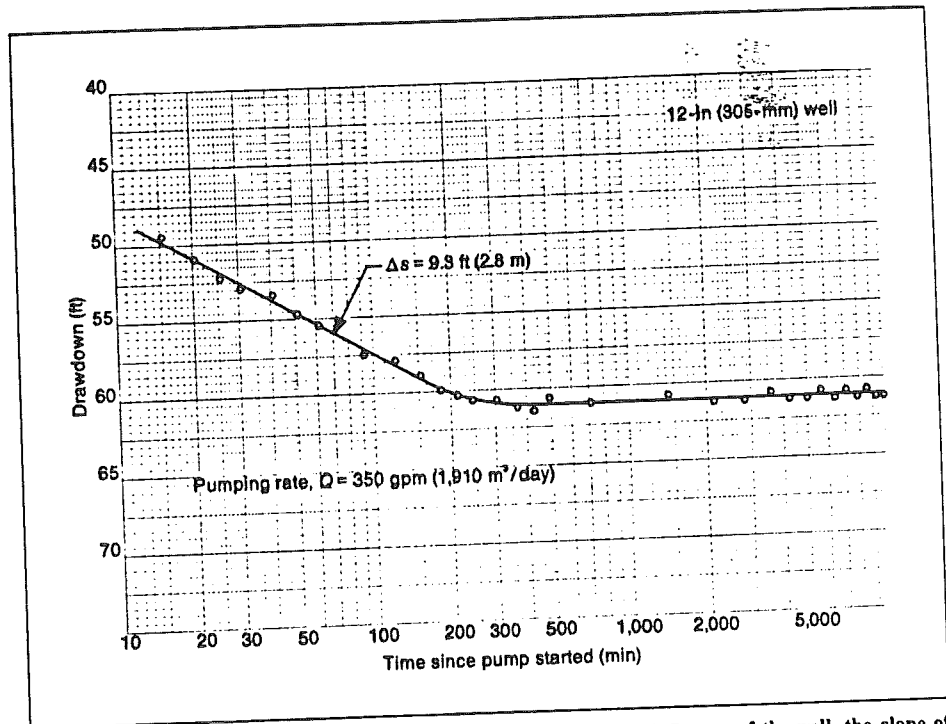


Figure 9.15. When recharge to the aquifer occurs within the zone of influence of the well, the slope of the time-drawdown curve becomes flatter. The horizontal leg indicates that recharge equals well discharge after 240 minutes of pumping.

drawdown graph such as Figure 9.14. Obviously, a well that is pumped on a cycle of 12 hours on and 12 hours off benefits from the recovery of water level during the 12-hour idle period.

If insufficient recharge takes place while the pump is off, the water level will 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. A method used to estimate the cumulative drawdown under intermittent pumping conditions is discussed later in this chapter.

Drawdown will stabilize 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 in Figure 9.15.

The first part of the curve in Figure 9.15 shows that the cone of depression was enlarging during the first 240 minutes of pumping. After 240 minutes, 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 over a period of time; it does not occur instantaneously. Recharge may be from a lake or river located in only a part of the cone of depression. After the recharge boundary is encountered, the drawdown will increase slowly in areas away from the recharge source until equilibrium is established.

It sometimes happens that the recharge rate within the cone of depression is lower

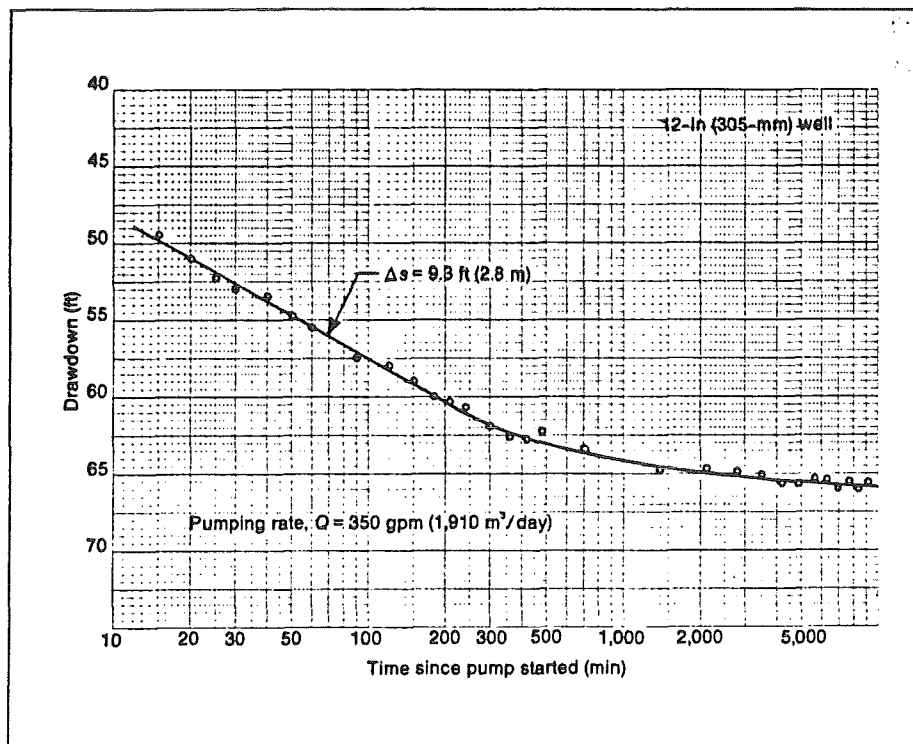


Figure 9.16. Recharge rate is somewhat less than the pump discharge; thus, the second part of the drawdown curve does not become horizontal.

than the pumping rate of the well. While this changes the slope of the time-drawdown curve, the second part may 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 9.16). Future drawdown in a well may be predicted by extending the straight line of the second leg of the curve and reading the drawdown indicated for any future time.

When the slope of the drawdown curve changes after a period of continuous pumping, only graphical methods can be used to predict future drawdown. If the slope does not change, the future drawdown can be computed from the Theis equation or found by the graphical method. If the slope changes, however, the Theis equation cannot be applied to any time after the change from the initial slope.

It should be pointed out that calculation of the transmissivity, T , of a water-bearing formation must be made from Δs based on the first part of the time-drawdown curve. A numerical value that may represent the slope of the second part of the graph is of no significance in analyzing the pumping test data; the new or second Δs value cannot be used because physical characteristics of the aquifer are somewhat different farther from the well. For example, the aquifer may thicken in one or more directions away from the well, 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 anomaly within the radius

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of influence. Similarly, if a zone of higher hydraulic conductivity is encountered within the cone of depression, the time-drawdown curve will flatten somewhat. Considerable variation in aquifer thickness and hydraulic conductivity violates two of the Theis assumptions. The time-drawdown curve will reflect this departure from idealized conditions.

Recharge from a River

Equilibrium conditions that stabilize the cone of depression around a pumping well may develop in several general situations. One of these is when an aquifer is recharged from a river or lake. Figure 9.17 illustrates this situation after equilibrium has been reached.

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 9.15. 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, river water will percolate downward through the pervious 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 of the drawdown curve in Figure 9.15 and the situation shown in Figure 9.17.

Extension of the drawdown curve in Figure 9.15 shows that the predicted drawdown after 5,000 minutes of continuous pumping would be 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 9.14 after the same period of continuous pumping.

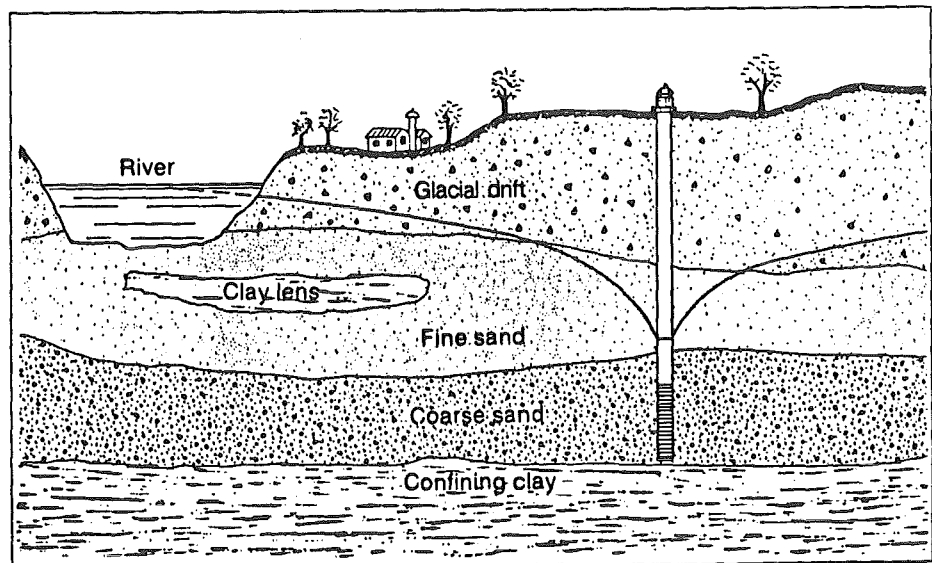


Figure 9.17. Cone of depression expanding beneath a riverbed creates a hydraulic gradient between the aquifer and river. This can result in induced recharge to the aquifer from the river.

Recharge from Vertical Infiltration

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.

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 may have a considerably lower hydraulic conductivity than the deeper material in which the well terminates. The difference in hydraulic conductivity between the upper and lower strata may be so great that the upper material is not considered as part of the aquifer. When the area of the cone of depression covers tens of thousands of square feet, the total vertical seepage from the upper material, even though the material is of low hydraulic conductivity, can equal the well discharge and thus bring about equilibrium in the pumping levels. This situation may also develop in lenticular formations in which only the lower part of the formation is screened.

Effect of Recharge on Drawdown in Observation Wells

Discussion of the recharge effect on the shape of the time-drawdown curve has been confined to the graph for a pumped well (Figure 9.15). It should be understood that measurements in an observation well would produce a similar time-drawdown curve, except that the time at which the change in slope occurs would differ according to the relative positions of the pumped well, the observation well, and the recharge source.

For an observation well between the pumped well and the river in Figure 9.17, the flattening of the drawdown curve would occur sometime before 240 minutes. If the observation well were on the opposite side of the pumped well, away from the river, the change in slope would occur sometime after 240 minutes of pumping.

Data from observation wells are usually more reliable and accurate than data from pumped wells, so the time-drawdown plots from observation wells are most often relied upon to reveal the performance of an aquifer. Data from observation wells are not affected by minor changes in well discharge caused by variations in pump speed, or by uncertain measurements of the true water level because of turbulence in the well bore. When transmissivity is calculated, the value of Δs corresponding to the first part of the time-drawdown curve must be used. Theoretically, the drawdown curves of the pumped well and observation well should produce the same value of Δs .

It should be evident from this discussion that accurate and frequent measurements of drawdown at the start of a constant-rate pumping test are extremely important. Sufficient dependable data must be obtained during the early part of the test to reflect clearly both the well and aquifer performances before recharge effects or other outside influences, which invalidate the basic nonequilibrium theory, begin to distort the data.

Sloping Water Table or Potentiometric Surface

Groundwater flow toward and past a well produces another situation that can bring about virtual stabilization of the pumping level. As explained in Chapter 5, groundwater moves because hydraulic gradients develop between areas of recharge and areas of discharge. The gradient is represented by the slope of the water table or potentiometric surface. Most natural slopes of these surfaces are relatively flat and do not affect appreciably the well performance curves. A relatively steep slope, however, causes distortion of the cone of depression around the well. Rather than being circular, the area of influence of the well becomes elliptical. More of the pumped water comes from the up-gradient direction than from all directions equally.

Slow Drainage

In unconfined conditions, slow drainage within the cone of depression can affect the validity of early time-drawdown data. Figure 9.18 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.

The causes of this phenomenon are 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 even clay.

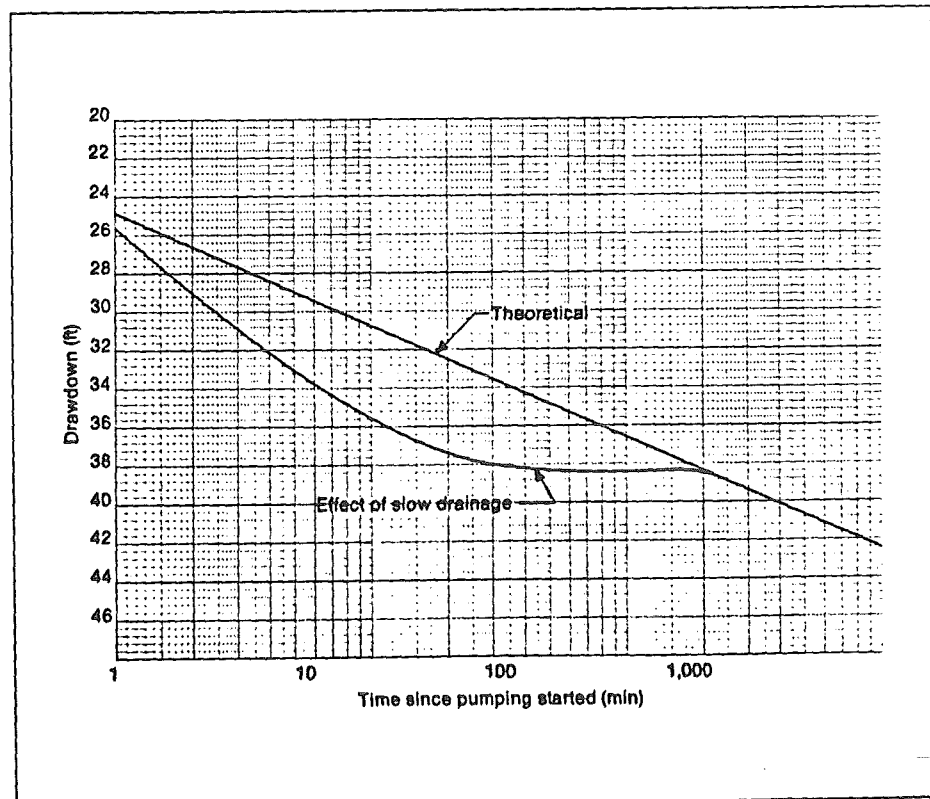


Figure 9.18. Time-drawdown curve showing the effect of slow drainage on the early part of the curve.

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 curve.

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 may 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 at a later time. Vertical leakage occurs when an aquifer is confined geologically by two aquitards. Commonly these aquitards are limestone layers that have numerous vertical joints or faults. Under ordinary conditions, vertical leakage into the confined aquifer is minimal. During pumping, however, reduction in head within the cone of depression may cause leakage from both above and below the aquifer.

Detailed analyses of leakage effects on aquifers have been developed by Jacob (1946a), Walton (1960), and others. Figure 9.19 shows distortion of the Jacob curve by vertical leakage. In a practical sense, it is difficult to account generally for leakage ef-

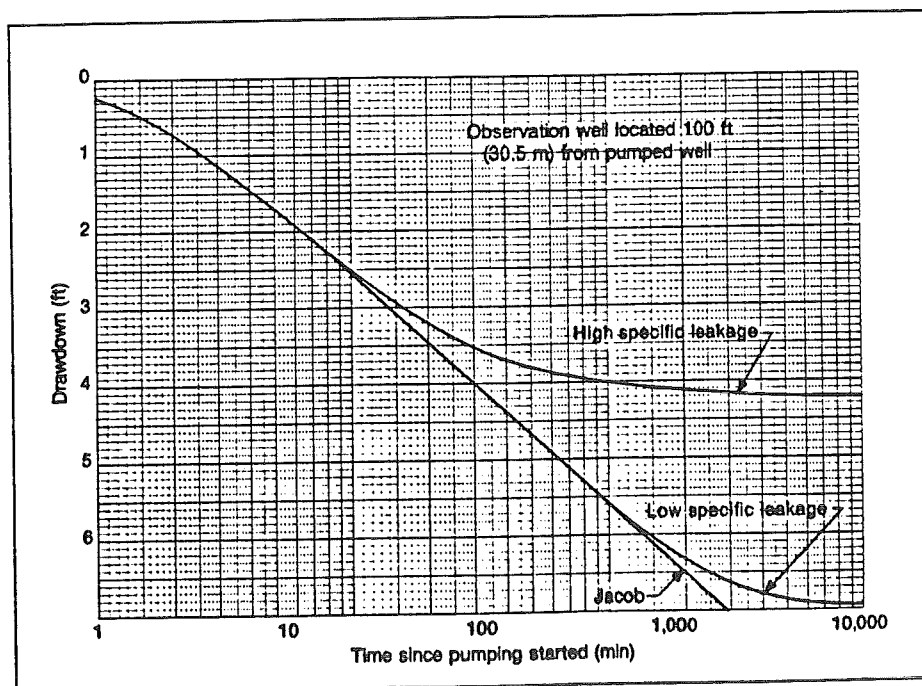


Figure 9.19. Drawdown graphs showing both low and high specific leakage.

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fects mathematically because they are often masked by other physical irregularities of the aquifer.

Impervious Boundaries

Relatively few aquifers conform to the basic assumption of infinite extent in all directions from the pumped well. In many localities, definite geologic and hydraulic boundaries limit aquifers to areas ranging from a few to many square miles. This is especially true in glaciated regions.

The effect of an impervious boundary on the time-drawdown graph is opposite to the effect of aquifer recharge. The boundary causes the slope of the drawdown plot to steepen instead of flatten. This is easily understood if we consider how water stored within the aquifer is supplied to the well.

Under the basic assumptions mentioned above, water flows equally from all directions toward a well. When the expanding cone of depression encounters an impervious boundary on one side of a pumped well, it can expand no farther in that direction and no additional water can be supplied from that locality. The cone must expand and deepen more rapidly in all other directions to maintain the yield of the well. The effect on the semilog time-drawdown curve is to steepen its slope, as shown in Figure 9.20.

Care must be used in analyzing aquifer test data when either nearby boundaries or recharge sources are suspected. Aquifer coefficients must be calculated from the early test data if boundary effects are apparent. In no case should calculations be made from the slope of any part of the time-drawdown plot that reflects a boundary. The latest established slope of the time-drawdown plot should be extended to predict water levels after longer periods of continuous pumping.

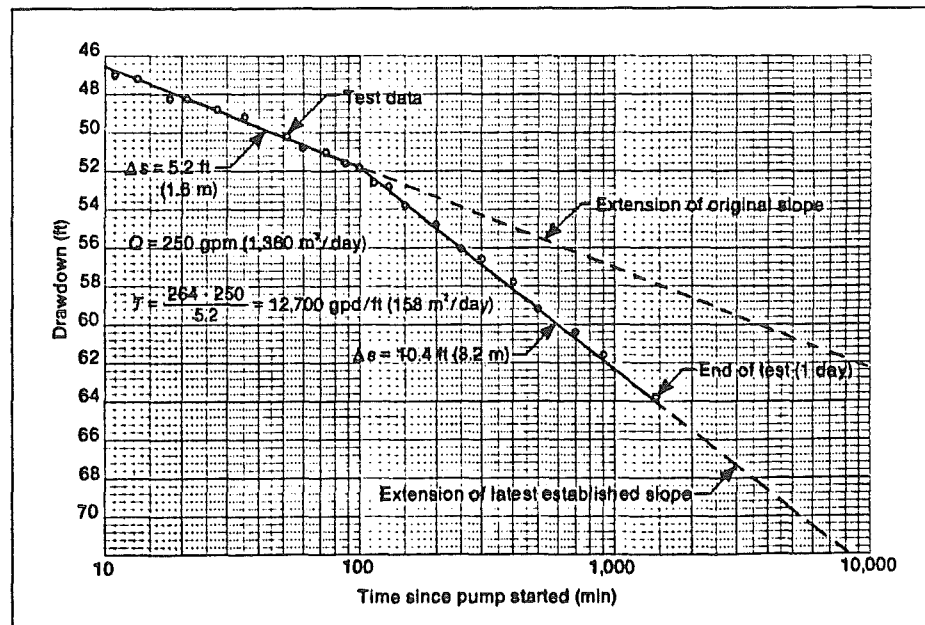


Figure 9.20. Steepening of the time-drawdown, semilog curve indicates a limited aquifer. The expanding cone of depression encountered one or more impervious boundaries at the time shown by the change in slope.

It is also pertinent to note the reason for the earlier suggestion that pumping tests last at least 1 day for wells in confined aquifers and at least 3 days for wells in unconfined aquifers. Figure 9.20 shows evidence of a boundary effect 100 minutes after the pump was started. If the test had been conducted for only 100 minutes, the boundary would not have been revealed. Moreover, extension of the initial slope indicates a drawdown of 62 ft (18.9 m) after 7 days (10,000 minutes) of pumping at 250 gpm (1,360 m³/day), whereas the correct estimate is 73 ft (22.3 m) as determined from the extension of the second leg of the curve.

In a confined aquifer, enlargement of the cone of depression during a 24-hour pumping period is usually extensive enough to encounter boundaries that could appreciably affect drawdown predictions from the semilog diagram. In an unconfined aquifer, the cone of depression expands more slowly, and so a longer pumping period is required to detect boundaries that may exist.

Casing Storage

Schafer (1978) suggests that in many instances early pumping test data may not fit Jacob's modification of the nonequilibrium theory, and that calculations based on this early Δs value will be erroneous. These early data reflect the removal of water stored in the casing. When pumping begins, water in the casing is removed first. As the water level in the casing falls, water begins to enter the well from the surrounding formation. Gradually, a greater percentage of the well's yield will be from the aquifer. The Δs value will be higher during the time required to exhaust the casing storage, giving an erroneously low transmissivity value in the early stages of the pumping test. Figure 9.21 shows data from a typical pumping test in which casing storage has

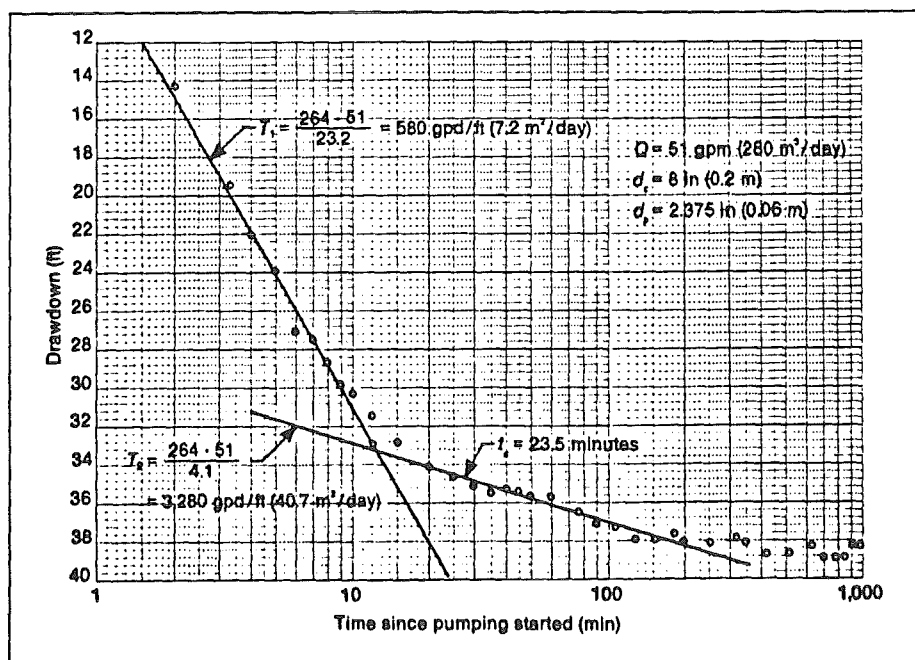
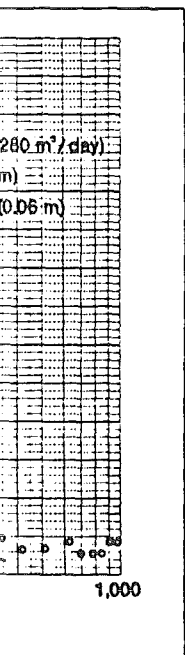


Figure 9.21. Pumping test data in which casing storage has altered the early part of the time-drawdown plot.

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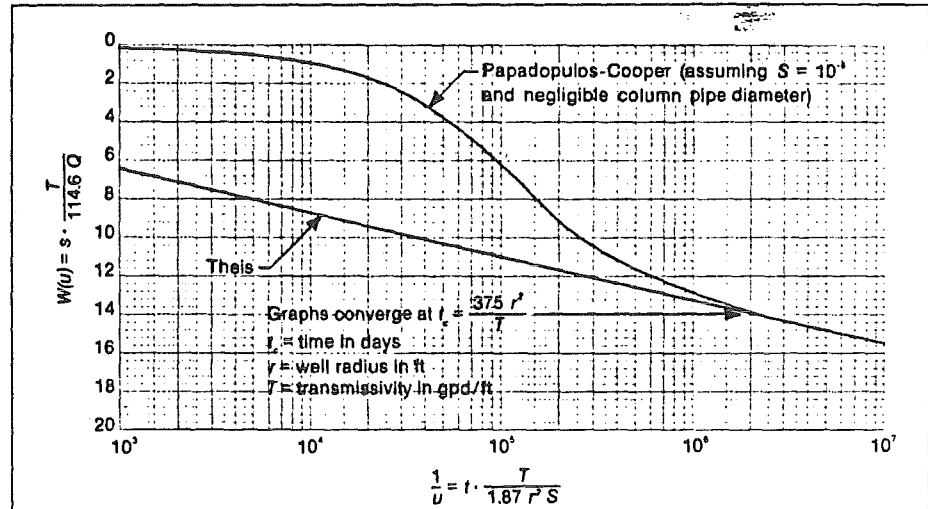


Figure 9.22. Graphic representation of the Papadopoulos-Cooper equation which takes into account casing storage.

distorted the early part of the time-drawdown curve.

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

Papadopoulos 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 (Papadopoulos-Cooper equation for determining t_c is shown in Figure 9.22). 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 will represent the true physical conditions within an aquifer. Unfortunately, these equations can be used only if the transmissivity and well efficiency are known in advance.

Schafer suggests that the critical time can be calculated by the equation:

$$t_c = \frac{0.6 (d_c^2 - d_p^2)}{Q/s} \quad t_c = \frac{0.017 (d_c^2 - d_p^2)}{Q/s} \quad (9.9)$$

where

t_c = time, in minutes, when casing storage effect becomes negligible
 d_c = inside diameter of well casing, in inches
 d_p = outside diameter of pump column pipe, in inches
 Q/s = specific capacity of the well in gpm/ft of drawdown at time t_c

where

t_c = time, in minutes, when casing storage effect becomes negligible
 d_c = inside diameter of well casing, in mm
 d_p = outside diameter of pump column pipe, in mm
 Q/s = specific capacity of the well in m³/day/m of drawdown at time t_c

Determination of the true transmissivity value depends on being able to identify whether a casing storage effect has occurred or a recharge boundary has been encountered early in the pumping test. As an example, let us assume that it is unknown whether the pumping test data presented in Figure 9.21 represent boundary conditions or casing storage. If an iteration using Equation 9.9 is done, it is possible to delineate whether the change in the slope of the curve is caused by a casing storage effect or actually represents a recharge boundary. If the change in slope is caused by a casing storage effect, the time of the change can be predicted by Equation 9.9.

Equation 9.9 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 which can be used in Equation 9.9 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. The use of this equation does not require knowledge of either the transmissivity or well efficiency.

To use Equation 9.9, an initial drawdown value is selected and a t_c value is calculated. For illustration, assume the initial s is 20 ft; the estimated t_c is then calculated:

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

At 13.7 minutes, the drawdown is 33 ft (read from time-drawdown graph). Another calculation is made using 33 ft instead of 20 ft, resulting in a t_c value of approximately 22.7 minutes. The drawdown at 22.7 minutes is 34.2 ft. Substituting this value in Equation 9.9 leads to a t_c value of 23.5 minutes. Note that the change in the t_c value between the last two iterations has been only 0.8 minutes, a small value. Thus, the three iterations using Equation 9.9 suggest that the casing storage effect would have become negligible at approximately 23 minutes. Thus, the initial slope provides an erroneous T value and any predictions of the well's performance should be based on the T value calculated on the basis of the latter part of the curve. If the change in slope were caused by a boundary effect, the time of the change in slope could not be predicted by applying Equation 9.9 and the initial T value must be used to predict well performance.

Analyses of pumping tests in which casing storage is a factor indicate that T_1 and T_2 can be related by the equation:

$$T_2 = \frac{4 T_1}{E} \quad (9.10)^*$$

where E is the efficiency of the well, T_2 is the transmissivity reflecting the true aquifer characteristics, and T_1 is the apparent transmissivity calculated from the portion of the graph affected by casing storage. This equation can be used to check calculated transmissivity values and well efficiencies derived from pumping tests, especially when data from the pumped well are the only data available. The numerical value of 4 on the right side of the equation is based on the value of the exponent of the storage coefficient, that is, $S = 10^{-4}$. It will change as the exponent varies.

Careful collection of early time-drawdown and recovery values can enhance the data

*This equation can be used with either SI or English terms.

base used to evaluate wells and aquifers. The effect, however, of casing storage on the early measurements cannot be ignored and must be incorporated into the overall data analysis. Estimation of t_c by Equation 9.9 aids in the interpretation by determining which data are influenced by casing storage and are therefore not subject to conventional analysis. Equation 9.10 then provides a useful check on obtained values of transmissivity and efficiency.

CALCULATING DRAWDOWN FOR INTERMITTENT PUMPING SITUATIONS

For intermittent pumping situations, direct calculation of the cumulative drawdown is quite cumbersome after a large number of pumping cycles. If boundary conditions or recharge are encountered, calculation is impossible. Graphical methods as discussed above are not applicable in cases of intermittent pumping. However, an abbreviated method can be used to estimate the drawdown resulting from pump cycling.

To obtain this estimate, the cycling well is assumed to be replaced by two imaginary wells which pump continuously without cycling. One well is assumed to pump continuously throughout the entire operating period at a rate that would produce a volume equal to the volume produced by the cycled well. The other imaginary well is assumed to pump for one pumping cycle only and at a rate equal to the difference between the actual pumping rate and that of the first imaginary well. The drawdowns in each imaginary well are determined and added together to give a reliable estimate of the drawdown in the real well.

As an illustration, assume that a 1,000-gpm (5,450-m³/day) well pumps 75 percent of the time, 18 hours on and 6 hours off each day for 90 days. The drawdown at the end of the 18-hour pumping cycle on the 90th day can be estimated by thinking of the single well as two imaginary wells, one pumping at three-fourths of the actual discharge for the entire duration of pumping and the other pumping at one-fourth the actual discharge for just the last cycle. In other words, one imaginary well pumps 750 gpm (4,090 m³/day) for 89 days, 18 hours (89.75 days), and the other imaginary well pumps 250 gpm (1,360 m³/day) for 18 hours (0.75 days). The drawdowns from the imaginary wells are then added together to give the desired estimate. Although this procedure is not mathematically precise, it is intuitively acceptable and produces drawdown values within 1 or 2 percent of those obtained mathematically.

DISTANCE-DRAWDOWN GRAPHS

Applying the second relationship outlined on page 219, a semilog distance-drawdown graph can be constructed. Simultaneous drawdown measurements in at least three observation wells, each at a different distance from the pumped well, are needed to construct an accurate distance-drawdown graph. If these values are plotted on ordinary graph paper, as in Figure 9.23, the familiar trace of the cone of depression near a pumped well is obtained. This is similar to the curves in Figures 9.4, 9.8, 9.9, and 9.17.

If the drawdowns in the same three wells are plotted on a semilog diagram, however, the drawdown curve becomes a straight line, as in Figure 9.24. Points representing drawdowns in other observation wells farther away from the pumped well would fall a little below the straight line in Figure 9.24, because at some distance from the pumped well u is larger than 0.05. If u is greater than 0.05, the straightline relationship of s to $\log r$ no longer holds.

(9.10)*

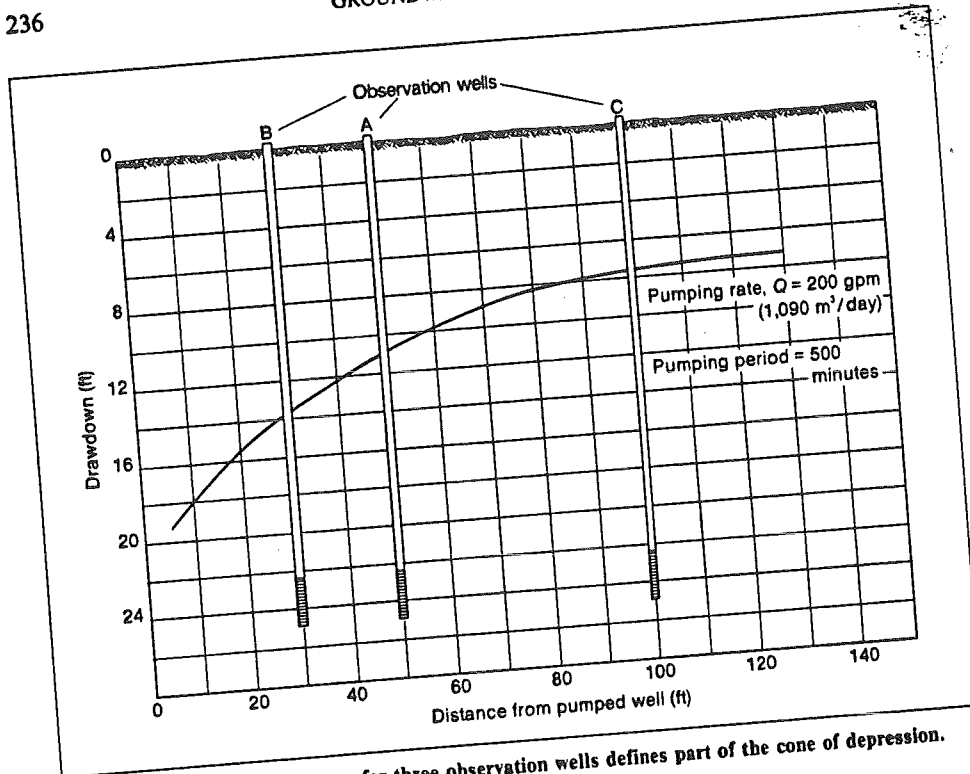


Figure 9.23. Plotting drawdowns for three observation wells defines part of the cone of depression.

The semilog plot of the cone of depression (distance-drawdown diagram) simplifies application of the distance-drawdown relationship. The straight line may be extended to the right to determine the effect of pumping at any distance from the pumped well. For example, Figure 9.24 shows that when the test well was pumped at 200 gpm (1,090 m³/day) for 500 minutes, a drawdown of 2.6 ft (0.8 m) would have been produced in another well 300 ft (91.5 m) away.

Transmissivity

A simple transformation of Equation 9.6 permits calculation of transmissivity from the distance-drawdown diagram. The slope of the straight line is used in a manner similar to the procedure already explained for use of the time-drawdown diagram. The equation for this method is:

$$T = \frac{528 Q}{\Delta s}$$

$$T = \frac{0.366 Q}{\Delta s} \quad (9.11)$$

where

T = coefficient of transmissivity, in gpd/ft

Q = pumping rate, in gpm

Δs = slope of the distance drawdown graph expressed as the change in drawdown, in ft, between any two

where

T = coefficient of transmissivity, in m²/day

Q = pumping rate, in m³/day

Δs = slope of the distance drawdown graph expressed as the change in drawdown, in m, between any two

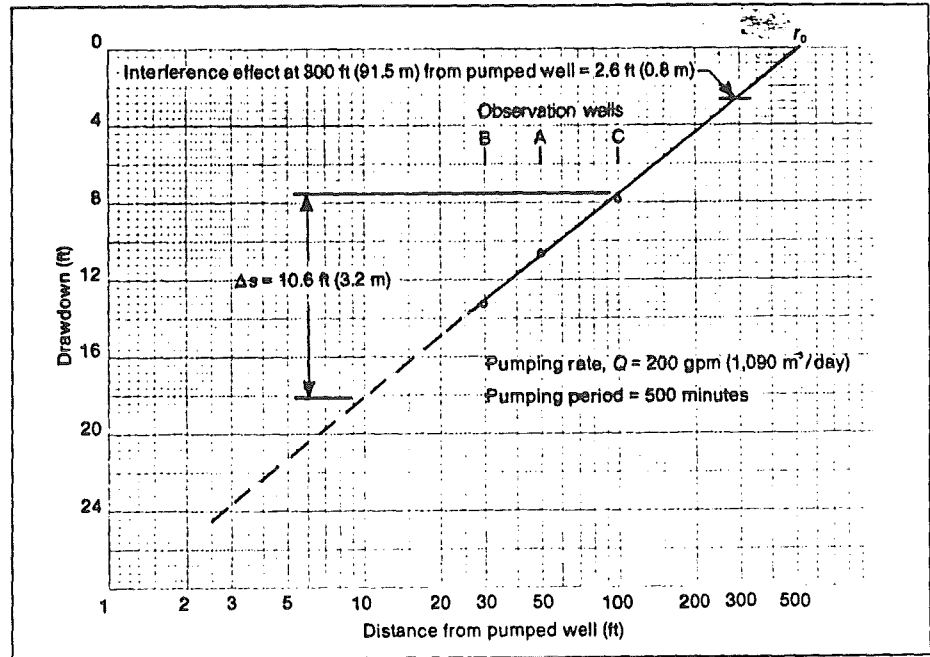


Figure 9.24. Trace of the cone of depression plotted on semilogarithmic graph paper becomes a straight line. Drawdown in each observation well was measured 500 minutes after start of the pumping test.

values of distance on the log scale
whose ratio is 10

values of distance on the log scale
whose ratio is 10

For the example shown in Figure 9.24:

$$T = \frac{528 \cdot 200}{10.6}$$

$$= 9,960 \text{ gpd/ft}$$

$$T = \frac{0.366 \cdot 1,090}{3.2}$$

$$= 125 \text{ m}^2/\text{day}$$

Coefficient of Storage

The coefficient of storage can be obtained from the distance-drawdown diagram by using the following equation derived from Equation 9.6:

$$S = \frac{0.3 T t}{r_0^2}$$

$$S = \frac{2.25 T t}{r_0^2} \quad (9.12)$$

where

S = coefficient of storage

T = coefficient of transmissivity, in
gpd/ft

t = time since pumping started, in days

r_0 = intercept of extended straight line at
zero drawdown, in ft

where

S = coefficient of storage

T = coefficient of transmissivity, in
 m^2/day

t = time since pumping started, in days

r_0 = intercept of extended straight line at
zero drawdown, in m

From Figure 9.24, the value of r_o is 500 ft (152 m), T is 9,960 gpd/ft (124 m²/day), and t is 500 minutes (0.347 days). Therefore:

$$S = \frac{0.3 \cdot 9,960 \cdot 0.347}{(500)^2}$$

$$= 4.1 \times 10^{-3}$$

$$S = \frac{2.25 \cdot 124 \cdot 0.347}{(152)^2}$$

$$= 4.2 \times 10^{-3}$$

It is seen, then, that aquifer storage coefficients may be calculated from the following two relationships, revealed by an aquifer test:

1. Rate of lowering of the water level at any place within the cone of depression on the time-drawdown diagram.
2. 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 may be used to check the other.

Other Uses of Distance-Drawdown Graphs

Note that the numerical constant in Equation 9.11 is twice that in Equation 9.7. This occurs because t appears to the first power in Equation 9.6, whereas r appears to the second power. Since $\log r^2$ is the same as $2 \log r$, it follows that the value of Δs for the distance-drawdown graph is twice the Δs for the time-drawdown graph. For a given aquifer and a given pumping rate, this ratio for the slopes of the two straight lines is a fixed relationship. Therefore, when Δs is determined from a time-drawdown graph, the slope of the curve on the distance-drawdown graph should be twice as great if the

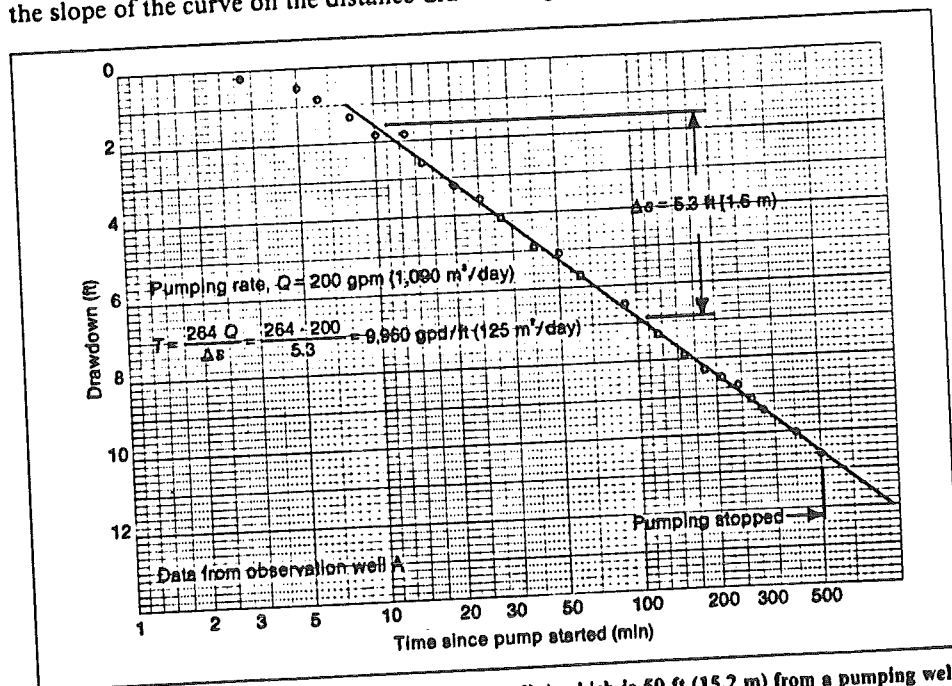


Figure 9.25. Time-drawdown curve for observation well A which is 50 ft (15.2 m) from a pumping well. Aquifer test was conducted at a constant pumping rate of 200 gpm (1,090 m³/day).

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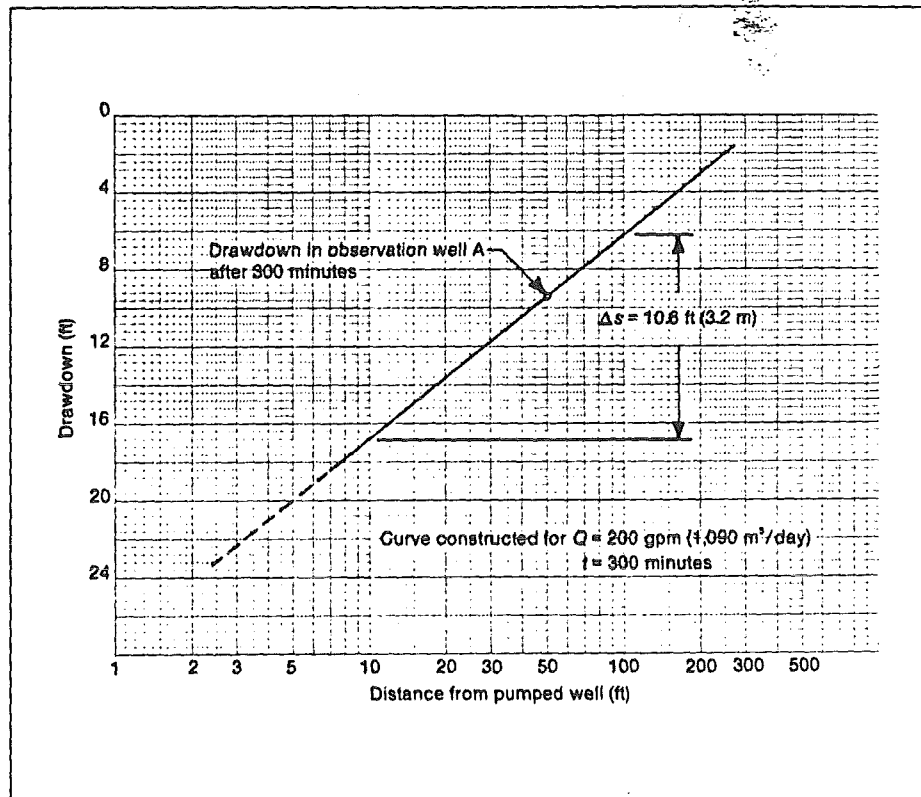


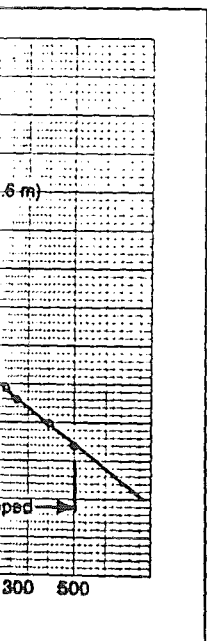
Figure 9.26. Distance-drawdown plot constructed from time-drawdown data, Figure 9.25, represents the position of the cone of depression after 300 minutes of pumping at 200 gpm (1,090 m³/day).

well is pumped at the same rate. This permits construction of the distance-drawdown graph from a time-drawdown graph derived from measurements in a single observation well. However, when drawdown is observed in only one well (for example, the production well), independent calculations of aquifer performance are not possible.

The following example explains how distance-drawdown graphs can be constructed from time-drawdown data. Figure 9.25 shows a semilog time-drawdown graph of data from observation well A, which is 50 ft (15.2 m) from the pumped well as shown in the two previous illustrations. The Δs from the time-drawdown plot is 5.3 ft (1.6 m), exactly one-half the value of Δs from the distance-drawdown curve in Figure 9.24.

If a distance-drawdown plot is constructed after 300 minutes of pumping, the drawdown in well A at 300 minutes is 9.4 ft (2.9 m) as shown in Figure 9.25. This measurement is plotted at the 50-ft distance on a new diagram, as shown in Figure 9.26. A straight line with $\Delta s = 2 \times 5.3 = 10.6$ ft (3.2 m) is then drawn through this point. Thus, a distance-drawdown plot is derived which represents the cone of depression after 300 minutes of pumping at 200 gpm.

In many cases, the drawdown must be calculated at pumping rates different from the one used in the pumping test. These data are needed to calculate the interference effect of one well upon another at various pumping rates. For example, it may be necessary to calculate the interference effect in a well 300 ft (91.5 m) away from a per-



) from a pumping well.
y).

manent well pumping 400 gpm (2,180 m³/day) that will be constructed at the site of the test well.

Equation 9.11 shows that Δs varies directly with the pumping rate, Q . For an assumed pumping rate of 400 gpm, therefore, the Δs of the distance-drawdown plot would be $2 \times 10.6 = 21.2$ ft (6.5 m).

From the data shown in Figures 9.25 and 9.26, a series of straight-line graphs can be developed, each one representing the relationship between distance and drawdown at a different pumping rate. To do this, two relationships are applied from the equations already discussed. The first of these relations is that the drawdown at any location outside the pumped well, such as at observation well A, varies directly with the pumping rate. The second is that Δs , the slope of the distance-drawdown graph, also varies directly with the pumping rate.

To illustrate this procedure, distance-drawdown graphs are constructed for pumping rates of 100 gpm (545 m³/day), 250 gpm (1,360 m³/day), and 400 gpm (2,180 m³/day) at 300 minutes. The first step is to determine one definite point through which each of the desired curves must pass. This is done by multiplying the measured drawdown in observation well A, taken from either Figure 9.25 or Figure 9.26, by the ratio of the pumping rates. The drawdown at observation well A, for a pumping rate of 100 gpm, would be one-half of the drawdown as measured at the pumping test rate of 200 gpm. The measured drawdown at 200 gpm was 9.4 ft (2.9 m), so the calculated drawdown for 100 gpm is 4.7 ft (1.4 m) for the same period of pumping. This value plotted on a semilogarithmic graph (Figure 9.27) is a definite point representing conditions when pumping 100 gpm, 50 ft (15.2 m) from the pumped well.

The next step is to calculate the slope of the distance-drawdown plot for 100 gpm.

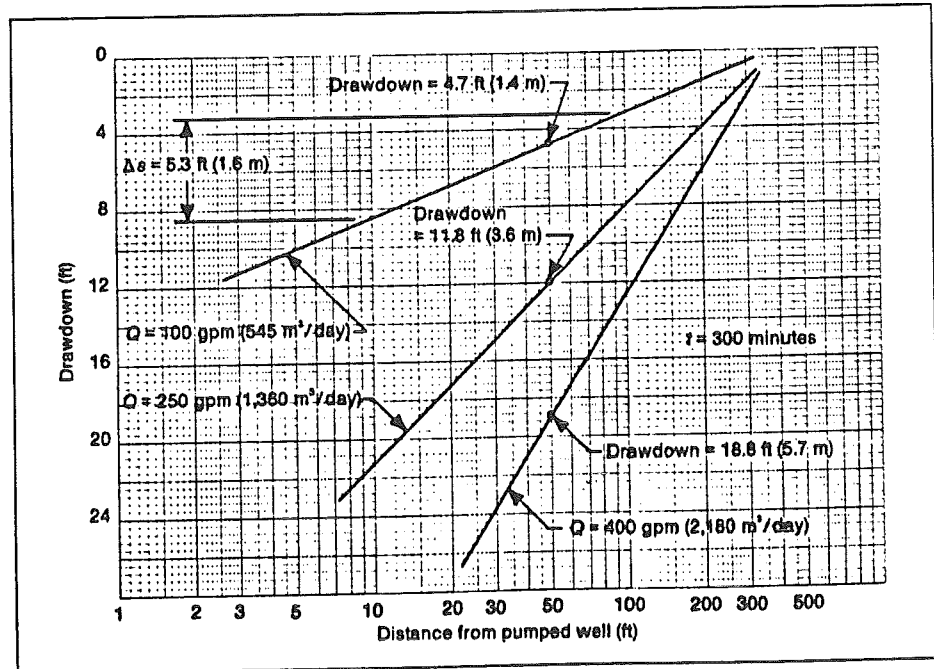


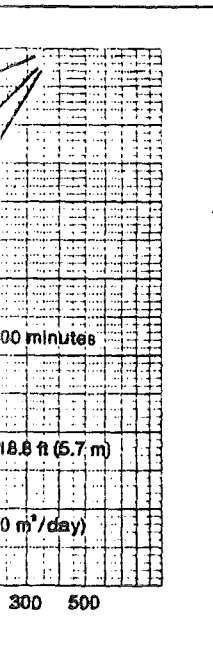
Figure 9.27. Distance-drawdown diagram for various rates of pumping, using data from Figure 9.25. Each plot represents conditions after 300 minutes of pumping at the rate indicated.

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This is done by multiplying the slope shown in Figure 9.26 by the ratios of the pumping rates. The value of Δs is 10.6 ft (3.2 m) when the pumping rate is 200 gpm. The value of Δs for a rate of 100 gpm is one-half as much, or 5.3 ft (1.6 m).

With the calculated drawdown of 4.7 ft at 50 ft from the pumped well and the calculated value of Δs , the distance-drawdown curve can now be drawn for a pumping rate of 100 gpm. This is done by constructing a straight line with a slope of 5.3 ft per log cycle through the point representing 4.7 ft of drawdown at 50 ft from the pumped well, as shown by the upper curve in Figure 9.27. Data for plotting the other curves in Figure 9.27 are given in Table 9.2. These were calculated in the manner described above.

Table 9.2. Data for Distance-Drawdown Graphs

Pumping Rate		Drawdown at 50 ft		Values of Δs	
gpm	m ³ /day	ft	m	ft	m
100	545	4.7	1.4	5.3	1.6
200	1,090	9.4	2.9	10.6	3.2
250	1,360	11.8	3.6	13.3	4.1
400	2,180	18.8	5.7	21.2	6.5

Each graph is constructed by plotting a point representing the drawdown for a certain time at a given distance from the pumped well (50 ft in this case) and drawing

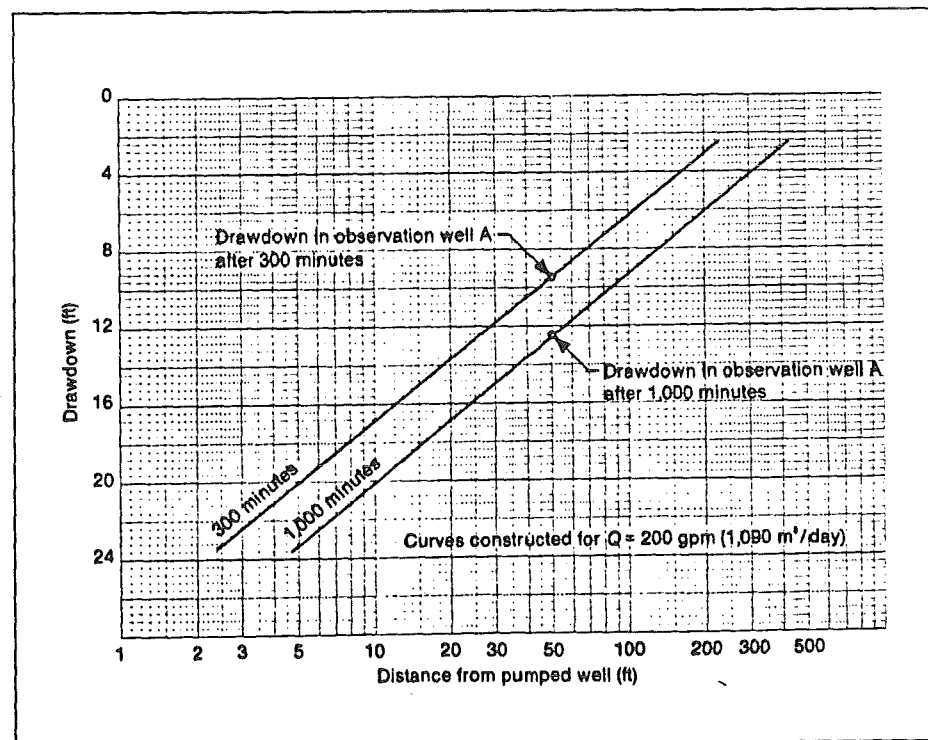


Figure 9.28. Distance-drawdown graph showing the positions of the cone of depression after pumping for 300 minutes and 1,000 minutes at a constant rate of 200 gpm (1,090 m³/day).

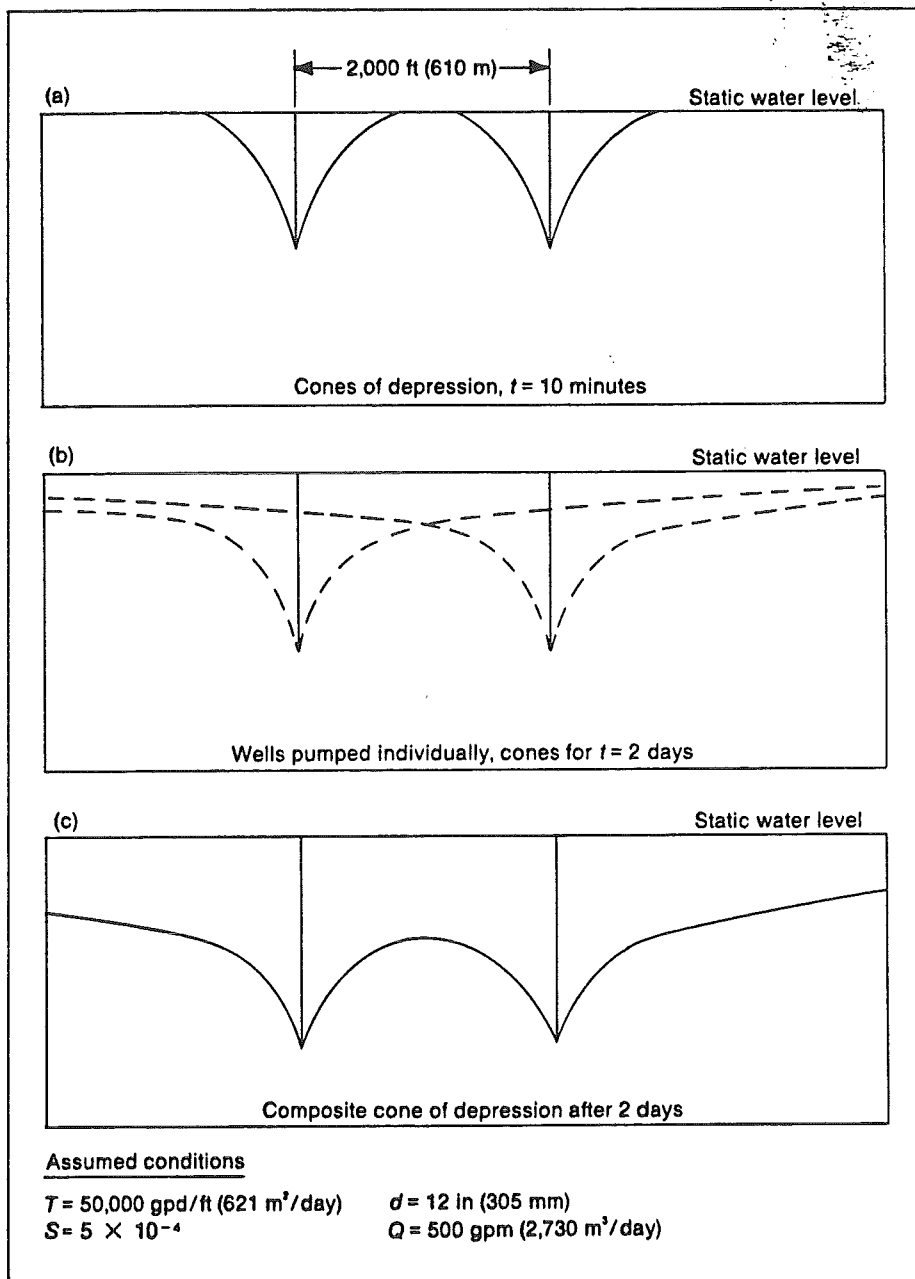


Figure 9.29. Interference between adjacent wells tapping the same confined aquifer. Composite cone is for both wells pumping simultaneously under the assumed conditions.

through this point a straight line having a Δs , or slope, of the calculated value.

WELL INTERFERENCE

The interference or drawdown in another well 300 ft (91.5 m) from the pumped well

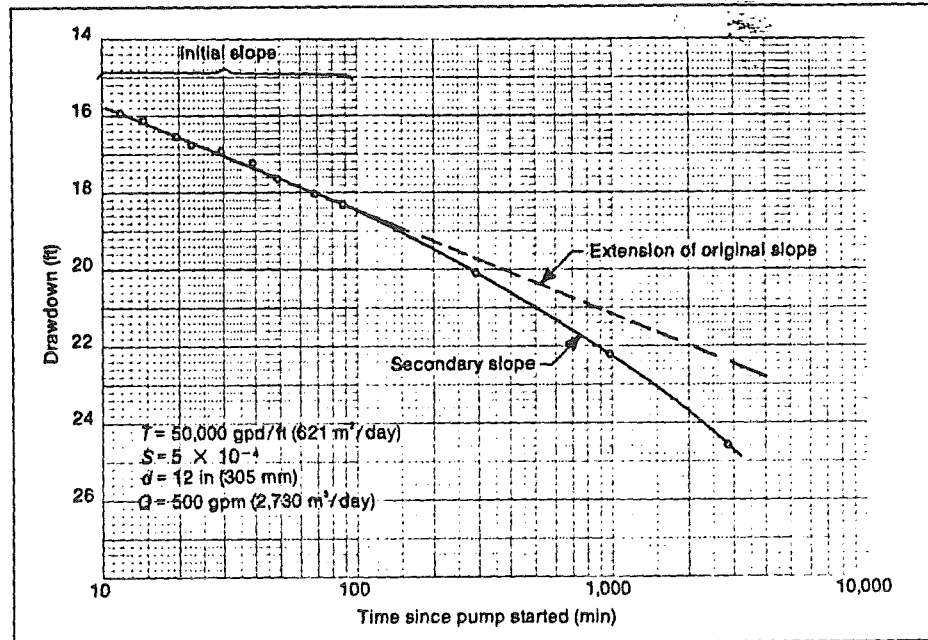


Figure 9.30. Time-drawdown graph for either well in Figure 9.29 shows effect of well interference when the cones of depression intersect.

used in the previous example is obtained directly from Figure 9.27. If the test well were pumped at 400 gpm ($2,180 \text{ m}^3/\text{day}$), the distance-drawdown plot shows that the water level 300 ft away would be drawn down 2.2 ft (0.7 m); the total drawdown would then be that caused by its own pumping plus the 2.2 ft of drawdown interference from the first well. Operation of the second well would also produce the same degree of interference on the first well if both were pumped at 400 gpm. The interference effects of several wells spaced at various distances can be evaluated in the same way. The total effect in any one well is the sum of the drawdown interferences produced by all the others in the group.

In this discussion, the calculations of well interference are for aquifer conditions after pumping continuously for 300 minutes. Figure 9.26 was constructed on this basis. The first point plotted was taken from the time-drawdown graph for observation well A at 300 minutes after pumping began. Because Figure 9.27 was constructed from Figure 9.26, all the distance-drawdown curves in Figure 9.27 are also valid only for 300 minutes after pumping began.

If interference data for a different period of continuous pumping are desired, one must start with another point taken from the original test data. The drawdown in observation well A after 1,000 minutes of pumping, shown in Figure 9.25 as 12.3 ft (3.8 m), might be used. The plot showing conditions after 1,000 minutes of pumping at 200 gpm ($1,090 \text{ m}^3/\text{day}$) is parallel to the plot for a pumping period of 300 minutes as shown in Figure 9.28. Another set of plots similar to those in Figure 9.27 can then be prepared, starting with the data from the 1,000-minute plot in Figure 9.28.

Figure 9.29 should be studied carefully for a clear picture of interference and just what it means in a well field where several cones of influence overlap. In part (a), the

profiles of two wells that are 2,000 ft (610 m) apart are shown, each pumping continuously at a rate of 500 gpm (2,730 m³/day) for 10 minutes. Note that at the end of 10 minutes the cones of depression have not reached each other. In part (b), the extent of the cones of depression after 2 days of continuous pumping is shown. If either of the two wells were pumped alone, the cone of depression would be represented by the dashed line for the well being pumped. When both wells are pumped, however, the net result will be as in part (c), which is the sum of the drawdowns at any point within the zone of influence of both wells.

A time-drawdown curve for each of the two wells in Figure 9.29 would appear as shown in Figure 9.30. The correct transmissivity of the aquifer can be obtained only from the initial slope of this curve. The secondary slope, which is about twice as steep, shows the effect of the other well. The net result of the second well, when pumping at the same rate, is mathematically equivalent to an impervious boundary.

WELL EFFICIENCY

The efficiency of a pumped well in some cases can be estimated from the distance-drawdown graph. This can be done by extending the straight line representing the profile of the cone of depression to show the drawdown in the aquifer just outside the well. Such an extension is shown in Figure 9.31.

The intersection of the extended line with the radius of the pumped well shows the theoretical drawdown for a 100-percent efficient well. (In a filter-packed well, the radius is taken from the center of the well to the outside of the filter pack.) The result is valid for a confined aquifer only when the full thickness of the aquifer is screened. The theoretical drawdown for the example shown is 26.3 ft (8 m); the actual drawdown in

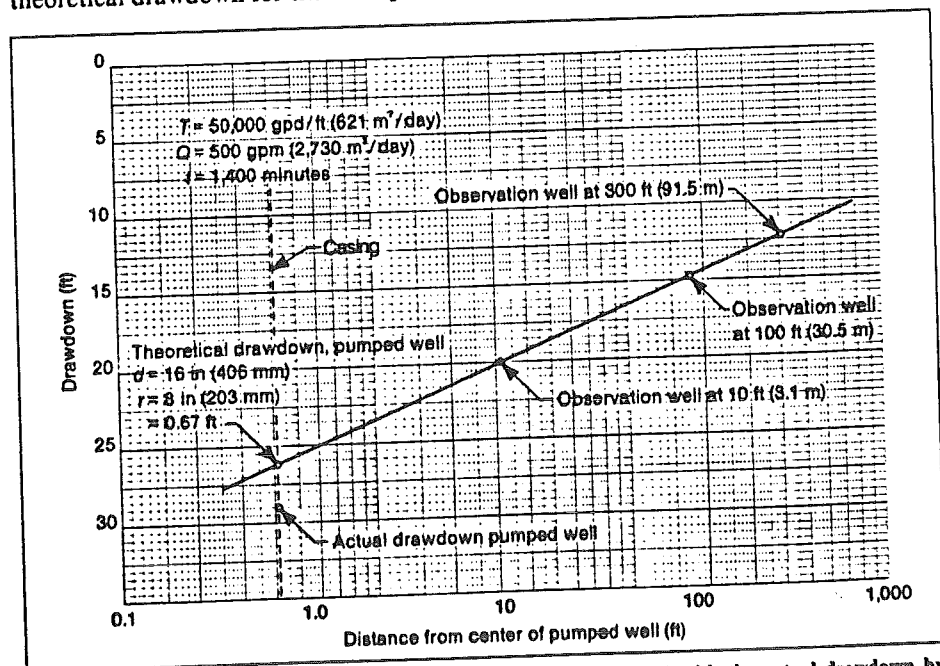


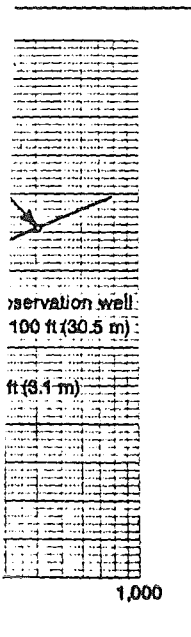
Figure 9.31. Theoretical drawdown of a pumped well can be compared with the actual drawdown by extending the straight line on the distance-drawdown diagram to a point where the radius of the well (outer face of the well) is indicated on the horizontal scale.

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the well is 29 ft (8.8 m). The well efficiency is calculated as $26.3/29 = 91$ percent.

Extra drawdown occurs when well operation causes considerable reduction in the saturated thickness of the aquifer. The extra drawdown, found primarily in wells in unconfined aquifers, should not be considered as representing inefficiency in the pumped well. An equation is given in Appendix 9.C to correct the theoretical drawdown so that well efficiency can be calculated more accurately when partial dewatering of the aquifer occurs.

The factors contributing to excess drawdown in wells (inefficiency) can be grouped into two classes. One class comprises those factors related primarily 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

1. Choice of a well screen with insufficient open area makes entrance velocities too high, resulting in greater-than-normal entrance (head) losses.
2. Poor distribution of screen openings causes excessive convergence of flow near the individual openings, and may produce twice as much drawdown as necessary. The accompanying diagrams of flow nets around well screens illustrate this point (Figure 9.32).
3. Insufficient length of well screen, resulting in partial penetration of the aquifer, distorts the flow pattern for some distance around the well (Figure 9.33). Flow to the well screen then includes major vertical components as well as the main horizontal component. Vertical hydraulic conductivity is generally lower than horizontal hydraulic conductivity, so considerable head losses result from the vertical flow. Even though extra drawdown may result, a shorter well screen may be used in many instances because of other design considerations. The effect of partial penetration of the aquifer is discussed later in this chapter; screen length is discussed in Chapter 13.
4. Improperly sized filter packs or those made from angular or platelike materials can restrict flow into a well screen. Particle shape, size, and grain size distribution affect the hydraulic conductivity of the pack.

Construction Factors

1. Inadequate development of a well may leave so much drilling fluid and small particles in the formation around the screen that the original permeability is reduced. Drilling contractors have great difficulty developing wells when they install screens that have insufficient open area or poor distribution of slot openings.
2. Improper placement of the well screen may put it at a depth that does not correspond to the best water-bearing stratum.

RADIUS OF INFLUENCE

The radius of influence of a well can be determined from the distance-drawdown graph in most instances. Many owners of high-capacity wells want this information because it indicates how far the cone of depression extends from the well. For all practical purposes, it is the distance indicated by extending the straight line on the distance-drawdown graph to the point of zero drawdown on the drawdown scale. Note that this distance is the same as r_0 in Equation 9.12.

Some believe that the radius of influence should be the determining factor in well spacing to avoid any interference between wells. This practice is often impractical, es-

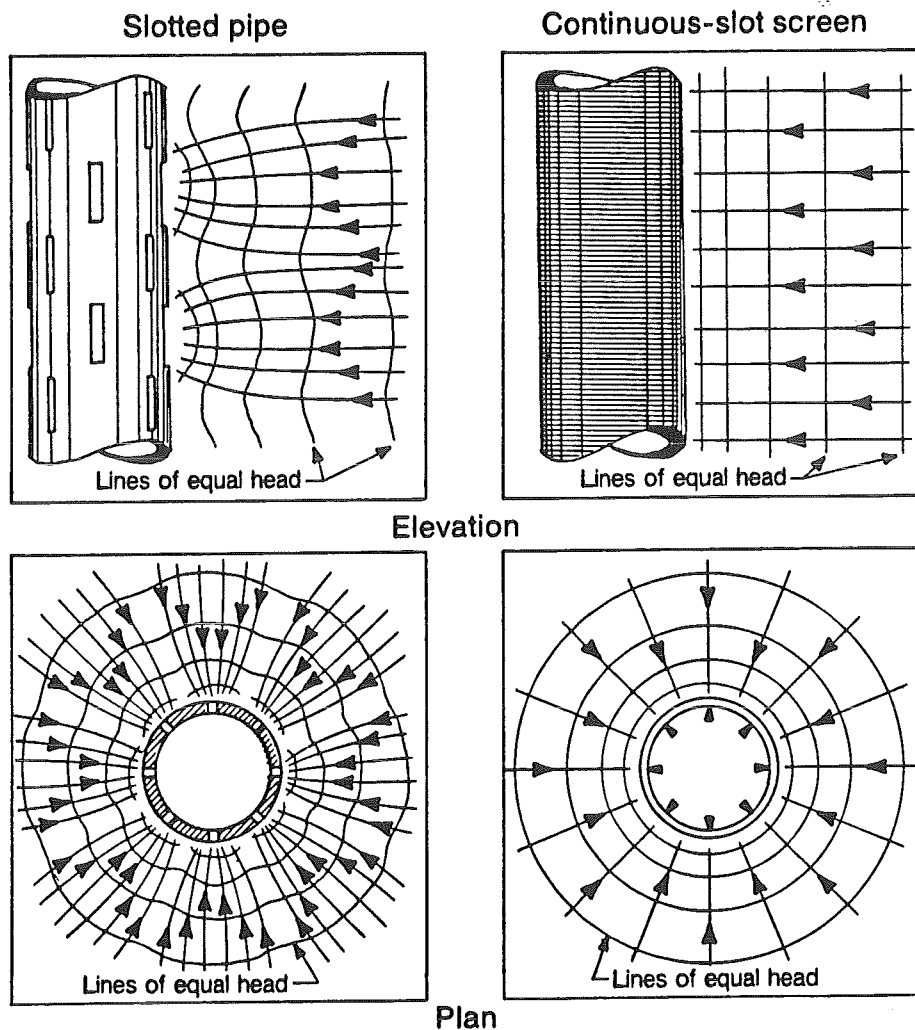


Figure 9.32. Flow nets around screen devices. Water approaches openings along lines indicated by arrows. Flow lines for slotted pipe converge to individual slots; flow lines for continuous-slot well screens are less distorted.

pecially in confined aquifers where cones of depression may extend 1 to 2 mi (1.6 to 3.2 km) or more for high-capacity wells.

As the cone of depression begins to stabilize during a pumping test, further increases in the radius of influence are usually slight. The cone of depression may stabilize because of recharge from precipitation, leakage from saturated strata above or below the aquifer, or natural groundwater flow toward and past the well. Equilibrium may also result from induced recharge from a lake or stream (considered to be a line source) when a hydraulic gradient develops between the line source and the pumped well.

RECHARGE AND BOUNDARY EFFECTS

In the case of induced recharge, the cone of depression is steeper toward the line

source than in other directions away from the well. If the lake or stream is nearby, measurements in observation wells between the pumped well and the recharge source produce a distance-drawdown curve that is steeper than normal. This occurs because the recharge effect results in less drawdown in the observation wells nearest the line source as compared with those observation wells on the opposite side of the pumped well. But the slope of the distance-drawdown curve derived from the pumping test will be almost the same in all directions if the recharge source is relatively distant and the observation wells are relatively close to the pumped well. Except in cases of nearby line sources of recharge, the slope of the distance-drawdown curve provides a reliable basis for calculating aquifer transmissivity regardless of recharge effect.

The same cannot be said for calculation of the storage coefficient when recharge occurs. Recharge affects the vertical position of the distance-drawdown curve, which makes the value of r_0 less than it would be in the absence of recharge. This, in turn, makes the calculated storage coefficient greater than its real value. In some cases, the calculated storage coefficient is greater than one — an impossible value — proving that recharge is occurring.

The effect of impervious boundaries on the distance-drawdown graph is essentially opposite to the effect of recharge. The slope of the curve is affected only slightly if the distance to any boundary is much greater than the distances to the observation wells. Observation wells close to a boundary, however, show drawdowns greater than normal, and the resulting distance-drawdown curve is a little flatter than it would be otherwise. This leads to a calculated value of transmissivity that is higher than the true value. The calculated value of the storage coefficient is lower than the true value because the distance-drawdown curve is displaced downward on the semilog diagram because of the influence of the aquifer boundary. An unreasonably low value for the storage coefficient usually indicates an impervious boundary within the zone of influence of the pumping well.

COMBINED USE OF SEMILOG GRAPHS

As shown above, calculations from time-drawdown graphs can be used to check cal-

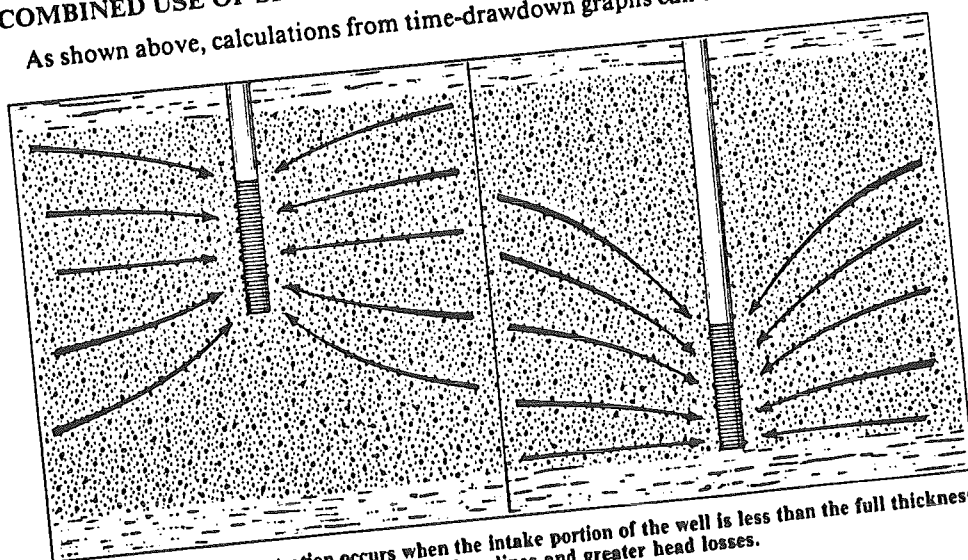
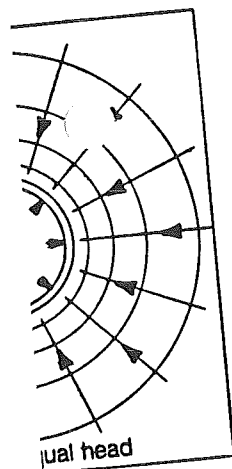
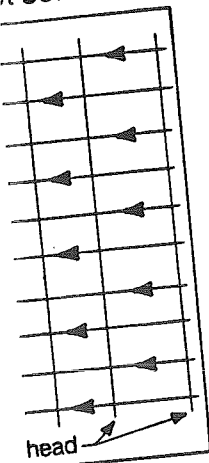


Figure 9.33. Partial penetration occurs when the intake portion of the well is less than the full thickness of the aquifer. This causes distortion of the flow lines and greater head losses.

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Table 9.3. Comparisons of Recharge and Boundary Effects on Semilog Diagrams

Recharge Effect During Pumping Test	
Time-drawdown graph	Distance-drawdown graph
1. Slope of the curve becomes flatter. Transmissivity calculated from the flatter slope will be higher than the true value.	1. Slope of the straight line remains almost unchanged if the recharge source is relatively distant and the observation wells are relatively close to the pumped well. Transmissivity calculated from the graph usually will be close to the true value.
2. Extending straight line of the flatter slope gives a value for r_0 that is too low. Storage coefficient calculated from this figure will be lower than the correct value.	2. Straight line is displaced upward. Extension to zero drawdown gives a value for r_0 that is too low. Storage coefficient calculated from this figure will be higher than the correct value.
Boundary Effect During Pumping Test	
Time-drawdown graph	Distance-drawdown graph
1. Slope of the curve becomes steeper. <u>Transmissivity calculated from the steeper slope will be lower than the true value.</u>	1. Slope of the straight line remains almost unchanged if the distance to a boundary is relatively great compared with the distance to the observation wells. Transmissivity calculated from the graph will be close to the true value.
2. Extending line of the steeper slope gives a value for r_0 that is too high. Storage coefficient calculated from this figure will be higher than the correct value.	2. Straight line is displaced downward. Extension to zero drawdown gives a value for r_0 that is too high. Storage coefficient calculated from this figure will be lower than the correct value.

culations from distance-drawdown graphs, and vice versa. If an extensive homogeneous aquifer supplies a well from storage only, both graphs should give identical results. Calculations from the one diagram are independent of those from the other, assuming that the distance-drawdown graph is drawn from measurements in three or more observation wells.

If a recharge effect appears during the pumping test, or if the expanding cone of depression encounters an impervious aquifer boundary, the effects on the two graphs are quite different. Table 9.3 summarizes these contrasts.

Knowledge of the differing effects is useful in aquifer-test interpretation. Suppose, for example, that a test has been performed and drawdown measurements from three observation wells are available. Suppose, also, that the time-drawdown graphs show only one clearly defined straight line. The transmissivity as calculated from the slope of these plots is about one-half of the transmissivity value from the distance-drawdown graph. This result leads to the suspicion that a boundary effect influenced the drawdown measurements so early in the pumping period that a break in the slope of the time-drawdown plots is not apparent. In the example shown in Figure 9.13, any abnormal deviation during the first 10 minutes of pumping would not be evident on

Log Diagrams

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the semilog diagram.

Comparison of calculations of the storage coefficient in our present example, however, supports the suspected boundary condition if the value obtained from the time-drawdown graph is higher than the value from the distance-drawdown graph. In this case, neither value is the true coefficient of storage; it would lie between the limiting values of the two calculated figures. The correct coefficient of transmissivity can be obtained from the distance-drawdown graph if the observation wells are close enough to the pumping well so that the boundary effect does not occur within the first 10 minutes of pumping.

Up to this point, most of our discussion of well performance has assumed that flow to a well bore is entirely radial. Figure 9.2 illustrates this condition, where all flow lines are horizontal and have no vertical component. The flow is radial here because it occurs in (1) a confined aquifer, (2) a screen whose length is the same as the aquifer thickness, and (3) a pumping condition in which the formation near the well is not dewatered.

EFFECT OF PARTIAL PENETRATION

When flow is not strictly radial, the flow pattern and drawdowns differ somewhat from those calculated from the equations presented thus far. Figure 9.34 shows a confined aquifer with only the upper 50 percent screened. Here the arrows represent some typical flow lines or paths of water particles as they move through the formation to the intake portion of the well. The departure from radial flow is evident.

Water in the lower part of the aquifer must move upward along the curved lines to reach the well screen. In doing so, this water must take paths that are somewhat longer than radial flow lines. Also, the flow must converge through a smaller cross-sectional area while approaching the short screen. The result of the longer flow paths and smaller cross-sectional area is an increase in head loss. For a given yield, therefore, the drawdown in a pumping well is greater if the aquifer thickness is only partially screened. For a given drawdown, the yield from a well partially penetrating the aquifer is less than the yield from one completely penetrating the aquifer.

Figure 9.34 shows that drawdowns near the pumped well vary with depth in the

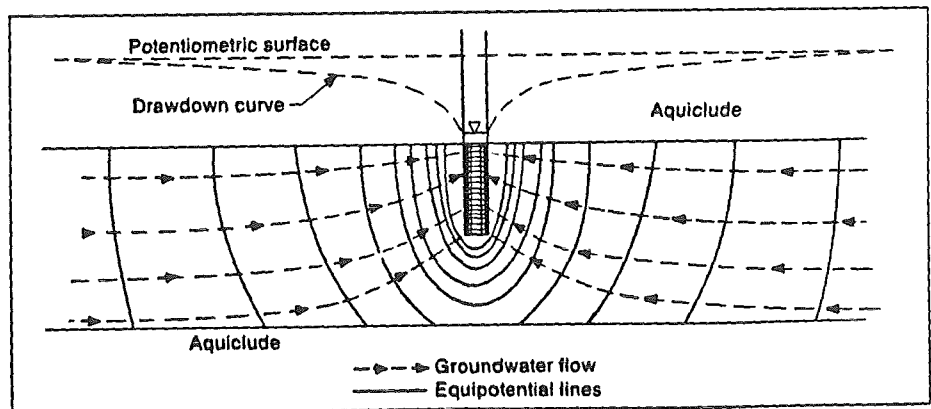


Figure 9.34. When the intake section of a well partially penetrates a confined aquifer, flow lines deviate somewhat from the radial flow pattern associated with a fully penetrating well. (Water and Power Resources Service, 1981).

confined aquifer; the potentiometric drawdowns shown here also indicate that pressure varies with depth. This variation in drawdown with depth is most pronounced adjacent to the well and decreases with distance from the well. At a distance equal to about twice the aquifer thickness, the line representing drawdown in the aquifer is vertical, showing that the drawdown is the same at all depths. Radial flow prevails at this distance and beyond.

The mathematical analysis of partial penetration problems is difficult for homogeneous aquifers, but the development of equations for stratified aquifers is almost impossible. The curves in Figure 9.35 provide a simple method for estimating results from partially penetrating wells in confined aquifers that are reasonably homogeneous. This diagram was developed from the Kozeny (1933) equation:

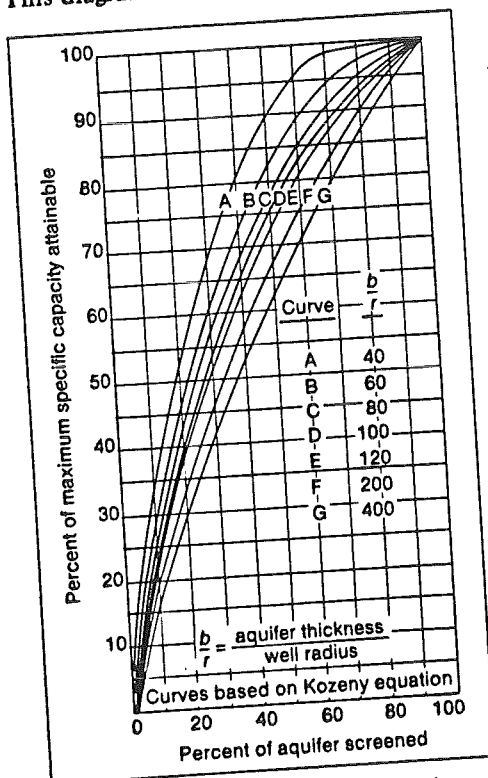


Figure 9.35. Relationship of partial penetration and attainable specific capacity for wells in homogeneous confined aquifers.

etrating well, which is theoretically impossible. The curves in Figure 9.35 represent conditions within the valid range of the equation.

To use the curves, the screen length must be expressed as a percentage of the aquifer thickness. After locating this value on the horizontal scale, move upward along a vertical line to the curve that represents the ratio of aquifer thickness to well radius for the case in question. Then move horizontally and read the percentage value on the vertical scale. This represents the specific capacity of the partially penetrating

$$\frac{Q/s_p}{Q/s} = L \left(1 + 7 \sqrt{\frac{r}{2bL} \cos \frac{\pi L}{2}} \right) \quad (9.13)^*$$

where

Q/s_p = specific capacity of a partially penetrating well, in gpm/ft ($\text{m}^3/\text{day}/\text{m}$)

Q/s = maximum possible specific capacity of a fully penetrating well, in gpm/ft ($\text{m}^3/\text{day}/\text{m}$). (Left-hand side of the equation is the ratio of the production of a partially penetrating well to that of a fully penetrating well.)

r = well radius, in ft (m)

b = aquifer thickness, in ft (m)

L = well screen length as a fraction of the aquifer thickness

This equation is not valid when the aquifer thickness is small, the percent of penetration is large, and the well radius is large. Calculated results for some combinations of these conditions would be higher than those for a completely pen-

*This equation can be used with either SI or English terms.

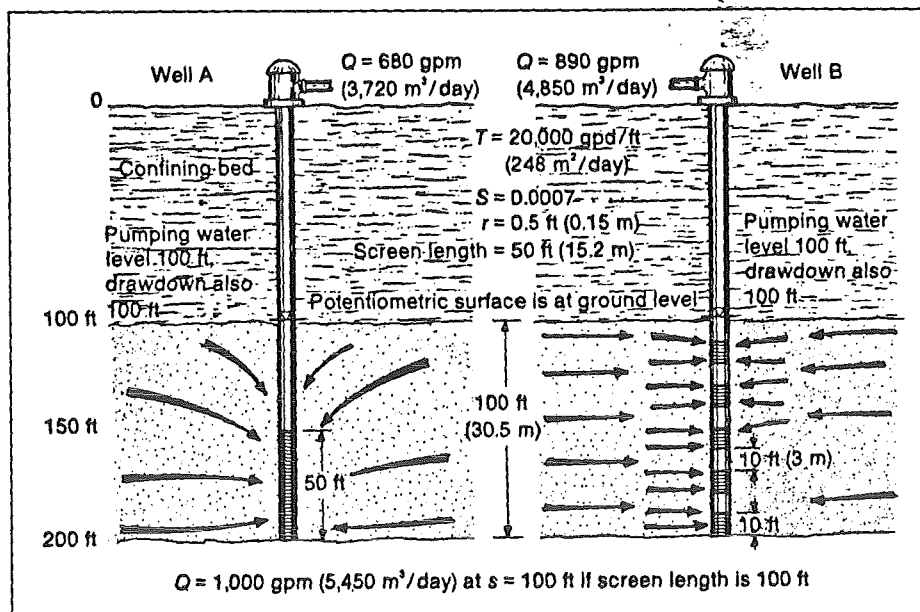


Figure 9.36. Well performance can be improved by using multiple sections of well screen in a thick aquifer to reduce the effect of partial penetration. Total screen length is the same in both wells.

well, expressed as a percentage of the specific capacity that could be obtained with a fully penetrating well. The specific capacity of a fully penetrating well is referred to here as its maximum possible specific capacity.

Analysis of the alternate well designs shown in Figure 9.36 provides a practical example of the use of the curves. Both wells have been constructed with 50 ft (15.2 m) of 12-in (305-mm) diameter well screen. A homogeneous confined aquifer is assumed. In well A, the screen is positioned in the lower half of the aquifer, making $b/r = 200$. Figure 9.35 shows that 68 percent of the maximum possible specific capacity could be expected from this well. Using Equation 9.5, the specific capacity of a fully penetrating well with a 6-in (152-mm) radius is calculated as 10 gpm/ft (179 $\text{m}^3/\text{day}/\text{m}$) of drawdown after 24 hours of pumping. The expected specific capacity for well A, therefore, is 68 percent of this figure, or 6.8 gpm/ft (122 $\text{m}^3/\text{day}/\text{m}$) of drawdown. The available drawdown to the top of the aquifer is 100 ft (30.5 m), so the yield would be $6.8 \times 100 = 680 \text{ gpm}$ ($122 \times 30.5 = 3,720 \text{ m}^3/\text{day}$).

In well B, the 50-ft length of screen has been split into 10-ft (3-m) sections. These are separated by four 10-ft sections of blank pipe. With this arrangement, the 100-ft aquifer can now be treated as five aquifers, each 20 ft (6.1 m) thick with each section 50 percent screened; this changes the b/r ratio. The advantage of this screen arrangement is apparent from the curves. If 50 percent of an aquifer that is 20 ft thick ($b/r = 40$) is screened, 89 percent of the maximum specific capacity can be obtained. From this observation, the estimated yield of well B is 890 gpm (4,850 m^3/day), a 31-percent improvement over well A. (To simplify this comparison, well entrance losses have been ignored.)

In a well in an unconfined aquifer, the problem of partial penetration must always be considered because the pumping well dewateres the upper part of the water-bearing

formation. This reduces the saturated thickness and necessarily shortens the intake portion of the well. This type of well is often pumped so that its drawdown is a large proportion of the aquifer thickness. This results in a greatly distorted flow pattern as compared with strictly radial flow. Figures 9.11 and 9.12 can provide a reasonable estimate of the effects of partial penetration and dewatering for wells in unconfined aquifers.

WATER-LEVEL RECOVERY DATA

When pumping is stopped, well and aquifer water levels rise toward their pre-pumping levels. The rate of recovery provides a means for calculating the coefficients of 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. Values obtained from analysis of the recovery record serve to check calculations based on the pumping record.

The water-level recovery data from an observation well will indicate the hydraulic characteristics of the aquifer if the well is located close enough to the pumped well so that the drawdown changes significantly (easily measured) during the pumping test. If no observation well is available, the water-level recovery data from the pumped well can be used for limited calculations of aquifer capability. One observation well should be used if at all possible; two or more are desirable. In all cases, water levels should be measured in the pumped well and in each observation well.

Recovery data can be analyzed as described below only when the test pumping is done at a constant rate. Recovery measurements following a variable-rate test, such as a step-drawdown test, cannot be used (see Chapter 16). The exact time of starting and stopping the pump must be recorded, along with any changes in pumping rate and the time each occurs. The recovery curves reflect the change in aquifer water

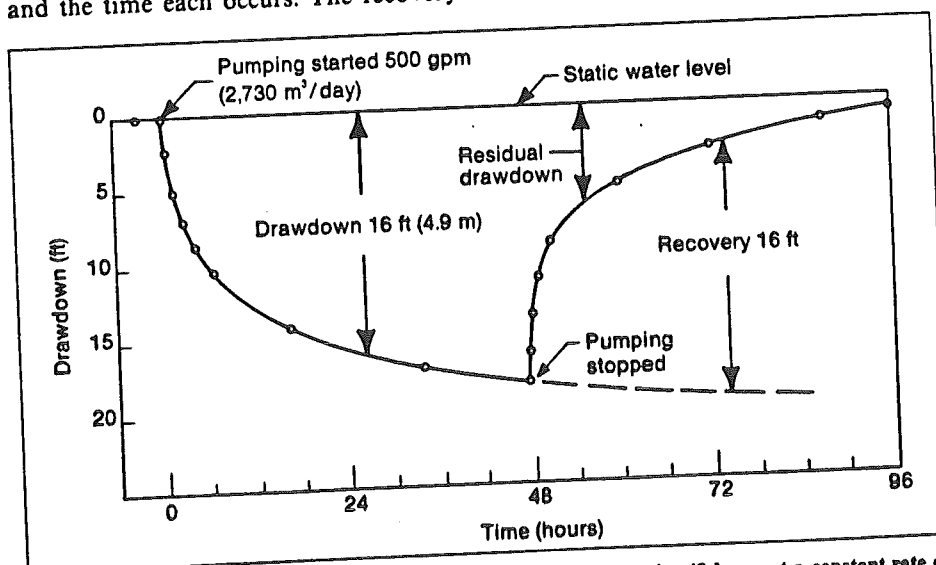


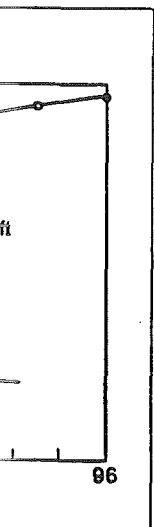
Figure 9.37. Typical drawdown and recovery plots for a well pumped for 48 hours at a constant rate of 500 gpm (2,730 m³/day) followed by a 2-day period for water-level recovery.

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level with time. An essential part of every recovery record, therefore, is the exact time at which each measurement is made. All wells do not have to be measured simultaneously, but the time intervals between readings should be about the same for each.

The elevation of the measuring point (often the top of the well casing) should be determined for each well. An electrical or acoustical sounding device or a weighted steel tape are needed to measure the depth to water accurately.

Figure 9.37 shows how water levels in a well change with time. The left half of the diagram corresponds to the pumping period, the right half to the recovery period. The recovery curve is practically 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 9.37) 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 recovery measurements of the water level must be based on the pumping water level. The hydraulic theory of well and aquifer performance assumes that water-level changes during the recovery period are the result of input from an imaginary recharge well. If such a well injects water into the aquifer at the rate the real well pumps it out, with both wells working simultaneously after a given instant, the recovery curve will be like the one in Figure 9.37. 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, then, means 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 is theoretically identical with the drawdown for the same time during the pumping period. Putting it another way, the recovery 24 hours after pumping stops should equal the drawdown that was measured 24 hours after pumping started. Complete recovery, however, generally requires a period considerably longer than the previous pumping period, except in cases where recharge to the aquifer occurs during the pumping and recovery periods.

Assume that a 6-in (152-mm) test well and an observation well 50 ft (15.2 m) away are available for an aquifer test, as shown in Figure 9.38. After pumping the

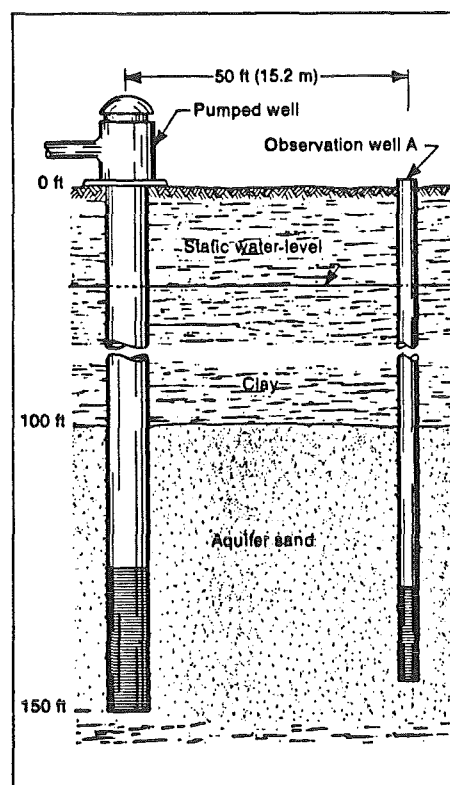


Figure 9.38. Production well and observation well as used in an unconfined aquifer test.

well at 200 gpm (1,090 m³/day) for 500 minutes, the pump is stopped and water-level measurements are made during the first 400 minutes of the recovery period.

Table 9.4 shows the depth-to-water measurements in the observation well and the residual drawdown for time intervals measured from both the beginning of the pumping test and the beginning of the recovery period. These intervals are designated t and t' , respectively. The ratios of the two time periods, t/t' , are also shown in the table.

Figure 9.39 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 shown in Table 9.4.

The recovery curve plotted in Figure 9.39 is difficult to portray by mathematical analysis. It can be simplified for analysis, however, in either of two ways: Theis' (1935) corollary to the nonequilibrium equation (presented below), or Jacob's (1946b) modification of the nonequilibrium equation. It has been shown that the time-drawdown curve for the pumping period becomes a straight line on a semilogarithmic 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 9.4 plotted in this way are shown in Figure 9.40. The result is similar to a time-drawdown plot for the pumping phase of the same aquifer test. Theo-

Table 9.4 Residual Drawdown and Calculated Recovery in the Observation Well

Time since pump started, t min	Time since pump stopped, t' min	Ratio, t/t'	Depth to water*		Residual drawdown*, s'		Drawdown, s , from pumping curve†		Calculated recovery ($s - s'$)	
			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
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

*Static water level, 8 ft (2.44 m)

†Average pumping rate during preceding pumping period was 200 gpm (1,090 m³/day)

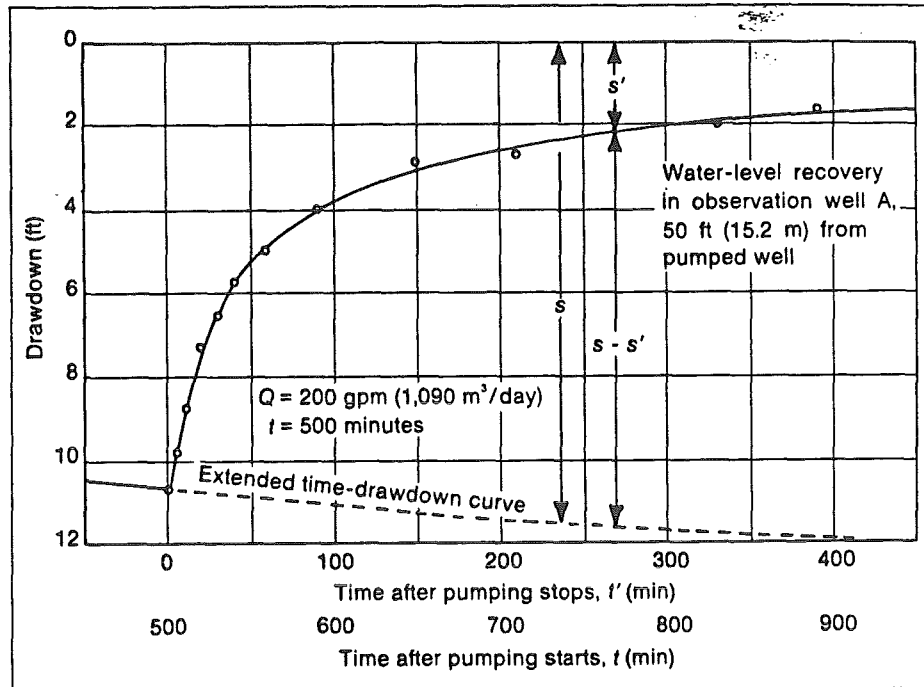


Figure 9.39. Residual-drawdown curve from observation well, with extended time-drawdown curve (on arithmetic scales) showing how calculated recovery is determined at any instant during the recovery period. Producing well pumped 200 gpm (1,090 m³/day) for 500 minutes.

retically, the drawdown and recovery plots should be identical if the aquifer conditions conform to the basic assumptions of the Theis concept.

The time-recovery data from the pumped well can also be plotted by using the method applied to the observation well. 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 can be made without being affected by pump vibrations and momentary variations in the pumping rate.

In analyzing the time-recovery plot, its slope is of primary interest. Two factors determine the slope of the straight line in Figure 9.40. One is the average pumping rate during the preceding pumping period, the other is the aquifer transmissivity.

In Figure 9.40, the slope of the straight line is expressed numerically as the change in the water-level recovery per logarithmic cycle. It is designated by $\Delta(s - s')$. Its value in Figure 9.40 is 5.2 ft (1.6 m), which is the recovery during the period from 10 minutes to 100 minutes after pumping stopped.

The next step is to calculate the transmissivity of the aquifer from the following equation:

$$T = \frac{264 Q}{\Delta(s - s')} \quad T = \frac{0.183 Q}{\Delta(s - s')} \quad (9.14)$$

Note that this equation is similar to Equation 9.7. Figure 9.40 shows the value of T to

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Observation Well

Calculated recovery (s - s')	
ft	m
0.00	0.00
0.05	0.01
0.10	0.03
0.21	0.06
0.52	0.15
0.90	0.28
1.41	0.43
2.00	0.61
3.40	1.03
4.20	1.28
5.10	1.55
5.85	1.78
6.95	2.12
8.35	2.55
8.65	2.64
9.50	2.89
9.80	2.99
10.35	3.15

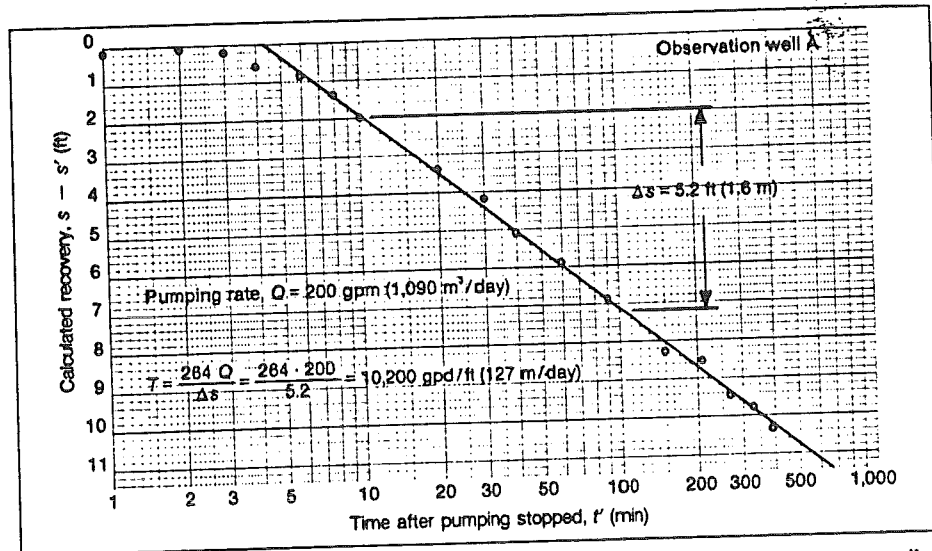


Figure 9.40. Time-recovery plot for observation well becomes a straight line when plotted on a semilog diagram, similar to the time-drawdown diagram for the preceding pumping period.

be about 10,200 gpd/ft (127 m³/day), which may be compared with T as calculated from the time-drawdown data plotted in Figure 9.25. If test conditions meet the required standards and measurements are taken carefully, the two results should agree reasonably well.

A second 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:

$$s' = \frac{264 Q}{T} \log t/t' \quad s' = \frac{0.183 Q}{T} \log t/t' \quad (9.15)$$

Mathematical development of this relationship is given in Appendix 9.D.

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 9.41 shows the data from Table 9.4 plotted on a semilog diagram, with s' indicated on the vertical arithmetic scale and t/t' on the horizontal logarithmic scale. The transmissivity is then calculated from the following equation:

$$T = \frac{264 Q}{\Delta s'} \quad T = \frac{0.183 Q}{\Delta s'} \quad (9.16)$$

Note from Figure 9.41 that time during the recovery period increases toward the left in this method of plotting, whereas on the time-drawdown and time-recovery plots time increases toward the right.

The residual-drawdown plot as shown in Figure 9.41 is preferred over the recovery plot, Figure 9.40, for calculating transmissivity. The method shown in Figure 9.41 provides a more independent check on the results calculated from the pumping period.

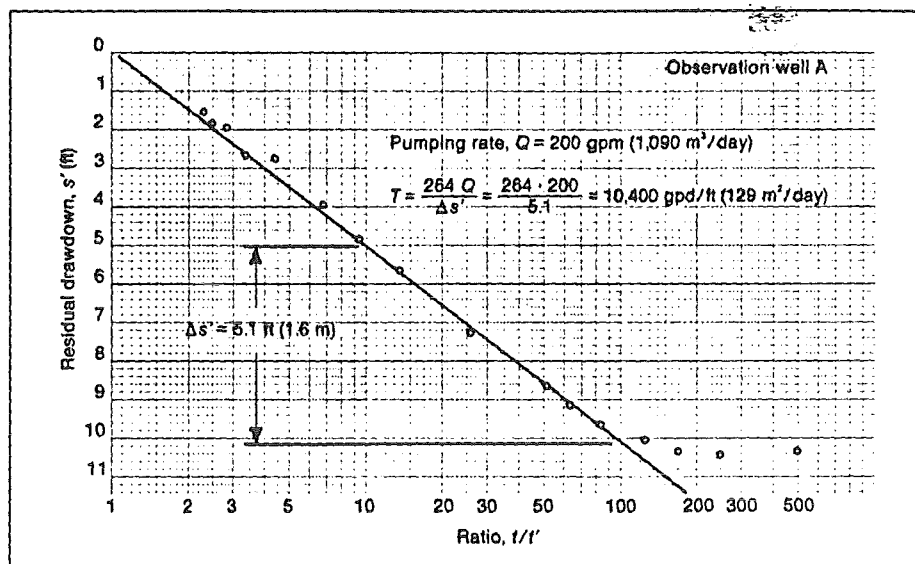


Figure 9.41. Residual drawdown plotted against the ratio t/t' becomes a straight line on semilog graph and permits calculation of transmissivity as shown. Time during recovery period increases toward the left in this diagram.

The method used in Figure 9.40 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, the recovery data from the pumped well usually provide the best basis for calculating the transmissivity of the aquifer. The residual-drawdown plot, as shown in Figure 9.41, should always be used in such a case.

Determining Storage Coefficient Using Recovery Data

If measurements are made in at least one observation well during the recovery period, the storage coefficient can be calculated from portions of these data. The data must be plotted as shown in Figure 9.40. The residual-drawdown plot cannot be used for determining the storage coefficient, even though that plot is valid for calculating the transmissivity.

Figures 9.42 and 9.43 show the similarity in calculations of the storage coefficient from time-drawdown and time-recovery diagrams. Using Equations 9.7 and 9.8, the time-drawdown data for an observation well, shown in Figure 9.42, give values of $T = 13,000$ gpd/ft ($161 \text{ m}^3/\text{day}$) and $S = 5.7 \times 10^{-4}$, respectively. Parallel calculations from Figure 9.43 using $\Delta(s - s')$ in place of Δs and t'_0 in place of t_0 , give values of $T = 13,700$ gpd/ft ($170 \text{ m}^3/\text{day}$) and $S = 4.4 \times 10^{-4}$, respectively. These two sets of results are considered to be in reasonable agreement.

It is apparent from the residual-drawdown curve in Figure 9.41 that t'_0 cannot be obtained from that diagram. The horizontal scale represents a ratio without units. The intercept of this curve at zero drawdown has an entirely different significance on this graph. It is necessary to review the basic assumptions listed on page 218 that were used in developing the equations for both the pumping period and the recovery period

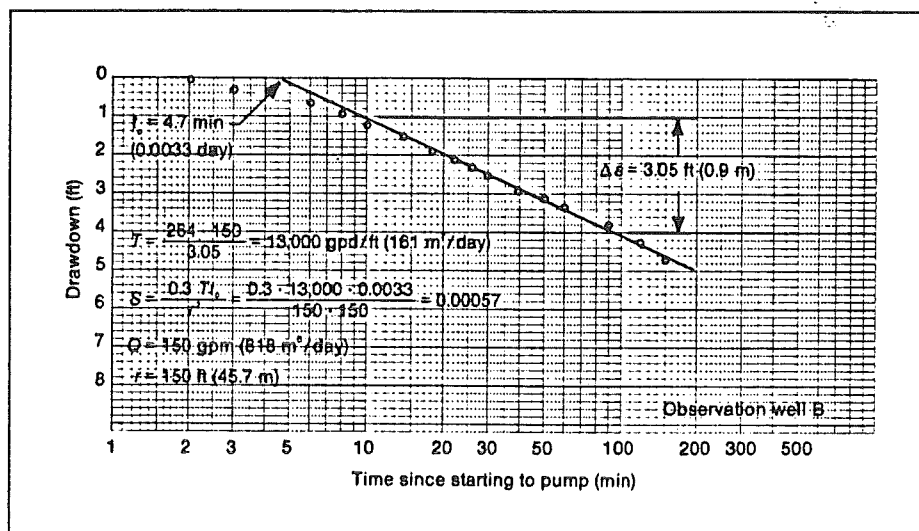


Figure 9.42. Time-drawdown curve plotted from measurements in an observation well 150 ft (45.7 m) from a pumped well. Value of t_0 for calculating the coefficient of storage, S , is obtained by extending straight line to the left to zero drawdown.

of an aquifer test. When an aquifer conforms to these assumptions, the residual-drawdown curve, when extended to the left as in Figure 9.41, should pass through the zero-drawdown point where the ratio t/t' reaches 1. The ratio t/t' approaches 1 as the recovery period is extended.

After a long period of recovery, the water level throughout the aquifer tends to return to the original static level, with the residual drawdown approaching zero as t/t' approaches 1. Theoretically, therefore, the residual-drawdown curve should pass through the upper left corner of the diagram as in Figure 9.41.

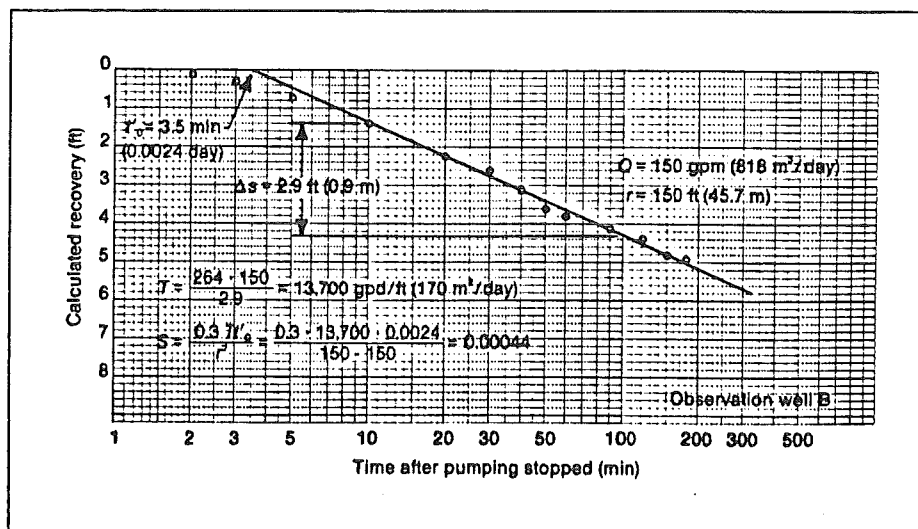


Figure 9.43. Time-recovery curve for an observation well, extended to the left to get t_0' for computing the storage coefficient, S .

A study of residual-drawdown curves from actual aquifer tests reveals that the curve does not always pass through this point, called the origin of the diagram. When the curve fails to pass through the origin, it is concluded that the aquifer conditions do not conform to the assumed idealized conditions.

Three ways in which the conditions differ from the theoretical aquifer may be indicated by the residual-drawdown plot. If the graph indicates zero drawdown at a t/t' value of 2 or more, it is concluded that some recharge water reached the aquifer during the pumping period. The result of the recharge is to bring about full recovery to the original static level during a relatively short recovery period, long before t/t' approaches 1. The upper plot in Figure 9.44 might be obtained for such a situation.

A different condition is indicated when the plot extended to the left shows a residual drawdown of several inches or more as t/t' approaches 1. This situation would occur in an aquifer of limited extent with no recharge, when pumping permanently lowers the static water level. The lowest plot in Figure 9.44 illustrates this type of result.

The third condition that can account for minor displacement of the residual drawdown plot results from a variation in the storage coefficient, S . In theory, the storage coefficient is assumed to be constant during both the pumping period and the recovery period of the test. In practice, however, S probably varies and is apt to be greater during the pumping period than during the subsequent recovery (Jacob, 1963).

The value of S for a confined aquifer depends upon the elastic properties of the formation. If the aquifer is not perfectly elastic, it does not rebound vertically during recovery of water levels (recovery of pressure) at the same rate that it is compressed as a result of the drawdown during the preceding pumping.

During pumping from an unconfined aquifer, air occupies the voids in the sands within the cone of depression, because that part of the formation is actually dewatered. The volume of water drained per cubic foot of the formation is the value of S . When pumping is stopped, the rising water table may trap some of the air as bubbles in the

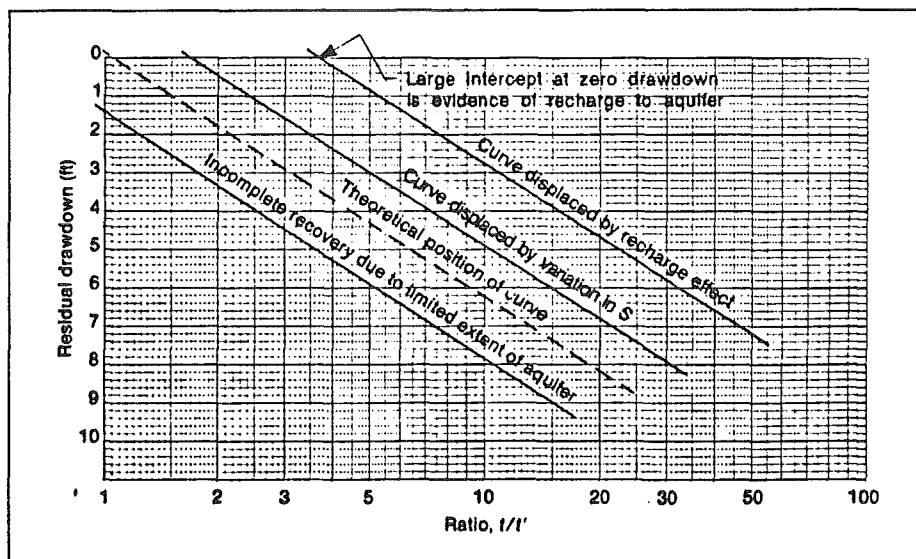


Figure 9.44. When real aquifer conditions differ from theoretical conditions, the residual-drawdown plot may be displaced in any of the three ways shown in this diagram.

voids of the sand. Thus, a slightly smaller volume of water will refill the dewatered portion of the formation, resulting in a correspondingly lower value of S during recovery. The effect of the variation in S on the residual-drawdown graph is shown in Figure 9.44. Note that this plot crosses the zero-drawdown line at a value of t/t' between 1 and 2.

Although analysis of residual-drawdown data is useful in interpreting constant-rate pumping tests, these data cannot be used to obtain a distance-drawdown plot. Readings in two or more observation wells taken at any instant during pumping can be plotted, as has been described, to illustrate the extent of the cone of depression. The nature of water-level recovery in the cone of depression causes the residual drawdown in observation wells away from the pumped well to equalize soon after pumping stops.

It may seem that much emphasis has been placed on variations in the value of S and the fact that S cannot always be calculated from the type of data available from a given pumping test. This does not imply, however, that these limitations invalidate the basic methods of aquifer analysis. Even a substantial error in S does not affect greatly the calculations of well performance and interference between wells.

If the storage coefficient cannot be calculated from the test data, other information from the test is still useful. An assumed storage coefficient can be used, based on whether a confined aquifer or unconfined aquifer is being considered; geologic information would indicate this in most cases. For a confined aquifer, an assumed value of $S = 5 \times 10^{-4}$ can be used; for an unconfined aquifer, an assumed value of $S = 0.1$ can be used. The calculations will be less accurate than in cases where the real values are known, but close enough for most situations.

Although most aquifers do not conform to all the theoretical conditions assumed by Theis and Jacob, results from the application of these equations and their graphic relationships have been quite satisfactory. In nonuniform aquifers, uneven hydraulic interconnections throughout the formations result in continuous adjustment of flow between local regions of differing hydraulic conductivity. The cone of depression then tends to deepen and spread in a way that reflects the overall transmissivity and storage characteristics of the aquifer. Because T and S are average values for large areas, some variations in yield at individual well sites should be anticipated when predicting future performance from aquifer tests.

THEIS NONEQUILIBRIUM WELL EQUATION

Considerable space has been devoted to applications of Jacob's modification of the more general Theis equation. In many cases it is not necessary to use the more laborious Theis method because the additional accuracy it provides is not significant. But in some cases the pumping time may be limited or the drawdown cannot be measured relatively near the well. Data collected when t is small or r is large do not plot on a straight line in Jacob's method. Therefore, in certain cases, the Theis method is necessary to obtain T and S .

Recall that the simplest form of the Theis equation (9.5) is:

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

$$s = \frac{1}{4\pi} \frac{Q}{T} W(u)$$

$W(u)$, the well function of u , is an abbreviation for the exponential integral:

$$\int_u^{\infty} \frac{e^{-x}}{x} dx = W(u) = -0.5772 - \log_e u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} + \dots \quad (9.17)$$

Equation 9.5a gave the value of u as:

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

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

where

r = distance, in ft, from the center of a pumped well to a point where the drawdown is measured

where

r = distance, in m, from the center of a pumped well to a point where the drawdown is measured

If the transmissivity and storage coefficients are known, values for these and other terms can be substituted in the equation to obtain an unknown.

For example, suppose the transmissivity of a confined aquifer is 50,000 gpd/ft (620 m²/day) and the storage coefficient is 5×10^{-4} , a typical value for confined conditions. What would be the specific capacity, Q/s , of a 12-in (305-mm) well after 1 day of continuous pumping? In this case, $r = 0.5$ ft (0.15 m) and $t = 1$ day. First, calculate u :

$$u = \frac{1.87 \cdot (0.5)^2 \cdot 5 \times 10^{-4}}{50,000 \cdot 1} = 4.7 \times 10^{-9}$$

$$u = \frac{(0.15)^2 \cdot 5 \times 10^{-4}}{4 \cdot 620 \cdot 1} = 4.5 \times 10^{-9}$$

Next, the value of $W(u)$ corresponding to this value of u is read from Appendix 9.E. $W(u)$ in this case is 18.60 (18.64). Rearranging Equation 9.5 gives:

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

$$= \frac{50,000}{114.6 \cdot 18.60}$$

$$= 23.5 \text{ gpm/ft of drawdown}$$

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

$$= \frac{4 \cdot 3.14 \cdot 620}{18.64}$$

$$= 418 \text{ m}^3/\text{day/m of drawdown}$$

Having calculated the specific capacity, the drawdown in the pumped well for any pumping rate is easily determined. If the pumping rate is 200 gpm (1,090 m³/day), then:

$$s = \frac{Q}{Q/s} = \frac{200}{23.5} = 8.5 \text{ ft}$$

$$s = \frac{Q}{Q/s} = \frac{1090}{418} = 2.6 \text{ m}$$

These calculations assume that the pumped well is 100-percent efficient. If the pumped well is discharging 200 gpm, what would be the drawdown 1,000 ft (305 m) from the pumped well after 1 day of pumping?

$$\begin{aligned}
 u &= \frac{1.87 r^2 s}{Tt} \\
 &= \frac{1.87 \cdot (1,000)^2 \cdot 5 \times 10^{-4}}{50,000 \cdot 1} \\
 &= 0.0187
 \end{aligned}$$

$$\begin{aligned}
 u &= \frac{r^2 s}{4Tt} \\
 &= \frac{(305)^2 \cdot 5 \times 10^{-4}}{4 \cdot 620 \cdot 1} \\
 &= 0.0188
 \end{aligned}$$

For this value of u , Appendix 9.E shows that $W(u) = 3.46$. Substituting this value in Equation 9.5 gives:

$$\begin{aligned}
 s &= \frac{114.6 Q W(u)}{T} \\
 &= \frac{114.6 \cdot 200 \cdot 3.46}{50,000} \\
 &= 1.59 \text{ ft}
 \end{aligned}$$

$$\begin{aligned}
 s &= \frac{1}{4\pi} \frac{Q}{T} W(u) \\
 &= \frac{1,090 \cdot 3.46}{4 \cdot 3.14 \cdot 620} \\
 &= 0.48 \text{ m}
 \end{aligned}$$

The calculated drawdown is independent of the efficiency of the pumped well.

Use of the Theis equation to determine T and S from a field pumping test requires measurements of drawdown in at least one observation well. These should be made at proper intervals after the pumping starts; measurements in more than one observation well also can be used. A direct calculation from Equations 9.5 and 9.17 is not possible.

Theis worked out a graphical solution which yields T and S if values of other terms are known. The method involves matching a curve plotted from specific pumping test data with a type curve. The type curve is prepared by plotting values of $W(u)$ against $1/u$ on graph paper with logarithmic scales, as shown in Figure 9.45. Values from Appendix 9.E are used for this. Data from a pumping test are plotted on similar graph paper with logarithmic scales and cycles identical to those of the type curve.

Table 9.1 (page 221) shows data from a pumping test. Drawdown was measured at frequent intervals in an observation well 400 ft (122 m) from a well that was pumped at a constant rate of 500 gpm (2,730 m³/day).

Figure 9.46 shows these data plotted on regular graph paper. The curve shows how drawdown increases rapidly at first, with the rate of increase diminishing as time progresses so that the water level would appear to stabilize with continued pumping. Because the measurements show that the water level was still dropping after 240 minutes of pumping, the equilibrium equation (9.2) cannot be applied. The test data in this case can be analyzed using the Theis nonequilibrium equation.

The test data may be plotted conveniently in any of the following ways:

Method	Vertical log scale	Horizontal log scale
(a)	Drawdown, s	t
(b)	Drawdown, s	t/r^2
(c)	Drawdown, s	$1/r^2$

Method (c) is restricted to cases where data are obtained from three or more obser-

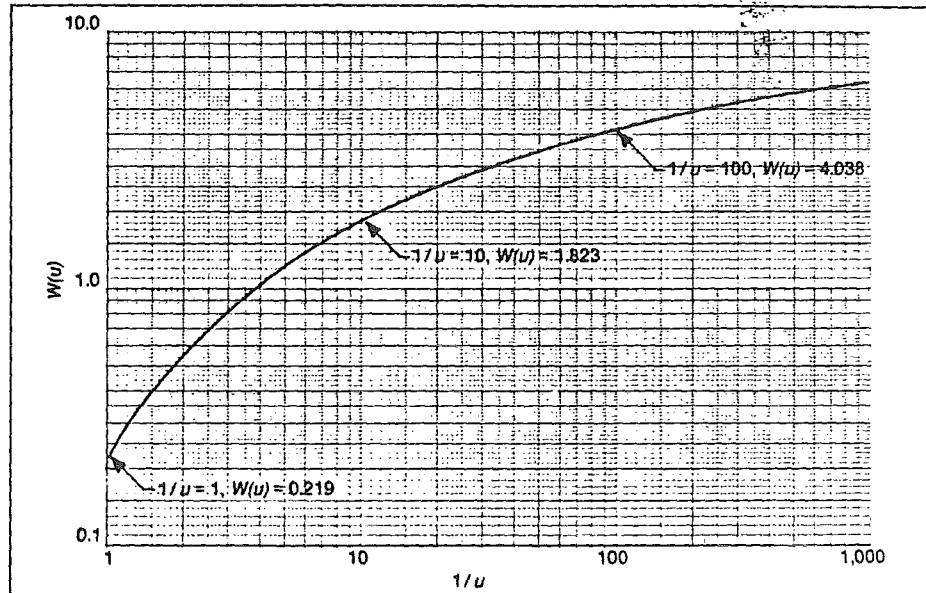


Figure 9.45. Type curve for graphic solution of Theis nonequilibrium equation shows values of $W(u)$, well function of u , corresponding to values of $1/u$. Curve is plotted on logarithmic graph paper.

vation wells during the test.

Use of the nonequilibrium equation requires that the data be plotted on logarithmic graph paper as shown in Figure 9.47. Plotting by method (a) is used here; drawdown

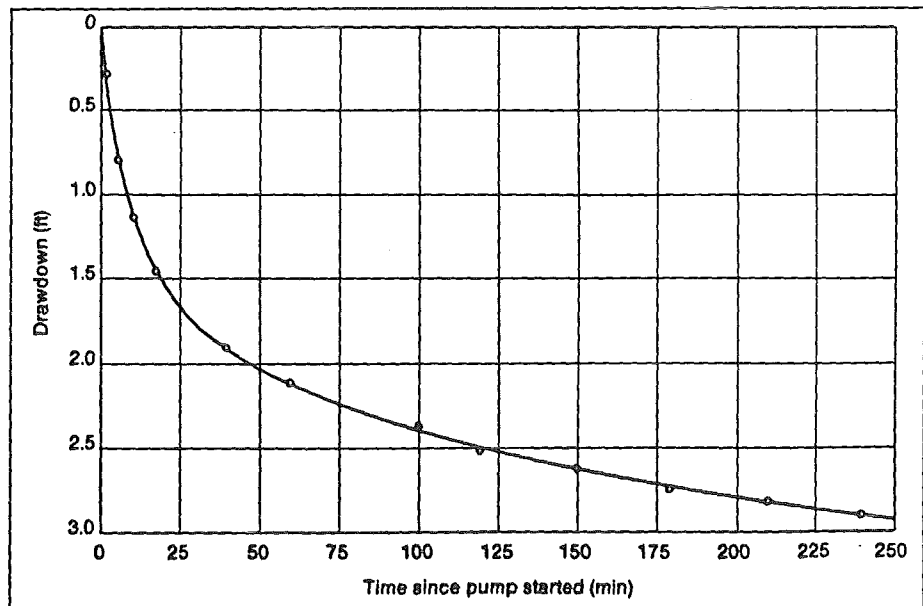


Figure 9.46. Data given in Table 9.1, plotted on ordinary graph paper, show how drawdown varies with duration of pumping. Water level drops rapidly at first, but rate of lowering decreases as pumping continues at 500 gpm (2,730 m³/day). Proper analysis of the test data requires use of the Theis nonequilibrium concept.

is on the vertical axis and the time since pumping began is on the horizontal axis. This graph is then superimposed on the type-curve sheet so the plotted points fall on or fit some portion of the type curve. In finding the position of best fit, the axes of both graphs must be kept parallel.

Once a good matching position is found, a match point is selected. The match point can be any convenient point on the graphs (that is, where u equals 1, 10, or 100, or s equals a whole number), but often the point is selected in the center of the area of best overlap as shown in Figure 9.48. The match point in Figure 9.48 is shown on the type curve where $1/u$ equals 100, and $W(u)$ equals 4.038. At the corresponding point on the time-drawdown diagram, s equals 2.3 ft (0.7 m) and t equals 83 minutes (0.058 days).

Substituting in Equation 9.5, we have:

$$\begin{aligned} T &= \frac{114.6 Q W(u)}{s} \\ &= \frac{114.6 \cdot 500 \cdot 4.038}{2.3} \\ &= 101,000 \text{ gpd/ft} \end{aligned}$$

$$\begin{aligned} T &= \frac{1}{4\pi} \frac{Q}{s} W(u) \\ &= \frac{1}{4 \cdot 3.14} \frac{2,730}{0.7} \cdot 4.038 \\ &= 1,250 \text{ m}^2/\text{day} \end{aligned}$$

After determining T , we can calculate S from the following relationship:

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

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

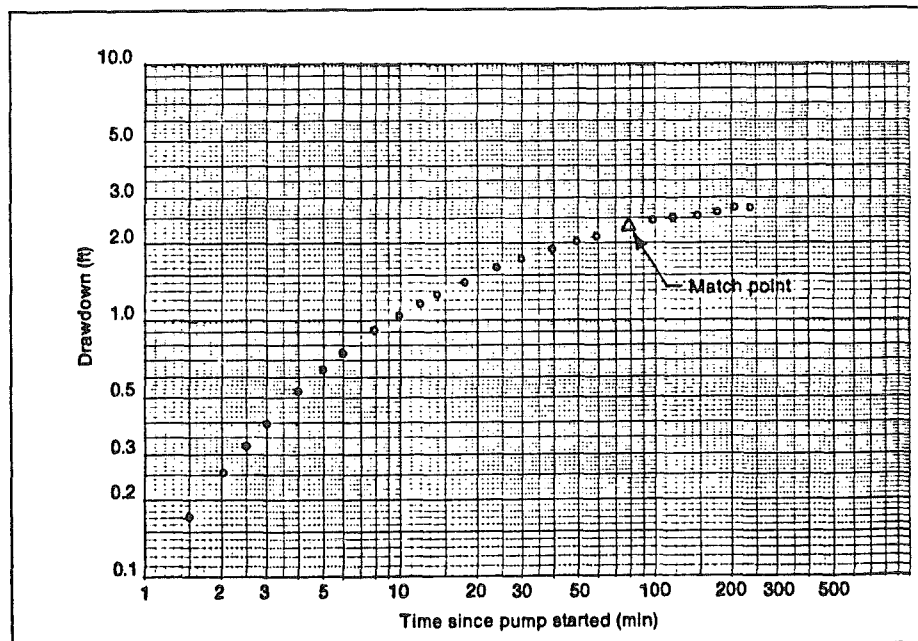


Figure 9.47. Data given in Table 9.1, plotted on logarithmic graph paper, define a curve similar in shape to the type curve in Figure 9.45.

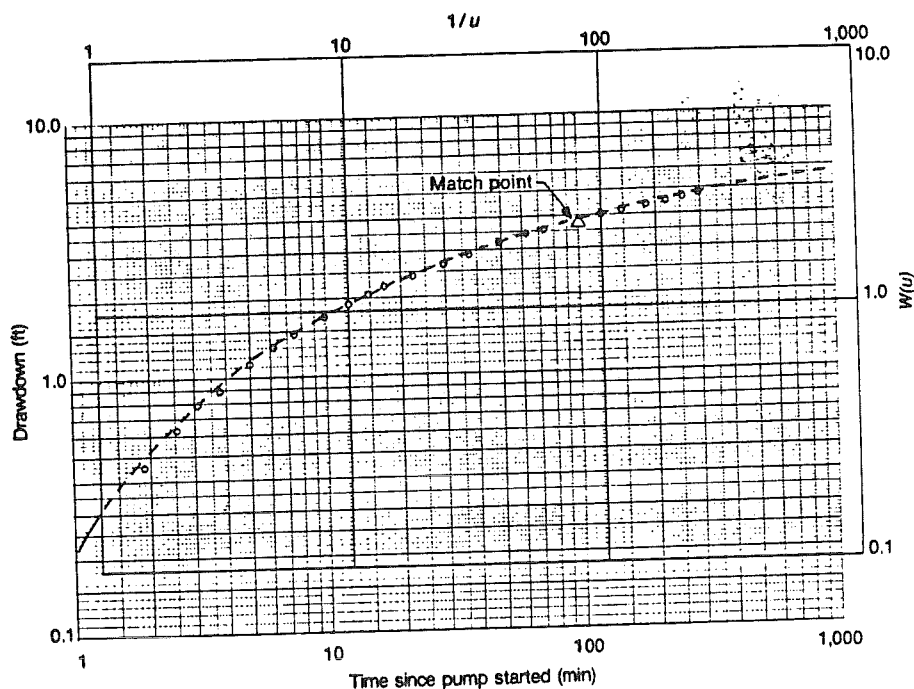


Figure 9.48. Diagram of plotted points representing pumping test data superimposed on the type curve. Match point chosen for $1/u = 100$.

The value of r in this example is 400 ft (122 m), the distance from the pumped well to the observation well. Thus:

$$S = \frac{1 \cdot 101,000 \cdot 0.058}{100 \cdot 1.87 \cdot (400)^2} = 1.9 \times 10^{-4}$$

$$S = \frac{4 \cdot 1 \cdot 1,250 \cdot 0.058}{100 \cdot (122)^2} = 1.9 \times 10^{-4}$$

Brown (1953) presents detailed explanations of other ways to solve the nonequilibrium equation by the curve-matching technique.

Under certain hydrogeologic conditions, pumping test data may deviate from the type curve. For example, Figure 9.49 shows that recharge and negative boundary effects can alter the pumping test data sufficiently to cause a distinct deviation from the type curve. Any variance from the type curve indicates that one or more of the assumptions upon which the Theis method is based have been violated. Leakage, casing storage, and mechanical difficulties encountered in the pumping test also can lead to data that do not follow the type curve exactly.

OTHER METHODS OF AQUIFER ANALYSIS

In recent years, many other analytical models (in addition to the Theis and Jacob methods) have been developed to aid in aquifer analysis. Analytical models, as presented in this chapter, are used to solve problems relating to small parts of a total aquifer system. Numerical computer models, on the other hand, are utilized to describe what is happening in the entire aquifer system. Although extremely useful to

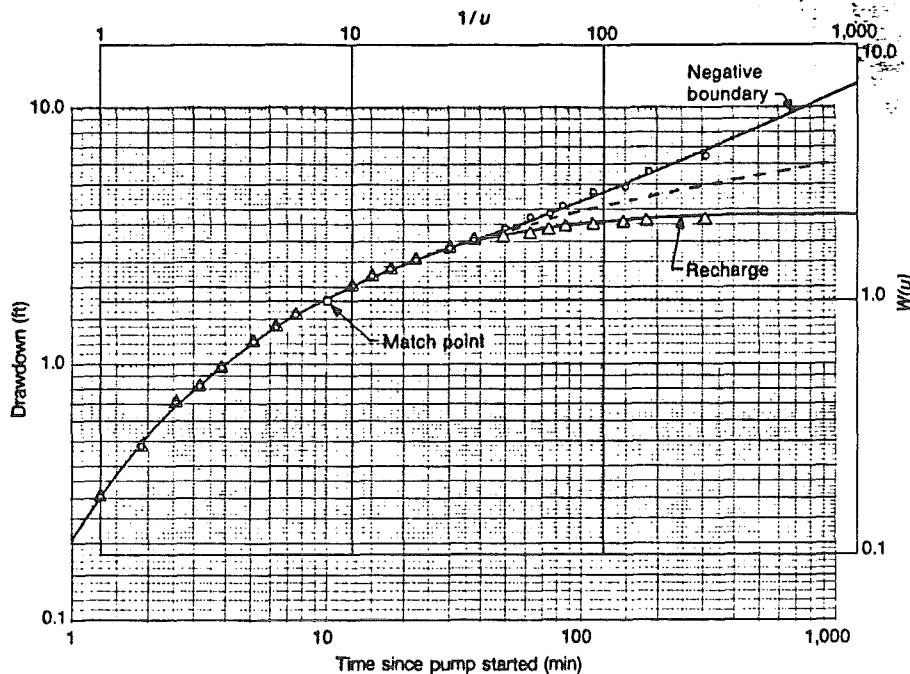


Figure 9.49. Effect of recharge and negative boundaries on pumping test data when using the Theis curve-matching method. In this instance, both boundaries are the same distance from the pumping well.

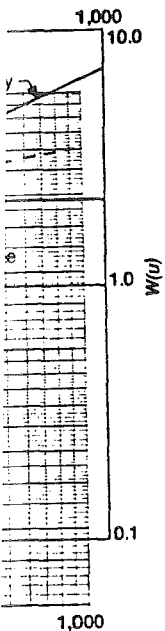
environmental planners, computer models are rarely of value to engineers in solving site-specific problems, as in the case of a single well. If computer models are based on a large and reliable data base, however, they can predict the response of an aquifer to large-scale (regional) pumping demands.

Analytical models will continue to be one of the most versatile methods for typical design engineers or well drilling contractors to analyze what is happening at specific sites. Generally, the models describe flow conditions in aquifers under more or less standard or ideal conditions. That is, the aquifer is (1) homogeneous, (2) infinite in areal extent, or has easily described geometric limits in the form of barriers or recharge boundaries, (3) uniform in thickness, (4) either isotropic or anisotropic, and (5) partially or fully penetrated by wells.

The models listed in Appendix 9.F describe nonequilibrium, time-drawdown, and distance-drawdown relationships in nonleaky, leaky, and unconfined aquifer systems. Use of these models where applicable may yield more accurate information than can be obtained by either the Theis or Jacob methods. The equations, however, may be somewhat more difficult to use. Long experience shows that judicious use of the Jacob method generally yields data of sufficient accuracy for engineering purposes, although greater accuracy may be obtained with one or more of the sophisticated models noted in Appendix 9.F.

SUMMARY

Three mathematical methods commonly used to calculate the principal aquifer characteristics, T and S , have been discussed in this chapter. Knowledge of these two



using the Theis curve
for a well.

Engineers in solving
models are based
on the response of an aquifer

Methods for typical
pumping at specific
under more or less
us, (2) infinite in
carriers or recharge
topic, and (5) par-

drawdown, and
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formation than can
however, may be
as use of the Jacob
purposes, although
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principal aquifer
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aquifer parameters permits calculation of specific capacity, potential yield, drawdown at any point away from a well, and well efficiency. In some cases, however, drawdown data from pumping tests are difficult to analyze, and there may be a high degree of uncertainty regarding the actual values of T and S , and hence the real hydraulic potential of the aquifer. The extreme heterogeneity of many geologic formations leads to data that seem to defy rational explanation. Earlier chapters described in detail the formation of various types of aquifers. Recollection of this information is vital in analyzing pumping test data, for it allows the engineer to understand anomalies in the data which occur so often. Chapter 16 presents numerous examples of pumping tests conducted in various geologic media. Combining geologic information with a thorough understanding of well hydraulic theory is essential when analyzing drawdown data for T and S .

tests be terminated prematurely, however, because the limited data collected may not reveal the true nature of the aquifer.

It is a good idea to plot preliminary drawdown data during the course of the pumping test. Anomalies in the data will become apparent, and the necessary adjustments can be made so that the remaining data will be more useful. Plotting the data will also indicate when equilibrium conditions have been reached for shallow aquifers or for aquifers with high hydraulic conductivities. In this case, the pumping portion of the test can be shortened with no loss of sensitive data.

Barometric or tidal changes can influence drawdown data. For example, a barometric pressure change of 1 in (25.4 mm) of mercury 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 aquifer's ability to transmit changes in atmospheric pressure. Record the nature and time of any weather changes on the drawdown data sheet. Unusually high or low oceanic tides can also affect drawdown data in wells near coastlines.

Recovery Data

Whenever possible, recovery data should be taken to verify the accuracy of pumping data. Often, the recovery data will be more reliable because no pumping is required and any previously inexperienced personnel will have learned proper measurement techniques by the time recovery data can be taken. Recovery measurements should be recorded with the same frequency as those taken during the pumping portion of the aquifer test.

WELL EFFICIENCY

In Chapter 9, the term "well efficiency" was defined and the two major factors influencing efficiency were discussed. For many years prior to the escalation of energy costs in the 1970's, well efficiency was not recognized as a serious well construction objective. Only those contractors who had to deliver the maximum volume of water from limited aquifers were interested.

In recent years, the cost of pumping water has risen precipitately, not only because energy costs have increased, but also because declining water tables in major aquifers have added to lift requirements. At 1984 energy costs, and assuming average efficiency for a pump running 33 percent of the time, the extra fuel expense for lifting 1,000 gpm (5,450 m³/day) an additional 10 ft (3 m) amounts to approximately \$1,000 per year. Thus, reducing drawdown should be a major consideration when designing and constructing all high-capacity wells.

The amount of drawdown required to produce a particular yield is determined by the hydraulic nature of the aquifer and the care with which the well was designed, constructed, and developed. Drawdown caused by friction losses in the aquifer as water flows to a well is unavoidable. But substantial head losses sustained as water passes through the disturbed zone around the borehole are avoidable. They are caused by drilling fluid left in the formation, damage to the formation caused by drilling, the presence of a poorly designed filter pack, or use of a well screen with limited open area. Good design practices and enlightened drilling methods can insure that head losses through the zone near the well bore will be minimal. Screens with maximum inlet area, surrounded by a suitable filter pack, and in turn surrounded by a formation developed properly to remove drilling fluid and fine material are necessary for min-

imizing head losses.

Mogg (1968) defined well efficiency as the ratio of the actual specific capacity at the designed well yield after 24 hours of continuous pumping, to the maximum specific capacity possible, calculated from formation characteristics and well geometry. In this method of defining efficiency, it is possible to identify how much of the total head loss is attributable to natural losses in the formation and those caused by well construction damage to the aquifer and by installation of a screen and filter pack.

The procedure for calculating well efficiency is as follows:

1. Graph the time-drawdown data.
2. Calculate Δs .
3. At a particular time, note the drawdown in an observation well.
4. On a distance-drawdown graph, plot the drawdown for the observation well (for the particular time) at the proper distance from the pumping well.
5. Complete the drawdown curve by using a slope of 2 times Δs (in the Jacob equation, $\log r^2 = 2 \log r$, so the value of Δs in the distance-drawdown graph is twice the value of Δs in the time-drawdown graph).
6. Extend the slope of the data to the radius of the well.
7. In a 100-percent efficient well, drawdown just outside the borehole should equal the drawdown inside the well. It is more likely, however, that the water level inside the well is lower. Therefore, efficiency equals drawdown outside the borehole divided by drawdown inside the casing, times 100.
8. An efficiency of 70 to 80 percent is usually obtainable if good design, construction, and development practices are followed.

An example of the efficiency calculation is shown in Figure 16.14.

STEP-DRAWDOWN TESTS

All conventional well hydraulics theory is based on the assumption that laminar flow conditions exist in the aquifer during pumping. If the flow is laminar, drawdown

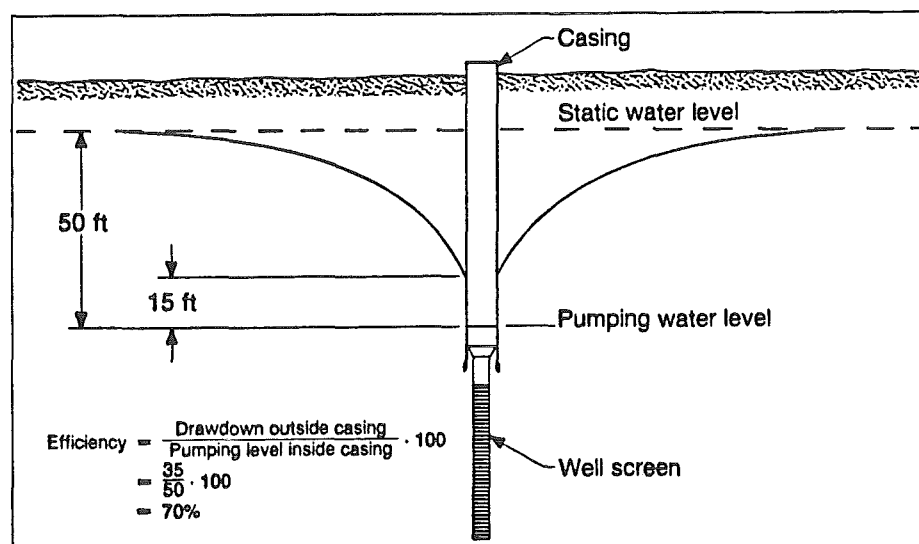


Figure 16.14. Calculating true well efficiency for a high-capacity well.

This figure is far less than the 69-percent value given for L_p . This means that much of the head loss caused by inefficiency is associated with laminar flow rather than with turbulent flow. Had L_p been used for the well efficiency, the well would have been judged to have a reasonably good efficiency value of 69 percent, whereas the true well efficiency is quite poor.

In summary, the step-drawdown test can be used to determine the following:

1. The 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, may not be sufficient to encounter boundary effects. Therefore, the actual long-term specific capacity at a certain discharge may be different than originally calculated from the step-drawdown data. Slow drainage can also distort the discharge data in highly stratified aquifers. Depending on the exact nature of the aquifer, the specific capacity may seem to improve with higher discharges and longer pumping time — a highly unlikely situation that will occur rarely, if ever, in natural geologic materials.

2. A ratio, L_p , denoting the percentage of the total head loss attributable to laminar flow, which is not to be confused with the true hydraulic efficiency of the well.

3. 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.

PROBLEMS OF PUMPING TEST ANALYSIS

Constant-rate and step-drawdown pumping tests offer the most powerful method for analyzing the hydrogeologic character of an aquifer. Measurements made during

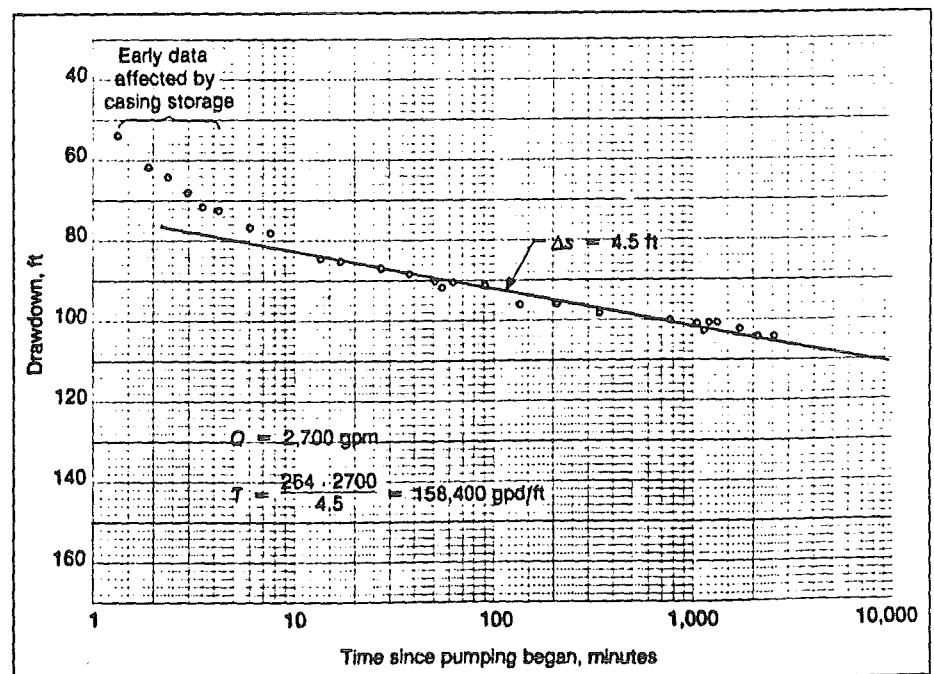


Figure 16.16. Time-drawdown plot for a well discharging 2,700 gpm (14,700 m³/day).

an aquifer test provide values for some of the terms in the Theis equation, permitting calculation of the transmissivity and storage capacity of the aquifer. (See Chapter 9 for a full description of plotting procedures.) The hydraulic conductivity values determined from pumping test data are far more accurate than hydraulic conductivities calculated on the basis of field samples tested in a laboratory, because the samples are rarely representative of the undisturbed formation. The aquifer test measures the performance of the aquifer in its natural state.

In addition to determining hydraulic conductivity, transmissivity, and coefficient of storage, pumping test data can be used to determine any interference between wells at various spacings and rates of pumping other than those employed in the test itself. Under some conditions, pumping test data will also indicate what drawdown may be expected from long-term pumping at different discharge rates, the existence of impervious boundaries that limit the extent of the aquifer, and the existence of recharge sources which may not be apparent otherwise.

In many cases, pumping test data can be interpreted in more than one way. Even in the best controlled test, the data may be confusing unless all hydraulic and geologic factors are taken into account. The analyst must be able to identify unreliable data so that the calculated values for transmissivity and storage coefficient will correctly predict aquifer performance.

The pumping test analyst can be faced with a large number of anomalies in the data, but in the discussions below it is assumed that the following physical conditions for a proper test have been met:

1. The diameter of the pumped well is large enough to accommodate the test pump and provide enough clearance so that the water level can be measured with an air line, electric or acoustic probe, or other device.
2. The test pump can deliver at least 10 percent of the well's maximum yield.
3. In unconsolidated formations, the well screen is at least one-third the thickness of the aquifer, except in extremely thin strata where three-fourths of the formation should be screened.

Although either a constant-rate or a step-drawdown test could have been conducted in the instances discussed below, most of the actual pumping tests are constant-rate tests because the values obtained for transmissivity, storage coefficient, and efficiency are generally more accurate, and different trends in the data are more readily discernible.

The pumping test examples cited below will serve to illustrate some of the major problems that can occur in data interpretation. It is assumed that the reader has thoroughly understood constant-rate pumping test analysis as discussed in Chapter 9. The analyst must also keep in mind several other factors during data interpretation. These include the effect of

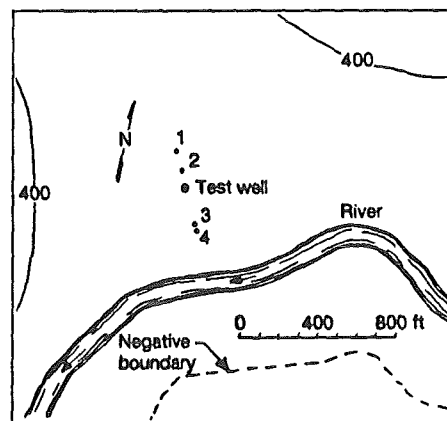


Figure 16.17. Arrangement of 12-in (305-mm) test well and four observation wells for a 72-hour pumping test. This test was made to get sufficient information for designing a well field for a total supply of 2,000 gpm (10,900 m³/day), with wells sized and spaced for best economy and minimum interference.

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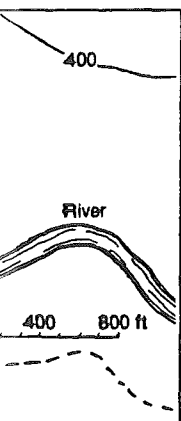
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the hydrogeologic character of the aquifer on the data, errors in data acquisition, potential omission of important environmental impacts, well design and construction practices, pumping rates, and the theoretical limits of the Jacob theory.

Example 1 — Test Conducted Near a River

In this case, the floodplain of a small river was being considered as the site for an industrial plant (Figure 16.17). A generalized cross section of the two aquifers in this area, and the positions of the various observation wells, are shown in Figure 16.18.

For the aquifer test, a 12-in (305-mm) well was drilled about 400 ft (122 m) from the bank of the river. Both aquifers were penetrated — an unconfined aquifer extending to a depth of about 46 ft (14 m), and a deeper confined aquifer extending from 50 to 105 ft (15.2 to 32 m). A 4-ft (1.2-m) clay layer, from 46 to 50 ft (14 to 15.2 m), separated the aquifers. It was assumed that the 4-ft clay layer effectively isolated the two aquifers. The static water level, however, in both aquifers was 5 ft (1.5 m) below ground level. The arrangement of observation wells for this pumping test illustrates some of the points discussed earlier in this chapter.

Three 2-in (51-mm) observation wells about 90 ft (27.4 m) deep were put down at distances of 100 ft (30.5 m) and 200 ft (61 m) from the 12-in (305-mm) well. Two of these wells were placed away from the river compared with the location of the test well, whereas one well was placed between the river and the test well. A shallow observation well, No. 4, was installed near observation well No. 3 for measuring any water-level changes that might occur in the upper aquifer. This was done primarily to check for any hydraulic connection between the two aquifers, because the continuity of the clay separation was unknown. Any water-level change in the shallow well caused by the pumping of the test well would indicate discontinuity in the clay layer.

The pumping test was conducted for 72 hours. During the early stages of the pumping test, a heavy rainstorm occurred in the upper reaches of the watershed, causing the river level to rise suddenly about 2 ft (0.6 m). No other unusual event took place

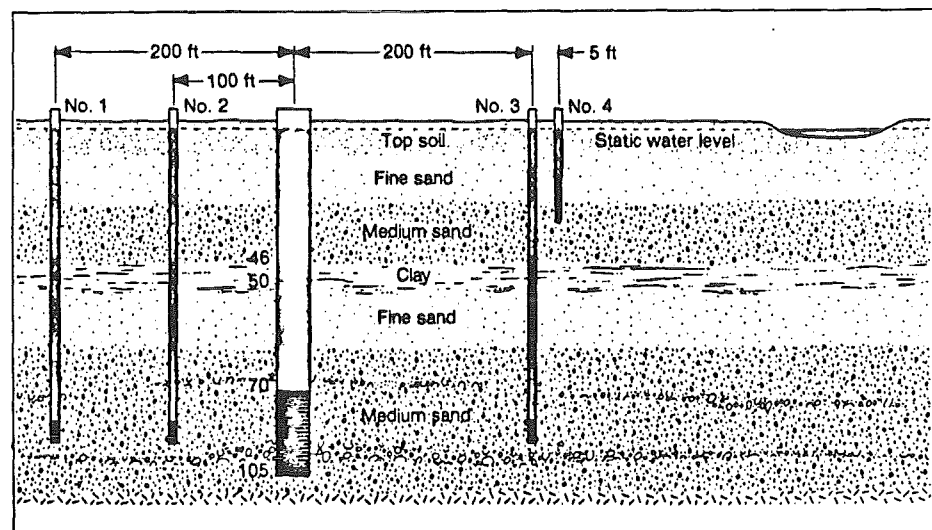


Figure 16.18. Geologic section on a line through test well and observation wells shown in Figure 16.17. The clay layer apparently separates the upper unconfined aquifer from the confined aquifer.

during the remainder of the pumping test.

The consulting engineer was asked to determine transmissivity and storage coefficient values for the aquifer. The consultant decided to use the drawdown data from observation well No. 3 to calculate the transmissivity. Evaluation of the data by the Jacob method indicated that five different slopes were evident in the time-drawdown graph (Figure 16.19). Because the engineer was required, under terms of the contract, to guarantee a certain pumping well efficiency, selection of the correct transmissivity value was important.

Interpretation of Data

After reviewing the data, the consultant reasoned correctly that:

1. Data taken during the first 10 minutes are not valid when using the Jacob technique. (Recall that u in the Theis equation must be less than 0.05 for the Jacob method to be valid.)
2. Slope #1 represents the true aquifer characteristics because the cone of depression had not yet been affected by the subsequent rise in river level or other boundary. If this value of Δs were used in the Jacob equation for transmissivity, the well efficiency could be calculated correctly.
3. Slope #2 is caused when the cone of depression intercepts the river, thereby producing a recharge effect on the drawdown data. Transmissivity values calculated from this Δs value would be too high.
4. Slope #3 represents the sudden rise of the river level, and causes an additional small, but temporary, recharge effect on the data. Transmissivity values calculated on the basis of this part of the curve would also be too high.
5. Slope #4 represents the negative boundary effect as the southern side of the cone

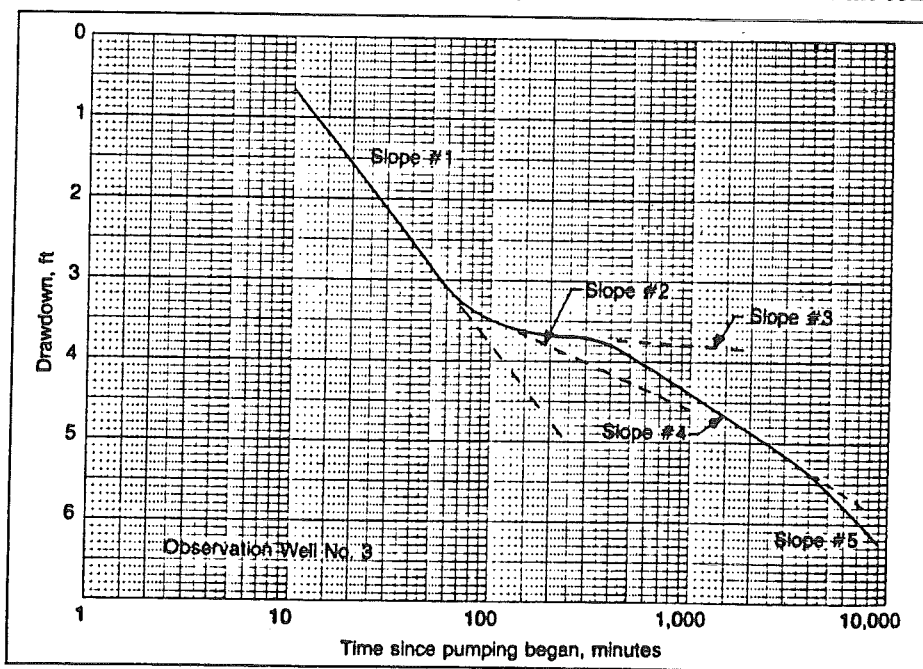


Figure 16.19. Time-drawdown graph for observation well No. 3 showing five different slopes.

of depression intercepts the impermeable rocks of the valley wall.

6. Slope #5 represents the reduction in river level to its pre-rainfall stage. This transmissivity value is still higher than the transmissivity calculated from Slope 1 because the recharge effect of the river is slightly greater than the effect of the negative boundary.

Determination of accurate transmissivity and storage coefficient values from pumping tests will depend not only on how well the test itself has been carried out but also on the analyst's ability to interpret the results. The drawdown data are strongly affected by the hydrogeological character of the aquifer, and the analyst must understand the nature of the aquifer before the data make sense. For example, the pumping test data for the shallow well (4) exhibit the same drawdown characteristics as for the nearby deeper well (3). Thus, the areal extent of the clay layer must be limited in comparison with the extent of the cone of depression, although the clay layer appears to be a major hydraulic factor in the geologic cross section shown in Figure 16.18. In floodplains, clay layers often result from the infilling of old, now-buried oxbow lakes by flooding events; the line of observation wells was, by chance, drilled along this former channel. Because the clay layer is limited (only as wide as the old channel), it does not inhibit recharge effects on the deeper observation and test wells caused by the sudden rise in river level.

The same drawdown trends that occur in well 3 will also occur in wells 1 and 2. But breaks in the drawdown slopes caused by recharge or boundary conditions would reach observation wells at times related to their distances from the river or boundary. For example, the observation well closest to the river would show the recharge effect first.

Example 2 — Pumping Test in Glacial Outwash

Pumping test data taken from wells in aquifers deposited as a result of melting glacier ice often show curious anomalies that at first glance seem to defy rigorous analysis. But experience suggests that each curve indicates important physical characteristics of the aquifer, as this example will demonstrate.

A 6-in (152-mm) test well was installed in glacial outwash (fine glacial gravel) for the city of Brewster, Minnesota. During a 24-hour pumping test, the pumping rate was maintained at 193 gpm (1,050 m³/day). Drawdown data are presented in Figure 16.20. The part of the graph that best represents the true transmissivity and storage coefficient of the aquifer material is not immediately clear, and the analyst is forced to draw on hydrogeological information to determine which part of the curve should be used.

Interpretation of Data

A quick appraisal of the graph suggests that a negative boundary was encountered at about 100 minutes into the pumping test. The nature of this boundary may not be immediately apparent unless the analyst recalls the way in which glacial outwash is deposited (see Chapter 3). At the glacier terminus, meltwater rushes through loosely consolidated glacial till deposits and entrains a full load of sediment. Upon reaching the land surface beyond the glacier front, which normally has a lower gradient, much of the bedload is deposited in topographically low areas, whereas finer, suspended material moves downriver. The areal extent of a certain lens of outwash is usually

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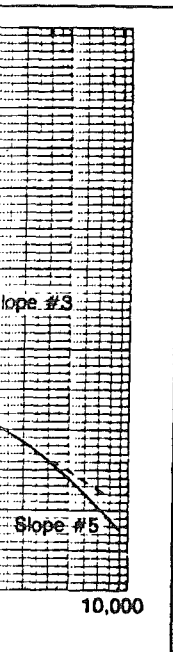
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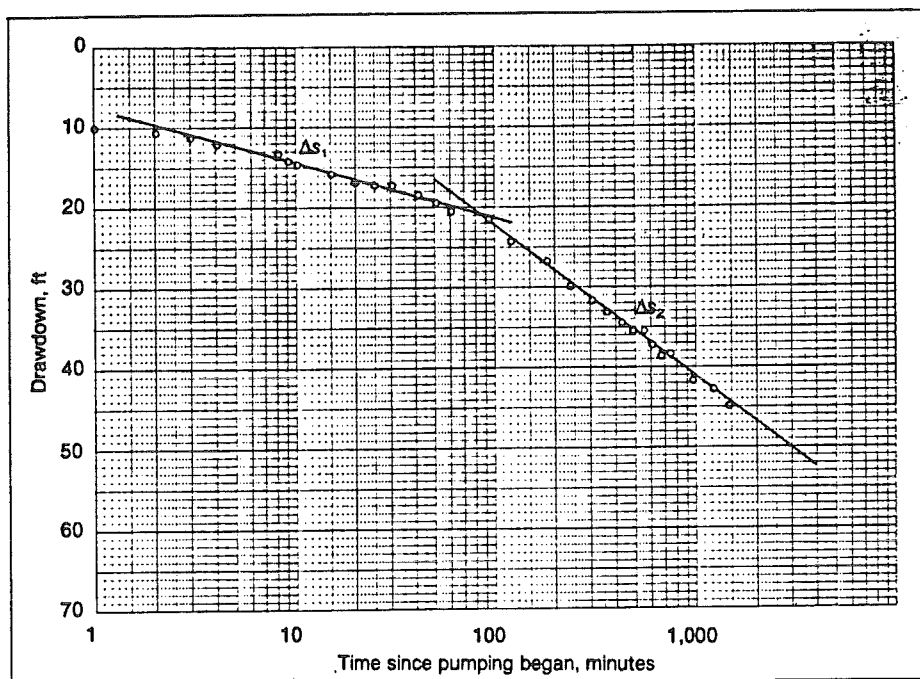


Figure 16.20. Drawdown data for 6-in (152-mm) test well in Brewster, Minnesota.

limited vertically or horizontally because of rapid changes in the stream's direction or ability to carry a certain size bedload. Although the entire outwash deposit may be quite large, the limited extent of a particular sand or gravel lens may have a large effect on drawdown during a pumping test.

During the spread of the cone of depression in the sediment in which the well has been drilled, the original slope (Δs_1) is maintained. At about 100 minutes, however, the cone spreads into finer materials that have a lower transmissivity value (Δs_2). In other words, the gravel has "pinched out" in one or more directions away from the well, and the finer sand of the surrounding material now controls flow through that portion of the aquifer and thus lowers overall transmissivity. A cross section through the well shows a thinning of the lens (Figure 16.21). This type of drawdown curve often occurs in pumping test data from glacial outwash deposits.

Knowledge of how geologic processes form aquifers is vital in analyzing pumping test information. A thorough understanding of the mathematical basis for equations used in pumping test analysis is usually not enough because the analyst must be able to visualize how the physical nature of the aquifer can produce hydraulic anomalies in the pumping test data. Alluvial deposits, for example, are usually highly elongate in character, so any pumping test reveals a negative boundary as the cone of depression encounters the valley walls. Depending on the extent and grain-size distribution of the alluvial material and on the pumping rate, the effect of the valley walls may affect the data only minutes into the test. Glacial outwash deposits, as illustrated above, are almost always deposited erratically, and the topography of the pre-existing land surface plays a major role in defining the thickness and lateral extent of the deposit. Significant variations in thickness can lead to both positive (apparently recharge) and

negative boundaries. Identification of the drawdown trend before these boundaries are encountered is important in predicting aquifer performance.

From this example, it may seem to many analysts that slope 2 may yield better results for predicting radius of influence, drawdown after a certain time, and potential yield. But in practice, the hydraulic characteristics of the aquifer will be indicated more accurately by using the Δs_1 value. It is clear, however, that the most realistic Δs value will fall somewhere between the two Δs values. Normally, however, the error induced by using Δs_1 will be less than by using Δs_2 . This point should always be kept in mind when the initial data are affected by boundaries and a prediction of aquifer performance must be made.

Example 3 — Pumping Test in Alluvial Deposits

The first two examples show that aberrations in data can be caused by variations in the geologic character of the aquifer materials created in the depositional process. Pumping test data can also be affected by the way the well and pumping equipment are constructed and installed.

In this example, an 8-in (203-mm) well, drilled in deep, fine-to-coarse alluvium, was test pumped at 5.2 gpm (28.3 m³/day). No lakes or rivers occur within the potential radius of influence. The time-drawdown data are plotted in Figure 16.22.

Interpretation of Data

A first hypothesis might suggest that a recharge boundary was encountered after approximately 80 minutes of pumping, but no recharge sources are known to exist. Another hypothesis might be that the pumping rate decreased; it did not.

What is unusual about this pumping test is the relatively low specific capacity. In pumping tests where the specific capacity is low in comparison to the size of the casing, the analyst should anticipate that the data may exhibit casing storage effects (discussed in Chapter 9). Recall that one of the basic assumptions made in deriving Theis's, and subsequently Jacob's, equation is that all of the water pumped from a well during a pumping test comes from the aquifer and none from storage within the well. Papadopoulos and Cooper (1967) presented an equation describing the discharge from a pumping well which takes into account the volume of water removed from casing storage when the well is efficient. Drawdown values calculated from this equation differ significantly from the Theis and Jacob equations during the early portion of the pumping test, when a relatively high percentage of the discharge comes from casing storage. During the later stages of the pumping test, when only a negligible

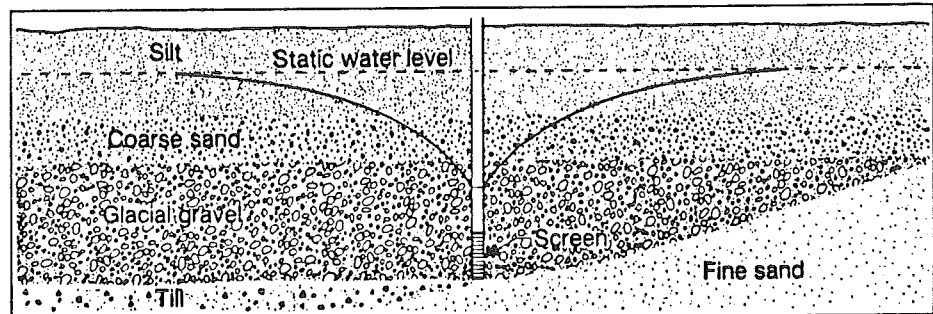


Figure 16.21. Cross section of well showing the thinning gravel deposit.

can alter water quality in local aquifer systems. The type of soils and their ability to adsorb contaminants, how far the landfill is situated above the water table or confined aquifer, and the hydraulic properties of the aquifer contribute to or reduce contaminant concentrations. Two typical geologic settings are presented in Figures 21.1 and 21.2 that demonstrate the interaction between the landfill leachates and the local hydrogeology. Other examples are given in Appendix 21.A. These illustrations are instructive in showing the potential flow paths taken by contaminants.

Drilling contractors and engineers should become familiar with potential contaminant flow paths in any waste project area, because regulations require that most monitoring wells be placed downgradient from the contaminant source. Initially, an estimate of the dimensions of the plume must be made on the basis of assumed hydraulic conductivity, porosity, and dispersion values for the aquifer. Any boundary conditions such as faults or changes in rock type must also be considered. It is important to recognize that some contaminants will become soluble in water, whereas others such as hydrocarbons will float on the surface of the groundwater. The density of a soluble

Table 21.3. Representative Ranges for Various Inorganic Constituents in Leachate from Sanitary Landfills

Parameter	Representative Range (mg/l)
Potassium (K^+)	200 - 1,000
Sodium (Na^+)	200 - 1,200
Calcium (Ca^{+2})	100 - 3,000
Magnesium (Mg^+)	100 - 1,500
Chloride (Cl^-)	300 - 3,000
Sulfate (SO_4^{-2})	10 - 1,000
Alkalinity	500 - 10,000
Iron (Fe) (total)	1 - 1,000
Manganese (Mn)	0.01 - 100
Copper (Cu)	< 10
Nickel (Ni)	0.01 - 1
Zinc (Zn)	0.1 - 100
Lead (Pb)	< 5
Mercury (Hg)	< 0.2
Nitrate (NO_3^-)	0.1 - 10
Ammonia (NH_4^+)	10 - 1,000
Phosphorus (P) as phosphate (PO_4)	1 - 1,000
Organic nitrogen	10 - 1,000
Total dissolved organic carbon	200 - 30,000
COD (chemical oxidation demand)	1,000 - 90,000
Total dissolved solids	5,000 - 40,000
pH	4 - 8

(Leckie et al., 1975; Griffin et al., 1976)

water/salt-water interface is 40 times the distance between the surface of the water table and the mean sea level. If this distance (h_f) is 4 ft (1.2 m), the interface is 160 ft (48.8 m) below the water table. Reduction of this 4-ft head during pumping causes the interface to rise significantly. For example, if the drawdown is 3 ft (0.9 m), h_f is 1 ft (0.3 m), and the interface would be only about 40 ft below the water table.

Although Equation 22.15 will give an approximate depth to the salt-water interface, two other factors may affect the actual distance. In many situations, the boundary between fresh and salt water is a diffused layer of mixed water that may alter the exact location of the interface. Flow of groundwater toward the sea also affects the position of the interface, as shown in Figure 22.29. When flow occurs (as it normally does), the interface is lower than Equation 22.15 suggests.

Maintenance of Infiltration Galleries

Maintenance of infiltration galleries may be difficult, depending on the configuration of the screen and where it is placed. Therefore, it is important to observe several operational criteria that will reduce or eliminate maintenance problems. These include the following:

1. Never exceed the designed pumping rate. Higher pumping rates may cause fine sediment to enter the filter pack, reducing its permeability. Eventually, sand may enter the screens and block part of the intake openings, causing even more sand pumping.

2. Do not let the screens become aerated, because iron bacteria may cause severe incrustation problems. Near-surface waters are often high in iron, and the rapid growth of iron-ingesting bacteria can clog the gallery. Inorganic deposits of magnesium, calcium, and other ions may also form.

3. Do not let the gallery go unused for long periods. Inactivity tends to lower the hydraulic conductivity of the filter pack and surrounding materials.

If maintenance is necessary, it usually involves backwashing and chemical treatment. For galleries installed beneath a water body, backwashing with 4 to 10 gpm of water per ft² (234 to 586 m³/day of water per m²) of filter pack provides a reasonable degree of agitation and sufficient flow to remove the fine material from the filter pack. Approximately 5 to 10 minutes of backwashing is usually required to clean the pack, assuming that the flow rate above the infiltration gallery is sufficient to carry away the fine material removed from the pack. The use of compressed air to backwash

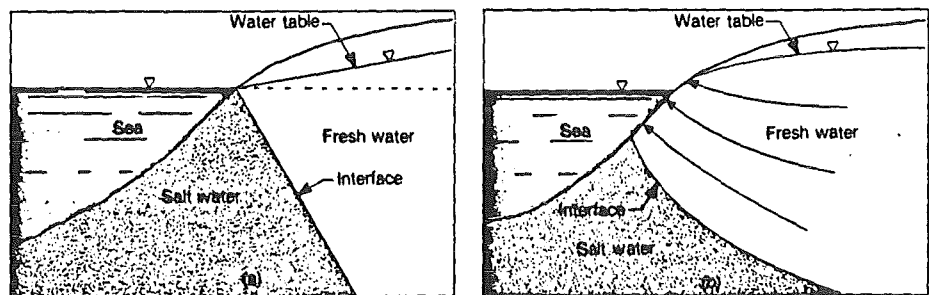


Figure 22.29. The position of the salt-water/fresh-water interface along a coast. In (a), the head created by the groundwater system just equals the head created by the ocean; thus no groundwater flow is occurring and the interface remains stationary. When the groundwater head exceeds that of the ocean, groundwater flows into the ocean forcing the interface to move seaward (b). (After Hubbert, 1940; Freeze and Cherry, 1979)

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Preface

Groundwater is a priceless resource lying beneath most of the Earth's land surface. In the United States, groundwater supplies about 25 percent of the nation's domestic, agricultural, and industrial water. Fifty percent of the United States' population depends on groundwater for its drinking water supply. Reliance on groundwater has increased greatly over the past 35 years because of population shifts to areas where surface water is often not plentiful. Similarly, substantial increases in groundwater withdrawals have occurred recently in almost every country of the world.

The vulnerability of groundwater to overuse and water-quality degradation was not widely understood until recently. In the future, as much attention will be paid to water-quality preservation and resource conservation as to resource development. The principal objective of this book is to provide the necessary technical knowledge to locate, extract, treat, and protect groundwater so that current and future generations can depend on this resource to enhance their quality of life.

Johnson Division is the world's largest manufacturer of water well screens. Since 1904, its technical staff in the United States and abroad has designed tens of thousands of high-capacity wells for every conceivable application. From this collective experience has come the development of effective well design procedures for all types of aquifers. Through the years, countless practical suggestions have been received from water well contractors, well design engineers, consultants, and scientists. This book summarizes this knowledge so that groundwater resources can be developed economically in the widest range of hydrogeologic settings without damage to aquifers through either overpumping or contamination.

Significant advances have been made in almost all phases of groundwater technology in recent years. Enhanced understanding of the groundwater environment, new geophysical exploration techniques, advances in well drilling methods, improved drilling fluid additives and screen designs, the development of groundwater quality monitoring technology, and remarkably effective new water treatment methods are just some examples. In addition, the majority of groundwater scientists and technicians have only recently received their formal training. Thus the need for sound, practical information is at a historic high. In response to this need, this edition of *Groundwater and Wells* provides both new and experienced members of the ground-

water industry with a single, comprehensive, up-to-date reference on all essential aspects of groundwater technology.

The approach of this book is pragmatic rather than theoretical. The material is presented so the interested reader, regardless of background, will gain a good understanding of the physics, chemistry, and hydraulics of groundwater and the various technologies used to develop this resource. In each subject area, a major effort is directed toward bringing out the practical elements; theoretical aspects are discussed wherever they serve to directly strengthen practical knowledge.

The book is written for four distinct reader groups — design engineers, water well contractors, government officials involved in well projects, and educators and students interested in the practical elements of hydrogeology. To be of maximum value to these four diverse reader groups, highly technical words from geology, hydrogeology, chemistry, physics, engineering, and hydraulics have been used sparingly. A certain number of technical words are necessary, but these words are generally defined where they are first used in the text. A large number of technical words are also defined in the glossary. To facilitate use of the book in countries using System International units, all basic equations are presented first in English units of measure and are followed immediately by the S.I. equivalent. Numerical values are given in both English and S.I. units. Problems, however, are worked out in English Engineering units only. References are listed at the end of the text. This reference list is not comprehensive but serves to indicate representative articles or textbooks that examine a particular subject in more detail.

The first edition of *Groundwater and Wells*, which has become a classic in its field with more than 60,000 copies sold worldwide, provided reliable technical information on many phases of water well technology. The Johnson Division staff is confident that this greatly expanded second edition will become an even more valuable resource for those involved in groundwater technology. If use of this book contributes to the public good, to the economic growth of any country or region, and to the advancement of the water-well industry, our efforts will have been rewarded.

JOHNSON DIVISION

Sam R. Goodman
General Manager and Vice President

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less reliable because of turbulence created by the pump. Thus, at least one observation well should be used when practicable. Furthermore, drawdown data from an observation well are required to calculate the storage coefficient accurately, whereas transmissivity values may be calculated on the basis of drawdown data taken from either a pumping well or observation well.

Observation wells should be just large enough to allow accurate and rapid measurement of the water levels. Small-diameter wells are best, because the volume of water contained in a large-diameter observation well may cause a time lag in drawdown changes. Most observation wells are constructed with screens 3 to 6 ft (0.9 to 1.8 m) long. Longer screens may be desirable, depending on the degree of stratification, but are not absolutely necessary.

When observation wells are too close to the pumped well, the drawdown readings may be affected by the stratification of the aquifer. Stratification distorts the distribution of hydraulic head and drawdown in the vicinity of the pumped well during the aquifer test. Recall that the vertical hydraulic conductivity of a stratified formation is usually much less than its horizontal hydraulic conductivity. This means that changes in head caused by pumping occur more slowly in the vertical direction than in the horizontal direction. At any moment after test pumping is started, the drawdown in a series of observation wells placed at a given distance from the pumped well may vary if the screens are set at different depths within the aquifer. These variations in drawdown become less as time of pumping increases. The distorted pattern of drawdown caused by stratification is eliminated at distances equal to three to five times the aquifer thickness.

For unconfined aquifers, observation wells should be placed no farther than 100 to 300 ft (30.5 to 91 m) from the pumped well. For thick confined aquifers that are considerably stratified, observation wells should be placed within 300 to 700 ft (91 to 214 m) of the pumped well. Locating the wells too far away is not good practice because the pumping test must be continued for a longer time to produce sufficient drawdowns at the most distant points, and small measurement errors may be a significant percentage of the total drawdown in the observation well. In addition, boundary effects may not be noticed.

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, because the distribution of drawdown is more distorted. If this procedure is followed, the reduction in pressure or water level at the observation well will usually occur within moments of its occurrence in the pumping well (assuming the observation well is spaced at the correct distance from the pumped well). Occasionally, observation wells are terminated in strata above or below the one tapped by the pumped well to see if there is any hydraulic interconnection between the formations. Naturally the response of these observation wells to pumping may be delayed significantly, depending on the degree of hydraulic connection.

The appropriate number of observation wells 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 location outside the pumped well permit calculation of the average hydraulic conductivity, transmissivity, and storage coefficient of the aquifer. If two or more observation wells are placed at different distances,

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This figure is less than the 69-percent value given for L_p . This means that much of the head loss is associated with laminar flow rather than with turbulence. Had L_p been used for the well efficiency, the well would have been judged to have a reasonably good efficiency value of 69 percent, whereas the true well efficiency is quite poor.

In summary, a step-drawdown test can be used to determine the following:

1. The 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, may not be sufficient to encounter boundary effects. Therefore, the actual long-term specific capacity at a certain discharge may be different than originally calculated from the step-drawdown data. Slow drainage can also distort the discharge data in highly stratified aquifers. Depending on the exact nature of the aquifer, the specific capacity may seem to improve with higher discharges and longer pumping time — a highly unlikely situation that will occur rarely, if ever, in natural geologic materials.
2. A ratio L_p , denoting the percentage of the total head loss attributable to laminar flow, which is not to be confused with the true hydraulic efficiency of the well.
3. 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.

PROBLEMS OF PUMPING TEST ANALYSIS

Constant-rate and step-drawdown pumping tests offer the most powerful method for analyzing the hydrogeologic character of an aquifer. Measurements made during

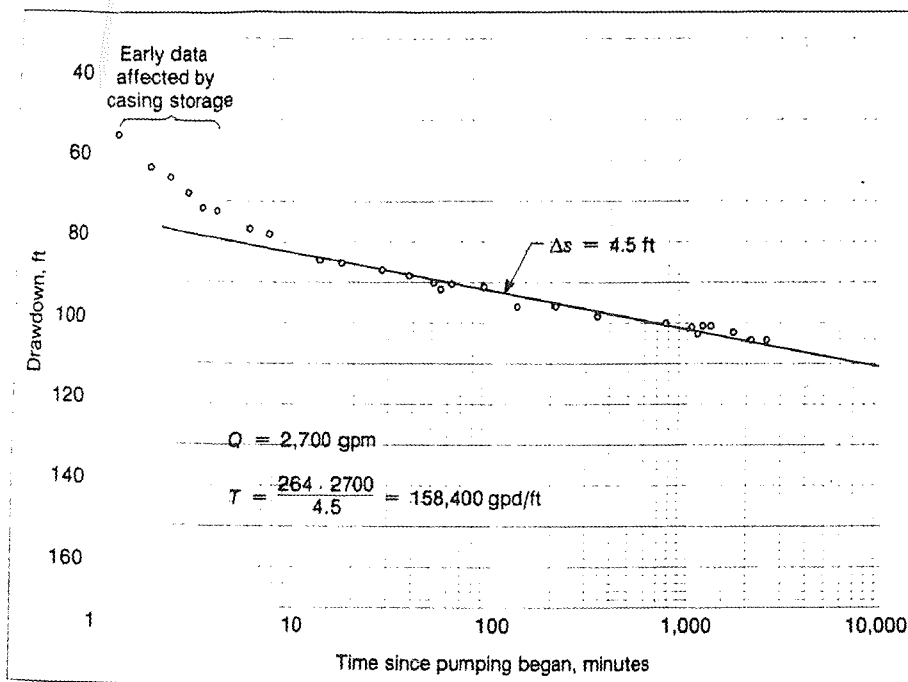


Figure 16.16. Time-drawdown plot for a well discharging 2,700 gpm (14,700 m³/day).

an aquifer test provide values for some of the terms in the Theis equation, permitting calculation of the transmissivity and storage capacity of the aquifer. (See Chapter 9 for a full description of plotting procedures.) The hydraulic conductivity values determined from pumping test data are far more accurate than hydraulic conductivities calculated on the basis of field samples tested in a laboratory, because the samples are rarely representative of the undisturbed formation. The aquifer test measures the performance of the aquifer in its natural state.

In addition to determining hydraulic conductivity, transmissivity, and coefficient of storage, pumping test data can be used to determine any interference between wells at various spacings and rates of pumping other than those employed in the test itself. Under some conditions, pumping test data will also indicate what drawdown may be expected from long-term pumping at different discharge rates, the existence of impervious boundaries that limit the extent of the aquifer, and the existence of recharge sources which may not be apparent otherwise.

In many cases, pumping test data can be interpreted in more than one way. Even in the best controlled test, the data may be confusing unless all hydraulic and geologic factors are taken into account. The analyst must be able to identify unreliable data so that the calculated values for transmissivity and storage coefficient will correctly predict aquifer performance.

The pumping test analyst can be faced with a large number of anomalies in the data, but in the discussions below it is assumed that the following physical conditions for a proper test have been met:

1. The diameter of the pumped well is large enough to accommodate the test pump and provide enough clearance so that the water level can be measured with an air line, electric or acoustic probe, or other device.
2. The test pump can deliver at least 10 percent of the well's maximum yield.
3. In unconsolidated formations, the well screen is at least one-third the thickness of the aquifer, except in extremely thin strata where three-fourths of the formation should be screened.

Although either a constant-rate or a step-drawdown test could have been conducted in the instances discussed below, most of the actual pumping tests are constant-rate tests because the values obtained for transmissivity, storage coefficient, and efficiency are generally more accurate, and different trends in the data are more readily discernible.

The pumping test examples cited below will serve to illustrate some of the major problems that can occur in data interpretation. It is assumed that the reader has thoroughly understood constant-rate pumping test analysis as discussed in Chapter 9. The analyst must also keep in mind several other factors during data interpretation. These include the effect of

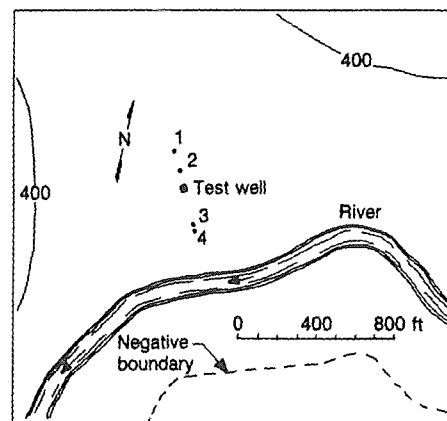


Figure 16.17. Arrangement of 12-in (305-mm) test well and four observation wells for a 72-hour pumping test. This test was made to get sufficient information for designing a well field for a total supply of 2,000 gpm (10,900 m³/day), with wells sized and spaced for best economy and minimum interference.

the hydrogeologic character of the aquifer on the data, errors in data acquisition, potential omission of important environmental impacts, well design and construction practices, pumping rates, and the theoretical limits of the Jacob theory.

Example 1 — Test Conducted Near a River

In this case, the floodplain of a small river was being considered as the site for an industrial plant (Figure 16.17). A generalized cross section of the two aquifers in this area, and the positions of the various observation wells, are shown in Figure 16.18.

For the aquifer test, a 12-in (305-mm) well was drilled about 400 ft (122 m) from the bank of the river. Both aquifers were penetrated — an unconfined aquifer extending to a depth of about 46 ft (14 m), and a deeper confined aquifer extending from 50 to 105 ft (15.2 to 32 m). A 4-ft (1.2-m) clay layer, from 46 to 50 ft (14 to 15.2 m), separated the aquifers. It was assumed that the 4-ft clay layer effectively isolated the two aquifers. The static water level, however, in both aquifers was 5 ft (1.5 m) below ground level. The arrangement of observation wells for this pumping test illustrates some of the points discussed earlier in this chapter.

Three 2-in (51-mm) observation wells about 90 ft (27.4 m) deep were put down at distances of 100 ft (30.5 m) and 200 ft (61 m) from the 12-in (305-mm) well. Two of these wells were placed away from the river compared with the location of the test well, whereas one well was placed between the river and the test well. A shallow observation well, No. 4, was installed near observation well No. 3 for measuring any water-level changes that might occur in the upper aquifer. This was done primarily to check for any hydraulic connection between the two aquifers, because the continuity of the clay separation was unknown. Any water-level change in the shallow well caused by the pumping of the test well would indicate discontinuity in the clay layer.

The pumping test was conducted for 72 hours. During the early stages of the pumping test, a heavy rainstorm occurred in the upper reaches of the watershed, causing the river level to rise suddenly about 2 ft (0.6 m). No other unusual event took place

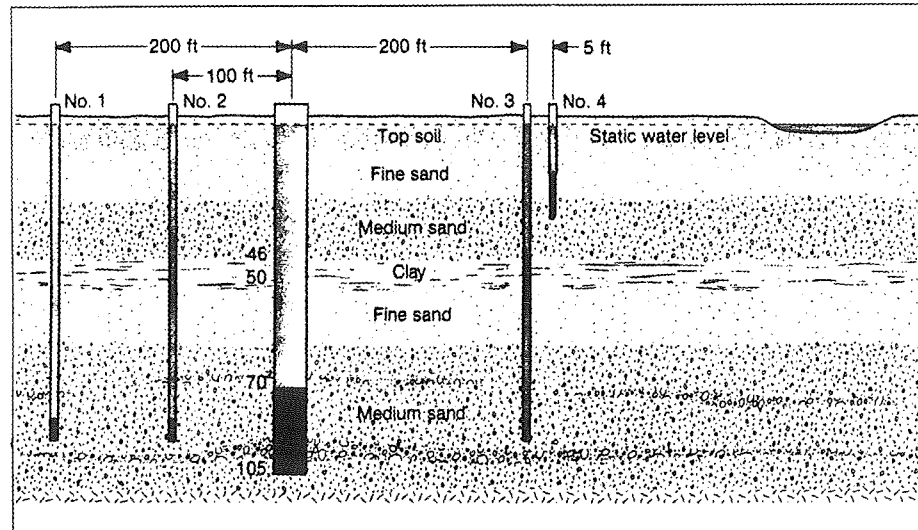
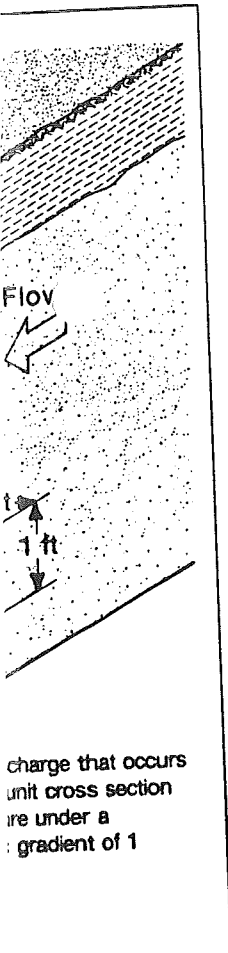


Figure 16.18. Geologic section on a line through test well and observation wells shown in Figure 16.17. The clay layer apparently separates the upper unconfined aquifer from the confined aquifer.

er unit change in
e aquifer. In con-
nsion of the con-
The coefficient of
from 0.01 to 0.3;
69) demonstrates
ned and confined

er 5, is the rate at
ide and extending
(100 percent). Fig-
issivity. Values of
o over 12,400 m²/
it can supply only



charge that occurs
unit cross section
are under a
gradient of 1

issivity. Hydraulic con-

enough water for domestic wells or other low-yield uses. When the transmissivity is 10,000 gpd/ft (124 m²/day) or more, well yields can be adequate for industrial, municipal, or irrigation purposes.

The transmissivity and storage coefficients are especially important because they define the hydraulic characteristics of a water-bearing formation. The coefficient of transmissivity indicates how much water will move through the formation, and the coefficient of storage indicates how much can be removed by pumping or draining. If these two coefficients can be determined for a particular aquifer, predictions of great significance can usually be made. Some of these are:

1. Drawdown in the aquifer at various distances from a pumped well.
2. Drawdown in a well at any time after pumping starts.
3. How multiple wells in a small area will affect one another.
4. Efficiency of the intake portion of the well.
5. Drawdown in the aquifer at various pumping rates.

CONE OF DEPRESSION

When water is pumped from a well, the initial discharge is derived from casing storage and aquifer storage immediately surrounding the well (Figure 9.6). As pumping continues, more water must be derived from aquifer storage at greater distances from the well bore. This means that the cone of depression must expand. The radius of influence of the well increases as the cone expands. Drawdown at any point also increases as the cone deepens to provide the additional head required to move the water from greater distances. The cone expands and deepens more slowly with time, however, because an increasing volume of stored water is available with each additional foot of horizontal expansion.

Figure 9.7 illustrates how the cone of depression expands during equal intervals of time. Assume that after 10 hours of pumping the radius of the cone is 400 ft (122 m) and its depth is 6 ft (1.8 m) at the well bore. At the end of 20 hours, the cone's radius has expanded to 570 ft (174 m) and its depth has increased to 6.3 ft (1.9 m). In the second 10 hours, the cone

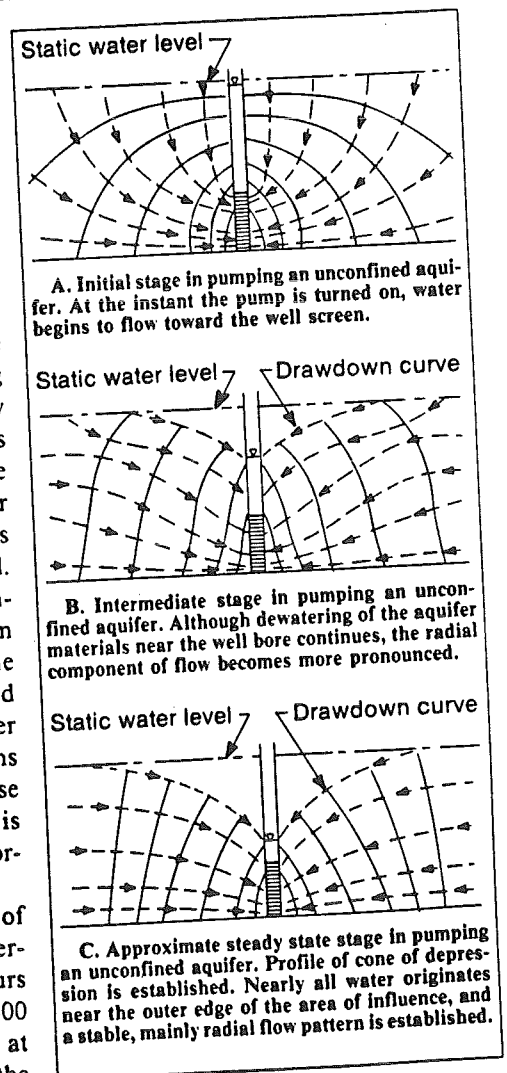


Figure 9.6. Development of flow distribution about a discharging well in an unconfined aquifer that is 33% screened. (Water and Power Resources Service, 1981)

TV.

N-756

**MIDDLEPORT SITE INVESTIGATION
FINAL REPORT
VOLUME I - TEXT**

**FMC Corporation
Middleport, New York**

May, 1987

**May 1987
Ref. No. 1539-08**

CONESTOGA-ROVERS & ASSOCIATES

7.0 MODELING OF THE FMC MIDDLEPORT PLANT SITE

7.1 INTRODUCTION

The following sections describe the numerical models used to simulate the groundwater flow and chemical transport in the vicinity of the Site.

The use of models is considered to be advantageous for at least two reasons:

- 1) They assist in understanding the response of the hydrogeologic regime to the system parameters.
- 2) They assist in the identification of those processes or process interactions which are most likely to affect the migration of chemicals from the Site, and thereby guide the data collection and analysis program.

A two-dimensional variably saturated cross-section modeling approach was used for the Site. The model was based on the assumption of equivalent porous media flow for fractured media. The varying hydrogeologic conditions with depth could be incorporated into the vertical plane model through the discretization process required by the finite element method.

7.2 GROUNDWATER FLOW AND CHEMICAL TRANSPORT IN VARIABLY SATURATED CROSS-SECTIONS

7.2.1 Groundwater Flow Model

Steady-state groundwater flow was assessed for cross-section C-C' (see Figures 5.1 and 5.12) of the Plant Site. The governing variably saturated flow equation is:

$$\frac{\partial}{\partial x_i} \left\{ K_{ij} K_r(S_w) \left(\frac{\partial \psi}{\partial x_j} + \frac{\partial Z}{\partial x_j} \right) \right\} = 0 ; i, j = 1, 2 \quad (1)$$

with boundary conditions defined as

$$\psi = \hat{\psi} \text{ on } \Gamma_1 \quad (2a)$$

$$K_{ij} K_r(S_w) \left(\frac{\partial \psi}{\partial x_j} + \frac{\partial Z}{\partial x_j} \right) \cdot n_i = \hat{q}_n \text{ on } \Gamma_2 \quad (2b)$$

subject to $\Gamma = \Gamma_1 + \Gamma_2$

In Equations (1) and (2),

$K_r(S_w)$ = relative hydraulic conductivity, a function of the degree of water saturation S_w with $0 < K_r(S_w) < 1.0$

ψ = pressure head (L)

Z = datum (L)

K_{ij} = saturated hydraulic conductivity tensor (L/T)

Equation (2a) represents prescribed pressures on surface Γ_1 and Equation (2b) represents a prescribed flow normal to boundary Γ_2 . In this study, out-of-plane flow and mass transport were not considered. It was felt that the data for the Site could be adequately explained with a cross-sectional description.

Equations (1) and (2) assume that in an unsaturated zone, water is the wetting fluid at pressure ψ and air the non-wetting phase at atmospheric pressure throughout the unsaturated zone. The solid matrix is assumed to be rigid and stable. The pore fluid is assumed to have a constant density and viscosity and to be incompressible over the range of pressures considered.

The solution of Equation (1) with (2) is based on the Galerkin finite element method and the isoparametric quadrilateral element (Sykes, 1980). Element sides can be linear, quadratic or cubic. Components of the saturated hydraulic conductivity tensor and areal recharge/discharge can vary from element to element. The relative hydraulic conductivity $K_r(S_w)$ is a functional coefficient in the model.

Sensitivity of a performance measure or system response function to the system parameters is accomplished using the adjoint method (Sykes et al, 1985). Two performance measures are considered:

$$P = \{g\}^T \{h\} \quad (3)$$

and

$$P = (q_1^2 + q_2^2)^{1/2} \quad (4)$$

where $\{g\}^T$ is a row vector of user-defined nodal weights; $\{h\}$ is a column vector of nodal pressure heads; and q_1 and q_2 are the components of Darcy velocity in a user-selected element. Equation (3) represents a pressure head performance measure while Equation (4) is the magnitude of Darcy velocity in an element. The marginal sensitivity of P to a parameter θ_k is given by $dP/d\theta_k$. The normalized sensitivity coefficients are defined by $(dP/d\theta_k)(\theta_k/P)$. The adjoint model calculates the sensitivity of the selected performance measures to the element longitudinal hydraulic conductivities and the prescribed pressure heads $\hat{h}$ on Γ_1 . Sensitivity with respect to areal recharge/discharge was performed by perturbation of the recharge/discharge values.

The primary flow problem of Equation (1) has been validated in Sykes (1980).

7.2.2 Chemical Transport Model

The governing equation for chemical transport in variably saturated media is (Sykes, 1986)

$$\frac{\partial}{\partial t}(\bar{C}) + (\theta + \rho_b k_d) \frac{\partial C}{\partial t} = \frac{\partial}{\partial x_i} (D_{ij} \frac{\partial C}{\partial x_j}) - q_i \frac{\partial C}{\partial x_i} \quad (5a)$$

with
$$\frac{\partial}{\partial t}(\bar{C}) = -\theta \lambda (C_{\max} - C) \quad (5b)$$

where

θ = moisture content

C = concentration (M/L³)

t = time (T)

D_{ij} = hydraulic dispersion tensor (L²/T)

$C_{\max}$ = maximum dissolved concentration of species of interest (M/L³)

x_i = components of cartesian coordinate directions x_1 and x_2

q_i = components of Darcy velocity (L/T)

Q = areal recharge/discharge (L/T)

ρ_b = soil bulk density (M/L³)

k_d = linear adsorption isotherm (L³/M)

$\bar{C}$ = mass of constituent at source in solid or adsorbed form (assumed to be zero for nonsource areas) (M/L³)

λ = rate coefficient for leaching of source

The initial and boundary conditions used for the solution of Equations (5) are:

$$C(x_i, t) = \hat{C}_0 \quad t = 0 \quad (6a)$$

$$C(x_i, t) = \hat{C} \text{ on } \Gamma_3 \quad (6b)$$

$$J = (-D_{ij} \frac{\partial C}{\partial x_j} + q_i C) \cdot n_i \text{ on } \Gamma_4 \quad (6c)$$

where:

$\hat{C}_0$ = initial concentration

$\hat{C}$ = prescribed concentration on surface Γ_3

J = prescribed mass flux normal to surface Γ_4

n_i = components of unit inward vector normal to surface Γ_4

For the spatial domain the external boundary, Γ , is defined by $\Gamma = \Gamma_3 + \Gamma_4$. Equations (5) are valid when a constant fluid viscosity and density are maintained throughout the modeled domain. Equation (5a) represents a conservation of mass for the species of interest and includes terms describing leaching, adsorption, dispersion and advection. Equation (5b) represents the conservation equation for the species of interest at the source; the rate of leaching is assumed to be proportional to the difference between the limiting solute concentration of the species and the actual concentration.

The hydrodynamic dispersion tensor is
(Bear, 1972)

$$D_{ij} = \alpha \bar{V} \delta_{ij} + (\alpha_L - \alpha_T) \bar{V}_i \bar{V}_j / \bar{V} + D_d \delta_{ij} \quad (7)$$

for an isotropic medium with the principal direction of the dispersivity tensor coinciding with velocity direction. In Equation (7), α_L and α_T are the longitudinal and transverse dispersivities (L), respectively; $\bar{V}$ is the magnitude of the linear velocity (L/T); $\bar{V}_i$ and $\bar{V}_j$ are the components of the velocity vector (L/T); and D_d represents molecular diffusion (L²/T). For this study, the effect of D_d was considered to be negligible as the regime being studied is dominated by advective flow conditions.

The solution of Equations (5) with (6) is based on the Galerkin finite element method and the isoparametric quadrilateral element (Sykes 1980). For the temporal domain, the fully implicit finite difference formulation is used. The model incorporates spatially varying moisture contents (porosities for saturated media). Stability criteria with respect to the cell Peclet number ($\frac{\Delta x}{\alpha_l}$) as referenced by Conner and Christodoulou (1978), and with respect to the Courant number ($\frac{q \Delta t}{\Delta x}$) as referenced by O'Neill (1981), were adhered to during the problem solution. In the above, Δx and Δt are the spatial and temporal discretization, respectively, and q is the Darcy velocity.

The Darcy velocities that were determined by simulation of the groundwater flow system were utilized in the chemical transport model.

7.3 CONCEPTUALIZATION OF GROUNDWATER FLOW

The regional flow system has been previously presented in Section 3.

The formations in the vicinity of the Site were modeled as equivalent porous-medium, phreatic aquifers and aquitards. This conceptualization was chosen because of the following:

- 1) there were insufficient data describing fracture widths, orientation and interconnection,
- 2) varying hydrogeologic conditions with depth could be incorporated into the vertical plane model through the discretization process required by the finite element method, and
- 3) the measured potentiometric surfaces at the Site indicated a phreatic system.

The material properties of porosity and hydraulic conductivity can vary spatially to describe the complex lithology and layering at the Site. Within any layer, however, homogeneous conditions were assumed. Stratification that occurs in sediment deposition was accounted for through the use of anisotropic hydraulic conductivity. Sedimentary rocks are usually considerably less permeable in the vertical direction.

The conceptualization of cross-section C-C' is shown on Figure 7.1. This section was chosen to minimize the possible effect of out-of-plane flow that would occur if a cross-section was selected that was located where the potentiometric contours were not perpendicular with the cross-section. The hydrogeologic units were determined by:

- i) interpretation of the stratigraphic logs for boreholes in the vicinity of cross-section C-C',
- ii) review of the packer-pressure injection tests performed in ten-foot increments, and
- iii) extrapolation of the regional dip and strike from historical reports and records.

The vertical discretization consists of seven units:

LEGEND

ZONE LITHOLOGY

K_H (cm/s)

K_V (cm/s)

θ

1 1 OVERBURDEN

10^{-4}

10^{-5}

0.30

2 2 LIMESTONE

10^{-3}

10^{-4}

0.01

3 3 SANDSTONE

10^{-4}

5×10^{-6}

0.05

4 4 FRACTURE ZONE

10^{-2}

5×10^{-4}

0.075

5 & 7 5 & 7 SANDSTONE

3×10^{-4}

7.4×10^{-6}

0.05

6 6 SHALE

9×10^{-10}

5×10^{-11}

0.01

ELEVATION IN FEET (AMSL.)

JEDDO CREEK

ERIE CANAL

FMC PLANT SITE

QUEENSTON SHALE

N

S

ELEVATION IN FEET (AMSL.)

figure 7.1

SITE CONCEPTUALIZATION
FMC-Middleport Plant Site

- 1) a glacial till overburden,
- 2) a limestone unit,
- 3) an upper sandstone unit,
- 4) a fracture zone,
- 5) a middle sandstone unit,
- 6) a shale layer, and
- 7) a lower sandstone unit.

The base of the aquifer was considered to be the elevation of the top of the Queenston Shale, as shown in Figure 7.1. The lithology and stratigraphy north of the Erie Canal was based on extrapolation of the region south of the canal.

The fracture zone was interpreted from the logs of Wells R-523, R-525, R-527, R-530, R-531 and R-533. The bedrock cores from these boreholes had fractures that were in the same zone when the regional dip and strike were considered. The shale layer was interpreted from the logs of Wells R-8, R-10, R-12, R-14, R-521 and R-533. The bedrock cores from these boreholes had shale in the same zone when the regional dip and strike were considered. It was assumed, for both the fracture zone and shale layer, that they were locally laterally extensive, thus, their presences were extrapolated, following the regional dip and strike, beyond the region where data were available. The fracture zone was assigned a vertical thickness of approximately five feet to

correspond to the interval over which fractures were noted. The shale layer was assigned a thickness of approximately seven feet. This represents an average of the thickness noted in the core samples.

The geographic area simulated by the model was approximately 6,300 feet in length. The discretization consisted of 827 elements and 904 nodes.

Various types of boundary conditions can be employed in a finite element flow model. These include prescribed (constant) head, prescribed flow, seepage and no-flow boundaries. In this evaluation, the north and south boundaries were specified as prescribed pressure-head boundaries. Groundwater flow is expected to occur through the north and south boundaries; therefore, a no-flow boundary is inappropriate. Since there are no data available on the quantity of flow across these boundaries, a prescribed flux boundary could not be specified. A seepage boundary is a surface condition and thus, is inappropriate. Therefore, the only appropriate boundary condition left, that would allow flow across these boundaries, is a prescribed head boundary. At the boundaries, data were available regarding only near-surface groundwater levels. No data regarding the deeper formation at the boundaries was available. Therefore, hydrostatic conditions were assumed. The head loss across the shale layer at the south boundary was extrapolated from

data near the middle of the section. In this study, the observed head loss of 50 feet between the shallow wells and deep wells was assumed to occur entirely across the conceptualized shale layer. In fact, the head loss would be distributed across all layers through which vertical flow occurs. However, the vertical hydraulic conductivity of the shale was approximately five orders of magnitude less than that of the underlying sandstone. As a result, for a constant vertical flux through the sandstone and shale, the head loss in the sandstone would be negligible when compared to that in the shale.

The top boundary was considered to be a mixture of specified pressure-head, seepage and prescribed recharge boundaries. Recharge from precipitation infiltration occurs over the Site. Therefore, the top boundary was described by a specified-flux boundary. Since the Queenston Shale, assumed as a no-flow boundary due to its low hydraulic conductivity and large thickness, rises to the north and the land surface dips to the north, the cross-section available for groundwater flow decreases in depth to the north. Due to this thinning of the flow regime, it was anticipated that seepage of groundwater at the surface may occur in the northern area of the modeled cross-section. Therefore, as indicated on Figure 7.2, some nodes were assigned as seepage nodes. This is an optional condition which is not implemented until the calculations indicate that

LEGEND



LOCATION OF VELOCITY
PERFORMANCE MEASURE



LOCATION OF PRESSURE
PERFORMANCE MEASURE

PREScribed PRESSURE HEAD (feet)
89.08 RUN 1
(79.08) RUN 2

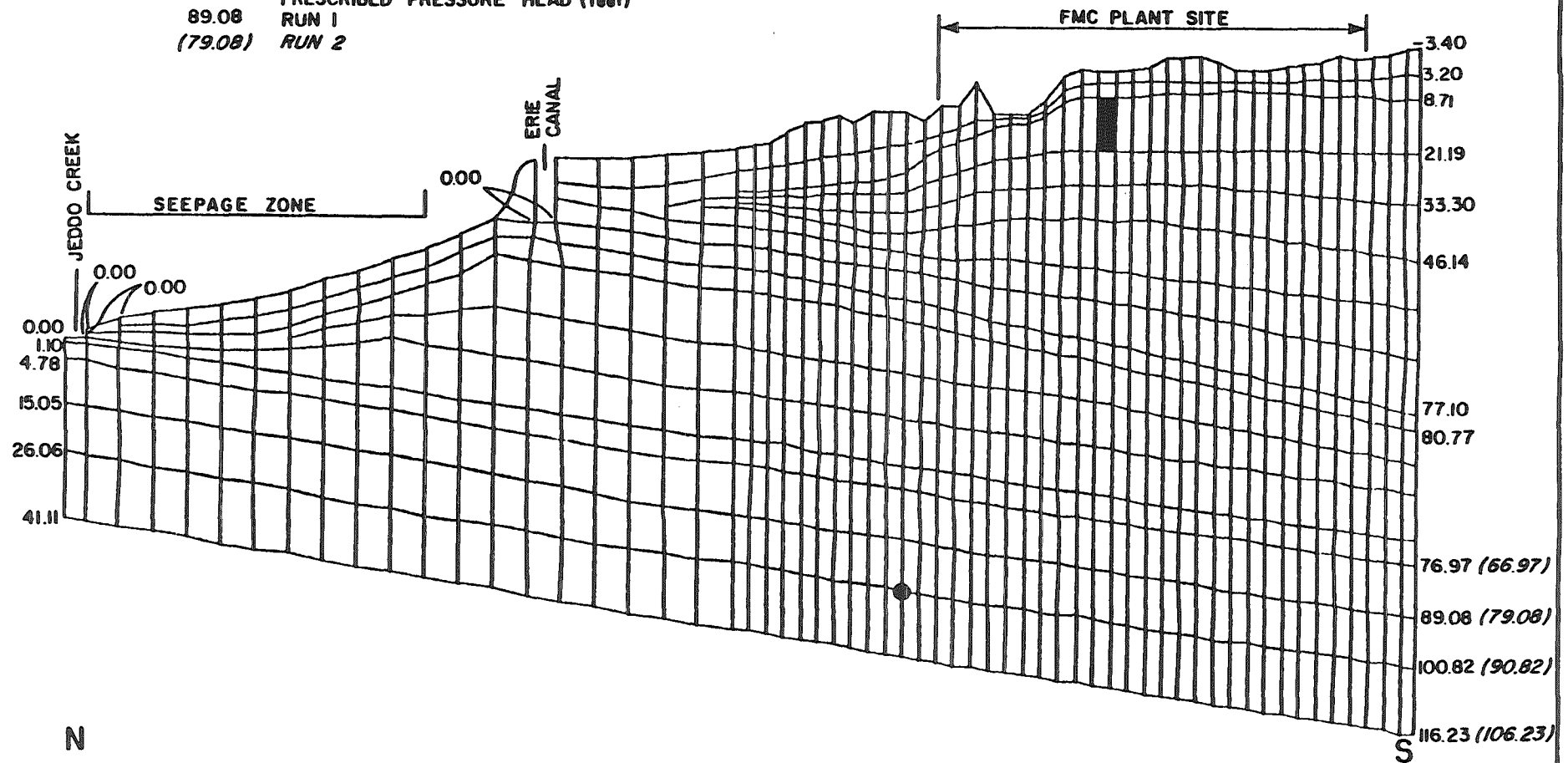


figure 7.2

PERFORMANCE MEASURE LOCATIONS
FMC-Middleport Plant Site

CRA

LEGEND

-  MONITORED INTERVAL
- 520 SIMULATED POTENTIOMETRIC CONTOUR (feet AMSL)
- 519.08 MEASURED POTENTIOMETRIC HEAD (feet AMSL)

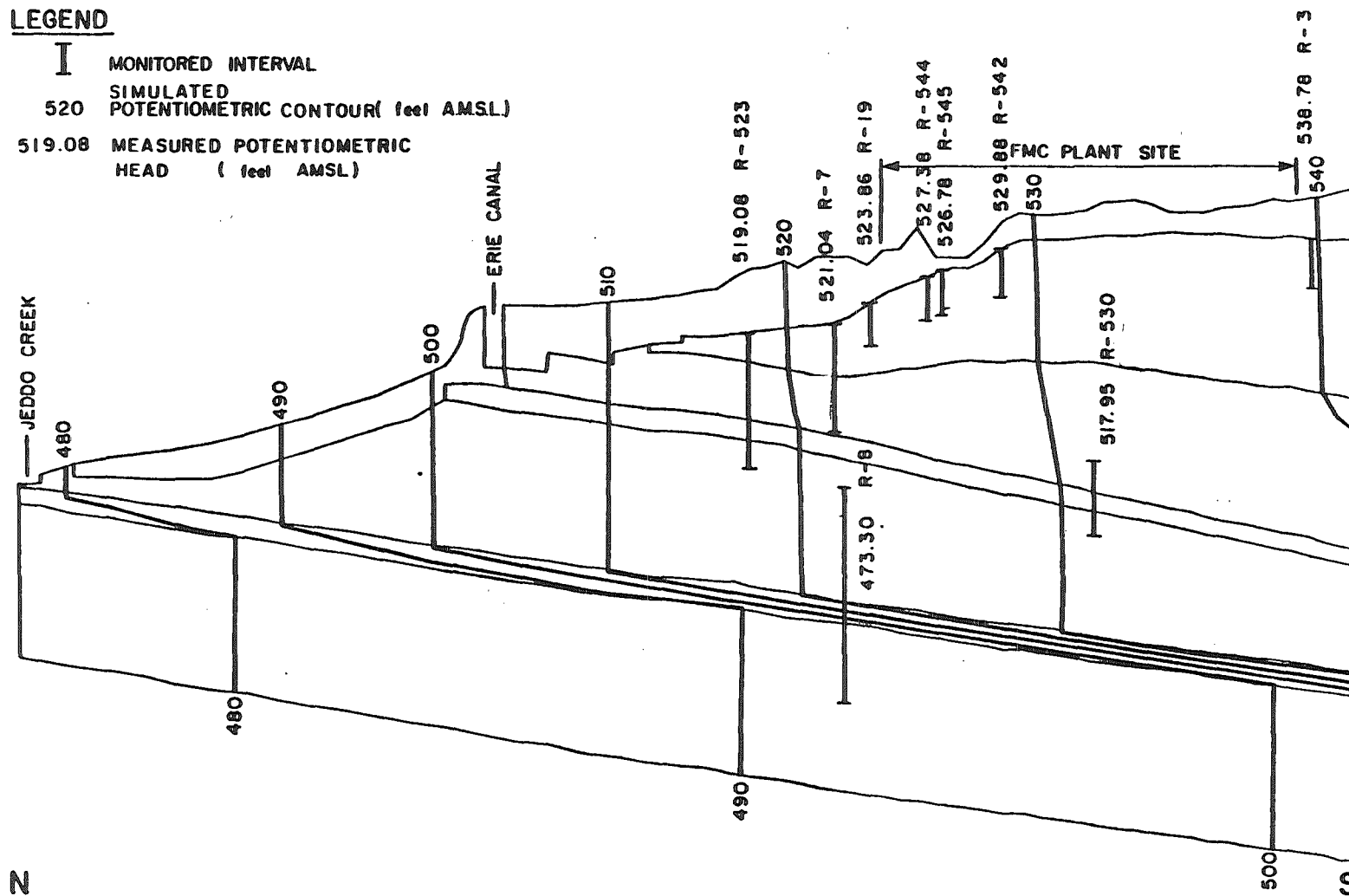


figure 7.3

POTENTIOMETRIC CONTOURS - RUN I
FMC-Middleport Plant Site

groundwater is discharging to the surface. The two nodes at the base of the Erie Canal and those nodes in the vicinity of Jeddo Creek were specified as constant pressure-head nodes thus reflecting the presence of surface water.

All the boundary conditions were based on interpretation of available data and thus are appropriate for this Site.

Initial hydraulic conductivity values for each hydrogeologic unit were calculated as the geometric mean of the appropriate values obtained from the slug tests for the overburden and the pressure-injection tests for the bedrock units.

7.4 GROUNDWATER FLOW ASSESSMENT

Preliminary calibration results of the analysis utilizing the parameters shown on Figures 7.1 and 7.2, are presented on Figures 7.3 to 7.5. The calibrated recharge value was 1.0 in/year. Although this value is lower than the 1.5 in/year estimated using the HELP model, it is considered to be consistent because of the conservative nature of the HELP estimate (see Section 5.5).

LEGEND

**10 SIMULATED PRESSURE
HEAD CONTOUR (feet)**

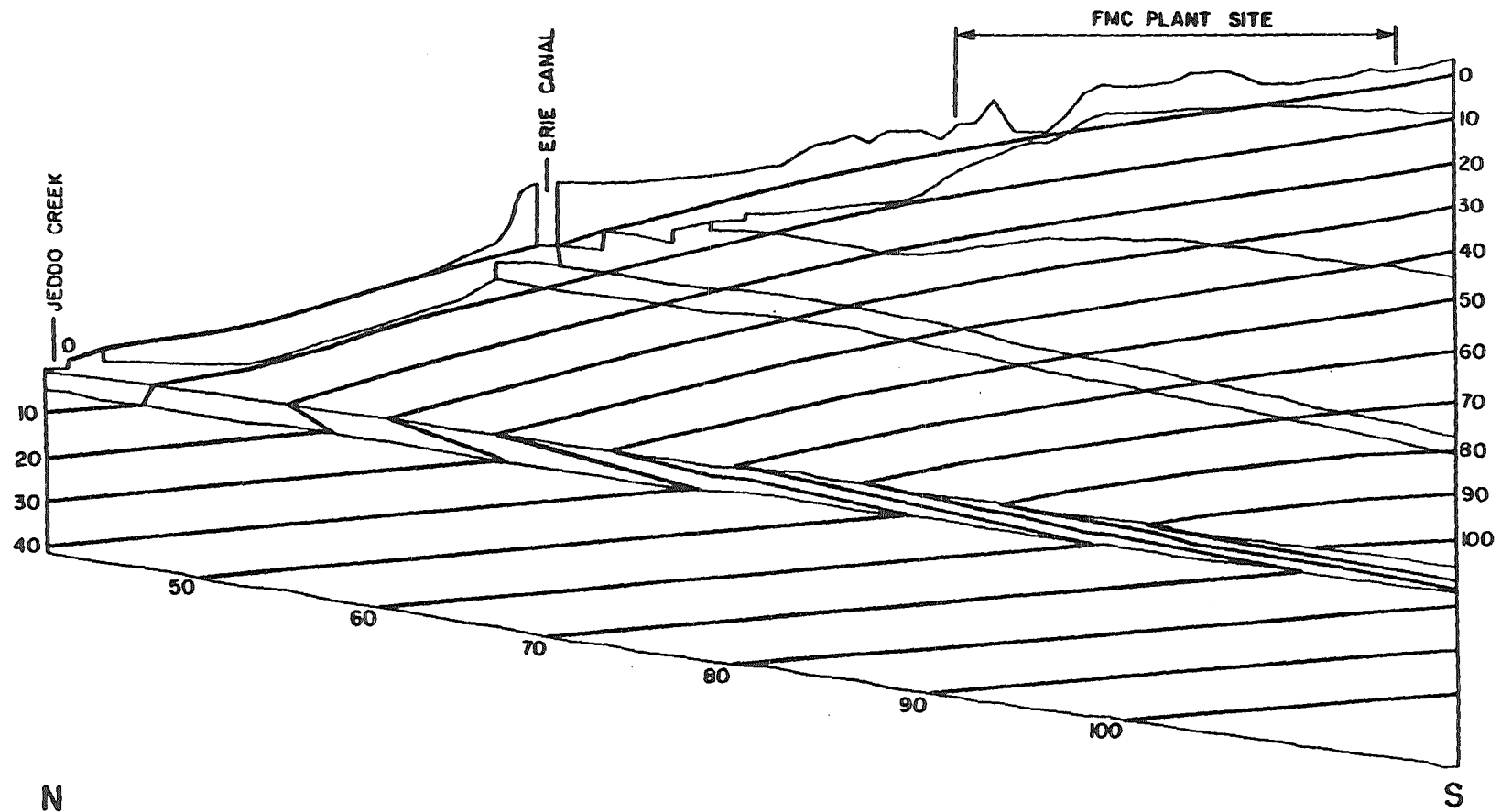
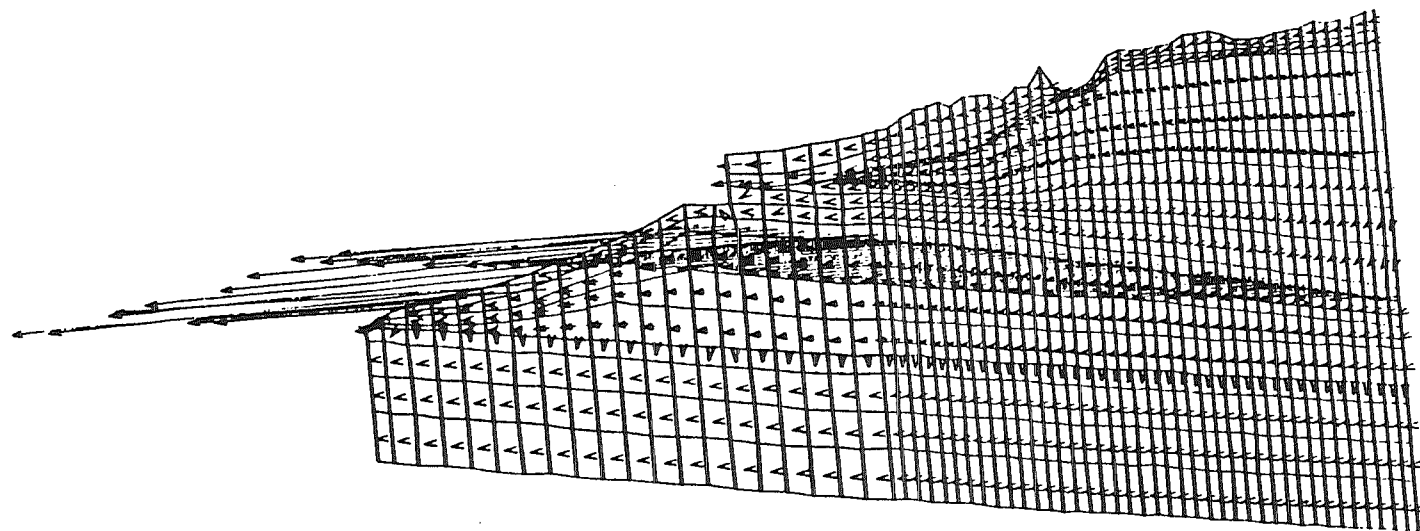


figure 7.4

PRESSURE HEAD - RUN 1
FMC-Middleport Plant Site

CRA



SCALE FOR VELOCITY VECTORS



figure 7.5
DARCY VELOCITIES - RUN 1
FMC - Middleport Plant Site

The calibrated hydraulic conductivity value of 0.28 ft/day (10^{-4} cm/sec) for the Overburden Unit was consistent with the slug test results shown on Table 4.5. For the Limestone Unit, the calibrated value of 2.83 ft/day (10^{-3} cm/sec) fell within the range of values derived from the pressure-injection tests. The same statement is valid for the upper and middle sandstone units. The hydraulic conductivity of the fracture zone was evaluated at approximately twice the highest measured value during the pressure-injection tests. As the hydraulic conductivities were calculated using 10-foot increments and the modeled thickness was 5 feet, the calibrated value is considered to be reasonable. No field data were available for the hydraulic conductivities of the shale and lower sandstone units. The calibrated hydraulic conductivity for the shale is within the range reported in the literature (Freeze and Cherry, 1979) while the lower sandstone unit was assigned the same value as the middle sandstone unit.

The anisotropy ratio of 1:10 that was used for $K_v:K_h$ for the overburden material is consistent with previous analyses of similar materials. The same ratio was used for the limestone due to the extensive fracturing of the principally horizontal bedding planes. A ratio of 1:20 was assumed for the sandstone units due to less fracturing and interbedding of shales with the sandstone matrix. The fracture unit was assigned a 1:20 ratio because the observed

fractures were horizontal and the same ratio, approximately, was assumed for the shale unit to simulate the horizontal bedding planes of the unit.

The results on Figure 7.3 show that the potentiometric head in the shallow bedrock was simulated quite well. As the depth was increased, the simulated results maintained the general trend of the observed heads in the deeper zones; however, there were some discrepancies. These discrepancies can be explained as follows:

- i) Firstly, the potentiometric head measured in Well R-8 represents an average head over the 50 feet that the well monitors. The measured value could be the effect of the dewatering of upper zones and the recharging of lower zones in the monitored interval by the water from the upper zones. Due to the discrete nature of the flow zones in a rock system, no information is available as to which zone or zones over the 50-foot monitored interval the measured potentiometric value represented.
- ii) Secondly, the Site system is topographically controlled. The flow model results for the lower zone are very sensitive to changes in the prescribed pressure-head boundaries below the shale unit at both the northern and southern boundaries as shown in Figures 7.6 and 7.7. There is little field information available to aid in the conceptualization of these boundaries.

LEGEND

PERFORMANCE MEASURE LOCATION - ▲
 PARAMETERS - HYDRAULIC CONDUCTIVITY (K_{xx})
 - PRESCRIBED PRESSURE HEAD BOUNDARY (2.96×10^{-3})

SENSITIVITY MAGNITUDE	+ve	-ve
$ s < 10^{-6}$		
$10^{-6} \leq s < 10^{-3}$		
$10^{-3} \leq s < 10^{-2}$		
$\geq 10^{-2}$		

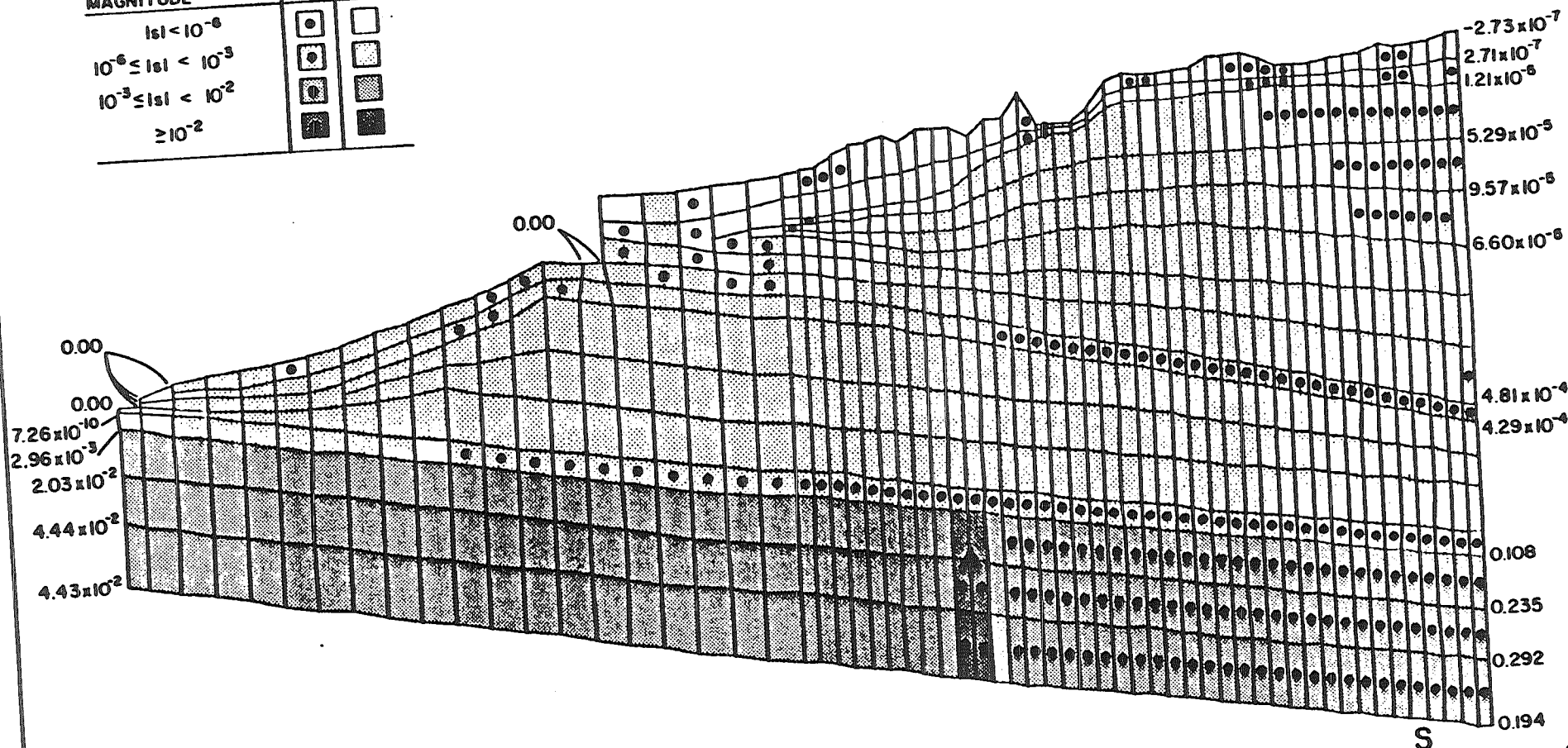


figure 7.6
 NORMALIZED SENSITIVITY COEFFICIENT
 PERFORMANCE MEASURE - PRESSURE HEAD
 FMC-Middleport Plant Site

LEGEND

PERFORMANCE MEASURE LOCATION-▲
 PARAMETERS-HYDRAULIC CONDUCTIVITY(K_{xx})
 -PRESCRIBED PRESSURE HEAD BOUNDARY (-2.11×10^{-5})

SENSITIVITY MAGNITUDE	+ve	-ve
$ s < 10^{-6}$		
$10^{-6} \leq s < 10^{-3}$		
$10^{-3} \leq s < 3 \times 10^{-2}$		
$ s \geq 3 \times 10^{-2}$		

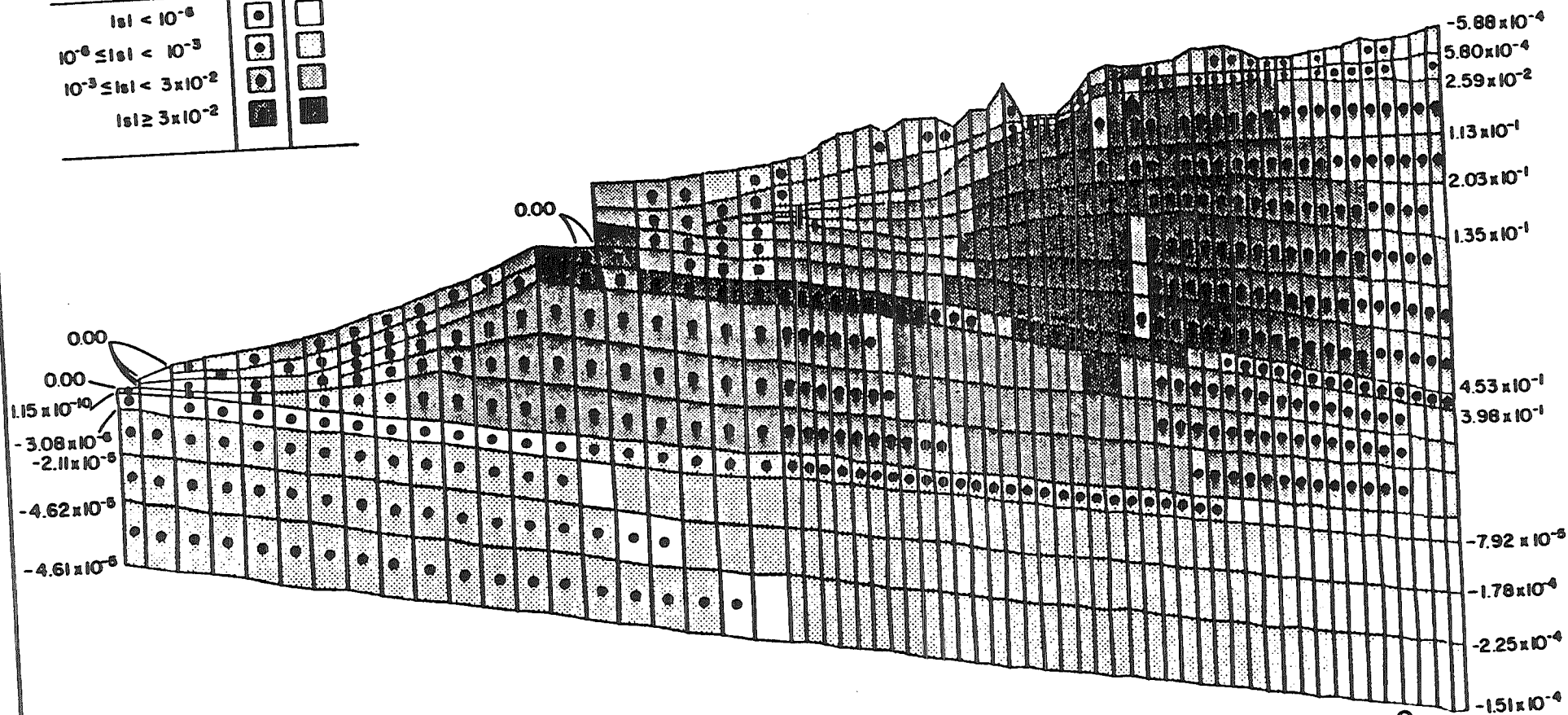


figure 7.7
 NORMALIZED SENSITIVITY COEFFICIENT
 PERFORMANCE MEASURE - DARCY VELOCITY
 FMC-Middleport Plant Site

Figures 7.6 and 7.7 show the normalized sensitivity coefficients for the performance measures of pressure heads at the indicated nodes and Darcy velocities in the indicated elements, respectively. The normalized sensitivity coefficient for each parameter represents the percent change in the performance measure for a one percent change. These results indicate that the simulated area behaves as two systems, one above the shale layer and one below. This is confirmed by the change in the sensitivity coefficient values by at least an order of magnitude at the shale unit.

To evaluate the importance of the boundary condition conceptualization, the potentiometric loss across the shale layer on the upgradient boundary was increased from 40 feet to 50 feet and all nodes below were changed correspondingly as shown on Figure 7.2. This head loss is consistent with the potentiometric difference noted between Wells R-8 and R-530. The information presented in Section 7.3 indicates that almost all of the head loss in the lower zones occurs through the shale layer.

Utilizing the calculated normalized sensitivity coefficients for the upgradient boundary nodes below the shale layer, the predicted pressure head at the performance measure was calculated to be 68.18 feet while the simulated value was 67.99 feet. This close agreement demonstrates the validity of the adjoint method.

Simulation of the above change produced the results shown on Figures 7.8 and 7.9. Comparison of Figures 7.3, 7.4, 7.8, and 7.9 indicates that the system does behave as two systems separated by the shale unit. The only significant change was the potentiometric contours in the lower sandstone unit.

Next, the sensitivity of the flow system to perturbations in recharge was investigated. Recharge values of 0.5 inches/year and 3.0 inches/year were simulated. These values were selected to observe the effect of decreased and increased recharge from the calibrated value of 1 inch/year. The results are shown on Figures 7.10 to 7.13, inclusive. For these simulations, the four nodes on the upgradient boundary that were initially prescribed as "no-flow" nodes, were re-designated as "prescribed head" nodes. They were assigned values based on the prior simulated results.

Comparison of the figures indicates that the lower sandstone unit is insensitive to perturbations in the recharge value. The units above the shale unit do respond to changes in recharge values. As shown in Figures 7.10 and 7.12, the largest changes occur for the 520-foot and 530-foot contours. The magnitudes of the changes in the potentiometric contours decrease downgradient of the 520-foot contour with the 480 and 490-foot contours changing insignificantly. Comparison of Figures 7.11 and 7.13

LEGEND

**520 SIMULATED POTENTIOMETRIC
CONTOUR (feet A.M.S.L.)**

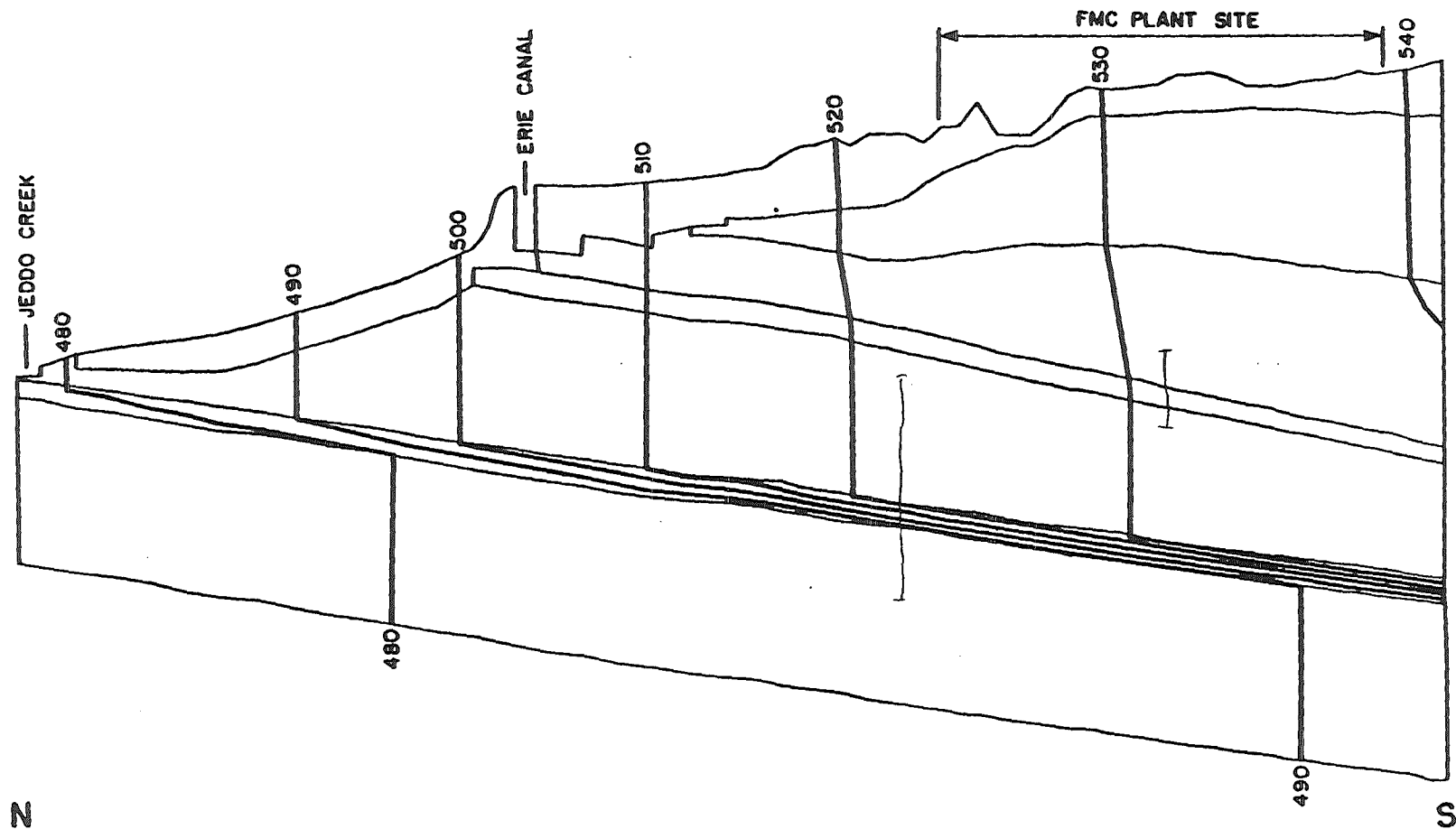


figure 7.8

POTENTIOMETRIC CONTOURS - RUN 2
FMC-Middleport Plant Site

LEGEND

10 SIMULATED PRESSURE
HEAD CONTOUR (feet)

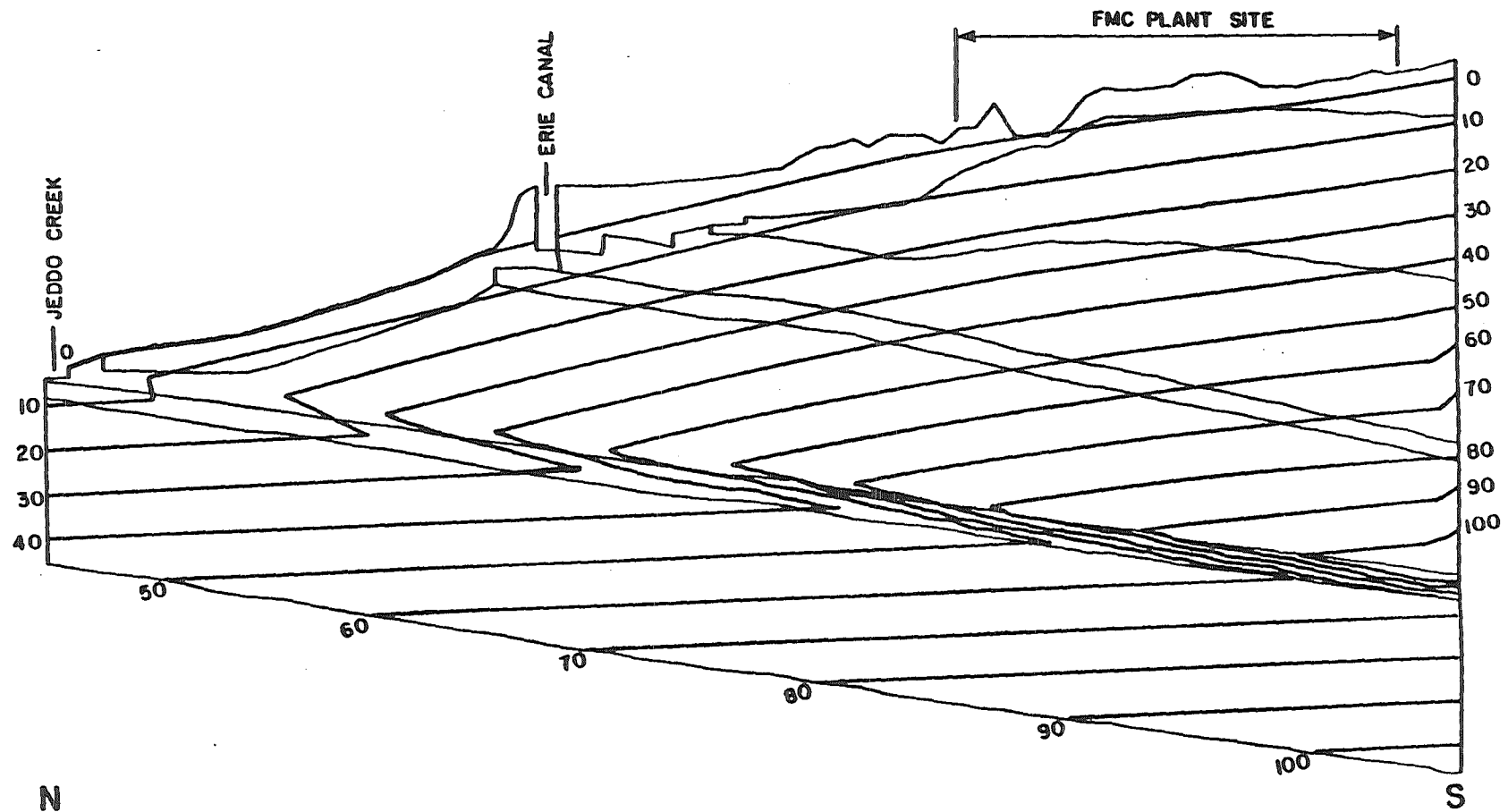


figure 7.9

PRESSURE HEAD-RUN 2
FMC-Middleport Plant Site

LEGEND

520 SIMULATED POTENTIOMETRIC
 CONTOURS (feet A.M.S.L)

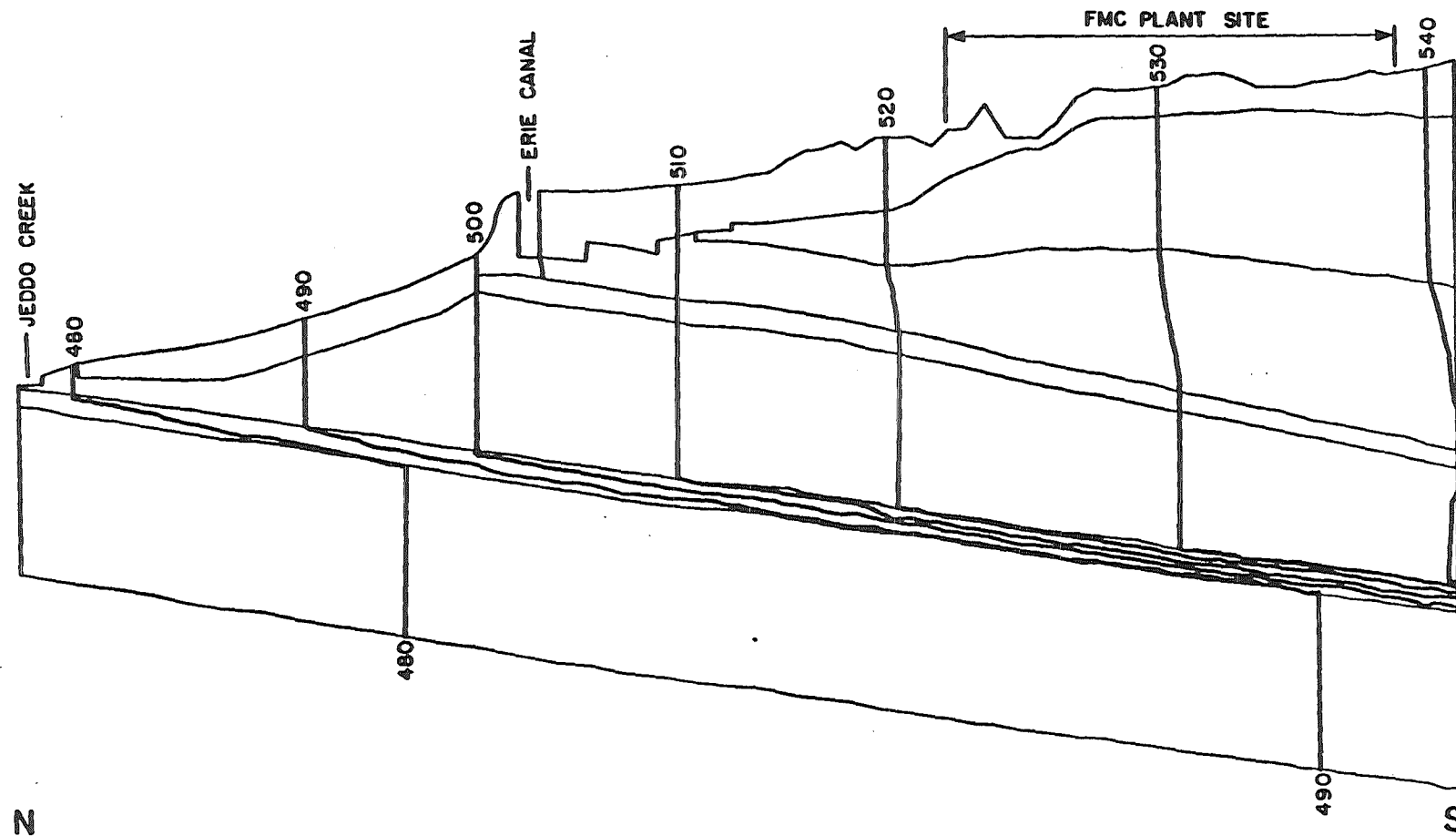


figure 7.10

POTENTIOMETRIC CONTOURS-RUN 3
FMC-Middleport Plant Site

LEGEND

10 SIMULATED PRESSURE
HEAD CONTOUR (feet)

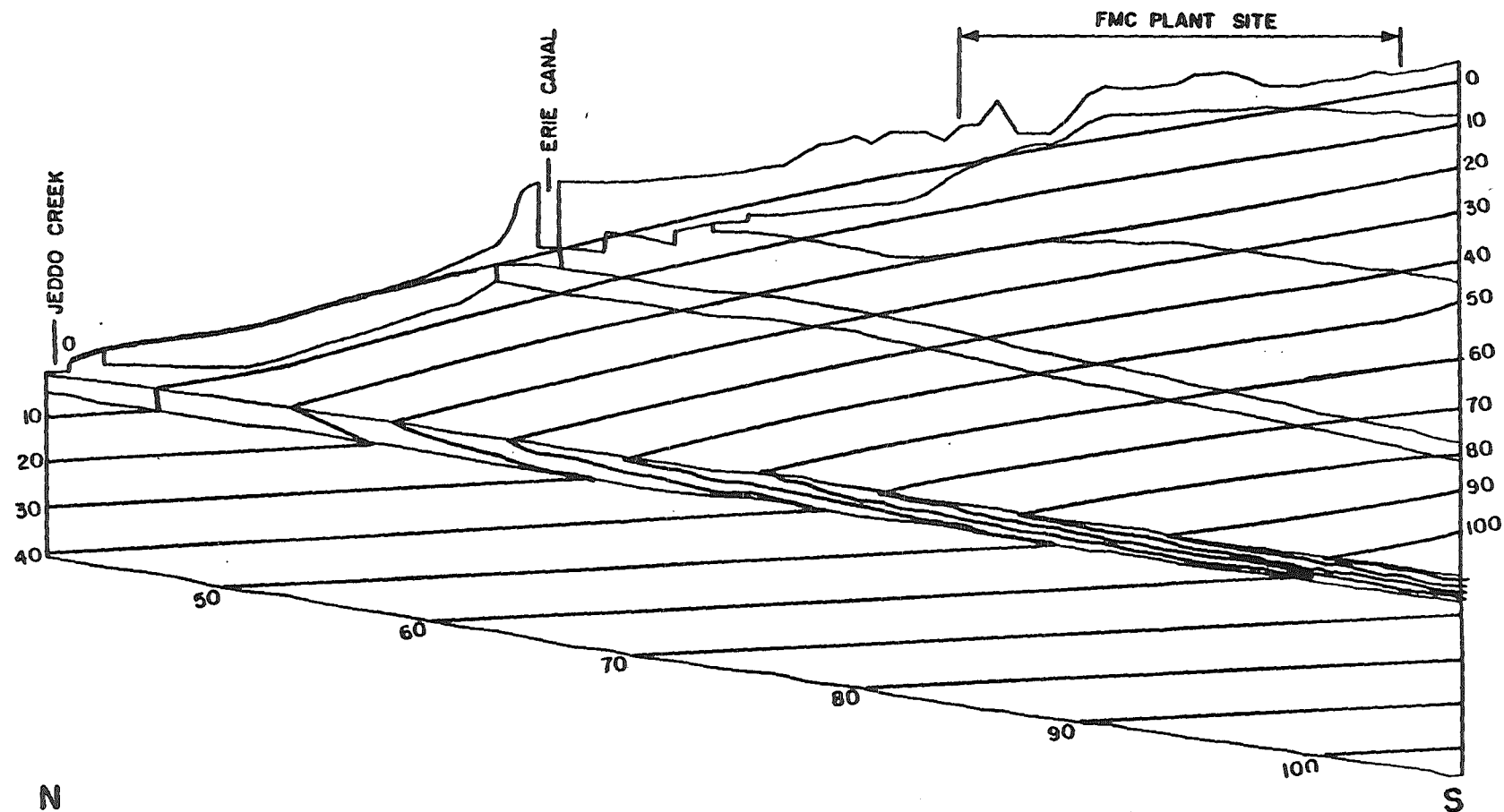


figure 7.11

PRESSURE HEAD-RUN 3
FMC-Middleport Plant Site

LEGEND

**520 SIMULATED POTENTIOMETRIC
CONTOURS (feet A.M.S.L.)**

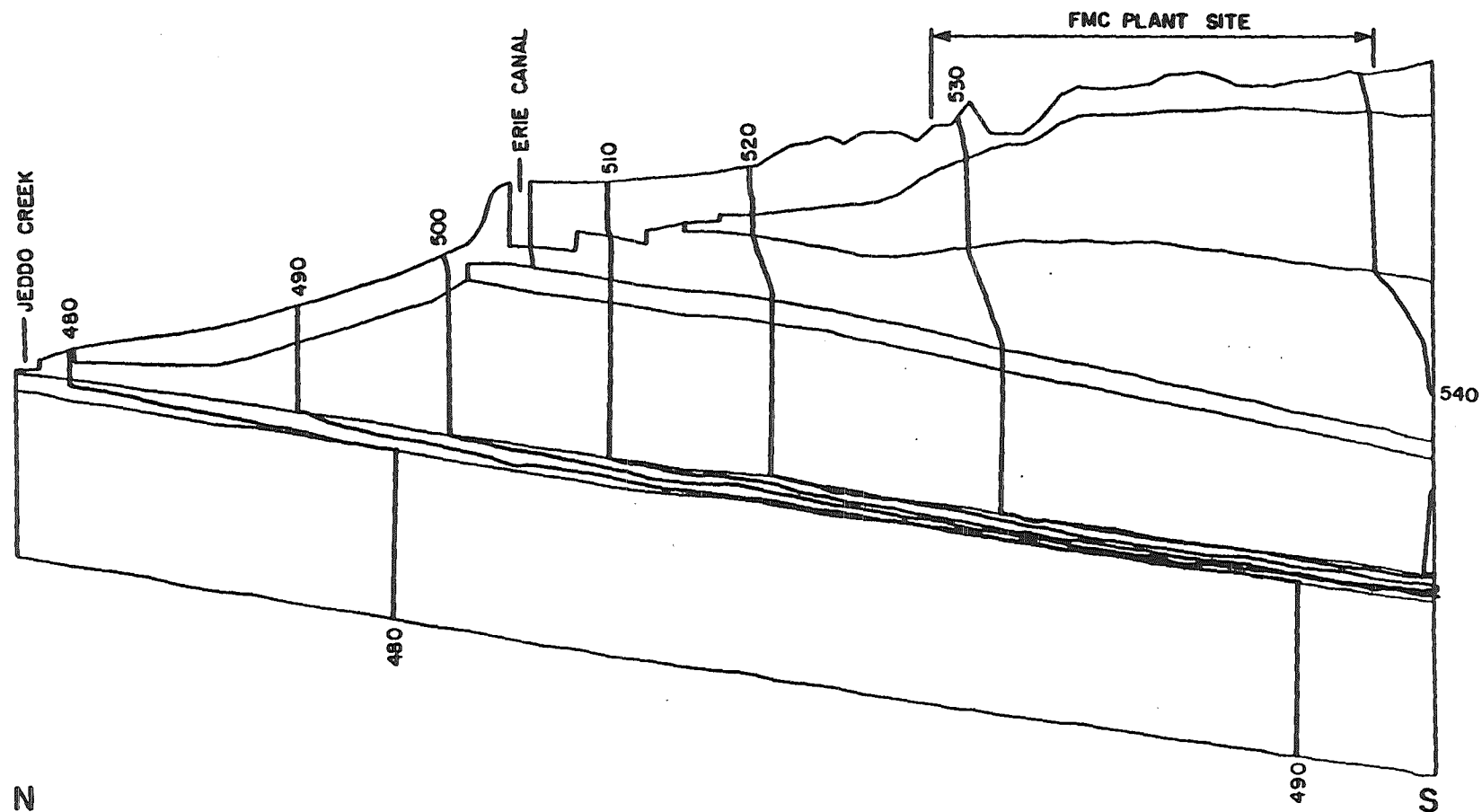


figure 7.12

POTENTIOMETRIC CONTOURS-RUN 4
FMC-Middleport Plant Site

LEGEND

10 SIMULATED PRESSURE
HEAD CONTOUR (feet)

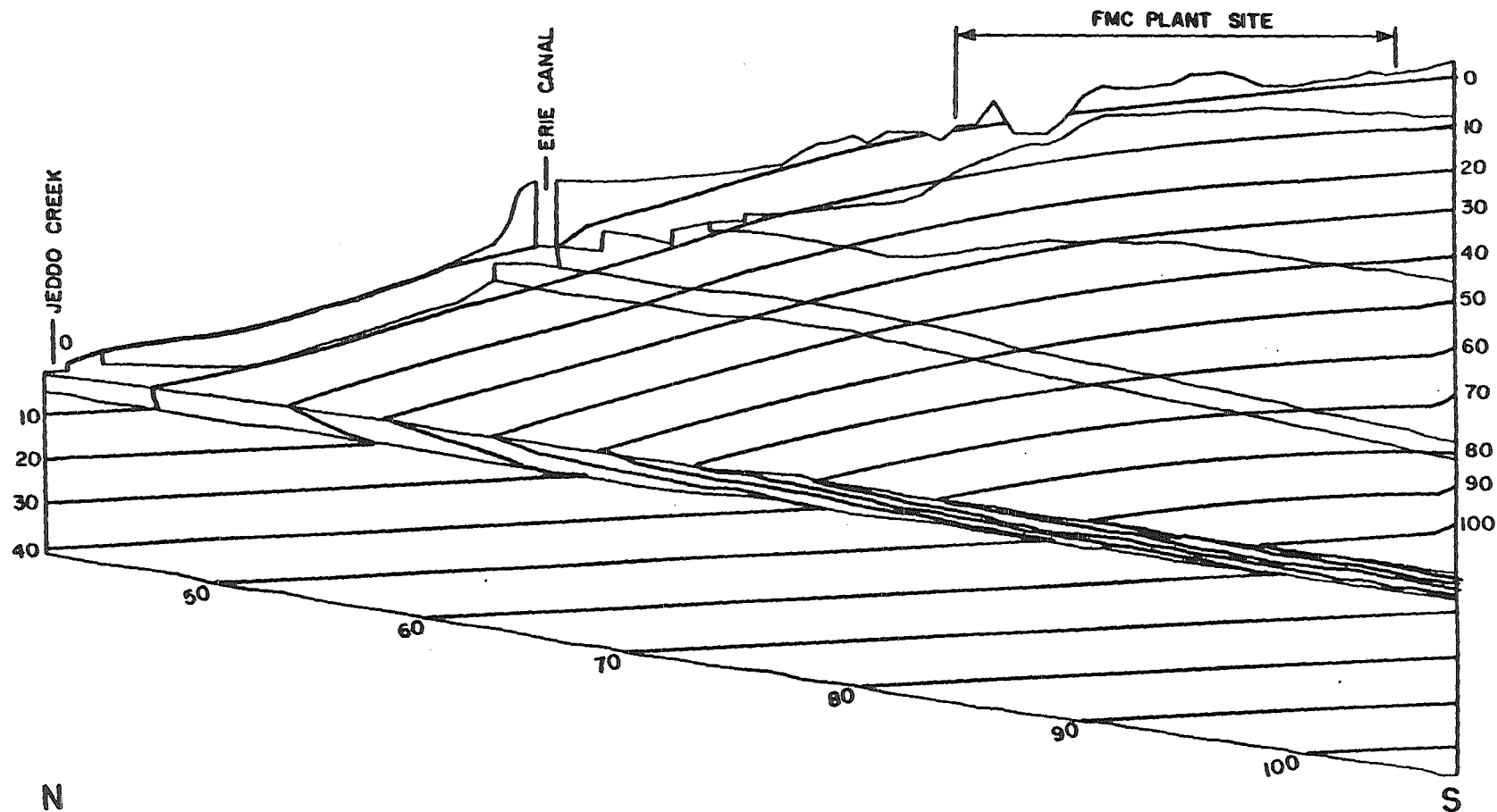


figure 7.13

PRESSURE HEAD-RUN 4
FMC-Middleport Plant Site

indicates that an increase in the recharge from 0.5 to 3.0 inches/year creates a slight mounding of the groundwater beneath the Site, with discharge of groundwater then occurring into the Central Impoundment. As indicated by the discussion presented in Section 5.3.3, this potential for discharge to the Central Impoundment would be possible during periods of high groundwater level and corresponding low surface water level in the Central Impoundment.

In general, the overall pattern of the groundwater flow system in the units above the shale remained essentially the same and is slightly sensitive to changes in recharge values.

To more accurately simulate the measured potentiometric head in the lower sandstone unit, a head loss of 15 feet was applied across the shale layer on the downgradient boundary. It was deemed that a 50-foot head loss at this point was not appropriate because:

- i) a 50-foot head loss on the downgradient boundary would mean that most of the lower sandstone unit would be in an unsaturated condition, and
- ii) the close proximity of the shale unit to the surface increases the probability that the shale would be more fractured at the downgradient boundary than the shale at depth.

The simulation results are shown on Figures 7.14 through 7.16. Comparison of these figures with the prior simulations indicate:

- i) minor changes in the 540-foot contour as a result of redefinition of the four nodes initially defined as no-flow nodes to prescribed head nodes, and
- ii) a significant change in the potentiometric contours in the lower sandstone unit but insignificant changes in the units above the shale unit. This change included the simulation of a small zone with unsaturated flow conditions at the downgradient boundary. The presence of an unsaturated zone in the lower sandstone at the downgradient boundary would not influence chemical transport in the overlying units. An increase in vertical gradient through the shale unit did not significantly increase the quantity of groundwater flowing through the shale. Little flow occurs through the shale. The flow lines simulated from the upgradient and downgradient boundaries of the Site, above the shale, remained unchanged throughout the modeled cross-section. Below the shale, the velocities approximately doubled throughout the lower sandstone unit. This uniform change reflects influx of water from the upgradient lower sandstone boundary.

LEGEND

10 SIMULATED PRESSURE
HEAD CONTOUR (feet)

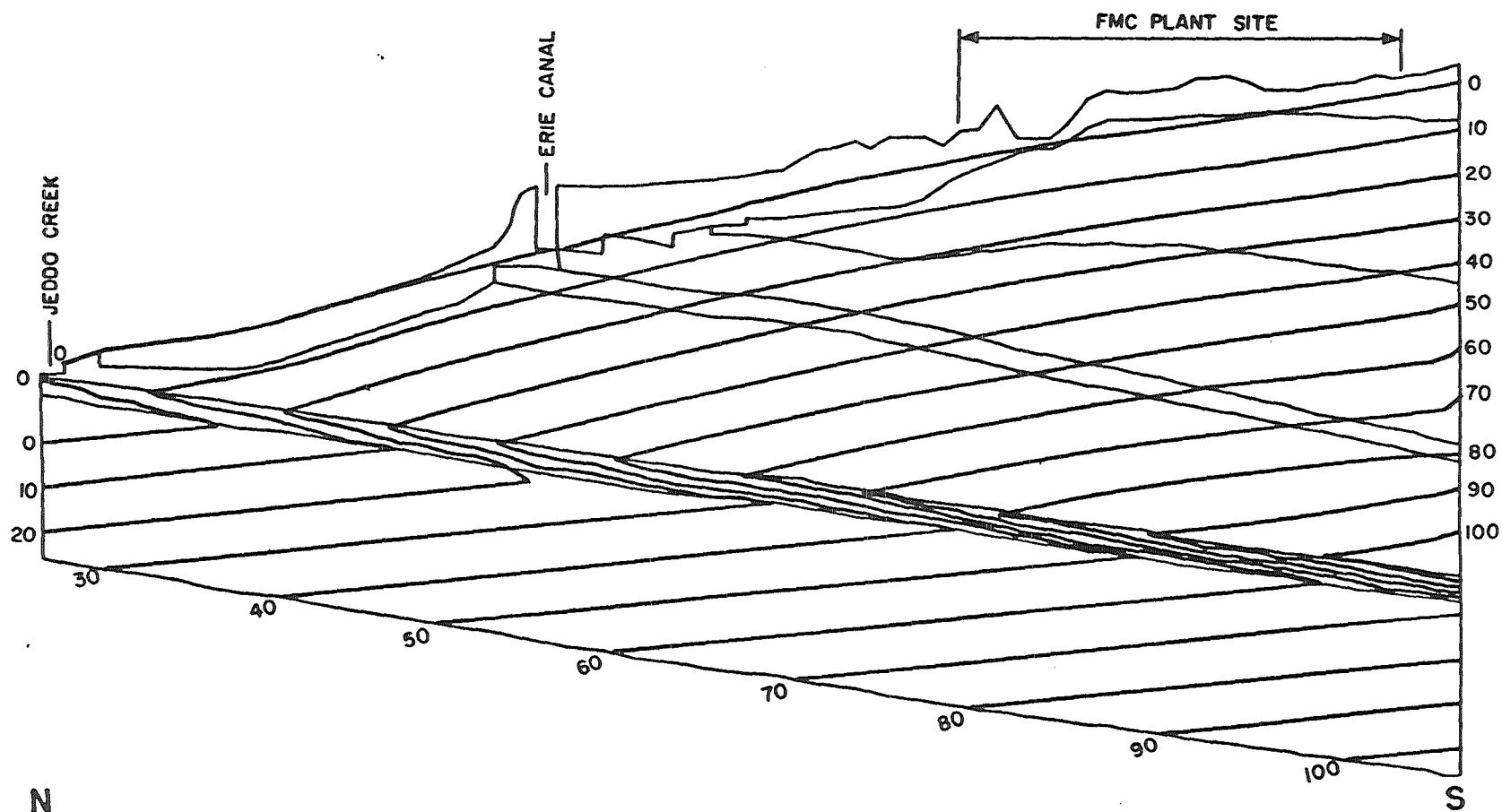
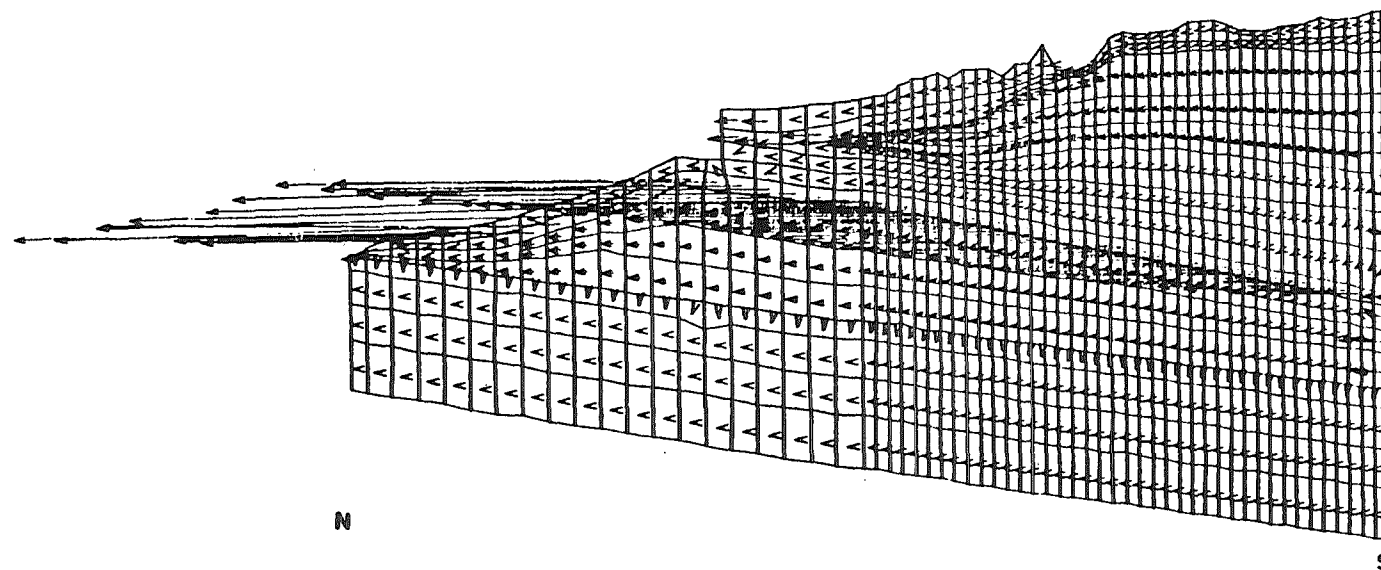


figure 7.15

PRESSURE HEAD - RUN 5
FMC-Middleport Plant Site



SCALE FOR VELOCITY VECTORS



ft / day

CRA

1539-00-1/03/07

figure 716
DARCY VELOCITIES - RUN 5
FMC - Middleport Plant Site

In summary:

- i) The simulated system behaved as two systems separated by the shale unit.
- ii) Changes in the parameters of the hydrogeologic units above the shale unit did not significantly impact the lower sandstone unit and vice versa.
- iii) The system is topographically controlled.
- iv) The groundwater simulation (model) adequately described the measured potentiometric data in the upper bedrock zones.

To add accuracy to the simulated potentiometric surface at Well R-8, when compared to the measured values, the prescribed head nodes below the shale might be further adjusted. However:

- i) no actual potentiometric data were available for the lower sandstone unit at the boundaries,
- ii) the changes would not significantly alter the flow pattern in the units above the shale, as shown and discussed in the prior pages, and

iii) it was not known which hydrogeologic unit the measured potentiometric value at Well R-8 represented due to averaging over the 50-foot monitored interval.

Therefore, no further modifications of the flow system parameters were simulated, particularly because chemical migration would be limited to the bedrock units above the shale unit, as shown in the subsequent sections.

7.5 CONCEPTUALIZATION OF CHEMICAL TRANSPORT

The areas where wastes were historically disposed of or deposited are shown on Figure 1.3.

For simulation of chemical transport, the following assumptions were made:

- i) The entire northern portion of the Site, as shown on Figure 7.17, was assumed to have contributed chemicals to the system from time zero. Time zero represents the time when deposition of wastes containing arsenic first occurred. This is a conservative assumption because,
(1) the arsenic-containing wastes were deposited from approximately 1921 to 1965 and therefore, all arsenic-containing waste did not start to contribute at time zero; (2) wastes were not deposited over the entire northern portion of the Site.

N-757

SITE "F"

HYDRAULIC CONDUCTIVITIES PER PUMPING TEST RESULTS

PUMPING TEST NO. 1

TRANSMISSIVITY CALCULATIONS BASED ON EARLY TIMES

<u>WELL NO.</u>	<u>*T (EARLY TIME) m²/s</u>
OW1-86	7.9×10^{-5}
MB16-2	4.1×10^{-5}
MB16-3	3.8×10^{-4}
PW 1	1.6×10^{-5}
MB 13-2	9.2×10^{-5}
MB 13-3	1.3×10^{-4}
WAT 7-1	1.4×10^{-4}
WAT 7-4	3.7×10^{-5}
WAT 7-5	2.9×10^{-5}
WAT 7-6	2.1×10^{-5}

Geometric Mean (Based on Early Times): $6.1 \times 10^{-5} \text{ m}^2/\text{s}.$
 ** Trow Mean: $4.1 \times 10^{-5} \text{ m}^2/\text{s}.$
 *** CRA Suggested Mean: $1.3 \times 10^{-5} \text{ m}^2/\text{s}.$

Hydraulic Conductivity

$$K = \frac{6.1 \times 10^{-5}}{(5 \text{ m}) (\text{per CRA})^{***}} = 1.2 \times 10^{-5} \text{ m/s.}$$

or

$$1.2 \times 10^{-3} \text{ cm/s.}$$

TROW
 10^{-2} - high
 10^{-3} - iso
 10^{-4} - mod
 2

* Ex. H-40A, Appendix 10	K per Trow: $1.3 \times 10^{-2} \text{ cm/s.}$ (Ex. H-40, p. 53)
** Ex. H-40, p. 53	K per CRA: $2.6 \times 10^{-4} \text{ cm/s.}$
*** Ex. B 617, p. 44	(Transcript Vol. 106, p. 25476
**** (Transcript Vol. 106, p. 25475)	and Ex. B 617, p. 44)

SITE "F"

HYDRAULIC CONDUCTIVITIES PER PUMPING TEST RESULTS

PUMPING TEST NO. 1

TRANSMISSIVITY CALCULATIONS BASED ON EARLY TIMES
USING SELECTED OBSERVATION WELLS ONLY, AS SELECTED
BY CRA*

<u>WELL NO.</u>	<u>**T (EARLY TIME) m²/s</u>
OW1-86	7.9×10^{-5}
MB16-2	4.1×10^{-5}
PW 1	1.6×10^{-5}
MB 13-2	9.2×10^{-5}
MB 13-3	1.3×10^{-4}

Geometric Mean (Based on Early Times): $5.7 \times 10^{-5} \text{ m}^2/\text{s}.$
*CRA Suggested Mean: $1.3 \times 10^{-5} \text{ m}^2/\text{s}.$
***Trow Mean: $4.1 \times 10^{-5} \text{ m}^2/\text{s}.$

Hydraulic Conductivity

$$K = \frac{5.7 \times 10^{-5} \text{ m}^2/\text{s}}{5 \text{ m. (per CRA)****}} = 1.1 \times 10^{-5} \text{ m/s.}$$

or

$$\boxed{1.1 \times 10^{-3} \text{ cm/s.}}$$

* Ex. B 617, p. 44
** Ex. H-40A, Appendix 10
*** Ex. H-40, p. 53
**** Transcript Vol. 106
p. 25475

K per Trow: $1.3 \times 10^{-2} \text{ cm/s}$
(Ex. H-40, p. 53)

K per CRA: $2.6 \times 10^{-4} \text{ cm/s}$
(Transcript 106, p. 25476
and Ex. B 617, p. 44)

1

7

N759

**MIDDLEPORT SITE INVESTIGATION
FINAL REPORT
VOLUME I - TEXT**

**FMC Corporation
Middleport, New York**

May, 1987

**May 1987
Ref. No. 1539-08**

CONESTOGA-ROVERS & ASSOCIATES

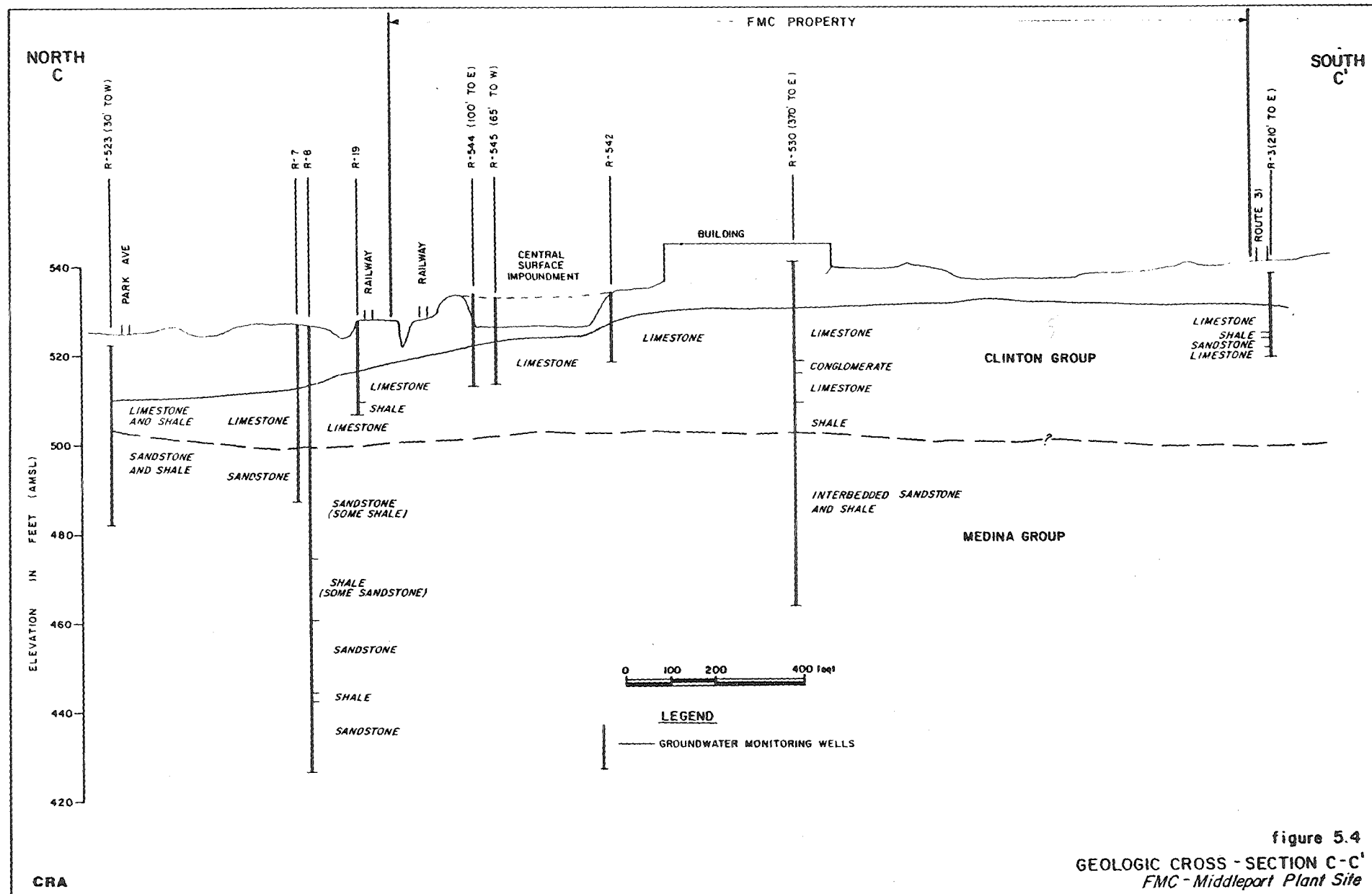


figure 5.4
GEOLOGIC CROSS - SECTION C-C'
FMC-Middleport Plant Site

In aquifers, groundwater flow is generally in a horizontal direction. Most evaluations of the hydraulic conductivity of geologic strata reflect the horizontal hydraulic conductivity. However, in clay rich strata, the dominant flow direction generally is vertical. The hydraulic conductivities determined by laboratory methods for such units are a measure of the vertical hydraulic conductivity.

There are four hydrostratigraphic units which have been defined at the Site, corresponding with the geologic units discussed previously. The hydrostratigraphic units, beginning with the uppermost, are:

- i) Overburden, an aquitard consisting of native and fill materials;
- ii) Upper Limestone Unit, an aquifer consisting of limestone with shale interbeds;
- iii) Interbedded Sandstone and Shale Unit, an aquifer consisting of fine-grained, thin to massive bedded, red sandstone; and
- iv) Queenston Shale, an aquitard that is considered the base of the flow system.

A series of hydrogeologic cross-sections have been prepared to illustrate the distribution of the hydrostratigraphic units. They are presented on Figures 5.10 through 5.13, inclusive. The location of the cross-sections are shown on Figure 5.1.

5.3.2 Overburden

Based upon this study and previous investigations, it has been shown that perched groundwater occurs within the overburden at the Site. The more permeable zones within the till and fill are saturated.

Horizontal hydraulic conductivity values of the overburden unit have been determined by slug-injection tests and estimated from grain-size distribution curves using the Hazen Method. These data are summarized on Tables 4.5 and 4.6. Six undisturbed samples of this unit were also tested during this study to determine the vertical hydraulic conductivity. In previous studies, six samples were also tested to determine vertical hydraulic conductivity. The laboratory testing results are summarized on Table 4.7.

Examination of the data on Table 4.5 indicates that the horizontal hydraulic conductivities, based upon the slug test data, vary from 1.2×10^{-3} to

2.5×10^{-5} cm/sec. The estimates based on the method of Hazen, although not totally applicable due to the clayey nature of the unit, are lower, varying from 1.1×10^{-4} to less than 1×10^{-7} cm/sec. Of interest is the fact that the estimates of hydraulic conductivity shown on Table 4.5 for the fill and native material are similar.

The vertical hydraulic conductivity values shown on Table 4.7 range from 2.3×10^{-7} to 8.4×10^{-9} cm/sec. These results indicate a large degree of anisotropy in the overburden unit.

Based upon the results of the hydraulic monitoring conducted as a part of the Investigation, groundwater water elevation contours have been prepared and are illustrated on Figures 5.14 through 5.16, for Site conditions on February 10, June 25 and November 3, 1986, respectively. Examination of these figures indicates that there are only minor elevation differences in the water table in the overburden unit over the time period represented by the contour maps. Horizontal groundwater flow within the overburden is generally in a northwesterly direction. Average horizontal hydraulic gradients vary from 0.011 to 0.016. Groundwater flow appears to converge in the vicinity of the Western Surface Impoundment. This is likely due to the effect of the underdrain system and sanitary sewer french drain, which are located at the northwest corner of the Site.

Using the range of horizontal gradients of 0.011 to 0.016, a conservative horizontal hydraulic conductivity of 1×10^{-3} cm/sec and an effective porosity of 0.25, average groundwater flow velocities of 4.4×10^{-5} to 6.4×10^{-5} cm/sec (0.12 to 0.18 ft/day) were calculated. The flow velocities increase in the immediate vicinity of the sanitary sewer french drain collection system due to the steeper slope of the groundwater table.

5.3.3 Upper Limestone Unit

Groundwater occurrence within the Upper Limestone Unit is primarily along bedding fractures and joints. Although not readily detected in coring, vertical fractures will also exist close to the bedrock surface. Some vugs were also encountered during the drilling program; these were generally infilled with calcite, and, therefore, would not transmit groundwater.

The hydraulic conductivity of the Upper Limestone Unit was determined using single- and double-packer tests. The packer test results are summarized on Tables 4.8 and 4.9, while the intervals tested and hydraulic conductivities determined are shown on the hydrogeologic cross-sections (Figures 5.10 through 5.13). These data indicate that the horizontal hydraulic conductivities vary

from 1.1×10^{-3} cm/sec to less than measurable by this method.

This wide variation of hydraulic conductivity is related to the degree, extent and continuity of the bedding plane and vertical fractures. There does not appear to be a trend in the hydraulic conductivity with depth within this unit. Rather, the range is due to the areal distribution of the fractures.

The groundwater level contours for the shallow bedrock are shown on Figures 5.17 through 5.19, inclusive. The majority of monitoring data used to construct these figures are derived from wells completed in the Upper Limestone Unit. However, some of the wells on the northern property boundary and off-Site are completed in both the Upper Limestone Unit and the underlying Interbedded Sandstone and Shale Unit. Due to similar hydrogeological characteristics of the shallow bedrock, these wells were grouped together.

Examination of these figures indicates that there are only minor differences in the distribution of hydraulic head within the shallow bedrock. Groundwater flow across the Site is generally in a northwest direction. The hydraulic gradient is relatively constant, varying from 0.0125 to 0.0107.

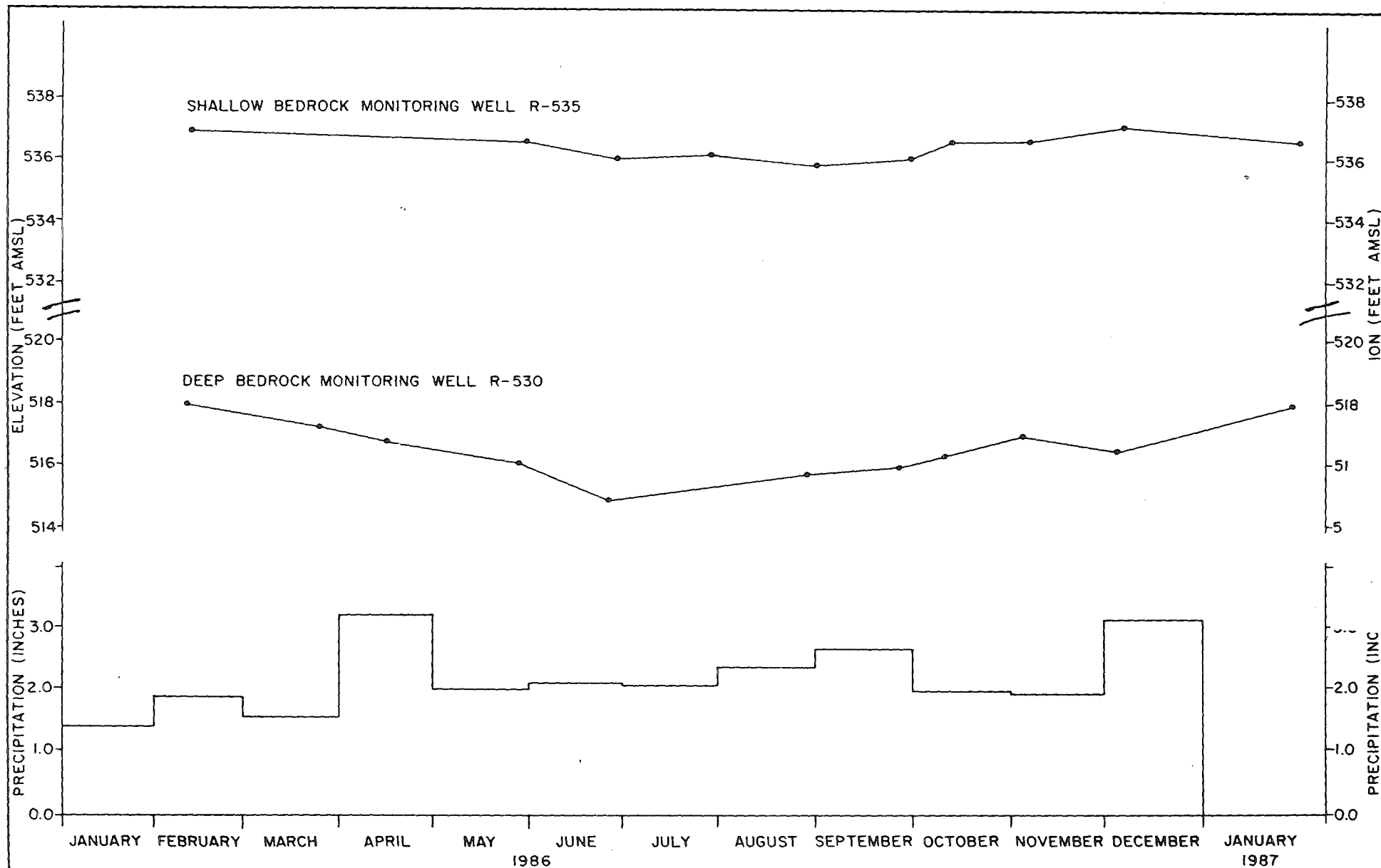


figure 5.20

WATER LEVEL HYDROGRAPH R-530 AND R-535
FMC - Middleport Plant Site

CRA

REVISION 1, AUGUST 10, 1987

the impoundment) at upgradient Wells R-534 (536.43) and R-539 (535.18) were still higher than the impoundment water level (535.07).

5.3.4 Interbedded Sandstone and Shale Unit

The Interbedded Sandstone and Shale Unit underlies the Upper Limestone Unit across the Site. North and northwest of the Site, the Upper Limestone Unit has been eroded away and the Interbedded Sandstone and Shale form the uppermost bedrock unit. Groundwater occurs within this unit primarily in bedding plane fractures. Within the sandstone beds, groundwater will also occur in intergranular pores. However, due to the relative fineness of the sand grains, this is a minor component of the total groundwater flow.

This unit is made up of discrete sandstone and shale beds that are likely continuous across the Site. A major feature of hydrogeologic importance is the fracture zone that was noted in the stratigraphic logs and during the pressure packer tests. Therefore, it is possible to divide this unit into a number of subunits, as described in the computer modeling section of this report (Section 7).

Hydraulic conductivity values for the Interbedded Sandstone and Shale Unit were determined by the

double-packer pressure tests in the deeper bedrock testholes. The test results are presented on Table 4.7 and are illustrated on the hydrogeologic cross-sections (Figures 5.10 to 5.13). The calculated hydraulic conductivities varied from 4.0×10^{-3} cm/sec to less than measurable with the monitoring equipment used. For the most part, average hydraulic conductivities for this unit were on the order of 1×10^{-4} to 1×10^{-5} cm/sec. There does not appear to be a definite trend in the hydraulic conductivity values with depth, other than the fact that the largest hydraulic conductivity values in the deep holes were encountered at the major fracture zone.

Water level measurements were taken in the intermediate and deep bedrock observation wells completed in this unit, and are presented in Appendix J. The resulting groundwater elevations for February, June and November 1986 are shown on Figures 5.24 through 5.26 respectively. While there are too few data points to construct contours, water levels are highest in the vicinity of Wells R-530, R-531 and R-637 and decrease in a northwest direction. It is noted that the water level in Well R-533 for November 1986 was anomalously low, probably due to non-recovery of this well from the pumping test conducted in August 1986.

Examination of the water level hydrographs for Wells R-530 and R-8 (Figures 5.20 and 5.21) illustrates

that the seasonal fluctuations in this unit are small (approximately three feet at Well R-530). However, water levels in the shallow bedrock (Upper Limestone Unit) are at higher elevations, indicating a downward gradient. Therefore, a potential exists for chemical migration to the deeper bedrock.

In order to determine the degree of hydraulic connection between the upper and lower bedrock units, pumping tests were conducted on deep bedrock Wells R-532 and R-533. The details of the pumping tests have been previously described in Section 4.5.2.2. The testing of Well R-533 indicated that very little recharge to the well occurred during the test as the water level was lowered to the top of the packer and did not recover during the test period. In fact, water levels in the well did not fully recover to pre-test levels during the remainder of the hydraulic monitoring (see Appendix J). These results indicate that the hydraulic conductivity of the deep bedrock at this location is very low, and that fractures in the rock, if any, are not continuous and extensive.

The results of the pumping test of Well R-532 and the associated water level monitoring are illustrated on Figure 4.13. This figure shows that the water level in the well declined very rapidly until the top of the packer was reached. At that level it was necessary to reduce the

pumping rate to approximately 0.25 gpm. This rapid drawdown is indicative of a low hydraulic conductivity material. Analysis of the earlier portion of the drawdown data by the modified non-equilibrium method (Cooper and Jacobs, 1946) yielded an average hydraulic conductivity of the monitored interval of 2×10^{-7} cm/sec. This low hydraulic conductivity would also indicate that the horizontal bedding plane fractures are not extensive in this area. This postulation is also borne out by the lack of any measurable drawdown in Well R-10, which is completed in the sandstone unit.

No measurable drawdown occurred in Wells R-15 and R-539 through R-541, all of which were completed in the Upper Limestone Unit. This lack of drawdown indicates that there is limited vertical fracturing in the Interbedded Sandstone and Shale Unit in this locality and, therefore, there is little potential for vertical groundwater flow between the upper and lower bedrock units.

5.4 GROUNDWATER FLOW SYSTEM

The Site is situated in a local groundwater recharge area. Precipitation infiltrating into the Overburden Unit will migrate downward into the underlying

Upper Limestone Unit. Due to the relatively low hydraulic conductivity of this unit, recharge is limited. There is also a horizontal component of groundwater flow in the overburden in a north-northwesterly direction.

Groundwater infiltrating into the Upper Limestone Unit will flow in a north-northwest direction, primarily along horizontal to sub-horizontal bedding plane fracture zones. There appears to be a vertical component of flow in the Upper Limestone Unit as evidenced by the lower hydraulic heads in the Interbedded Sandstone and Shale Unit. However, the interconnection pumping test conducted during the Investigation indicated very little hydraulic connection between these two units.

Groundwater flow within the Interbedded Sandstone and Shale Unit is also expected to be in a north-northwest direction. The flow will be primarily along the fractures in the sandstone beds.

Certain Site facilities have had an effect on the groundwater flow regime in the Overburden and Upper Limestone Unit. The most important alteration of the natural flow system is the underdrain beneath the Western Surface Impoundment. This underdrain and pumping system has created a cone of depression in the upper bedrock groundwater

- The native, fine-grained soils in the overburden have excellent attenuation capacity for arsenic. This is consistent with the measured concentrations of arsenic in the off-Site groundwater regime, as described in Section 6.7.

6.7 GROUNDWATER

Appendix P contains the historical groundwater analytical data for the Site. Appendix Q details the groundwater data collected during the Investigation, including samples collected from private wells. Appendix R presents the groundwater data collected during SWIM Program from Wells R-538 through R-549, inclusive. The following sub-sections of this report discuss the results of the groundwater sampling program on a parameter by parameter basis.

6.7.1 Arsenic

The arsenic results for Rounds 1 through 3 of the groundwater monitoring program are shown in Figures 6.6 to 6.8 for the overburden, shallow bedrock and intermediate/deep bedrock, respectively.

Examination of Figure 6.6 indicates that in the Overburden Unit, the measured concentrations of arsenic

this stream. Jeddo Creek is larger and could support game fish and could potentially be used for swimming and boating which could increase the potential for human contact. Neither the tributary nor Jeddo Creek is used as a potable water source. Because of the limited potential for exposure to these waters and the fact that surface discharges from the site to these receiving waters is regulated under a SPDES permit, these potential pathways will not be included in further assessment. A small component of surface run-on water from the northeast corner of the site which discharges under the railroad on the north boundary will be further addressed in a following section of this report.

Groundwater under the Site can be contaminated as a result of surface water leaching through contaminated soils and subsurface wastes or by leakage from surface impoundments. Overburden soils are relatively impermeable and horizontal flow in the discontinuous saturated fill areas is minimal; this zone is not considered to be a water bearing aquifer. Therefore, the exposure pathway is not complete via this zone. No groundwater from the bedrock aquifer is used on Site for human consumption; therefore, there is no potential human on-Site exposure to groundwater. Bedrock groundwater does flow off Site. The potential exposure of local residents and the environment to bedrock groundwater will be discussed.

A small area on the northeast boundary of the Site drains to a culvert under the Conrail mainline railroad tracks located north of the Site and discharges into the ditch north of the railroad. This ditch is heavily vegetated and overgrown but with difficulty is accesible to the general public.

The concentrations reported in water for sampling sites in the ditch are summarized in Table 8.8. Although this water would not be used for drinking, the concentrations reported were compared to drinking water standards as a conservative point of reference. Considering the low potential for exposure to this water, and the fact that concentrations approach criteria for drinking water in a comparatively small mixing zone, potential exposure to this water is not a significant health concern.

Off-Site Exposure - Bedrock Groundwater: Comparisons of the chemical concentrations detected in off-Site public and private bedrock wells with standards and criteria are presented in Table 8.9. None of the wells exceeded the criteria and there was no evidence of impact by indicator chemicals from the Site.

Total Recoverable Phenolics exceeded the criterion for phenol in one on-site well at one sampling period. The dilution and attenuation that would occur during any migration off-site would bring this concentration far below this criterion.

9.0 CONCLUSIONS

The Site Investigation has provided information necessary to assess and evaluate the extent of hazards and degree of risk posed to public health and the environment by past and present activities and conditions at the Site. Based on both historical information and data generated during the Investigation, the following conclusions are relevant:

- i) The Site, approximately 79 acres in size, contains surficial soil contamination and subsurface waste over large portions of its northern half. The southern half of the Site is apparently essentially uncontaminated.
- ii) Three Surface Impoundments and a system of open unlined channels collect and retain potentially contaminated surface water runoff from the northern half of the Site. Collected surface water runoff is treated by an on-site treatment plant located in the northwestern sector of the Site prior to discharge to a tributary of Jeddo Creek. The treated water discharge is regulated under a State Pollutant Discharge Elimination System permit.

A portion of the unlined channel system, which conveys surface water from the Eastern and Central Impoundments to the treatment plant, parallels the Conrail railway tracks which are adjacent to the northerly boundary of the Site. This channel intrudes on Conrail property north of the Site fence for a section of its length. Analyses of sediments collected from the channel ditch indicate the presence of elevated concentrations of arsenic and, to a lesser degree, lead.

A second channel originates north of the Eastern Impoundment south of the Conrail property. This channel collects surface water runoff from agricultural lands east of the Site and conveys it through a culvert under the Conrail railway to a ditch north of the Conrail railway tracks. Analyses of sediments collected in this ditch indicate the presence of elevated levels of arsenic and, to a lesser degree, lead and carbofuran. Concentrations of these parameters are generally highest near the culvert and decrease downstream. The chemical concentrations in the north ditch are considerably less than those measured in the south ditch.

- iii) On-Site surface water sampling and analytical results indicated the presence of arsenic and various pesticides in most samples. Six volatile organics were

also identified as present with only methylene chloride and chloroform present in more than one sample. Lead was identified in one sample. The concentrations of the surface water contaminants decreased during transport via the surface drainage ditch network to the water treatment plant, indicating that the processes of volatilization, adsorption and dilution were attenuating these materials.

Sampling and analyses of off-site surface waters in the ditch north of the Conrail railway revealed the presence of arsenic, BHC and carbofuran. The highest concentration occurred immediately at the railway culvert and reduced dramatically in a short distance downstream from the culvert to levels that approached or met drinking water standards.

- iv) Soil samples were collected at 24 locations at the Site in suspected areas of prior waste disposal or storage. Analyses of collected samples indicated that arsenic, and to a lesser degree, carbofuran/7-hydroxybenzofuran, were the major contaminants present. Site soils are subject to erosional and precipitation effects and, thus, have the potential to significantly impact surface water and groundwater.

- v) It was determined that the major geologic units beneath the Site were the overburden (both fill and native soil) underlain by the Upper Limestone Unit which is the first bedrock unit encountered which in turn is underlain by an interbedded sandstone and shale unit.

Due to the relatively fine-grained nature of the overburden material, the horizontal and vertical hydraulic conductivity of the Overburden Unit is relatively low. Over most of the Site, there is a downward vertical hydraulic gradient between the Overburden Unit and the underlying bedrock.

The Upper Limestone Unit is the uppermost aquifer unit identified beneath the Site. The hydraulic conductivity of this unit is highly variable due to the degree, extent and continuity of fractures. While this unit does permit groundwater flow, the unit is not considered to be a viable water supply aquifer and the volume of water available is limited in comparison to the major regional aquifer, the Lockport Formation. The Lockport, however, does not extend beneath the Site. Local use of the Upper Limestone Unit as a water supply is limited as a municipally supplied water system is available in Middleport.

The Upper Limestone Unit is influenced by the Western Impoundment underdrain system which extracts groundwater from the unit and by the Eastern Impoundment which contributes water into the Formation.

The Interbedded Sandstone and Shale Units also exhibit variable flow characteristics depending upon the fracture network. Testing indicates that the lower depths of this unit are not hydraulically connected to the upper units of the bedrock.

The direction of flow in the Overburden and Upper Limestone Units is north/northwesterly and is influenced significantly by the groundwater extraction systems located in the northwest corner of the Site.

- vi) Sampling and analysis of private wells in the vicinity of the Site has confirmed that past activities at the Site have not adversely impacted water quality in these wells.
- vii) Analyses of groundwater samples collected from overburden and bedrock monitoring wells at and in the vicinity of the Site have indicated that contaminants of relevance in the hydrogeologic environment are arsenic, carbofuran/7-hydroxybenzofuran and methylene

chloride. Significant groundwater contamination is generally limited to on-Site wells and is correlated to former waste storage or disposal practices.

Groundwater monitoring to date has confirmed little off-Site migration of contaminants. The minimal off-Site movement of contaminants in the groundwater is attributable to the following factors:

- i) the ability of the native overburden material to attenuate organic and inorganic contaminants;
- ii) the low permeability of the Overburden Unit, which limits recharge to the underlying bedrock and retards horizontal groundwater flow within the overburden regime;
- iii) the low groundwater flow velocities in the overburden and bedrock; and
- iv) the operation of the underdrain system at the Western Surface Impoundment, which has and continues to capture and remove significant volumes of contaminated groundwater from the Upper Limestone Unit.

Groundwater contamination, for all contaminants of relevance except methylene chloride, is restricted to

the Overburden and Upper Limestone Unit. Methylene chloride is found in a localized portion of the Site in the deeper bedrock.

- viii) It has been determined that underground utilities egressing the Site do not currently provide a preferential pathway for the off-Site migration of contaminants. In fact, past remedial actions undertaken by FMC, most notably the sanitary sewer french drain system and trench cutoff wall at the northwest corner of the Site, have actively controlled the migration of contaminants from the Site.

- ix) Groundwater flow was modeled in cross-section assuming equivalent porous media using the finite element technique for steady-state conditions. The simulated flow system was representative of the measured conditions. Chemical transport utilized the flow velocities from the calibrated flow model. After 50 years from time zero, the initial time when the compound first became available for leaching to the groundwater system, the predicted concentrations using worst case scenario are:

Discharge Point Concentrations (mg/L)
50 years from Time Zero
Erie Canal Jeddo Creek

Arsenic	0.19	0.09
Lead	NM	NM
Carbofuran	0.67	0.34
7-Hydroxybenzofuran	0.46	0.23
Methylene Chloride	138	69

NM - Not Modeled. Concentrations will be significantly less than arsenic.

These values are conservative values assuming no interaction of the compounds with the hydrogeologic environment.

- x) A public health evaluation was performed using environmental data collected from the Plant Site. During the evaluation, indicator chemicals, exposure pathways, potential human exposure points and exposure point concentrations were defined. Exposure point concentrations were then examined to determine if they exceeded applicable and relevant standards, criteria and guidelines.

For present conditions, and future conditions with the exception of bedrock groundwater, it was concluded that there was or would be no significant adverse impact to human health from expected environmental exposure to contaminants present at the Site.

Simulations of groundwater flow and contaminant transport from the Site for 50 years lead to the conclusion that the groundwater between the Site and Jeddo Creek and the Erie Canal would contain concentrations of contaminants, most notably methylene chloride, in excess of water quality criteria for drinking water. Thus, the use of wells from water supply from this area would be precluded. In addition, it is probable that groundwater discharges to surface waters (Jeddo Creek and Erie Canal) would result, at least for methylene chloride, in the exceedence of surface water quality criteria in the future.

- xi) No evidence exists that now or in the future, will the Site significantly adversely impact wildlife populations from areas adjacent to the Site.

The Site, being an industrial facility, inherently would not be expected to support wildlife populations.

xii) The investigation undertaken has provided the information necessary to accurately assess the hydrogeologic characteristics, geochemical conditions and the impact on the public welfare and the environment resulting from the Plant conditions.

All of which is respectfully submitted.

CONESTOGA-ROVERS & ASSOCIATES

Richard G. Shepherd, P. Eng.

James K. Kay, P. Eng.

Michael G. Mateyk, B.Sc.

Klaus Schmidtke, Ph.D., P. Eng.

NIAGARA ESCARPMENT

0 50 100 250 (M)

BURLINGTON SANITARY LANDFILL

BAYVIEW SANITARY LANDFILL

NORTH SERVICE ROAD

HWY 403

GL 22, GL 21, GL 1, GL 23, GL 7, GL 20, GL 29, GL 4, GL 6, GL 2, GL 30, GL 18, GL 31, GL 5, GL 3, GL 8A, GL 28, GL 10, GL 14, GL 9, GL 15, GL 24, GL 11, GL 12, GL 16, GL 13, GL 27, GL 17, GL 26, MB 4, MB 14, MB 6, 7, 8, 9, 10, MB 15, MB 1, MB 2, MB 3, MB 11, MB 12, MB 13, MB 16, MB 17, MB 18, WAT 2, WAT 3, WAT 4, WAT 5, WAT 7, LW 6, LW 5, LW 4, LW 3, LW 2, LW 1, PW 2, PW 3, PW 1, *3, *4, *5, *6, *7, *8, *9, *14, *15

changed by Rennie nodules for 7

reass. resp to pump at PW1

↳ meas. resp
to pump
at PW1

1354-10-13/07/87-34-D-1

EXTRACT FROM FIGURE
Ex. B 617, p. 19

N-761

detailed in Table 4 and
were calculated as

of hydraulic conductivity
section 4.5.2.1.

4.5.2.3 Vertical Ir.

more exhibits

ring Tests

Testing

WBCG

Testing

[illegible]

Several attempts were made to complete the pumping tests for Wells R-532 and R-533 in accordance with Appendix C of the approved Work Plan. The initial pumping test commenced on May 27, 1986. At this time, the wells had not yet been developed after their completion and gray silty sediment still present in the well clogged the submersible pump. The wells were pumped and flushed several times in late July in order to free the wells of any sediment.

During this development sequence, water was collected and discharged to an on-Site sump near tank T11-01 and transported off Site for treatment. As a result of the additional pumping, it was determined that Well R-533 would

not recharge sufficiently above the top of the packer even at a low pumping rate (e.g., 1 gpm). It was determined from the monitoring undertaken that pumping from this formation would not create a discernible change in shallow bedrock water levels in the area.

A single pumping test was completed on Well R-532 on August 19, 1986. Monitored observation wells included Wells R-6, R-15, R-16, R-539 through R-541 and R-543 for vertical definition, and Wells R-8, R-10, R-520 and R-531 for horizontal definition. In the event that a response was observed in either Well R-8 or R-520, it was planned that additional wells would be monitored to correlate with this response. These observation wells differed from those originally proposed in the approved Work Plan since hydraulic data made available from the Investigation testing programs identified the above wells to be more suitable.

Stevens Water Level Recorders (Model No. 68) were installed on Wells R-541, R-531 and R-6 six days prior to performing the actual pumping test at Well R-532. Data collected from the recorders are presented in Table 4.10. Water level recorders were installed on Wells R-539 and R-540 on the day of the test. Pumping was initiated at 10:15 a.m. on August 19, 1986 and continued for 12 hours using a 4-inch diameter submersible pump set at a pumping rate of one gpm. The well was evacuated to the top of the packer

Revision 1
August 10, 1987

not recharge sufficiently above the top of the packer even at a low pumping rate (e.g., 1 gpm). It was determined from the monitoring undertaken that pumping from this formation would not create a discernible change in shallow bedrock water levels in the area.

A single pumping test was completed on Well R-532 on August 19, 1986. *The open interval of well R-532 was from 479.8 to 460.8 feet AMSL.* Monitored observation wells

included Wells R-6, R-15, R-16, R-539 through R-541 and R-543 for vertical definition, and Wells R-8, R-10, R-520 and R-531 for horizontal definition. In the event that a response was observed in either Well R-8 or R-520, it was planned that additional wells would be monitored to correlate with this response. These observation wells differed from those originally proposed in the approved Work Plan since hydraulic data made available from the Investigation testing programs identified the above wells to be more suitable.

Stevens Water Level Recorders (Model No. 68) were installed on Wells R-541, R-531 and R-6 six days prior to performing the actual pumping test at Well R-532. Data collected from the recorders are presented in Table 4.10. Water level recorders were installed on Wells R-539 and R-540 on the day of the test. Pumping was initiated at 10:15 a.m. on August 19, 1986 and continued for 12 hours using a 4-inch diameter submersible pump set at a pumping rate of one gpm. The well was evacuated to the top of the packer

TABLE 4.10

WATER LEVELS OBSERVED PRIOR TO PUMPING TEST
FMC CORPORATION - MIDDLEPORT SITE INVESTIGATION

Pumped well: R-532
 Duration: 12 hours
 Observers: C. Cull, C. Padginton
 Observation wells: R-527, R-14, R-12, R-8, R-543, R-15, R-20, R-10,
 R-16, R-539, R-540, R-541, R-531, R-6, R-520

Water level data in Feet Above Mean Sea Level (AMSL)

<u>Well</u>	<u>8/9</u>	<u>8/10</u>	<u>8/11</u>	<u>8/12</u>	<u>8/13</u>	<u>8/14</u>	<u>8/15</u>	<u>8/16</u>	<u>8/17</u>	<u>8/18</u>	<u>8/19</u>
R-532											519.39
R-527											517.29
R-14											476.55
R-12											469.10
R-8											473.22
R-543											527.20
R-15											523.02
R-520											518.68
R-10											475.29
R-16											525.10
R-539											528.39
R-540											529.45
R-541	-	532.76	532.76	532.68	532.71	532.84	532.91	533.03	533.07	533.20	532.91
R-531	-	522.23	522.27	522.25	522.26	522.29	522.34	522.44	522.45	-	522.50
R-6	-	530.25	530.49	-	530.09	529.37*	-	529.62*	529.68*	-	528.67

Notes:

* questionable data - wires of recorder discovered to be tangled together

after approximately two hours and recharged at approximately one quart per minute thereafter. Pumping water was collected in a holding tank and discharged to the on-Site sump adjacent to Tank T11-01.

Prior to initiating the pumping test, a static water level was collected from the pumping well. Readings during the initial stages of the pumping test were collected from the pumping well each minute for the first 15 minutes, each five minutes for the next hour, every 15 minutes for the second hour, and once per hour thereafter. No readings were obtained for the half-hour prior to initiation of the pump test due to the insertion of the pump.

After the pumping test had been underway for two hours, static water levels were rechecked in surrounding Wells R-539 through R-541, R-10 and R-15 to determine if any change had occurred. These wells were selected on the basis of their close proximity to the pumping well. No response was observed in any of the monitored wells, indicating very little, if any, interconnectivity between the shallow and deep bedrock aquifers.

After the pump was turned off, readings were taken every minute for the first one and one-half hours. The pump was removed from the well and the water level recorder

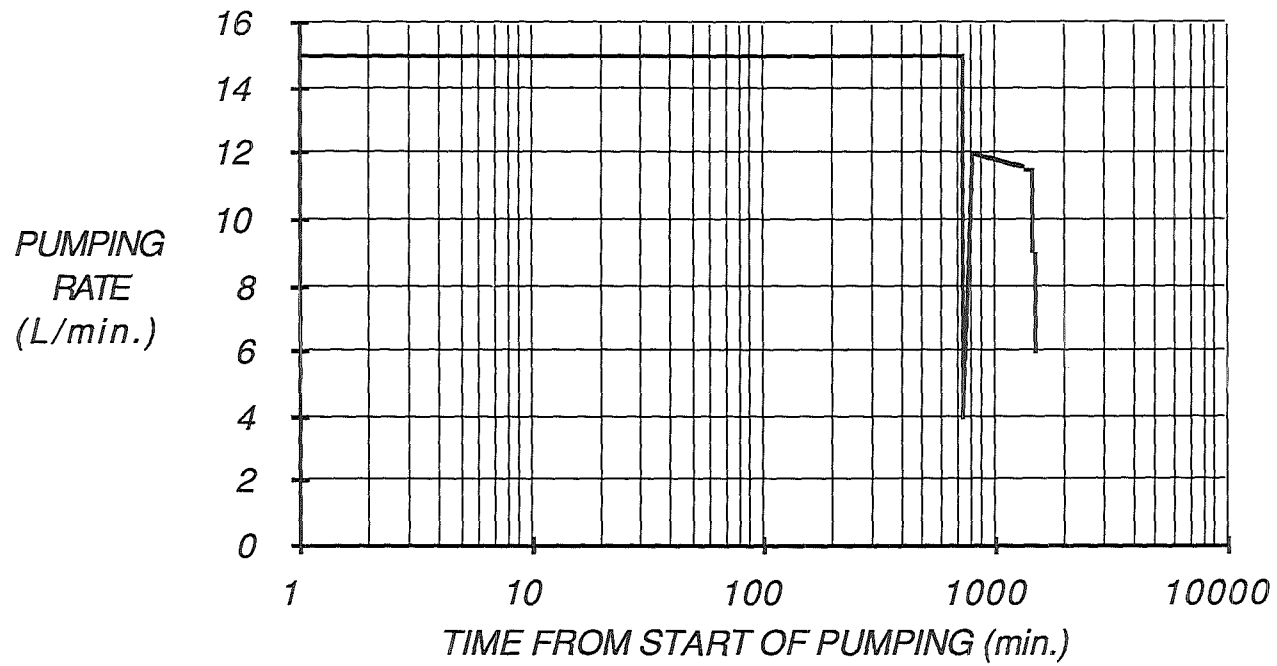
set up. The water level recorder malfunctioned, but additional readings were collected the next morning and the subsequent day. A water sample was collected at approximately 10:30 a.m. on August 20, 1986. After 48 hours of observation, the well had still not fully recovered to its original static water level. Figure 4.13 depicts the water level data obtained during the pumping test.

4.6 WASTE LEACHABILITY

4.6.1 Overview

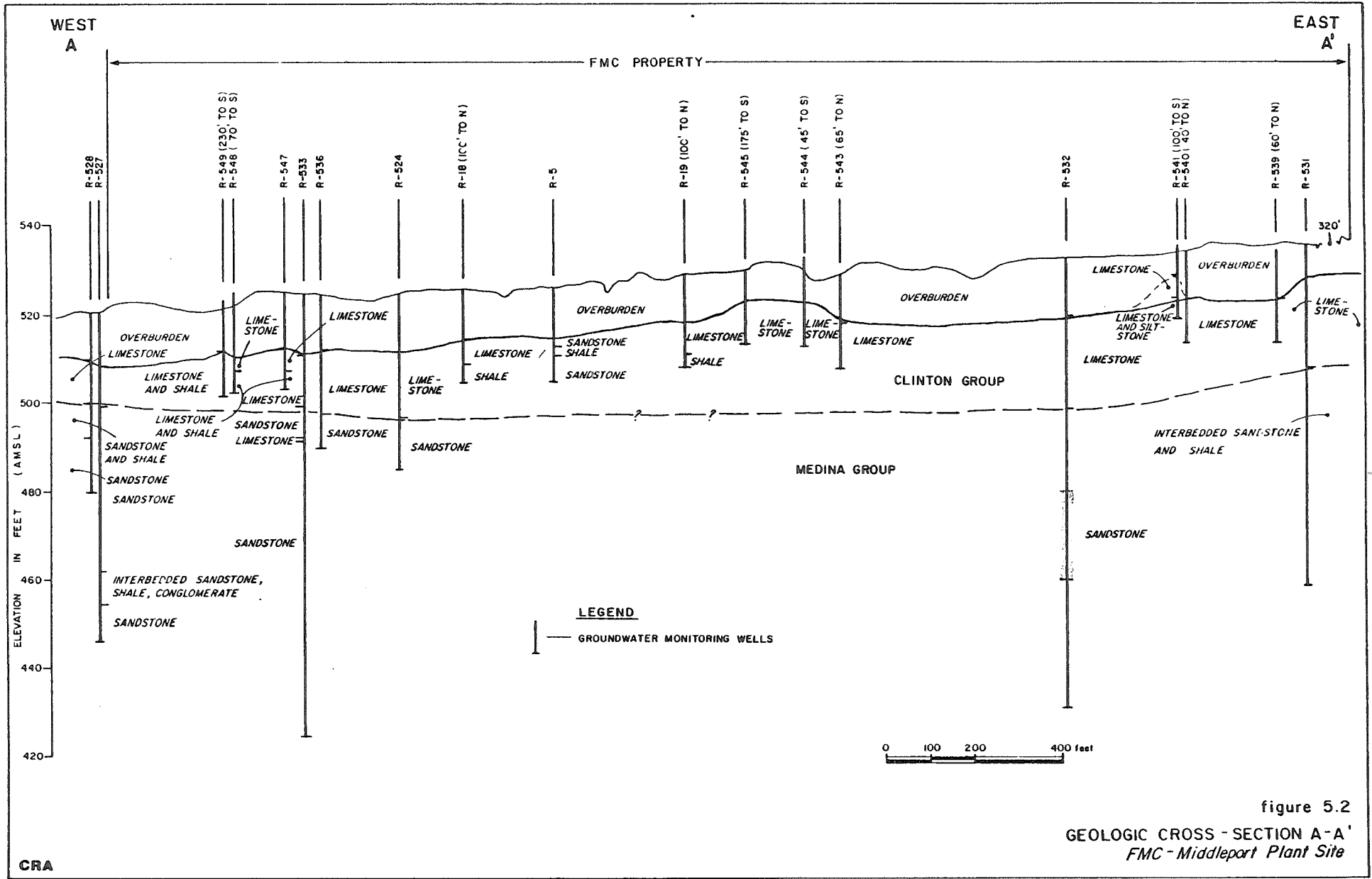
In order to evaluate leaching of the fill materials as a function of water infiltration, a series of four test samples of plant fill material was subjected to leaching tests. Native geologic material samples, which were located immediately below each fill sample site, were collected and used to evaluate the native soil's ability to attenuate leached site-specific chemicals. Physicochemical variables such as hydraulic conductivity, soil density (wet and dry) and infiltration water pH were evaluated and reproduced under laboratory conditions. This was done in order to produce an environment that simulated in situ conditions to the degree practical.

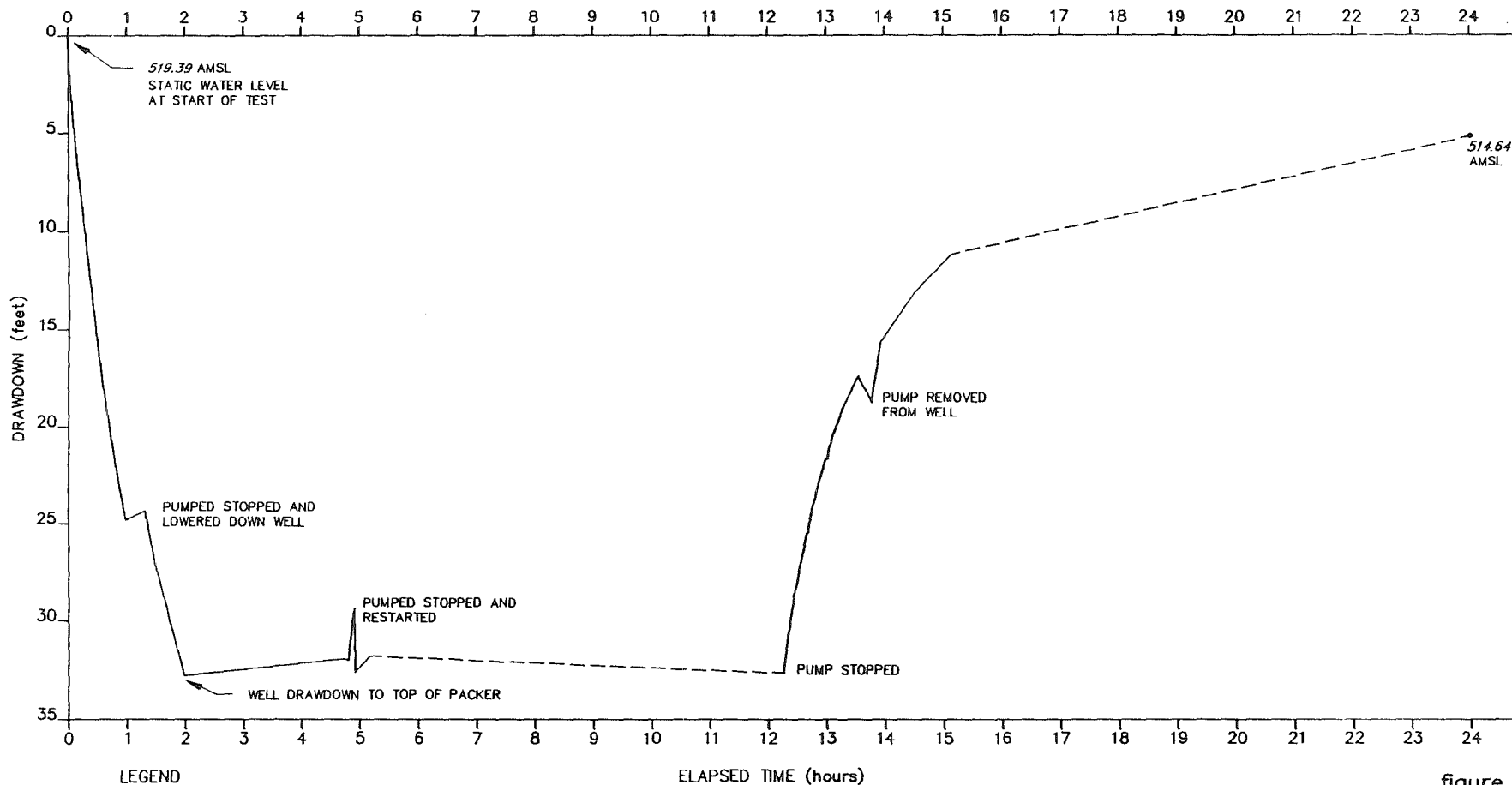
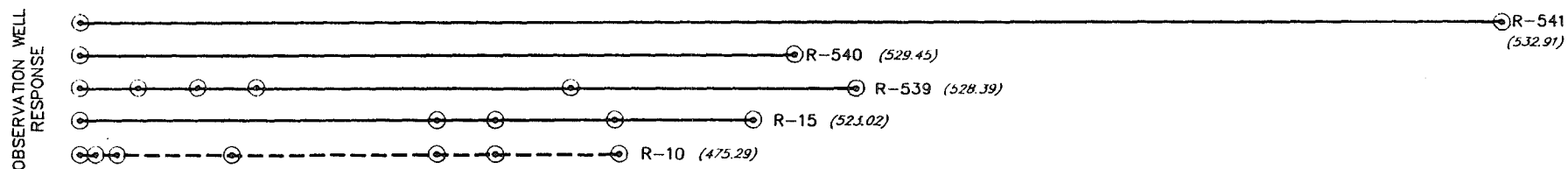
PUMPING RATE PW-1



4/7/88

N-7402





LEGEND

(538.39) WATER ELEVATION (FEET AMSL)

— SHALLOW BEDROCK WELL

- - - DEEP BEDROCK WELL

CRA

1539-08-1/05/87

figure 4.13
HYDROGRAPH OF PUMPING TEST — WELL R-532
FMC — Middleport Plant Site

STRATIGRAPHIC AND INSTRUMENTATION LOG

(BED ROCK)

PROJECT NAME: MIDDLEPORT

HOLE DESIGNATION: R-532 (Pg.1 of 2)

PROJECT NO.: 1539-10

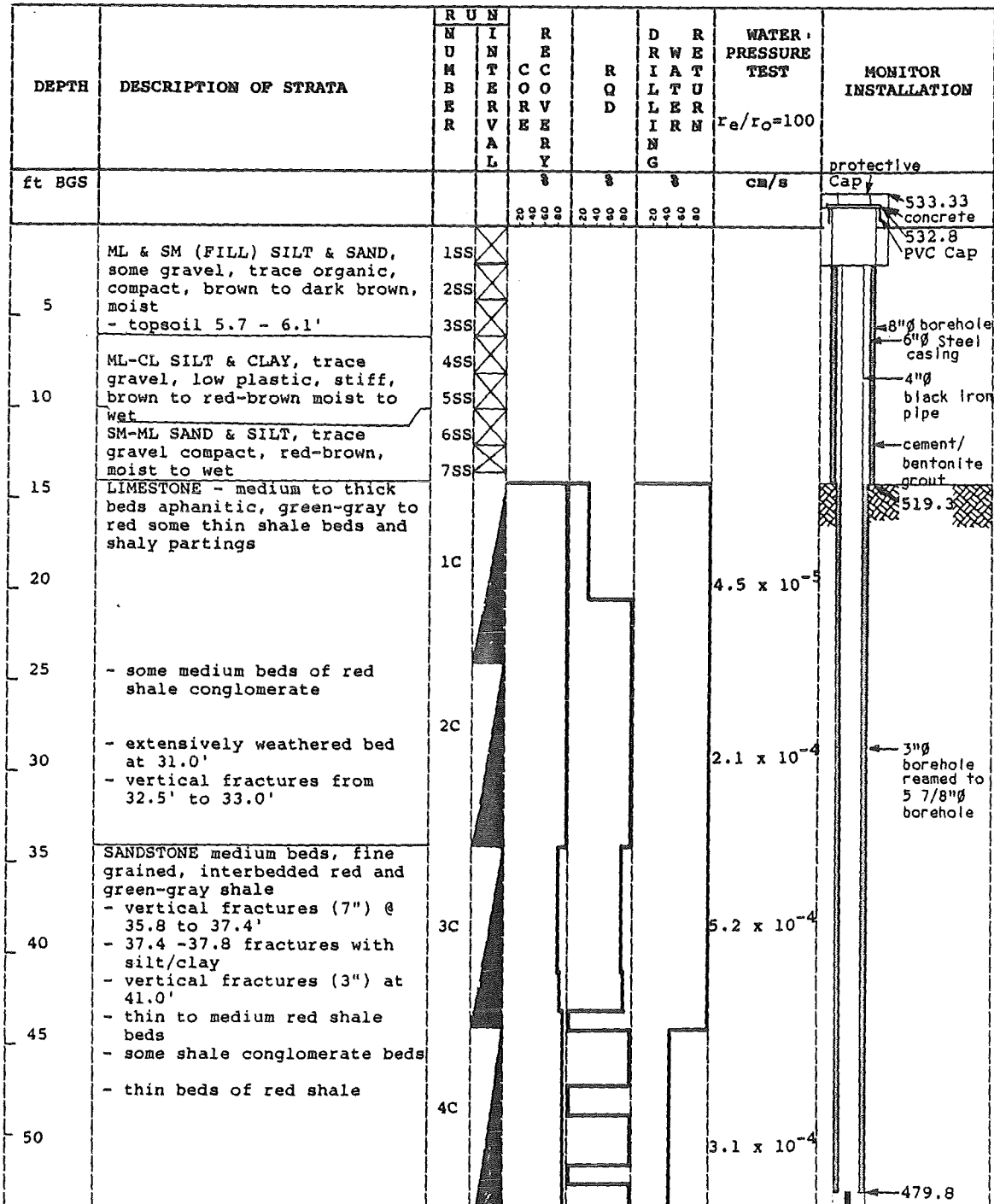
DATE COMPLETED: SEPTEMBER 24, 1985

CLIENT: FMC CORPORATION

DRILLING METHOD: HSA (4-1/4"ID)/
WR (NX CORE)

LOCATION: NORTH EAST CORNER OF CENTRAL PORTION
OF PLANT

CRA SUPERVISOR: DM/CAC/WSC



NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE

WATER FOUND

STATIC WATER LEVEL

NM - NOT MEASURED

STRATIGRAPHIC AND INSTRUMENTATION LOG
(BED ROCK)

PROJECT NAME: MIDDLEPORT

HOLE DESIGNATION: R-532 (Pg.2 of 2)

PROJECT NO.: 1539-10

DATE COMPLETED: SEPTEMBER 24, 1985

CLIENT: FMC CORPORATION

DRILLING METHOD: HSA (4-1/4" ID)/
WR (NX CORE)

LOCATION: NORTH EAST CORNER OF CENTRAL PORTION
OF PLANT

CRA SUPERVISOR: DM/CAC/WSC

DEPTH	DESCRIPTION OF STRATA	R U N N U M B E R	I N T E R V A L	R E C O V E R Y	R Q D	D R W E I A T U R E L E R I N G	WATER PRESSURE TEST $r_e/r_o=100$	MONITOR INSTALLATION
ft BGS							cm/s	
50				2222	2222	2222		
55	- some conglomerate beds - some thin red shaly partings	5C						479.8
60	Extensively weathered shale beds @ 58.7', 59.1' & 60.5'	6C					4.8×10^{-3}	deflated packer 3" borehole
65	Red to gray shale conglomerate beds; sandstone matrix	7C						
70	- massive beds - thin fossiliferous beds						7.4×10^{-4}	
75								460.8
80	- weathered shale bed at 78.6' - 78.8'	8C					1.5×10^{-5}	
85	medium beds of red shale							cement/ bentonite grout
90	- grades to thick sandstone beds with cross-bedding	9C					3.9×10^{-4}	
95		10C						
100	End of coring-100 ft. B G S							432.8

NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE

WATER FOUND

STATIC WATER LEVEL

NM - NOT MEASURED

WAT3

* N764

3-1-0
3-2-0
3-3-0
3-4-0
3-5-0
3-6-0
3-7-0
3-8-0
3-9-0
3-10-0
3-11-0
3-12-0

63m

⊕ TW4/86 (085) NST MONITORED

26.6m

32.5

⊕ TW3/86 (PW) HC18 (2.5)

8.6m

WAT S-1-0.4
WAT S-2-0.74
3-0-0
4-0-0
5-0
6-0
7-0
8-0
9-0
10-0
11-0
12-0

WAT S
70m

⊕ MB 11 MB 11-0-0
11-2-0 FLW
11-3-0

WAT6 ⊕ 5m

WAT6 a-0
b-0
c-0
d-0
e-0
f-0
g-0
h-0
i-0
j-0
k-0
l-0
m-0

TESTING OF TW 3/86

N764

WAT3

1-0
3-2-0
3-3-0
3-4-0
3-5-0
3-6-0
3-7-0
3-8-0
3-9-0
3-10-0
3-11-0
3-12-0

63m

⊕ TW4/86 (OBS) NOT MONITORED

26.6m

32.5

⊕ TW3/86 (PW) HC18 (2.5)

8.6m

WAT5-1-0.4
WAT5-2-0.74
3-0.0
4-0.0
5-0
6-0
7-0
8-0
9-0
10-0
11-0
12-0

WAT5
70m

⊕ MB 11 MB11-1-0
11-2-FLW
11-3-0

WAT6 ⊕ 5m

WAT6 a-0
b-0
c-0
d-0
e-0
f-0
g-0
h-0
i-0
j-0
k-0
l-0
m-0

TESTING OF TW 3/86

N 465

SUMMARY OF PUMPING TEST DATA - SITE "F"*

*Ex. H-676

**Ex. H-84(Add. I)

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 3 (TW 3/86)		2.50		Date of Test: March 26/86
	WAT 5(1)	0.395	8.6(S)**	Pumping Time: 420 min (7 hrs)
	5(2)	0.740		
	5(3)	0 (+0.03)		
	5(4)	0 (+0.04)		
	5(5)	0 (+0.01)		
	5(6)	0 (+0.02)		
	5(7)	0 (+0.01)		
	5(8)	0 (+0.01)		
	5(9)	0 (+0.02)		
	5(10)	0 (0.02)		
	5(11)	0 (0.02)		
	5(12)	0 (0)		
	MB 11(1)	0	78.6(S)**	
	11(2)	Flow		
	11(3)	0		
	WAT 6(1)	0	83.6(S)**	
	6(2)	0		
	6(3)	0		
	6(4)	0		
	6(5)	0		
	6(6)	0		

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 3 (TW 3/86)	WAT 6(7)	0	83.6(S)**	
	6(8)	0		
	6(9)	0		
	6(10)	0		
	6(11)	0		
	6(12)	0		
	6(13)	0		
	WAT 3(1)	0	89.6(N)**	
	3(2)	0		
	3(3)	0		
	3(4)	0		
	3(5)	0		
	3(6)	0		
	3(7)	0		
	3(8)	0		
	3(9)	0		
	3(10)	0		
	3(11)	0		
	3(12)	0		

- NOTES:**
- Figure in brackets () indicates the actual drop or rise (+) in water level during pumping test.
 - A reduction in water level of less than 0.10 is taken as zero drawdown, except where an adjacent piezometer (above or below) shows a recovery or no change. Where water levels rise, drawdown is shown as 0.

N + 67.

SUMMARY OF PUMPING TEST DATA - SITE "F"*

*Ex. H-676 + 766

**Ex. H-84 (Add. I)

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 4 (TW 4/86) (Test 1)	TW 3/86 WAT 5(1) 5(2)	0.65 0 0 (0.02) 0 (0.01)	-- 26.6(S)** 32.5(S)**	Date of Test: March 24/86 Pumping Time: 480 min (8 hrs)
PW 4 (TW 4/86) (Test 2)	TW 3/86 (OW1) WAT 5(1) 5(2) 5(3) 5(4) 5(5) 5(6) 5(7) 5(8) 5(9) 5(10) 5(11) 5(12) 5(13)	6.060 0 0 0 (0.08) 0 0 0 (+0.01) 0 (0.01) 0 0 (+0.03) 0 0 (0.01) 0 0	-- 26.6(S)** 32.5(S)**	Date of Test: March 25/86 Pumping Time: 898 min. (15 hrs)

PUMPING TEST	OBSERVATION WELL	TOTAL DRAWDOWN (m)	DISTANCE FROM PUMPING WELL (m)	REMARKS
PW 4(TW 4/86) (Test 2)	MB 11(1)	0	102.5(S)**	
	11(2)	0		
	11(3)	0		
	WAT 6(1)	0	107.5(S)**	
	6(2)	0		
	6(3)	0		
	6(4)	0		
	6(5)	0		
	6(6)	0		
	6(7)	0		
	6(8)	0		
	6(9)	0		
	6(10)	0		
	6(11)	0		
	6(12)	0		
	6(13)	0		
	WAT 3(1)	0	63(N)**	
	3(2)	0		
	3(3)	0		
	3(4)	0		
	3(5)	0		
	3(6)	0		
	3(7)	0		
	3(8)	0		
	3(9)	0		
	3(10)	0		
	3(11)	0		
	3(12)	0		

NOTES: 1. Figure in brackets () indicates the actual drop or rise (+) in water level during pumping test.

2. A reduction in water level of less than 0.10 is taken as zero drawdown, except where an adjacent piezometer (above or below) shows a recovery or no change. Where water levels rise, drawdown is shown as 0.

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from data provided by Trow for a 0.3 m thickness within this unit. CRA has demonstrated that this unit has hydraulic conductivities ranging over four orders of magnitude.

Trow states that this unit (Unit 3) has "good attenuation abilities" (p. 123, Exhibit H-40). CRA does not expect this to be the case for a fractured shale with hydraulic conductivities ranging up to 1×10^{-4} cm/sec. In such a case much of the contaminant migration is expected to be restricted to the interconnected fracture network where the effects of attenuation are expected to be insignificant.

CRA does not agree that the 'moderately fractured shale' (Unit 3) has "relatively good attributes for waste disposal" (Trow, p. 123, Exhibit H-40).

The base elevation of the proposed landfill excavation, as detailed by Trow (Drawing 5-r, Exhibit H-42B) was plotted by CRA on Trow's interpreted hydrostratigraphy (Figure B.7.1, p. 82). This figure demonstrates that a significant thickness of the 'moderately fractured and weathered shale' (Unit 2) remains below the excavation. Given that the hydraulic conductivity of this unit as determined from pumping tests is 1×10^{-4} cm/sec, the weathered and fractured shale may act as an underdrain for the landfill.

*Cunks
draft*

HYDRAULIC HEAD CALIBRATION - SUMMARY OF MODEL COMPARISONS

Model	Measured Heads				Density Corrected Heads			
	Absolute Differences (m)		Plus/Minus Differences (m)		Absolute Differences (m)		Plus/Minus Differences (m)	
	Sum	Mean	Sum	Mean	Sum	Mean	Sum	Mean
2 Trow #1 from H-85	46.18	2.72	-29.78	-1.75	45.06	3.00	-33.94	-2.26
3 Trow #1 from H-221	37.98	2.23	-21.48	-1.26	36.86	2.46	-25.64	-1.71
4 Trow #1 from B-617	45.54	2.68	-28.90	-1.70	44.45	2.96	-33.03	-2.20
5 Trow #1 Revised by CRA	34.37	2.02	-9.17	-0.54	32.64	2.18	-13.30	-0.89
6 Trow #2 from H-221	27.22	1.60	-2.68	-0.16	26.13	1.74	-6.15	-0.41
7 Trow #2 from B-617	26.88	1.58	-2.32	-0.14	25.79	1.72	-5.79	-0.39
8 Trow #2 Revised by CRA	29.94	1.76	10.70	0.63	28.03	1.87	7.23	0.48

HYDRAULIC HEAD CALIBRATION - TROW MODEL #1 FROM H-85

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.63	0.19	0.19	0.16	0.16
	2	180.14	180.16	177.66	2.48	-2.48	2.50	-2.50
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	117.35	5.61	-5.61	5.83	-5.83
	2	133.18	133.19	135.25	2.07	2.07	2.06	2.06
	3	133.60	133.61	135.35	1.75	1.75	1.74	1.74
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	112.68	2.29	-2.29	2.61	-2.61
	2	118.97	119.00	114.24	4.73	-4.73	4.76	-4.76
	3	124.70	124.71	118.91	5.79	-5.79	5.80	-5.80
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	111.43	1.97	1.97	1.60	1.60
	2	115.59	115.78	112.50	3.09	-3.09	3.28	-3.28
	3	117.87	117.97	114.63	3.24	-3.24	3.34	-3.34
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.44	2.65	-2.65	2.72	-2.72
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	105.72	2.59	-2.59	3.00	-3.00
	2	108.32	108.46	105.72	2.60	-2.60	2.74	-2.74
	3	108.63	108.64	105.72	2.91	-2.91	2.92	-2.92
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.98	1.49	1.49	NA	NA
	2	75.39	NA	76.12	0.73	0.73	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUMS
MEAN *

46.18	-29.78	45.06	-33.94
2.72	-1.75	3.00	-2.26

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

0127
0.43

HYDRAULIC HEAD CALIBRATION - TROW MODEL #1 FROM H-221

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.63	0.19	0.19	0.16	0.16
	2	180.14	180.16	177.66	2.48	-2.48	2.50	-2.50
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	117.47	5.49	-5.49	5.71	-5.71
	2	133.18	133.19	135.25	2.07	2.07	2.06	2.06
	3	133.60	133.61	135.25	1.65	1.65	1.64	1.64
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	112.84	2.13	-2.13	2.45	-2.45
	2	118.97	119.00	117.18	1.79	-1.79	1.82	-1.82
	3	124.70	124.71	121.51	3.19	-3.19	3.20	-3.20
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	111.58	2.12	2.12	1.75	1.75
	2	115.59	115.78	113.66	1.93	-1.93	2.12	-2.12
	3	117.87	117.97	115.75	2.12	-2.12	2.22	-2.22
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.44	2.65	-2.65	2.72	-2.72
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	105.77	2.54	-2.54	2.95	-2.95
	2	108.32	108.46	105.77	2.55	-2.55	2.69	-2.69
	3	108.63	108.64	105.77	2.86	-2.86	2.87	-2.87
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.98	1.49	1.49	NA	NA
	2	75.39	NA	76.12	0.73	0.73	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUMS
MEAN *

37.98	-21.48	36.86	-25.64
2.23	-1.26	2.46	-1.71

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

HYDRAULIC HEAD CALIBRATION - TROW MODEL #1 FROM B-617

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.63	0.19	0.19	0.16	0.16
	2	180.14	180.16	177.66	2.48	-2.48	2.50	-2.50
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	117.47	5.49	-5.49	5.71	-5.71
	2	133.18	133.19	135.25	2.07	2.07	2.06	2.06
	3	133.60	133.61	135.35	1.75	1.75	1.74	1.74
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	112.84	2.13	-2.13	2.45	-2.45
	2	118.97	119.00	112.84	6.13	-6.13	6.16	-6.16
	3	124.70	124.71	121.51	3.19	-3.19	3.20	-3.20
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	111.58	2.12	2.12	1.75	1.75
	2	115.59	115.78	112.62	2.97	-2.97	3.16	-3.16
	3	117.87	117.97	113.66	4.21	-4.21	4.31	-4.31
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.42	2.67	-2.67	2.74	-2.74
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	105.77	2.54	-2.54	2.95	-2.95
	2	108.32	108.46	105.77	2.55	-2.55	2.69	-2.69
	3	108.63	108.64	105.77	2.86	-2.86	2.87	-2.87
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.98	1.49	1.49	NA	NA
	2	75.39	NA	76.09	0.70	0.70	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA
SUMS					45.54	-28.90	44.45	-33.03
MEAN *					2.68	-1.70	2.96	-2.20

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

HYDRAULIC HEAD CALIBRATION - TROW MODEL #1 REVISED BY CRA

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3

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.71	0.27	0.27	0.24	0.24
	2	180.14	180.16	177.74	2.40	-2.40	2.42	-2.42
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	122.84	0.12	-0.12	0.34	-0.34
	2	133.18	133.19	135.19	2.01	2.01	2.00	2.00
	3	133.60	133.61	135.32	1.72	1.72	1.71	1.71
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	116.23	1.26	1.26	0.94	0.94
	2	118.97	119.00	116.23	2.74	-2.74	2.77	-2.77
	3	124.70	124.71	121.03	3.67	-3.67	3.68	-3.68
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	114.61	5.15	5.15	4.78	4.78
	2	115.59	115.78	114.65	0.94	-0.94	1.13	-1.13
	3	117.87	117.97	114.65	3.22	-3.22	3.32	-3.32
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.41	2.68	-2.68	2.75	-2.75
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	106.44	1.87	-1.87	2.28	-2.28
	2	108.32	108.46	106.41	1.91	-1.91	2.05	-2.05
	3	108.63	108.64	106.41	2.22	-2.22	2.23	-2.23
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.98	1.49	1.49	NA	NA
	2	75.39	NA	76.09	0.70	0.70	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUMS
MEAN *

34.37	-9.17	32.64	-13.30
2.02	-0.54	2.18	-0.89

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

HYDRAULIC HEAD CALIBRATION - TROW MODEL #2 FROM H-221

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.72	0.28	0.28	0.25	0.25
	2	180.14	180.16	178.04	2.10	-2.10	2.12	-2.12
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	122.04	0.92	-0.92	1.14	-1.14
	2	133.18	133.19	135.25	2.07	2.07	2.06	2.06
	3	133.60	133.61	135.38	1.78	1.78	1.77	1.77
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	116.01	1.04	1.04	0.72	0.72
	2	118.97	119.00	116.03	2.94	-2.94	2.97	-2.97
	3	124.70	124.71	125.35	0.65	0.65	0.64	0.64
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	114.38	4.92	4.92	4.55	4.55
	2	115.59	115.78	115.03	0.56	-0.56	0.75	-0.75
	3	117.87	117.97	116.00	1.87	-1.87	1.97	-1.97
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.14	2.95	-2.95	3.02	-3.02
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	107.22	1.09	-1.09	1.50	-1.50
	2	108.32	108.46	107.20	1.12	-1.12	1.26	-1.26
	3	108.63	108.64	107.23	1.40	-1.40	1.41	-1.41
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.61	1.12	1.12	NA	NA
	2	75.39	NA	75.80	0.41	0.41	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUMS
MEAN *

27.22	-2.68	26.13	-6.15
1.60	-0.16	1.74	-0.41

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

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HYDRAULIC HEAD CALIBRATION - TROW MODEL #2 FROM B-617

Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.72	0.28	0.28	0.25	0.25
	2	180.14	180.16	178.04	2.10	-2.10	2.12	-2.12
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	122.15	0.81	-0.81	1.03	-1.03
	2	133.18	133.19	135.25	2.07	2.07	2.06	2.06
	3	133.60	133.61	135.38	1.78	1.78	1.77	1.77
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	116.02	1.05	1.05	0.73	0.73
	2	118.97	119.00	116.03	2.94	-2.94	2.97	-2.97
	3	124.70	124.71	125.35	0.65	0.65	0.64	0.64
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	114.38	4.92	4.92	4.55	4.55
	2	115.59	115.78	115.20	0.39	-0.39	0.58	-0.58
	3	117.87	117.97	116.01	1.86	-1.86	1.96	-1.96
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.14	2.95	-2.95	3.02	-3.02
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	107.26	1.05	-1.05	1.46	-1.46
	2	108.32	108.46	107.23	1.09	-1.09	1.23	-1.23
	3	108.63	108.64	107.22	1.41	-1.41	1.42	-1.42
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.61	1.12	1.12	NA	NA
	2	75.39	NA	75.80	0.41	0.41	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUMS
MEAN *

26.88	-2.32	25.79	-5.79
1.58	-0.14	1.72	-0.39

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

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HYDRAULIC HEAD CALIBRATION - TROW MODEL #2 REVISED BY CRA

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Location	Well	Measured Head (m)	Density Corrected Head (m)	Model Result	Measured Heads		Density Corrected Heads	
					Absolute Differences (m)	Plus/Minus Differences (m)	Absolute Differences (m)	Plus/Minus Differences (m)
HC-1-85	1	177.44	177.47	177.83	0.39	0.39	0.36	0.36
	2	180.14	180.16	178.14	2.00	-2.00	2.02	-2.02
	3	183.98	183.98	183.98	0.00	0.00	NC	NC
HC-2-85	1	167.99	NA	167.99	0.00	0.00	NA	NA
HC-4-85	1	122.96	123.18	125.83	2.87	2.87	2.65	2.65
	2	133.18	133.19	135.18	2.00	2.00	1.99	1.99
	3	133.60	133.61	135.35	1.75	1.75	1.74	1.74
	4	135.44	135.44	135.44	0.00	0.00	NC	NC
HC-5-85	1	114.97	115.29	118.38	3.41	3.41	3.09	3.09
	2	118.97	119.00	118.39	0.58	-0.58	0.61	-0.61
	3	124.70	124.71	125.13	0.43	0.43	0.42	0.42
	4	125.83	125.83	125.83	0.00	0.00	NC	NC
HC-8-85	1	109.46	109.83	116.42	6.96	6.96	6.59	6.59
	2	115.59	115.78	116.57	0.98	0.98	0.79	0.79
	3	117.87	117.97	116.58	1.29	-1.29	1.39	-1.39
	4	117.84	117.88	117.84	0.00	0.00	NC	NC
HC-11-85	1	92.09	92.16	89.14	2.95	-2.95	3.02	-3.02
	2	89.48	89.48	89.48	0.00	0.00	NC	NC
HC-14-85	1	108.31	108.72	107.55	0.76	-0.76	1.17	-1.17
	2	108.32	108.46	107.45	0.87	-0.87	1.01	-1.01
	3	108.63	108.64	107.46	1.17	-1.17	1.18	-1.18
	4	109.14	109.16	109.14	0.00	0.00	NC	NC
HC-16-85	1	74.49	NA	75.61	1.12	1.12	NA	NA
	2	75.39	NA	75.80	0.41	0.41	NA	NA
	3	76.21	NA	76.21	0.00	0.00	NA	NA

SUM	29.94	10.70	28.03	7.23
MEAN *	1.76	0.63	1.87	0.48

NC - Fixed water table value, head correction not calculated.

NA - Density corrected head not available.

* Mean based on the number of non-fixed head piezometers.

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N-771

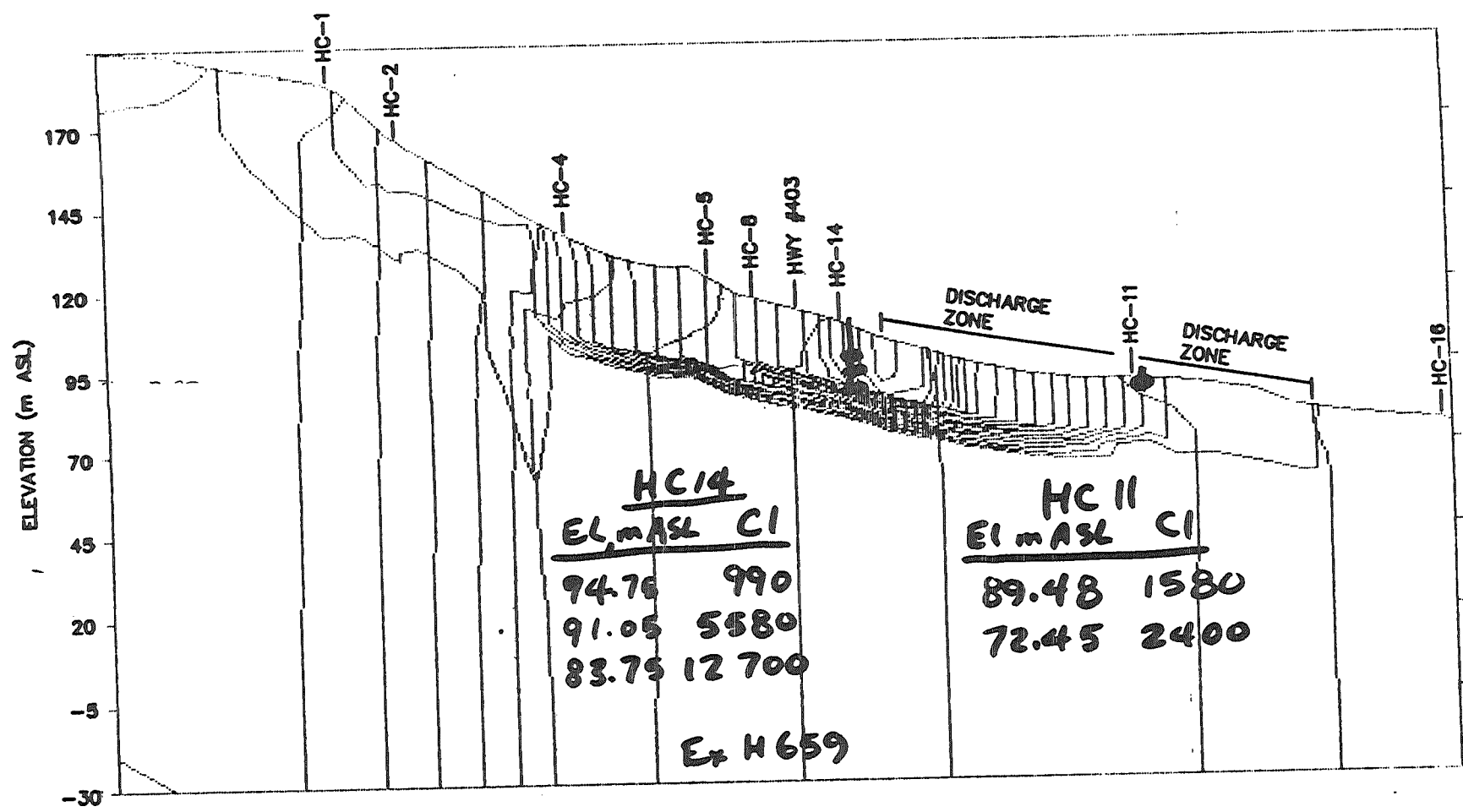
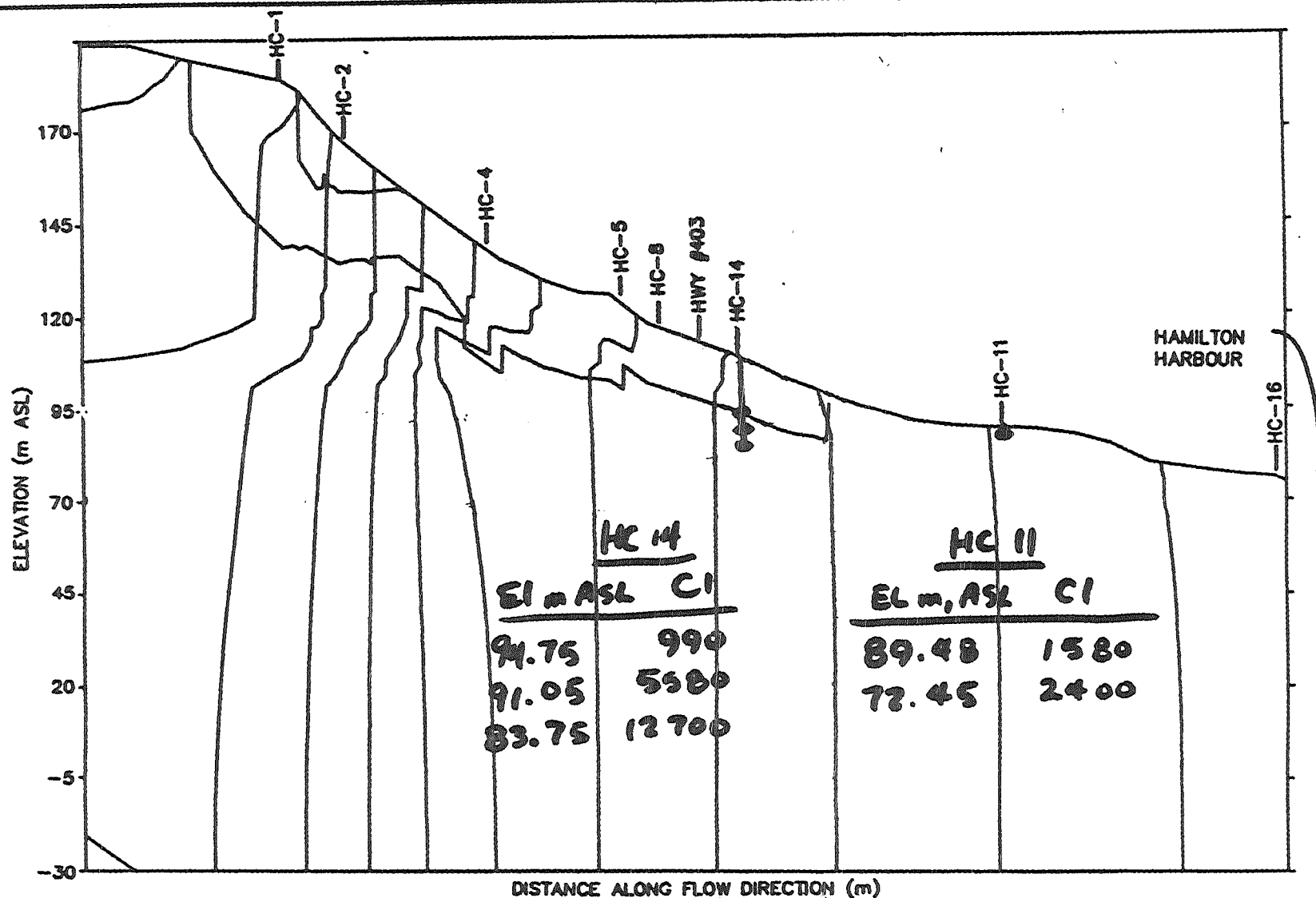


figure C.4.2

CALIBRATED FLOW MODEL
Site 'F' - Hydrogeology - City of Burlington

CRA

N-772



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: 30m³/yr

Revision 1
 TROW MODEL 2
 Site 'F'-Hydrogeology - City of Burlington

N 773

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*Woods Hole Oceanographic Institution
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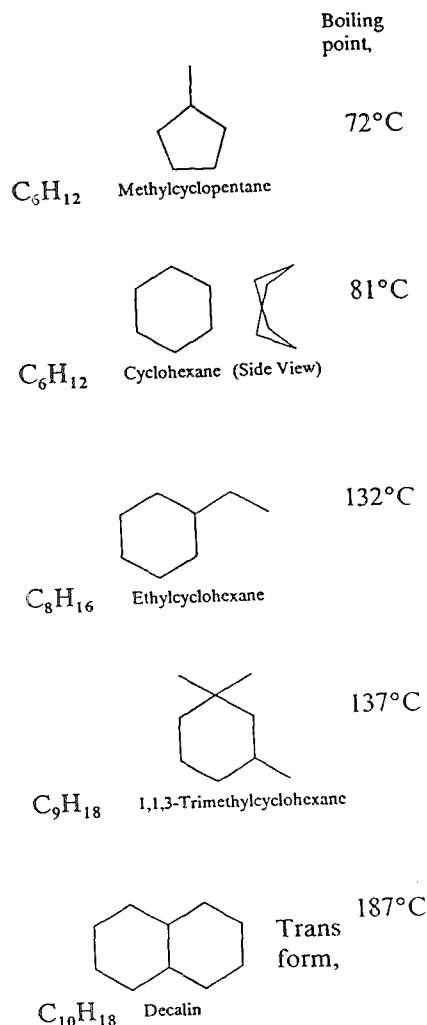
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*To those pioneering scientists who were in the
Geological Research Section of the former
Carter Research Laboratory of Tulsa, Oklahoma,
in the late 1940s and early 1950s.*

Naphthenes (Cycloparaffins)



Aromatics

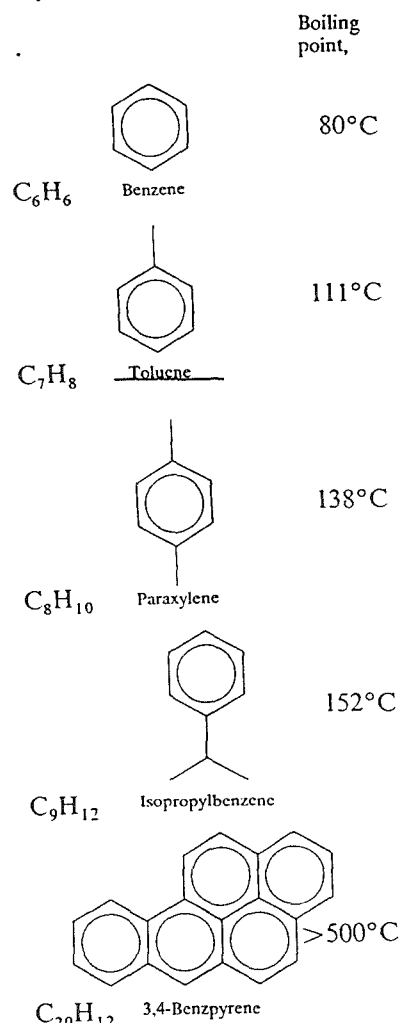


Figure 3-4
Skeletal formulas for cycloparaffin (naphthene) and aromatic hydrocarbons in petroleum.
All C_6 rings are warped. Cyclohexane is shaped like a twisted boat.

The average crude oil contains about 50 percent naphthenes with the quantities increasing in the heavier fractions and decreasing in the lighter fractions. In the heavier fractions, the naphthenes tend to fuse into polycyclic rings; that is, a group of rings in which two or more carbon atoms are shared between the rings. Decalin, in Figure 3-4, is an example. The most common

naphthenes are methylcyclopentane and methylcyclohexane, which together represent 2 percent or more of the average crude.

The naphthenes and paraffins are also referred to as *saturated hydrocarbons*, because all available carbon bonds are saturated with hydrogen. If hydrogen is removed from a paraffin, it will form one, two, or three double bonds, depending on whether 2, 4, or 6 hydrogen atoms are removed. Removing hydrogen from naphthenes forms either cycloolefins or aromatics. During World War II, methylcyclohexane was concentrated in refinery runs and stripped of half its hydrogen to form toluene, the starting material for trinitrotoluene (TNT).

Aromatic Hydrocarbons (C_nH_{2n-6})

The term *aromatic hydrocarbons* originated when some early, pleasant-smelling compounds such as cymene were isolated from natural fragrant oils. However, most hydrocarbons have very little odor in the pure state. The strong odor of petroleum is due to the nonhydrocarbons. All aromatic hydrocarbons contain at least one benzene ring. This is a flat 6-carbon ring (top of Figure 3-4) in which the fourth bond of each carbon atom is shared throughout the ring. For simplicity, the ring is shown with an inner circle, which indicates that the fourth bond's unpaired electrons are constantly oscillating between all carbon atoms in the ring. The aromatics are unsaturated hydrocarbons that will react to add hydrogen or other elements to the ring. The aromatics rarely amount to more than 15 percent of a total crude oil. They tend to be concentrated in the heavy fractions of petroleum, such as gas oil, lubricating oil, and residuum, where the quantity often exceeds 50 percent. Toluene (Figure 3-4) and metaxylene are the most common aromatic hydrocarbons in petroleum. Aromatics have the highest octane ratings of the hydrocarbon types, so they are valuable in gasoline blends. However, they are undesirable in the lubricating oil range, because they have the highest change in viscosity with temperature of all the hydrocarbons.

The heavy gas oil, lubricating oil, and residuum of petroleum contain increasing amounts of polycyclic (condensed-ring) hydrocarbons (Figure 3-4). McKay and Latham (1973) found that the 335–530°C boiling range of Recluse, Wyoming, crude contains four to eight condensed ring systems, and the higher boiling ranges appear to have even larger systems of condensed rings. It is probable that the number of rings in polycyclic molecules of residuum increase continuously up to the size of asphaltene particles (Figure 3-15).

Polycyclic aromatic hydrocarbons in natural products are being studied more intensively, because some of them, such as 3,4-benzpyrene, 3,4-

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INTRODUCTION TO GEOCHEMISTRY

Second Edition

Konrad B. Krauskopf
*Professor of Geochemistry Emeritus
Stanford University*

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CHAPTER

SEVEN

CLAY MINERALS AND SOILS

After a long digression on structural chemistry and the chemistry of colloids, we come back in this chapter to the products of weathering. The ultimate solid product of weathering is the material called soil, and chief among the constituents of most soils are the clay minerals. We start, then, by investigating the structure and properties of the clays, and go on to consider the role these minerals play in the formation of soils.

7-1 STRUCTURE

Introduction

Geologists have used the word "clay" in two senses: as a size term, to refer to material of any composition whose average grain size is less than 0.004 mm (this figure varies somewhat with different authors), and as a mineralogic term, to refer to a group of minerals with a specific range of composition and a particular kind of crystallographic structure. The two meanings often overlap or coincide, because the fine-grained part of a soil or sediment most commonly consists largely of clay minerals. The second meaning is the commoner one in recent literature, and it is the only one that will concern us here.

Chemically the clay minerals are best described as hydrous aluminum silicates. This simple definition must be qualified by adding that many clays contain other metals in addition to aluminum, particularly magnesium and iron, and that some extreme types have almost no aluminum at all. Crude molecular formulas of

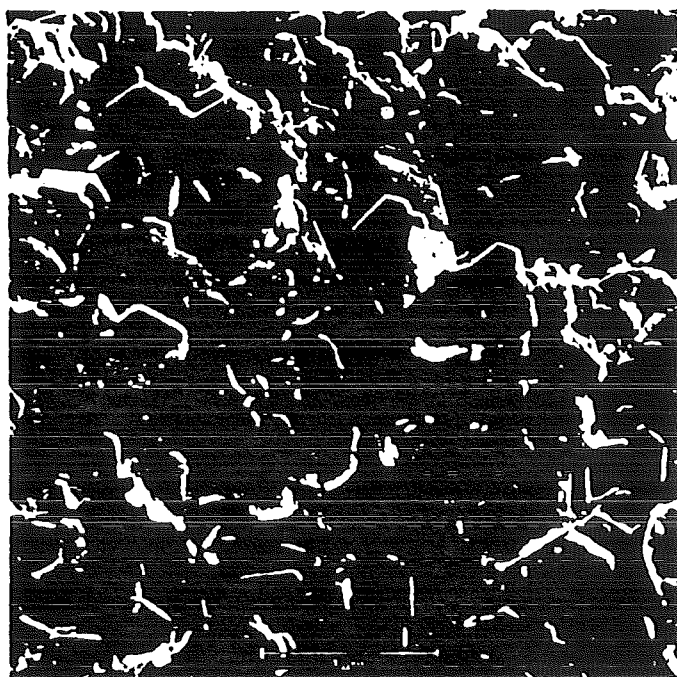
the principal clays may be written $H_4Al_2Si_2O_9$ (kaolin) and $HAISi_2O_6$ (montmorillonite), but these tell us practically nothing about the behavior of the minerals. Actual compositions are never this simple, but show variations in the Si/Al ratio, a variable quantity of water, and usually considerable amounts of magnesium, iron, calcium, and the alkali metals. Between composition and properties there is little correspondence. Two clays with similar proportions of the various elements may show great differences in plasticity and in capacity for ion exchange, while another pair with very different compositions may show strikingly similar properties. Chemical analyses by themselves obviously are of little help in understanding the clays; hence we turn instead to the structural considerations of Chap. 5.

Characteristically the clay minerals are extremely fine-grained, so fine that ordinary microscopes are inadequate for studies of crystal morphology. X-ray-diffraction cameras and electron microscopes are the instruments that have furnished most of our modern information about clays. By their aid the general patterns of clay structure have been deciphered, the apparent anomalies of chemical composition have been explained, and a classification of clay minerals has been established that fits fairly satisfactorily the observed macroscopic properties.

Octahedral and Tetrahedral Sheets

Clay minerals (with a few rare exceptions) are phyllosilicates, silicates with continuous sheet structures like the micas. This could be anticipated from the flakelike form of clay particles; the resemblance to micas is apparent also in the hexagonal shapes which the flakes of some clays assume (Fig. 7-1). Both micas and clays have a characteristic structure, made up of alternating layers or sheets of two kinds. One sheet consists of the ions Al^{3+} , O^{2-} , and OH^- ; the negative ions form octahedra around Al^{3+} , the relative numbers of O^{2-} and OH^- being adjusted to satisfy the valences of the entire structure; the O^{2-} and OH^- are shared between adjacent octahedra, so that the structure is continuous in two dimensions (Fig. 7-2). This pattern by itself, not as part of a clay structure, is identical with the structure of gibbsite $[Al_2(OH)_6]$; it is often referred to as the "gibbsite sheet," or the "octahedral sheet," of the clay structure. The second kind of sheet is made up of Si^{4+} , O^{2-} , and OH^- ions; each Si^{4+} is in the center of a tetrahedron of oxygen ions; the tetrahedra all face the same direction, and the oxygens at their bases are linked so as to form hexagonal rings (Fig. 7-3). This sheet is the "silica sheet," or the "tetrahedral sheet," of the clay structure. The complete clay structure (or mica structure) consists of one of several possible combinations of the octahedral and tetrahedral sheets. The simplest combination, the layer structure of kaolinite, is shown in Fig. 7-4: a single octahedral sheet is linked to a single tetrahedral sheet by sharing some of the oxygen ions. The double layer extends indefinitely in two dimensions, and the clay crystal is built up of a succession of these layers one on top of another.

The diagrammatic representations in Figs. 7-2 to 7-4 show the geometry correctly but give a false impression of ionic sizes. The big units of the structure are, of course, O^{2-} and OH^- ; these have almost identical sizes (radius 1.32 Å), since the

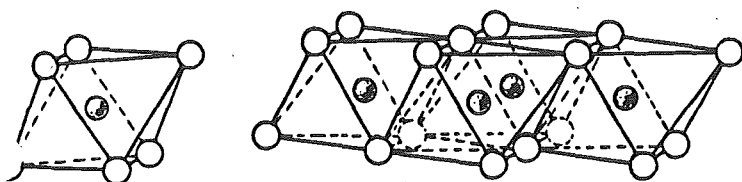


(a)



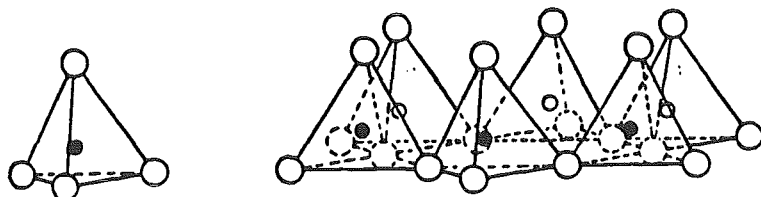
(b)

Figure 7-1 Electron photomicrographs of kaolinite: (a) replica; (b) transmission. Note the hexagonal shapes of many of the grains. The magnification is about 43,000 times. (Source: Kenneth Towe, Smithsonian Institution, Washington, D.C.)



○ and ○ = Hydroxyls ● Aluminums, magnesiums, etc.

Figure 7-2 Diagram of the octahedral sheet, or gibbsite sheet, of phyllosilicate structures. (The left-hand diagram shows a single octahedral unit.) When Al^{3+} occupies the centers of the octahedra, only two-thirds of the possible sites are filled; when Mg^{2+} occupies these positions, all sites are filled. (Source: Grim, 1968, page 52.)



○ and ○ = Oxygens ○ and ● = Silicons

Figure 7-3 Diagram of the tetrahedral sheet, or silica sheet, of phyllosilicate structures. (The left-hand diagram is a single tetrahedral unit.) (Source: Grim, 1968, page 52.)

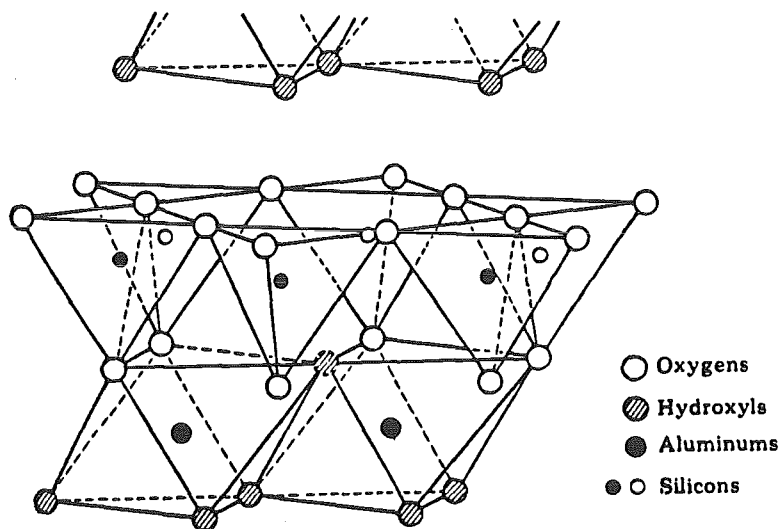


Figure 7-4 Diagram of the structure of kaolinite. (Source: Grim, 1968, page 58.)

size of the added proton in OH^- is negligible. These large anions should be thought of as in contact or nearly so, clustered around the smaller cations. Only four can fit around the tiny Si^{4+} (0.42 Å), but six can be accommodated around the larger Al^{3+} (0.51 Å).

Kaolinite and Montmorillonite

Another way in which the octahedral and tetrahedral sheets can be put together is shown in Fig. 7-5. Here, in montmorillonite, an octahedral sheet is sandwiched between two tetrahedral sheets, so that the theoretical Si/Al ratio is twice the ratio in kaolin. A convenient way to summarize the difference between the kaolinite and

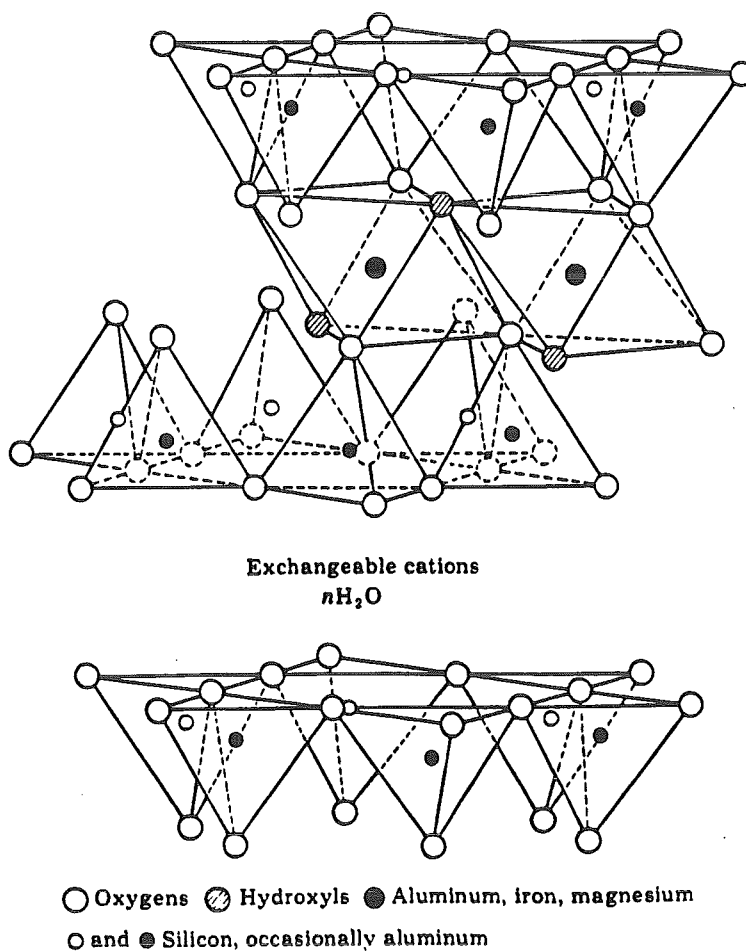


Figure 7-5 Diagram of the structure of montmorillonite. (Source: Grim, 1968, page 79.)

montmorillonite structures is simply to tabulate the numbers of ions at different levels in a unit of structure:

Kaolinite	Montmorillonite
6O^{2-}	6O^{2-}
4Si^{4+}	4Si^{4+}
$4\text{O}^{2-} + 2\text{OH}^-$	$4\text{O}^{2-} + 2\text{OH}^-$
4Al^{3+}	4Al^{3+}
6OH^-	$4\text{O}^{2-} + 2\text{OH}^-$
	4Si^{4+}
	6O^{2-}

In this representation, the layers designated $4\text{O}^{2-} + 2\text{OH}^-$ are common to the octahedral and tetrahedral sheets. A kaolin-type clay is often said to have a two-layer structure (one gibbsite sheet and one silica sheet), whereas a clay with the montmorillonite pattern is said to have a three-layer structure. Other combinations of the unit sheets are possible, but these two are the most common.

The conventional formula for kaolinite, $\text{Al}_4\text{Si}_4\text{O}_{10}(\text{OH})_6$, is clearly a condensation of the layer arrangement just described. The conventional formula for montmorillonite is $\text{Al}_4(\text{Si}_4\text{O}_{10})_2(\text{OH})_4$; since adsorbed water is always present, a more accurate formula is $\text{Al}_4(\text{Si}_4\text{O}_{10})_2(\text{OH})_4 \cdot n\text{H}_2\text{O}$.

Successive unit layers of the clay structure are stacked one on top of another. Differences in the geometry of stacking give rise to clays with different properties; thus the hydrothermal clay minerals dickite and nacrite have the same two-sheet structure as kaolinite, but differ in the position of one layer with respect to the next. Whatever the arrangement, the distance from the O^{2-} or OH^- ions of one layer to the O^{2-} or OH^- of the next is greater than that between the anions within each layer, so that the mineral splits between the layers much more easily than across them. This, of course, is reflected in the macroscopic (001) cleavage of the clay minerals and micas. Successive layers are more easily separated in the montmorillonite structure than in the kaolinite structure, the difference being correlated with the fact that adjacent layers in kaolinite have O^{2-} ions in one facing OH^- in the next, while in montmorillonite O^{2-} of one layer always faces O^{2-} of another. This difference is often expressed by saying that the kaolinite structure is asymmetric (gibbsite-silica, gibbsite-silica, ...) and the montmorillonite structure symmetric (silica-gibbsite-silica, silica-gibbsite-silica, ...). The attraction of O^{2-} for OH^- is much greater than that of O^{2-} for other O^{2-} ions; one can think of the H^+ from the OH^- as shared between the O^{2-} of adjacent layers of kaolinite (forming a "hydrogen bond"), and so linking the layers together. Much of the difference in macroscopic properties between the two types of clay can be ascribed to the greater ease with which the layers of the symmetric structure separate from one another.

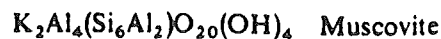
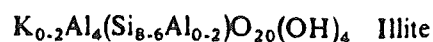
Isomorphous Substitution

Theoretically one might expect a great deal of isomorphous substitution in the clay-mineral structures, since Si^{4+} and Al^{3+} seemingly should be replaceable by many other ions with coordination numbers 4 and 6. The expectation is abundantly fulfilled in montmorillonite clays, but not in kaolinite. This difference in the amount of isomorphous substitution is another of the important differences between the two kinds of structure: kaolinite *always* has a composition approximating the ideal formula $\text{Al}_2\text{Si}_2\text{O}_5(\text{OH})_4$, but montmorillonite *never* has the ideal formula $\text{Al}_2(\text{Si}_2\text{O}_5)_2(\text{OH})_4$ because of extensive substitution. The common substitution in the tetrahedral sheets of montmorillonite is Al^{3+} for Si^{4+} , the amount of substitution being limited to about 15%. In the octahedral sheets a much greater variety of substitution is possible, and the amount may be anywhere from very small to 100%; the most common substitutions are Mg^{2+} and Fe^{3+} , and rarer ones are Zn^{2+} , Ni^{2+} , Li^+ , and Cr^{3+} . A formula for montmorillonite more realistic than the ideal one would be $(\text{Al}, \text{Mg}, \text{Fe}^{3+})_4(\text{Si}, \text{Al})_8\text{O}_{20}(\text{OH})_4 \cdot n\text{H}_2\text{O}$.

The substitution of Al^{3+} for Si^{4+} and of Mg^{2+} for Al^{3+} leaves a deficiency of positive charge in the montmorillonite layers. The deficiency may be compensated in various ways: by replacement of O^{2-} by OH^- , by introduction of excess cations into the octahedral layer (Al^{3+} in the ideal structure fills only two-thirds of the available positions), and by adsorption of cations onto the surface of individual layers. Some of the compensation is always accomplished by the last method, so that a constant characteristic of montmorillonite clays is the presence of abundant adsorbed ions. These ions are held loosely to the clay structure, and are readily replaced by others, so that montmorillonite clays typically have a high capacity for ion exchange.

Illite and Muscovite

One of the ions most firmly held by montmorillonite is potassium, K^+ , a big ion whose size enables it to fit snugly between layers of the clay structure. Clays which have a deficiency of positive charge due largely to substitution in the tetrahedral sheets (hence close to the surface of the layers) hold the K^+ especially tightly, so that only part of it is replaceable by other ions. These clays, with successive layers held together by K^+ ions, have properties different from montmorillonite and are given the name *illite*. (The term *hydromica*, often used in European literature, is practically a synonym.) The illite structure is transitional to that of muscovite, in which the amount of K^+ is greater and bears a constant relation to the amounts of Si and Al. The difference in structure between montmorillonite, illite, and muscovite may be symbolized by the ideal formulas



The term "illite," therefore, is less a name for a definite mineral than a name for a group of substances with compositions intermediate between montmorillonite and muscovite. Recent work, in fact, suggests that much so-called illite is a mechanical mixture of fine-grained montmorillonite and muscovite, or a clay containing alternate layers having a montmorillonite and a muscovite structure. The name is convenient, however, as a general term to designate clay materials with less potassium than that corresponding to the mica formula.

7-2 CLASSIFICATION OF CLAY MINERALS

The structural ideas of the last section provide a good basis for theoretical classification, and structures correlate well enough with macroscopic properties to make the classification a generally useful one.

We divide the clays first into two-layer and three-layer types: those (like kaolin) whose layers consist of one tetrahedral and one octahedral sheet, and those (like montmorillonite and illite) whose layers have an octahedral sheet between two tetrahedral sheets. In general, the kaolinite clays have layers bound more tightly together and permit less substitution of other ions for Al and Si. These structural differences are reflected in less ion-exchange capacity for the kaolinite clays and less plasticity because of a smaller capacity for adsorbing water.

The two-layer clays are further classified according to the degree of filling of the octahedral sites in the gibbsite sheet. In the kaolinite group of minerals Al^{3+} fills only two-thirds of the available positions (hence these are called "dioctahedral" clays); in the serpentine (or septeclorite) group Mg^{2+} and other cations fill all available positions (three per half unit cell, hence "trioctahedral" clays). The kaolinite group is further subdivided, according to the way successive layers are stacked, into kaolinite (by far the commonest mineral of this group), dickite, nacrite, and halloysite.

Among the three-layer clays, an important grouping is based on the ease with which layers are separated from one another. Thus clays whose layers are held together by the attraction of K^+ for strong negative charges in the tetrahedral sheets (illite and muscovite) can be distinguished from clays with more diffuse negative charges originating in the central octahedral sheets and therefore with less tendency for successive layers to be tightly bound by cations (montmorillonite). The latter clays (often called *smectite* as a group, with montmorillonite a principal variety) are commonly described as having expanding lattices, because their layers are readily pushed apart by adsorbed water. This property is strikingly shown by the ability of montmorillonite to swell when placed in water (and other polar liquids), and by its very large ion-exchange capacity. Varieties of smectite may be distinguished on the basis of composition, particularly by the nature of the principal cation in the octahedral sheets: montmorillonite (Al), nontronite (Fe^{3+}), saponite (Mg), sauconite (Zn), hectorite (Li). A related mineral is vermiculite, a

three-layer clay with all octahedral positions occupied by Mg^{2+} and Fe^{2+} and with more substitution of Al for Si than is usual in smectite; it is much less expansible, but has a similar high ion-exchange capacity.

In a broad classification, several other hydrous-silicate minerals produced by weathering, or at least by near-surface processes, may be grouped with the clay minerals. Especially important are the chlorite minerals, minerals whose structure consists of mica-like sheets [with the composition $(\text{Mg,Fe})_6(\text{Si,Al})_8\text{O}_{20}(\text{OH})_4$] alternating with sheets having the structure of brucite [composition $(\text{Mg,Al})_6(\text{OH})_{12}$]. Another way of looking at this structure is to regard it as similar to illite, with successive layers linked by $(\text{Mg,Al})(\text{OH})^+$ ions instead of by K^+ . Glauconite is a variety of illite, with considerable replacement of the Al of the octahedral sheet by Fe^{3+} and of the interlayer K^+ by Ca^{2+} and Na^+ . Another group of hydrous silicates are referred to by the names palygorskite, sepiolite, and attapulgite; their structures are not completely known but are probably based on silicon-oxygen double chains, like those in the structure of amphiboles, rather than on mica-like sheets. The fibrous nature of these minerals is a reflection of the chain structure.

One further complexity of structure is shown by clay minerals whose crystals consist of alternating layers of different kinds, the *mixed-layer* clays. The interlayering may follow a regular pattern—so many layers of one kind, then so many of the other—or it may be completely random. The chlorite minerals may be considered mixed-layer structures with a regular alternation of mica and brucite layers, and some illite is a mixed-layer structure of mica and montmorillonite.

Finally, clays should be mentioned which appear amorphous even to x-rays, clays given the general name *allophane*. Even in these clays there is probably some ordering of Al and O into octahedral patterns and of Si and O into tetrahedral patterns, but the units are so small and so poorly oriented with respect to one another that x-ray reflections cannot be obtained.

7-3 RELATION OF STRUCTURE TO PROPERTIES

In discussing colloidal properties of clays in the last chapter, and details of structure in this chapter, we have already had occasion to point out some of the principal effects of crystal structure on megascopic properties. Here we need do little more than summarize the preceding discussions.

The properties of clays in general depend on their small grain size. In most samples of clay the particles have dimensions entirely or partially within the colloidal range, which means that they remain suspended in water for long periods, that they may be flocculated by electrolytes, that they adsorb ions out of solution, and that the adsorbed ions may be replaced by others when the concentration of the solution changes. Also, by virtue of their small size, clay particles have the ability to adsorb water and organic materials and so to become plastic, in other words, to remain coherent and capable of being molded when wet. These

properties, although possessed in some degree by all clays, differ markedly from one clay mineral to another.

Minerals of the kaolinite group, because layers in the structure are held tightly together by the opposition of O^{2-} and OH^- and because little substitution of other cations for Al^{3+} and Si^{4+} is possible, show less capacity for adsorbing ions and water than do the other clays. Montmorillonite, with layers more easily separable and with extensive substitution, has abundant adsorbed ions and shows the capacity to adsorb water to an extreme degree. Montmorillonite therefore is much more plastic than kaolinite; its plasticity depends somewhat on the kind of adsorbed ions, being greatest for sodium and hydrogen montmorillonites and least for calcium montmorillonites. The extensive use of montmorillonite clays in drilling muds and in filters to remove undesirable ions and organic coloring materials from water and other liquids depends likewise on their adsorptive capacity. Illite clays and chlorites, with structural layers partly bound together by nonexchangeable cations, show intermediate plasticity and ion-exchange capacity. Vermiculite may have an ability to adsorb ions as great as that of montmorillonite, but its capacity for swelling by adsorption of water is more limited.

Most clays in nature are mixtures of two or more clay minerals, and their properties are accordingly intermediate between the extremes. It is surprising, however, how greatly the properties of a clay sample may be influenced by a minor constituent; in particular, a small percentage of montmorillonite in a clay can radically change its plasticity. This ability of certain clays to modify the properties of mixtures makes difficult predictions about the behavior of a given sample from its composition alone. Much of our information about the behavior of clays, particularly about mixtures of different clay minerals, remains empirical. The more obvious properties we can link in general terms to crystal structures, but detailed correlation is still impossible.

7-4 FORMATION OF THE CLAY MINERALS

The clay minerals are formed by alteration of aluminum silicates, both in weathering and in low-temperature hydrothermal processes. This much is obvious simply from geologic relationships, but details of the alteration process remain obscure. How does the chain structure of an amphibole, or the three-dimensional framework of a feldspar, convert itself into the hexagonal sheets of the clay minerals? Is it by a progressive modification of the preexisting structure, or do hydrated Al-O groups and Si-O groups detach themselves from the structure and later recombine into clays? If the second alternative is possible, can clays then precipitate from solution—say by reaction between dissolved silica and colloidal alumina in lakes or in the sea? These are some of the major conundrums about clay minerals which are still to be answered.

An obvious approach to such questions is through laboratory experiments. As so often happens in geochemistry, experiments carried out at low temperatures, under conditions duplicating as closely as possible those in nature, give unsatisfac-

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