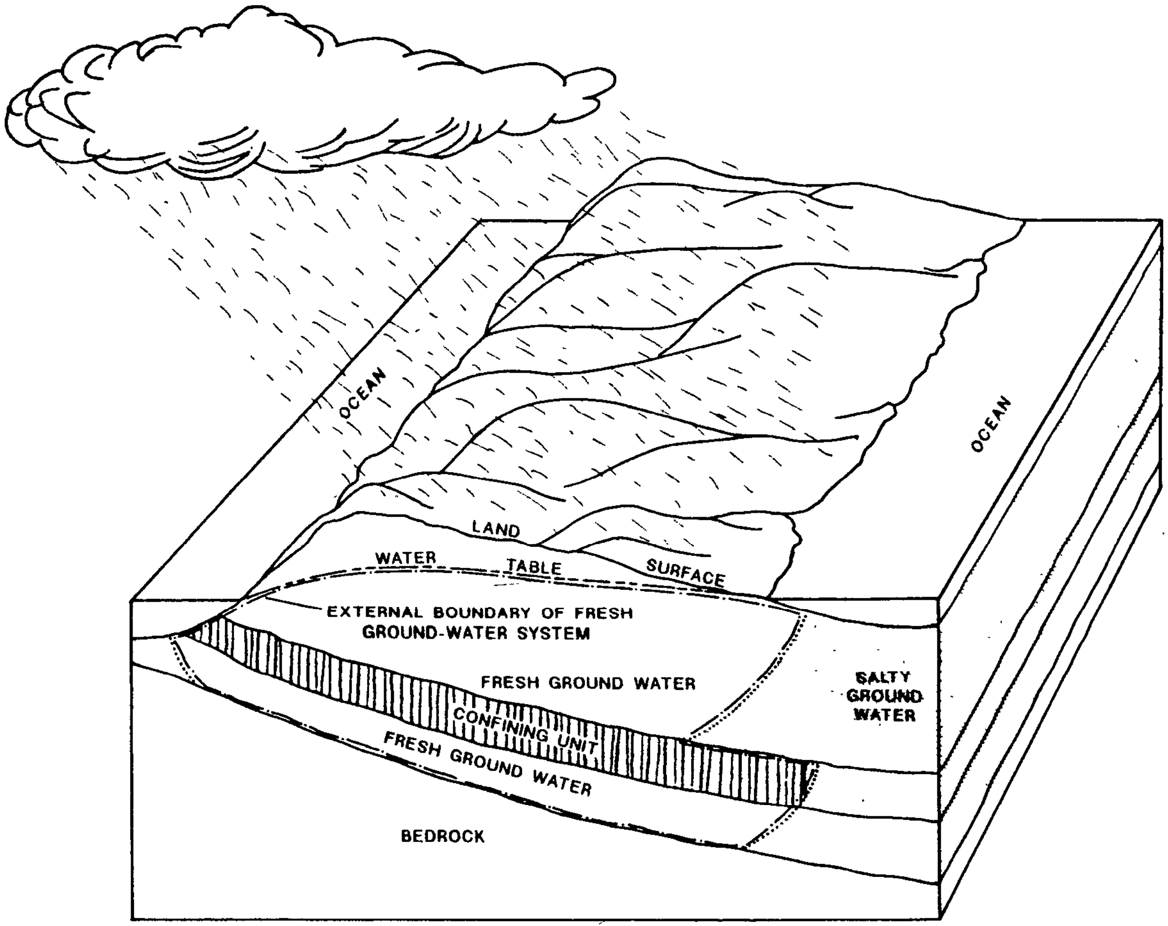
STUDY GUIDE FOR A BEGINNING COURSE IN GROUND-WATER HYDROLOGY: PART II -- INSTRUCTOR'S GUIDE



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Concept of Superposition and its Application to Well-Hydraulic Problems

Assignments

*Study Fetter (1988), p. 201-204; Freeze and Cherry (1979), p. 327-332; or Todd (1980), p. 139-149.

*Study Note (4-5)--Application of superposition to well-hydraulic problems.

*Work Exercise (4-4)--Superposition of drawdowns caused by a pumped well on the pre-existing head distribution in an areal flow system.

Superposition is a concept that has many applications to ground-water hydrology as well as to other physical systems that are described by linear differential equations. We use superposition when we analyze (most) aquifer tests, perhaps without realizing this fact, and in the theory of images and image wells. Superposition also has applications to the numerical simulation of ground-water systems, a topic that is not discussed in this course.

Reference

Reilly, Franke, and Bennett (1987)

Comments

A comprehensive overview of the principle of superposition is provided by Reilly and others (1987a). Todd (1980) offers a thorough review of image-well theory, which is a first-priority extension of this course on the topics of superposition and radial flow because it deals with the effects of hydrogeologic boundaries on the drawdown response of water levels to a pumped well.

Answers to Exercise (4-4)--Superposition of Drawdowns Caused by a Pumped Well on the Pre-Existing Head Distribution in an Areal Flow System

The next pages contain (1) tabulated calculations in table 4-3, (2) tabulated calculations in table 4-4, (3) contoured new potentiometric surface in response to pumping on figure 4-10, and (4) answers to two questions.

Table 4-3.--Format for calculation of drawdowns at specified distances from the pumped well

[r_e is distance from pumped well at which drawdown is negligible; r_1 is distance from pumped well at which drawdown equals s_1; *ln* is natural logarithm; Q is pumping rate of well; T is transmissivity of aquifer]

Preliminary calculation: $\dfrac{-Q}{2\pi T}$ = constant = $\dfrac{-9{,}090\ ft^3/d}{2\pi \bullet 1{,}000\ ft^2/d}$ = -1.447 ft

r_1 (feet)	$ln\ (r_e/r_1)$	s_1 (feet) = $-\dfrac{Q}{2\pi T}\ ln\ (r_e/r_1)$
250	3.00	4.34
500	2.30	3.33
707	1.96	2.84
1,000	1.61	2.33
1,118	1.50	2.17
1,414	1.26	1.82

Answers to Exercise (4-4) (continued)

Table 4-4.--Format for calculation of absolute heads at specified reference points

Well identifier	Initial prepumping head	Distance from well (r)	Drawdown due to pumping	Head = initial head − drawdown
A	16.00	1,414	1.82	14.18
B	17.00	1,118	2.17	14.83
C	18.00	1,000	2.33	15.67
D	19.00	1,118	2.17	16.83
E	20.00	1,414	1.82	18.18
F	15.00	1,118	2.17	12.83
G	16.00	707	2.84	13.16
H	17.00	500	3.33	13.67
I	18.00	707	2.84	15.16
J	19.00	1,118	2.17	16.83
K	16.50	250	4.34	12.16
L	14.00	1,000	2.33	11.67
M	15.00	500	3.33	11.67
N	15.50	250	4.34	11.16
O	16.50	250	4.34	12.16
P	17.00	500	3.33	13.67
Q	18.00	1,000	2.33	15.67
R	15.50	250	4.34	11.16
S	13.00	1,118	2.17	10.83
T	14.00	707	2.84	11.16
U	15.00	500	3.33	11.67
V	16.00	707	2.84	13.16
W	17.00	1,118	2.17	14.83
X	12.00	1,414	1.82	10.18
Y	13.00	1,118	2.17	10.83
Z	14.00	1,000	2.33	11.67
AA	15.00	1,118	2.17	12.83
BB	16.00	1,414	1.82	14.18
CC	15.29	250	4.34	10.95
DD	14.57	500	3.33	11.24

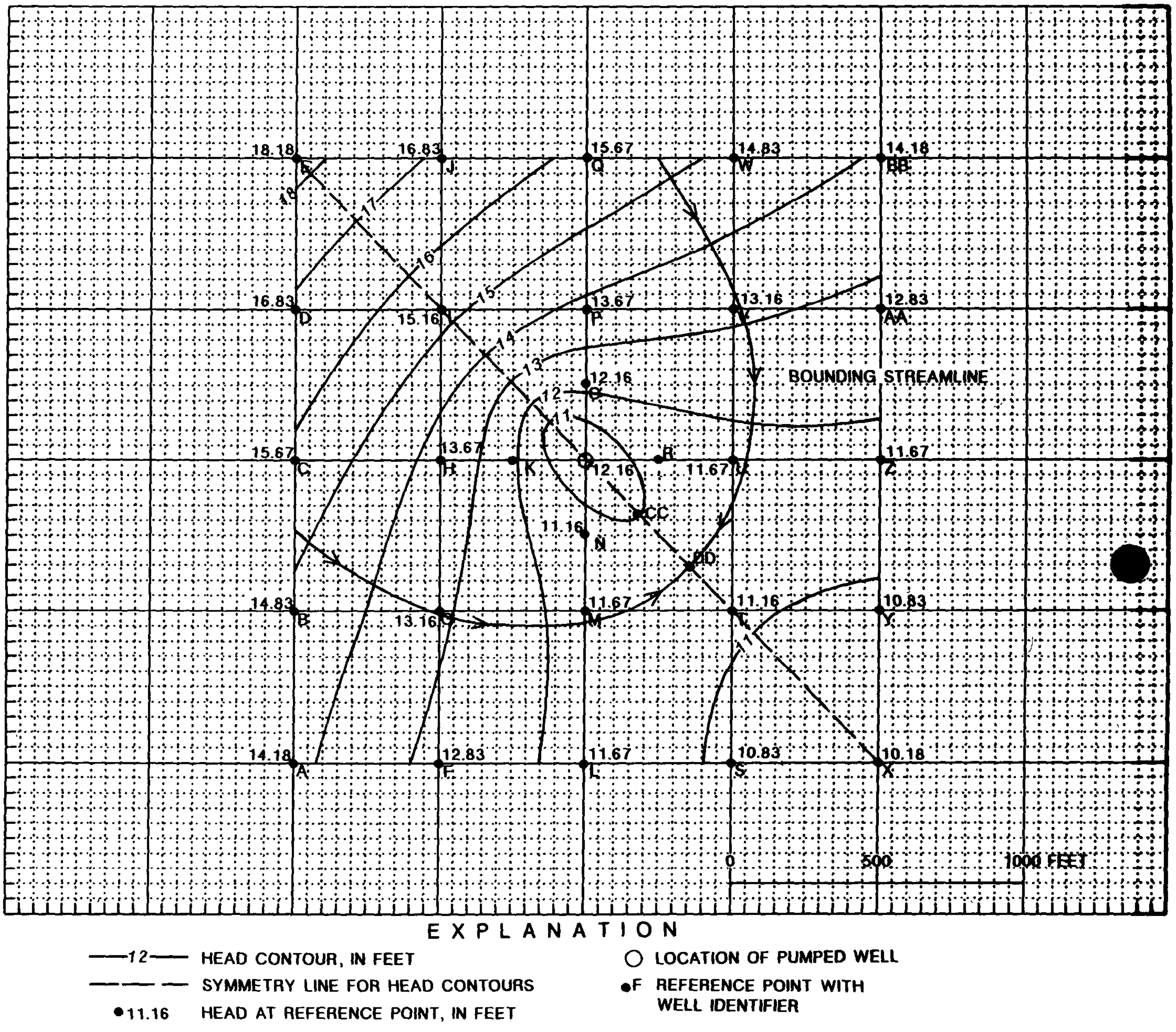


Figure 4-10.--Head distribution in confined areal flow system resulting from pumping.

Answers to Exercise (4-4) (continued)

Question (1). The first two streamlines drawn on figure 4-10 represent a type of ground-water divide that is analogous to the hydrologic situation depicted in figure 3-35, Exercise (3-3). Between these two bounding streamlines, all streamlines in the aquifer terminate at the pumped well. Outside the area bounded by these two streamlines, all streamlines in the aquifer continue to flow downgradient beyond the well as part of the regional flow system. The area between the two bounding streamlines is called the <u>area of diversion</u> of the pumped well.

Consider the steady-state, three-dimensional configuration of potential surfaces and related streamlines that are found in a ground-water system in equilibrium with a single pumped well. Conceptually trace upgradient all the streamlines that terminate at the pumped well to their point of entry into the saturated ground-water system. In real systems, this point of entry is generally at the water table or at the bottom or bank of a surface-water body. The shape of the volume of saturated earth material that is defined by this "bundle" of streamlines can be complex, particularly for wells screened near the bottom of thick unconfined aquifers or in confined aquifers between leaky confining units.

The term "contributing area" usually is used to define the area through which water enters the ground-water system and is synonymous with the term "recharge area." Thus, this area constitutes the starting points for the "bundle" of streamlines that enter the ground-water system through a boundary surface. The area of diversion, however, is the projected area in map view of the entire bundle of streamlines as they flow to their point of discharge at the well.

In terms of the class problem under discussion, it is useful, as always, to review its boundary conditions and implicit assumptions--(1) the pumped well is screened in a confined aquifer; thus, the saturated thickness of the pumped aquifer is assumed to remain constant; (2) the analysis is done with the assumption of two-dimensional flow in plan view; this assumption is best approximated in real systems if the pumped well completely penetrates the confined aquifer; and (3) the source of water to the regional flow system before pumping and to the pumped system is a plane constant-head boundary located at a great distance upgradient from the pumped well; no water enters this system by leakage through an overlying or underlying confining unit.

The area of diversion in the class problem, therefore, is the surface projection of the area encompassed by the two bounding flow lines drawn on figure 4-10. The location of the recharge area or contributing area depends on the actual source of water entering the ground-water system. This source of water is not explicitly stated in our problem; however, because the equipotential lines in the undisturbed system are evenly spaced, we can assume, as stated above, that there is no local source of water entering the system and the source of water must be an upgradient-plane constant-head boundary. The contributing or recharge area can not be defined for the class problem as given, and it would exist beyond the area shown in figure 4-10.

The existing terminology as used in many reports for terms such as zone of contribution, contributing area, and area of diversion is frequently confusing because it is based on two-dimensional systems, and it is imprecisely defined for three-dimensional systems. In three-dimensional systems it is desirable to identify the volume of earth material and contained fluid that is associated with flow to a pumped well, and to envision the changing shape of the actual "bundle" of flow tubes that constitute this volume from its entry into the flow system to its discharge from the system at the well. This conceptualization should be clearly explained in reports, instead of relying on terminology that is frequently misleading.

Question (2). The purpose of this question is to emphasize the difference between the area of diversion of the pumped well and the area of influence of the pumped well. Theoretically, the area of influence of the pumped well extends to the aquifer boundaries; in a practical sense, however, we can define the area of influence as the area of the aquifer in which we can measure drawdowns resulting from the influence of the pumped well that are greater than or equal to 0.01 ft. Our calculated data and contour map (fig. 4-10) show that (1) the ground-water divide between the two "areas" exists between reference points S, T, X, and Y and the pumped well; and (2) quantitatively significant drawdowns, as exemplified by the calculated drawdowns at these four points, are found inside the area of influence but outside the area of diversion of the pumped well.

Aquifer Tests

Assignments

*Study Fetter (1988), p. 204-209; Freeze and Cherry (1979), p. 335-343, 349-350; or Todd (1980), p. 45-46, 70-78.

*Study Note (4-6)--Aquifer tests.

One of the main activities of ground-water hydrologists is to estimate physically reasonable values of aquifer parameters for different parts of the ground-water system under study. The most powerful and direct field method for obtaining aquifer parameters is a carefully designed, executed, and analyzed aquifer test. Unfortunately, aquifer tests are labor- and time-intensive. Often, the most important decision concerning an aquifer test is whether or not to perform one--in other words, whether the value of the test results equals the cost of obtaining those data. This question generally is difficult to answer.

References

Heath and Trainer (1968), p. 83-84, 119-127.
Lohman (1972a), p. 52-54.
Stallman (1971).

Comments

Our goal in this subsection is to initiate a discussion of aquifer tests--what they are and what we seek to accomplish by undertaking them, their advantages and disadvantages, and their implementation in three phases--design, field measurements, and data analysis. In addition, adequate information is available in the keyed course textbooks and other listed references for a discussion of other ways in which hydrogeologists estimate aquifer and confining-unit coefficients. Introduction of this information is appropriate at this time.

SECTION (5)--GROUND-WATER CONTAMINATION

The goal of this section of the course is to introduce the physical mechanisms of solute movement in ground water. Further treatment of the vast and rapidly developing area of science and technology related to ground-water contamination can be found in the extensive literature that is available or in additional training courses.

Background and Field Procedures Related to Ground-Water Contamination

Assignments

*Study Fetter (1988), p. 367-389, 406-442; Freeze and Cherry (1979), p. 384-457; or Todd (1980), p. 344-346.

The depth of topical coverage in this section of the course will depend primarily on the time available and the interests of the instructors and participants. A useful and readable discussion on the conceptualization and organization of a field study involving solute transport, along with a pertinent bibliography, is provided by Reilly and others (1987).

Reference

Reilly, Franke, Buxton, and Bennett (1987)

Comments

The focus of this course is hydrogeology and the hydraulics of ground-water flow. A section on ground-water contamination is included primarily because of its present-day topical interest. The keyed course textbooks and the reference above provide much more information on ground-water contamination than can be discussed in this course. Freeze and Cherry (1979, p. 384-401) provide a thorough introduction to the physical mechanisms of solute transport.

Physical Mechanisms of Solute Transport in Ground Water

Assignments

*Study Fetter (1988), p. 389-405.

*Study Note (5-1)--Physical mechanisms of solute transport in ground water

*Work Exercise (5-1)--Ground-water travel times in the flow system beneath a partially penetrating impermeable wall

*Work Exercise (5-2)--Advective movement and travel times in a hypothetical stream-aquifer system

*Study Note (5-2)--Analytical solutions for analysis of solute transport in ground water

*Work Exercise (5-3)--Application of the one-dimensional advective-dispersive equation

The background for this section is provided in Note (5-1), which is an introductory discussion of the basic physical mechanisms of solute movement--advection and dispersion. Exercises (5-1) and (5-2) consider only advective movement of ground water and involve calculation of travel times by using the average linear velocity (Darcy velocity divided by porosity). In Exercise (5-1), travel times are calculated in a vertical section of a simple flow system; in Exercise (5-2), travel times are calculated in plan view.

Comments on the field application of analytical solutions to the advective-dispersive differential equation are provided in Note (5-2), and Exercise (5-3) involves numerical calculations with one of the simplest analytical solutions.

Answers to Exercise (5-1)--Ground-Water Travel Times in the Flow System Beneath a Partially Penetrating Impermeable Wall

With reference to the plotted time-of-travel values (as calculated in table 5-1) and related equal-time contours in figure 5-5, the total time of travel from the recharge boundary to the discharge boundary along the longest bounding streamline is about 20 times greater than the total travel time along the shortest bounding streamline around the impermeable wall. The time of travel for increments of the longest streamline vary widely. The longest travel times per unit length of streamline are found in the lower left-hand corner, in the lower right-hand corner, and at the right-hand vertical boundary. This observation is predictable from the low head gradients in these regions. The shortest times of travel in this system are found beneath the impermeable wall, where head gradients are greatest.

Hydrologists are not accustomed to calculating time-of-travel contours and visualizing their general pattern in ground-water systems. The pattern of these contours does not bear a visually obvious relation to the more familiar head contours and streamlines. Because of the present-day prevalence of contamination studies and the advent of particle-tracking algorithms in association with digital flow models, we can expect ever-increasing applications of time contours and "surfaces" in ground-water studies.

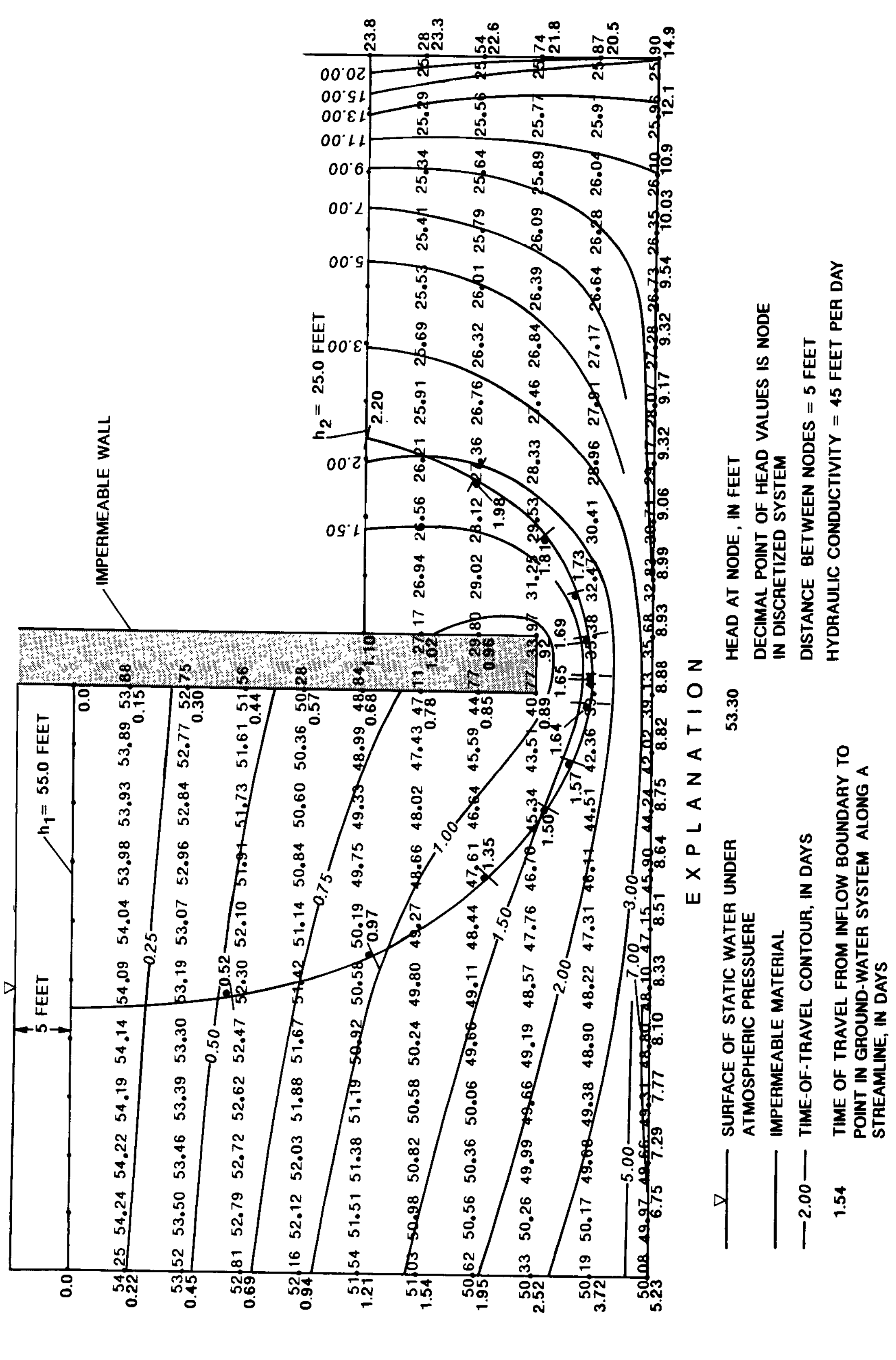


Figure 5-5.--Contours of equal time of travel from the upper left-hand inflow boundary in the impermeable-wall ground-water system.

Table 5-1.--Format for calculation of time of travel along selected flowlines in impermeable-wall problem (page 1 of 3)

[h is head at a node or other point in flow system; L is distance between two points on a flowline at which head is known; Δh is difference in head between two points on a flowline; t is time of travel between two points on a flowline; Σt is time of travel from inflow boundary to point on flowline; Ψ is stream function]

	h (feet)	L (feet)	Δh (feet)	t (days) = $\frac{6.67 \times 10^{-3} L^2}{\Delta h}$	Σt (days)
Flowline (a), Ψ = 0	55.00	--	--	--	--
	54.25	5.0	0.75	0.223	0.223
	53.52	5.0	.73	.229	.452
	52.81	5.0	.71	.235	.687
	52.16	5.0	.65	.257	.944
	51.54	5.0	.62	.269	1.213
	51.03	5.0	.51	.327	1.54
	50.62	5.0	.41	.407	1.947
	50.33	5.0	.29	.576	2.523
	50.19	5.0	.14	1.193	3.716
	50.08	5.0	.11	1.518	5.234
	49.97	5.0	.11	1.518	6.752
	49.66	5.0	.31	.539	7.291
	49.31	5.0	.35	.477	7.768
	48.80	5.0	.51	.327	8.095
	48.10	5.0	.70	.239	8.334
	47.15	5.0	.95	.176	8.510
	45.90	5.0	1.25	.134	8.644
	44.24	5.0	1.66	.101	8.745
	42.02	5.0	2.22	.075	8.820
	39.13	5.0	2.89	.058	8.878
	35.68	5.0	3.45	.048	8.926
	32.83	5.0	2.85	.059	8.985
	30.71	5.0	2.12	.079	9.064
	29.17	5.0	1.54	.108	9.172
	28.07	5.0	1.10	.152	9.324
	27.28	5.0	.79	.211	9.535
	26.73	5.0	.55	.304	9.839
	26.35	5.0	.38	.439	10.278
	26.10	5.0	.25	.668	10.946

Table 5-1.--Format for calculation of time of travel along selected flowlines in impermeable-wall problem (page 2 of 3)

	h (feet)	L (feet)	Δh (feet)	t (days) = $\frac{6.67 \times 10^{-3}\ L^2}{\Delta h}$	Σt (days)
	25.96	5.0	0.14	1.193	12.139
	25.90	5.0	.06	2.783	14.922
	25.87	5.0	.03	5.567	20.489
	25.74	5.0	.13	1.285	21.774
	25.54	5.0	.20	.835	22.609
	25.28	5.0	.26	.642	23.251
	25.00	5.0	.28	.596	23.847
			30.00		
Flowline	55.00	5.0	--	--	--
(f),	53.88	5.0	1.12	.149	.149
Ψ = 1.0	52.75	5.0	1.13	.148	.297
	51.56	5.0	1.19	.140	.437
	50.28	5.0	1.28	.130	.567
	48.84	5.0	1.44	.116	.683
	47.11	5.0	1.73	.097	.780
	44.77	5.0	2.34	.071	.851
	40.77	5.0	4.00	.042	.893
	33.97	5.0	6.80	.025	.918
	29.80	5.0	4.17	.040	.958
	27.17	5.0	2.63	.063	1.021
	25.00	5.0	2.17	.077	1.098
			30.00		

Answers to Exercise (5-1) (continued)

Table 5-1.--Format for calculation of time of travel along selected flowlines in impermeable-wall problem (page 3 of 3)

	h (feet)	L (feet)	Δh (feet)	t (days) = $\frac{6.67 \times 10^{-3} L^2}{\Delta h}$	Σt (days)
Flowline (c), Ψ = 0.40	55.00	--	--	--	--
	52.50	14	2.50	0.52	0.52
	50.00	13	2.50	.45	.97
	47.50	12	2.50	.38	1.35
	45.00	7.5	2.50	.15	1.50
	42.50	5	2.50	.07	1.57
	40.00	5	2.50	.07	1.64
	37.50	2	2.50	.01	1.65
	35.00	4	2.50	.04	1.69
	32.50	4	2.50	.04	1.73
	30.00	5.5	2.50	.08	1.81
	27.50	8	2.50	.17	1.98
	25.00	9	2.50	.22	2.20

Answers to Exercise (5-2)--Advective Movement and Travel Times in a Hypothetical Stream-Aquifer System

I. For Point A (as shown on figure 5-6):

1. Length:

$L_a = 1.65 \text{ mi} = 8{,}712 \text{ ft}$

2. Velocity:

$$v_A = \frac{K}{n}\frac{dh}{dl} = \frac{125 \text{ ft/d}}{.33} \bullet \frac{16 \text{ ft}}{1.65 \text{ mi}} \bullet \frac{1 \text{ mi}}{5{,}280 \text{ ft}} = 0.70 \text{ ft/d}$$

3. Time of travel:

$$t_A = \frac{L_A}{v_A} = \frac{8{,}712 \text{ ft}}{0.70 \text{ ft/d}} = 12{,}446 \text{ d} = 34.1 \text{ yr}$$

II. For Point B (as shown on figure 5-6):

1. Length:

$L_B = 2.90 \text{ mi} = 15{,}312 \text{ ft}$

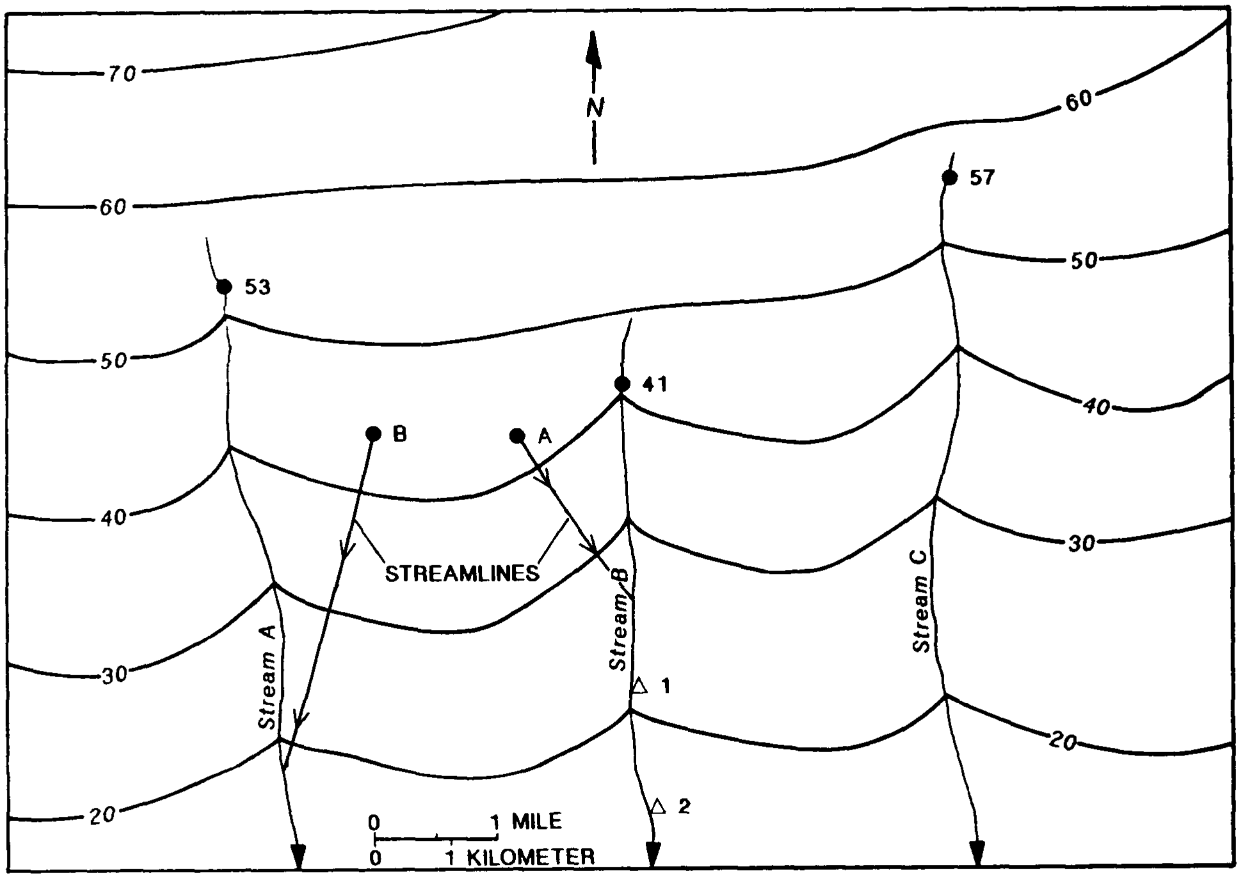
2. Velocity:

$$v_B = \frac{125 \text{ ft/d}}{.33} \bullet \frac{26 \text{ ft}}{2.90 \text{ mi}} \bullet \frac{1 \text{ mi}}{5{,}280 \text{ ft}} = 0.64 \text{ ft/d}$$

3. Time of travel:

$$t_B = \frac{L_B}{v_B} = \frac{15{,}312 \text{ ft}}{0.64 \text{ ft/d}} = 23{,}925 \text{ d} = 65.5 \text{ yr}$$

Comment: Estimates of travel time based on water-table maps and available estimates of hydraulic conductivity are simple to calculate, require minimal time, and provide an approximate but generally reliable frame of reference for travel time that is the foundation of any investigation involving ground-water contamination.



EXPLANATION

—20— WATER-TABLE CONTOUR -- Shows altitude of water table. Contour interval 10 feet. Datum is sea level

● 41 LOCATION OF START-OF-FLOW OF STREAM -- Number is altitude of stream, in feet above sea level

△ 2 LOCATION AND NUMBER OF STREAM-DISCHARGE MEASUREMENT POINT

Figure 5-6.--Hypothetical water-table map of an area underlain by permeable deposits in a humid climate showing streamlines from point A to stream B and from point B to stream A.

Answers to Exercise (5-3)--Application of the One-Dimensional Advection-Dispersion Equation

The following pages contain calculations of concentration as a function of distance from the source in tables 5-2 and 5-3, and a plot of these data in figure 5-9.

The curves in figure 5-9, based on the calculations in tables 5-2 and 5-3, provide a visual summary of classical advection-dispersion theory and the role of the dispersion coefficient. The frame of reference is the vertical line representing a "sharp front" between contaminated and uncontaminated ground water at a distance, L = 2,000 ft, from the contaminant source. The existence of a sharp front implies pure advective transport, or no mixing across the front.

The principal reference point on the vertical sharp front line is the point at which the relative concentration $C/C_0 = 0.50$. Curves of relative concentration for dispersion coefficients are symmetrical about this point for conditions where the simplified equation (3) is valid (i.e. the dispersion coefficient is small, or the distance is far from the boundary, or the time is large). For smaller coefficients of dispersion, at a given time and distance from the source, the symmetrical mixing zone relative to the sharp-front reference line is relatively narrow. For larger dispersion coefficients, at a given time and distance from the source, the zone of mixing is broader and may extend to the contaminant source.

Table 5-2.--Format for calculating solute concentrations when the dispersion coefficient $D = 10$ square feet per day and the elapsed time $t = 1,000$ days

[d, days; ft/d, feet per day; ft^2/d, square feet per day; mg/L, milligrams per liter]

Formula for calculations: $C = \frac{C_0}{2} \operatorname{erfc}\left(\frac{L - vt}{2\sqrt{Dt}}\right)$ where

- C = concentration of solute at point in plume at specified time, in mg/L
- C_0 = solute concentration of source, in mg/L
- L = distance from source, in feet
- v = average linear velocity of ground water, in ft/d
- t = elapsed time since introduction of solute at source, in d
- D = dispersion coefficient, in ft^2/d
- erfc = complementary error function (see Fetter, 1988, p. 562)

Preliminary calculation:

$$\text{For } D = 10\ ft^2/d,\ \left(\frac{L - vt}{2\sqrt{Dt}}\right) = \frac{L - 2ft/d \bullet 1,000\ d}{2\sqrt{10ft^2/d \bullet 1,000\ d}} = \frac{L - 2,000}{200}$$

L (feet)	$\frac{L-2,000}{200}$	$\operatorname{erfc}\left(\frac{L-2,000}{200}\right)$ [1]	$C = 50\ mg/L\ \operatorname{erfc}\left(\frac{L-2,000}{200}\right)$
1,500	-2.5	1.999	100. mg/L
1,600	-2.0	1.995	99.75 mg/L
1,700	-1.5	1.966	98.3 mg/L
1,800	-1.0	1.8427	92.1 mg/L
1,900	- .5	1.5205	76.0 mg/L
2,000	0.0	1.000	50.0 mg/L
2,100	.5	.4795	24.0 mg/L
2,200	1.0	.1573	7.9 mg/L
2,300	1.5	.0339	1.7 mg/L
2,400	2.0	.0047	.24 mg/L

[1] $\operatorname{erfc}(-x) = 1 + \operatorname{erf}(x)$

Table 5-3.--Format for calculating solute concentrations when the dispersion coefficient D = 100 square feet per day and the elapsed time t = 1,000 days

[d, days; ft/d, feet per day; ft²/d, feet squared per day; mg/L, milligrams per liter]

Formula for calculations: $C = \frac{C_0}{2} \operatorname{erfc}\left(\frac{L - vt}{2\sqrt{Dt}}\right)$ where

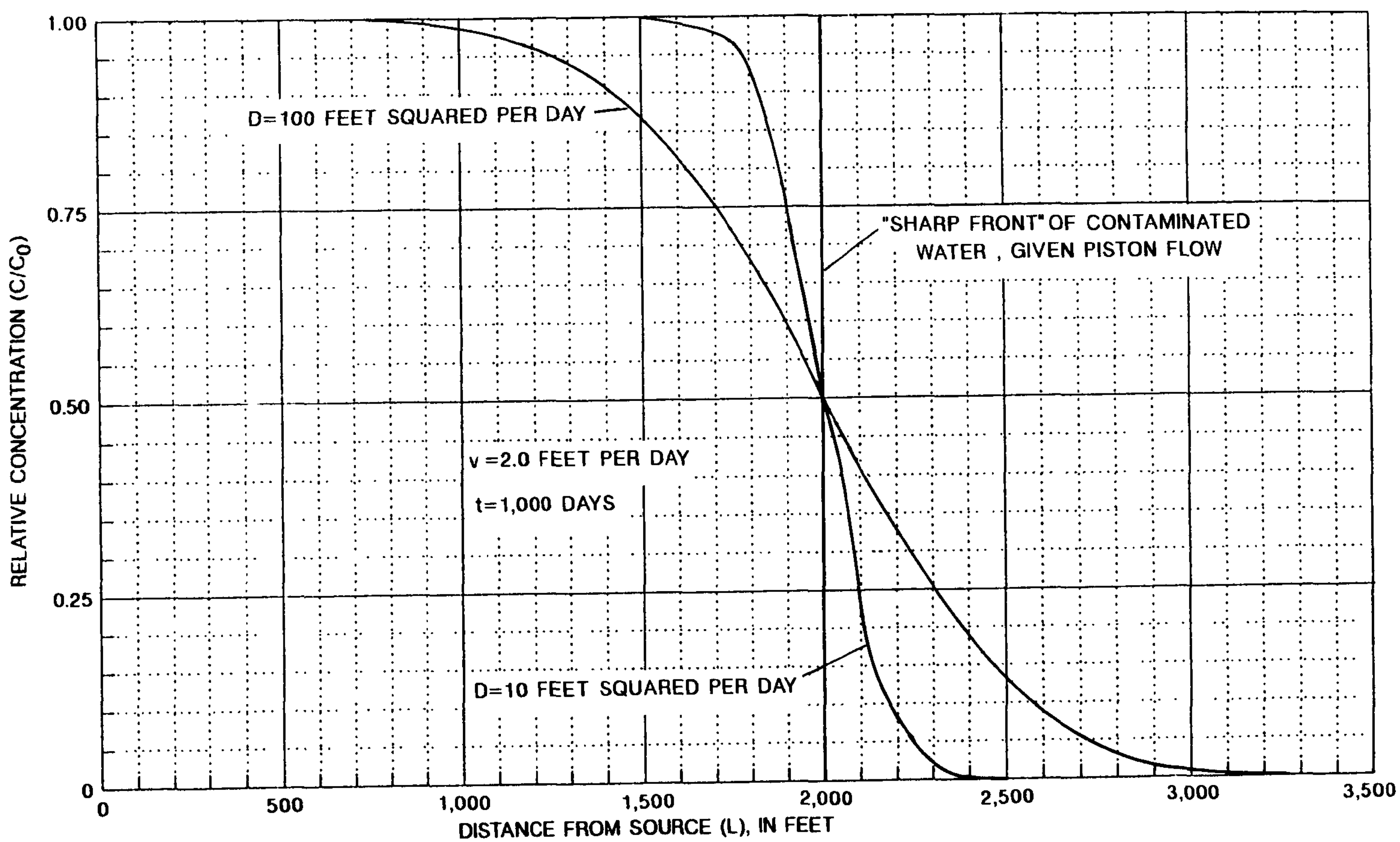
C = concentration of solute at point in plume at specified time, in mg/L
C_0 = solute concentration of source, in mg/L
L = distance from source, in feet
v = average linear velocity of ground water, in ft/d
t = elapsed time since introduction of solute at source, in days
D = dispersion coefficient, in ft^2/d
erfc = complementary error function (see Fetter, 1988, p. 562)

Preliminary calculation:

$$\text{For } D = 100\ ft^2/d,\ \left(\frac{L - vt}{2\sqrt{Dt}}\right) = \frac{L - 2ft/d \bullet 1{,}000\ d}{2\sqrt{100ft^2/d \bullet 1{,}000\ d}} = \frac{L - 2{,}000}{632.5}$$

L (feet)	$\frac{L-2{,}000}{632.5}$	$\operatorname{erfc}\left(\frac{L-2{,}000}{632.5}\right)$ [1]	$C = 50\ mg/L\ \operatorname{erfc}\left(\frac{L-2{,}000}{632.5}\right)$
1,000	-1.58	1.974	98.7 mg/L
1,250	-1.185	1.905	95.3 mg/L
1,500	- .791	1.736	86.8 mg/L
1,750	- .40	1.428	71.4 mg/L
2,000	0.0	1.000	50. mg/L
2,250	.40	0.572	28.6 mg/L
2,500	.791	0.264	13.2 mg/L
2,750	1.185	0.095	4.75 mg/L
3,000	1.58	0.026	1.3 mg/L

[1] erfc(-x) = 1 + erf(x)



EXPLANATION

C_0 = SOLUTE CONCENTRATION OF SOURCE
C = CONCENTRATION OF SOLUTE AT POINT IN PLUME
D = DISPERSION COEFFICIENT, IN FEET SQUARED PER DAY

Figure 5-9.--Plot of relative concentration against distance from source for two values of the dispersion coefficient D and an elapsed time of 1,000 days.

SELECTED REFERENCES

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