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VERTICAL MOVEMENT OF GROUND WATER UNDER THE MERRILL FIELD LANDFILL, ANCHORAGE, ALASKA

By Gordon L. Nelson

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CONVERSION TABLE

Multiply	By	To obtain
inch (in.)	25.40	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
acre	0.4047	hectare (ha)
square foot (ft^2)	0.0929	square meter (m^2)
foot per day (ft/d)	0.3048	meter per day (m/d)
foot squared per day (ft^2/d)	0.0929	meter squared per day (m^2/d)
gallon per minute (gal/min)	0.06309	liter per second (L/s)
million gallons per day (Mgal/d)	0.04381	cubic meters per second (m^3/s)

Note: National Geodetic Vertical Datum of 1929 (NGVD of 1929) is a geodetic datum derived from a general adjustment of the first order level nets of both the United States and Canada. It was formerly called Sea Level Datum of 1929 or mean sea level. NGVD of 1929 is referred to as sea level in this report.

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ABSTRACT

Shallow ground water under the Merrill Field sanitary landfill at Anchorage is polluted by leachate. Wells, including three municipal-supply wells, obtain water from two confined aquifers 100-300 feet beneath the landfill area. Aquifer-test data and information on subsurface geology, ground-water levels, and properties of materials were used to estimate vertical gradients and vertical permeabilities under the landfill. The author's "best estimates" of vertical permeabilities of two confining units are 1×10^{-2} feet per day and 2×10^{-4} feet per day. Theoretical travel-time calculations indicate that minor amounts of pollutants may reach the upper confined aquifer after many tens of years, but that water of the composition of the leachate probably would not reach the aquifer for more than three centuries. The range of error in the theoretical travel-time calculations is likely to be plus or minus a factor of two or three.

INTRODUCTION

The Merrill Field solid-waste landfill (fig. 1) has been utilized for refuse disposal since about 1950. When the site is fully utilized, approximately 150 acres will be covered with soil and refuse to an average thickness of about 30 ft. Some of the refuse has been buried below the water table, thereby creating an environment in which the refuse is continuously leached.

Three municipal-supply wells withdraw water from a lower confined aquifer 200 to 300 ft below the land surface in the vicinity of the landfill. Other wells near and downgradient from the landfill obtain water from an upper confined aquifer 75 to 190 ft below land surface. The purpose of this study was to use existing aquifer-test data to estimate the hydraulic gradients and hydraulic properties that affect the rate at which the polluted shallow ground water migrates downward from the landfill to the aquifers that supply the municipal wells.

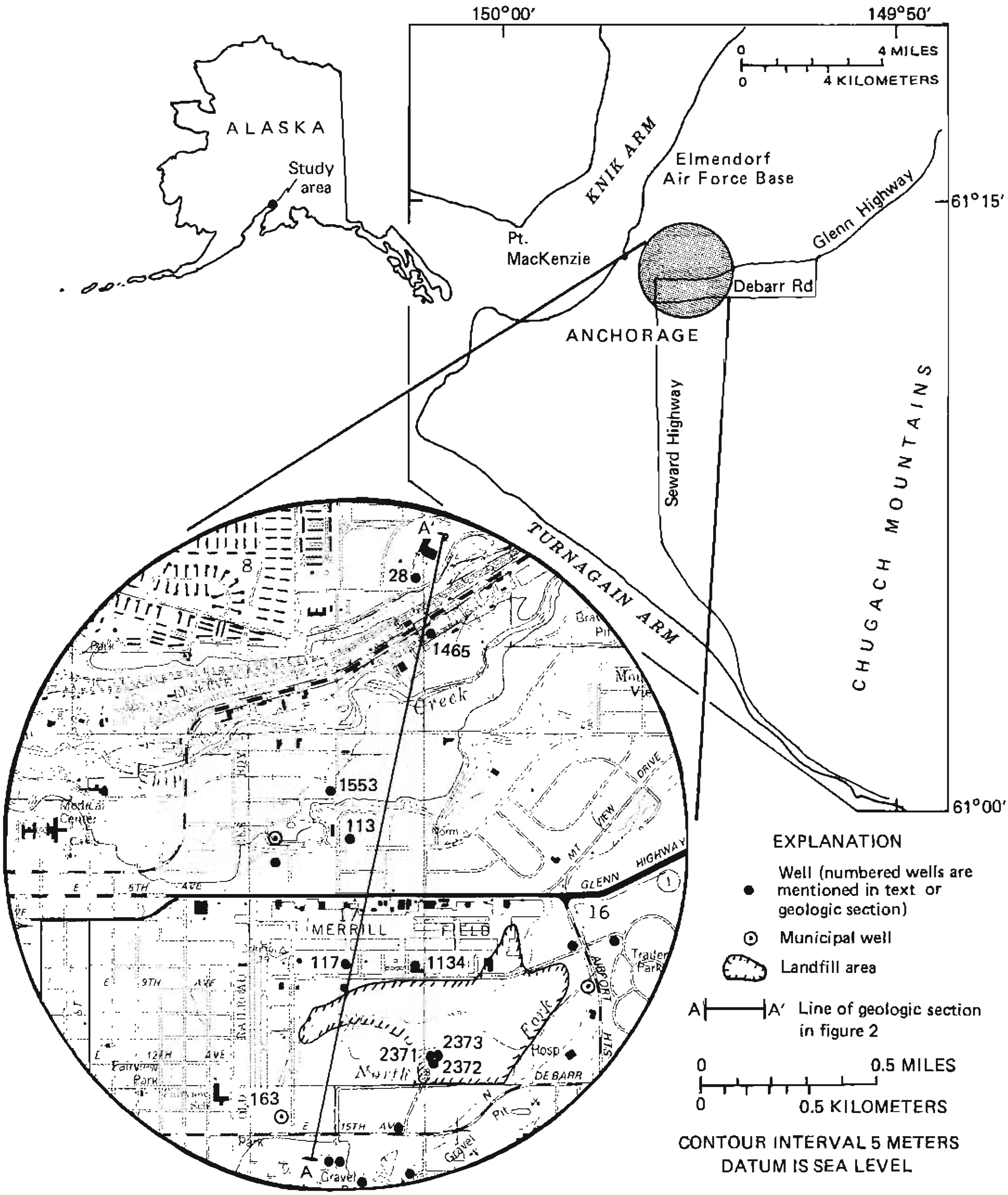

Figure 1.--Location of study area and wells.

The U.S. Geological Survey and the Municipality of Anchorage funded this study. (In this report the term Municipality applies both to the present municipal government and the former City of Anchorage government.) The Municipality drilled the test wells and provided information on topography and subsurface materials in the study area.

The author gratefully acknowledges the work of Larry Dearborn of the Alaska Division of Geological and Geophysical Surveys. Mr. Dearborn, a former employee of the U.S. Geological Survey, analyzed the compaction data and assembled all the pumping and aquifer-test data in this report.

GEOLOGY

The landfill area is underlain by a complex sequence of stratified glacial, fluvial, and lacustrine sediments which the author generalized into seven units (fig. 2). The saturated part of Unit I is the unconfined aquifer. Units III and V are lithologically similar, are generally of low permeability, and act as confining units. However, both contain a few thin stringers of sand and gravel that are low-yield aquifers. The hydraulics analysis in this report is based in part on the premise that the vertical hydraulic conductivities and storage properties of Units III and V are similar. Units IV and VI are the principal aquifers and are commonly referred to as the upper and lower confined aquifer.

HYDRAULIC CHARACTERISTICS OF THE SEDIMENTS

Vertical Gradients

Before 1958, pumping from the confined aquifers was insignificant. The potentiometric surface of the upper confined aquifer was higher than the water table, the gradient was upward, and thus there was no potential for downward migration of pollutants. Since 1958, the potentiometric surface has been lower than the water table, thereby producing a downward gradient. This condition is shown (for 1974) by the hydrographs in figure 3. The gradient between the upper and lower confined aquifers may be upward or downward, depending on distribution of pumping from them. The potentiometric surface of the upper confined aquifer under the landfill reached a record low level of 39 ft above sea level in April 1975.

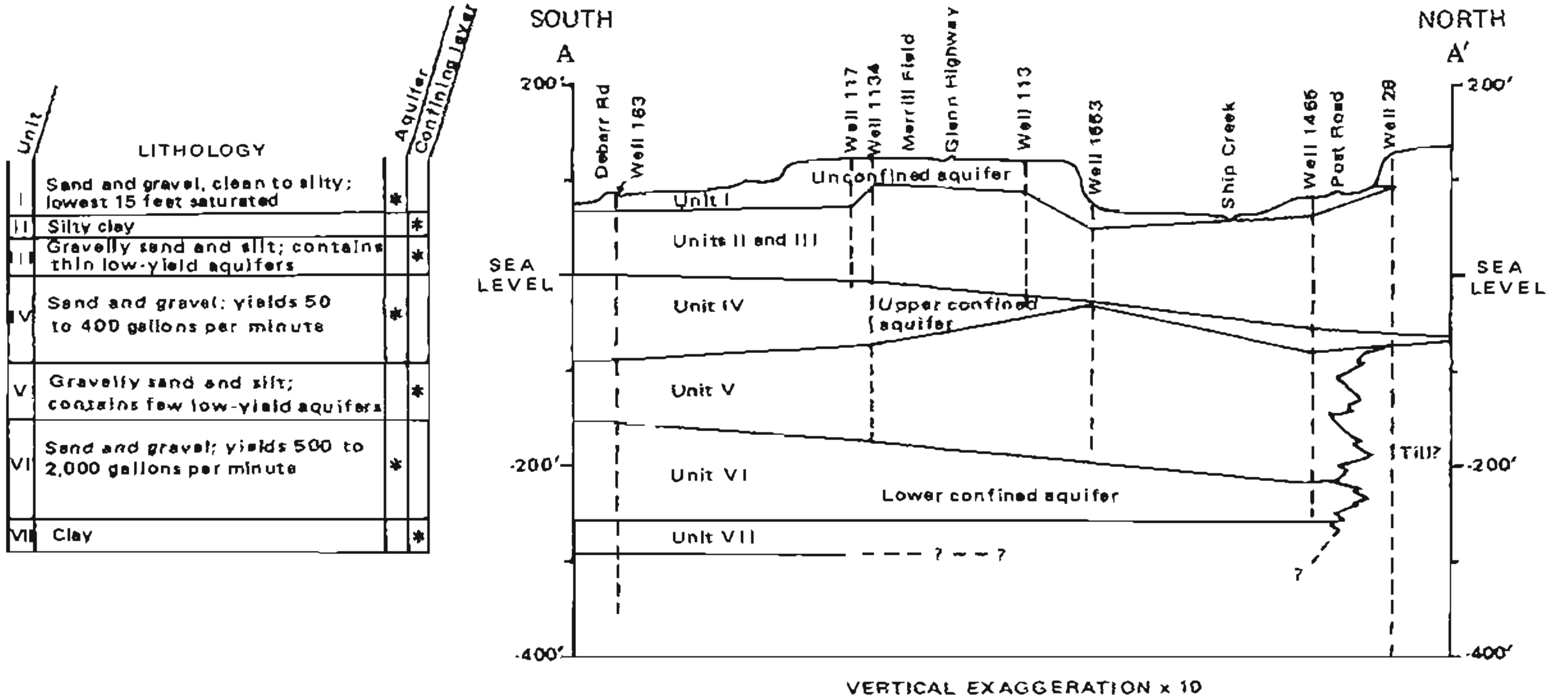

Figure 2.--Generalized stratigraphic column and geologic section at Merrill Field landfill.

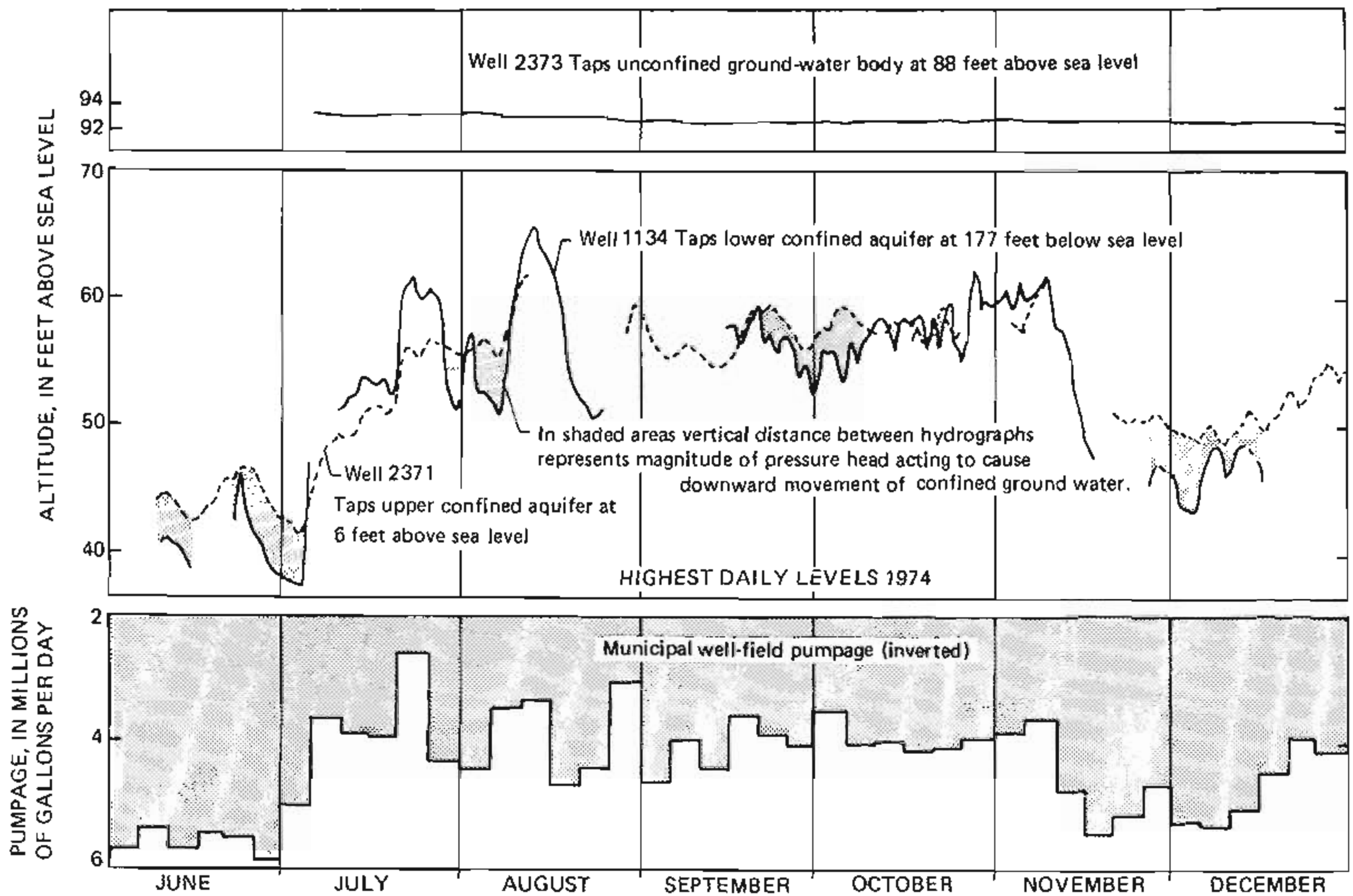

Figure 3.--Hydrographs of selected wells and histogram of pumpage from confined aquifers in the municipal well field, 1974.

During much of the time since 1971, the potentiometric surface of the upper confined aquifer has been below the base of Unit II. When well 2371 was drilled in July 1974, the potentiometric surface of the upper confined aquifer (Unit IV) was about 20 ft below the base of the clay of Unit II. The sediments at the top of Unit III yielded no water to the well during drilling. A diagram of the vertical gradients under the landfill in July 1974 (fig. 4), is based on the following assumptions:

1. Pore pressures in aquifers increase hydrostatically with depth. This assumption is strictly true only if there is no vertical flow in the aquifers, but it is approximately true if head lost by vertical flow in the coarse materials is very small.

2. Where pore pressure is negative at the boundary of Units II and III, the maximum tension is -1.5 ft. E. P. Weeks (written commun., 1976) determined a tension of -1.5 ft under similar conditions.

3. Units are homogeneous, so pore pressures change linearly within the units.

Total head was calculated as the sum of the gravity head, which decreases directly with depth, and the pore pressure. Water levels were plotted for wells 2371, 2372, and 2373 on figure 4. The slope of the pore-pressure curve in Unit III, which was calculated on the basis of the water level in well 2372, shows that the pore pressures reach zero at 9 ft below the base of Unit II. This is consistent with the observation during drilling that the upper sediments in Unit III yielded no water. The diagram indicates that the total-head gradient through Unit II (1.6) is much greater than through the saturated part of Unit III (0.033). These conditions are considered to be typical for periods when the municipal wells are pumping.

At steady flow the ratio of the hydraulic conductivities of Units II and III (K_{III}/K_{II}) is equal to the inverse of the ratio of the gradients through the units (1.6/0.03). That ratio indicates that the hydraulic conductivity of Unit III is about 50 times that of Unit II.

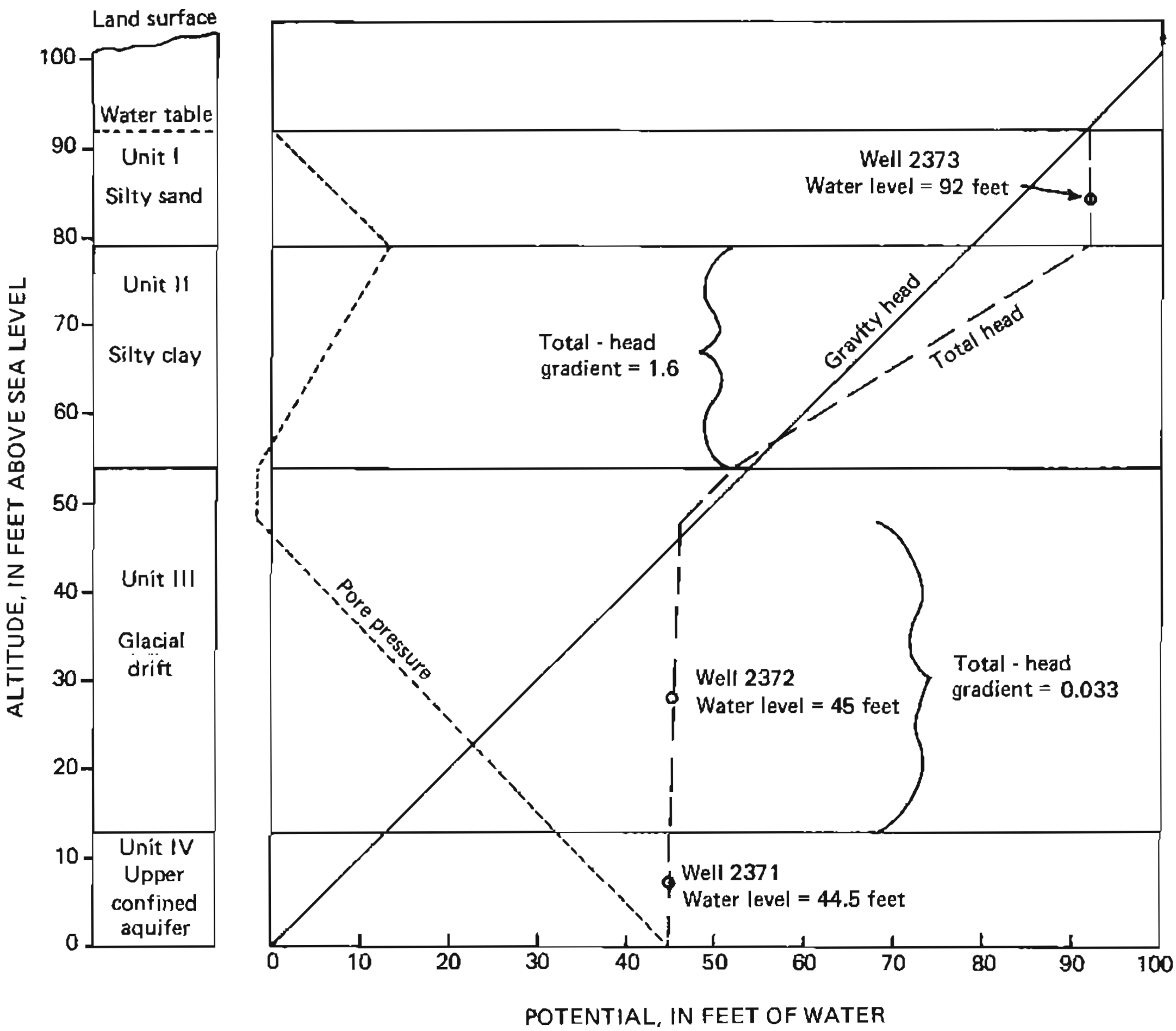

Figure 4.–Total-head gradients across Units I-IV in July 1974.

Extensometer Analysis

Figure 5 illustrates an extensometer installation at well 1134. An extensometer measures the compaction of sediments as the pore pressures are reduced by pumping. Readings from the shallow extensometer reflect seasonal changes in the altitude of the land surface that are caused by freezing or thawing of the ground. These readings are subtracted from the compaction values calculated from the record of the deep extensometer to obtain a corrected value for compaction of materials within the depth interval of +88 ft to -175 ft relative to sea level. A graphic technique described by Riley (1969 and oral commun., 1976) indicates an elastic compaction of 0.022 ft with 50 ft of decline in the potentiometric surface.

The average storage coefficient of the sediments between the extensometers may be determined from an equation given by Lohman (1972):

$$\Delta m = \gamma_w \, \Delta h \left(\frac{\bar{S}}{\gamma_w} - \frac{\theta m}{E_w} \right) \tag{1}$$

where Δm is measured compaction

γ_w is specific weight of water

Δh is change in head

θ is average porosity (assumed 25 percent)

m is total thickness of sediments

E_w is bulk modulus of elasticity of water

$\bar{S}$ is the average storage coefficient of the units.

The total thickness of sediments refers to the interval in which a pressure reduction occurs. The extensometers measure compaction of the interval from +88 ft to -175 ft, or about 263 ft. However, as indicated in figure 4, the base of Unit II is under tension. If sediments in the zone of tension are unsaturated, then these sediments separate Unit II from the effects of pressure reduction in the confined aquifer. Similarly, any sediments in Unit III that are unsaturated may not be considered in the compaction calculations. For these calculations, the phreatic surface in Unit III is assumed to be at the top of the unit, at 54 ft above sea level. The total thickness, m, is then +54 ft to -175 ft, or 229 ft. No correction is made for the potentially dewatered part of Unit III, because the altitude of the phreatic surface was not accurately defined.

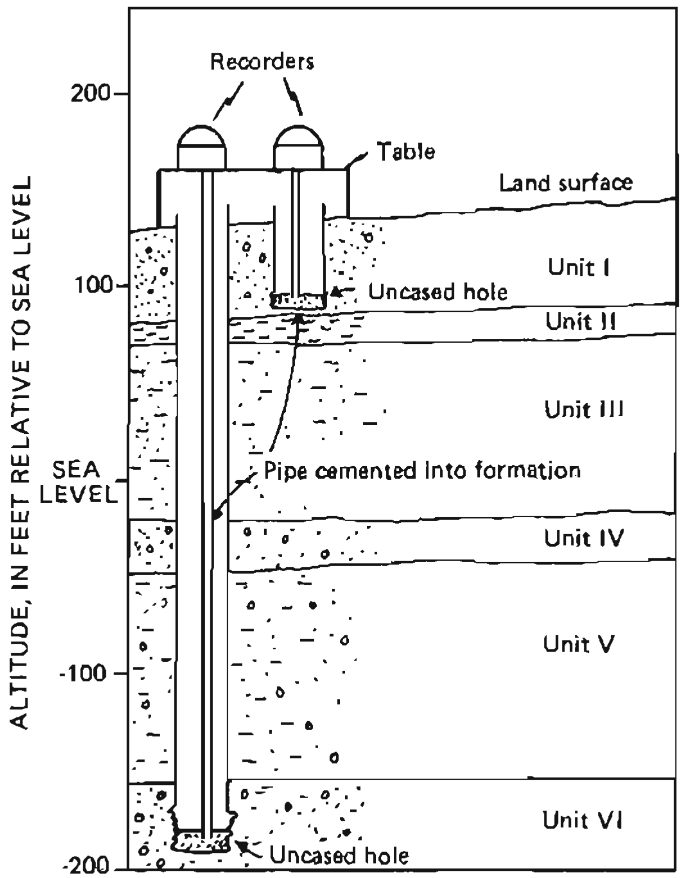

Figure 5.--Extensometer installation at well 1134. Vertical movement of pipes is recorded to indicate the changes in thickness between land surface and bottom of the holes.

The pressure reduction in the upper confined aquifer is assumed equal to the pressure reduction in the entire 229-foot interval. The change in head in the upper confined aquifer is 50 ft. The average storage coefficient ($\bar{S}$) of Units III, IV, and V can be computed using equation 1:

$$0.022 \text{ ft} = (62.4)(50)\left[\frac{\bar{S}}{62.4} - (0.25)(229)(2.3 \times 10^{-8})\right] .$$

Solving for $\bar{S}$ yields

$$\bar{S} = 5.22 \times 10^{-4}.$$

The specific storage of the sediments is: $\bar{S}_s = \bar{S}/m$.

Then $\bar{S}_s = 5.22 \times 10^{-4}/229 = 2.3 \times 10^{-6}\ \text{ft}^{-1}$.

If the assumption that Unit II does not contribute to the compaction calculations is wrong, equation 1 must be applied to the entire 263-foot interval spanned by the extensometer. Using a similar solution of equation 1, an $\bar{S}_s$ of 2.03×10^{-6} ft^{-1} was calculated. This value is not significantly different from the initial estimate.

Aquifer Tests

Two aquifer tests provided information on the hydrologic properties of the aquifers and confining layers. Data from both tests were analyzed using the aquitard-storage method of Hantush (1964). Hantush's equation that describes the aquifer response to constant pumping is:

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

where: s is drawdown in observation well

Q is pumping rate

T is transmissivity of the aquifer

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

erfc is conjugate error function

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

$$\beta = \frac{r}{4b}\left(\sqrt{\frac{K'S'_s}{KS_s}} + \sqrt{\frac{K''S''_s}{KS_s}}\right). \qquad (2)$$

In equation 2

r is radius to observation well

S is storage coefficient of the aquifer

t is time since pumping began

b is aquifer thickness

K is the hydraulic conductivity of the aquifer

$\left.\begin{matrix}K'\\K''\end{matrix}\right\}$ are vertical hydraulic conductivities of confining layers above and below, respectively, the tested aquifer

S_s is specific storage of aquifer

$\left.\begin{matrix}S'_s\\S''_s\end{matrix}\right\}$ are specific storages of confining units above and below, respectively, the tested aquifer.

Assumptions in Hantush's analysis are:

1. Aquifer is homogeneous and isotropic.
2. Flow in the aquifer is radial toward wells.
3. Flow in confining layers is vertical.
4. Drawdown effects do not fully penetrate the confining layer.
5. Well discharge is constant.
6. The aquifer responds to pumping as if the wells were fully penetrating.

The grain size of the clay (Unit VII) underlying the lower confined aquifer is much finer than that of the overlying confining layer (Unit V). Thus, the assumption was made that K" is much less than K', and that S_s' and S_s'' are not greatly different. Then it follows that $K'S_s' \gg K''S_s''$. For the test of the lower confined aquifer, equation 2 becomes:

$$\beta = \frac{r}{4b}\sqrt{\frac{K'S'_s}{KS_s}} \qquad (2a')$$

If the condition $K'S_s' \gg K''S_s''$ is not met, the error introduced is not great. Even if $K''S_s'' = K'S_s'$, the error in $K'S_s'$ is only a factor of four. $K'S_s'$ will be greatly in error only if $K'S_s' < K''S_s''$, a condition that can be true only if the product of hydraulic conductivity and specific storage of the clay exceeds that of the gravelly sand and silt of Unit V. That possibility is considered untenable.

Figure 6 is a graph of the drawdown in wells 28 and 1134 produced by pumping well 163. The graph is a composite of data for two separate periods of pumping, one in 1956 and one in 1974. The test indicates that transmissivity is 2,650 ft²/d and storage coefficient is 2.4×10^{-5}. Both pairs of values for β and r should yield identical solutions for K' from equation 2a. In order for this to be true, the condition $\beta_1/\beta_2 = r_1/r_2$ must be met. However, in this test, the ratio of β's is not equal to the ratio of radii, and there is not a unique solution. The possible solutions from equation 2a can be calculated

where S_s' is 2.3×10^{-6} from the extensometer analysis
K is T/b = 2,650/30 ft/d
b is 30 ft
S_s is $(2.4 \times 10^{-5}/30)$ ft^{-1}.

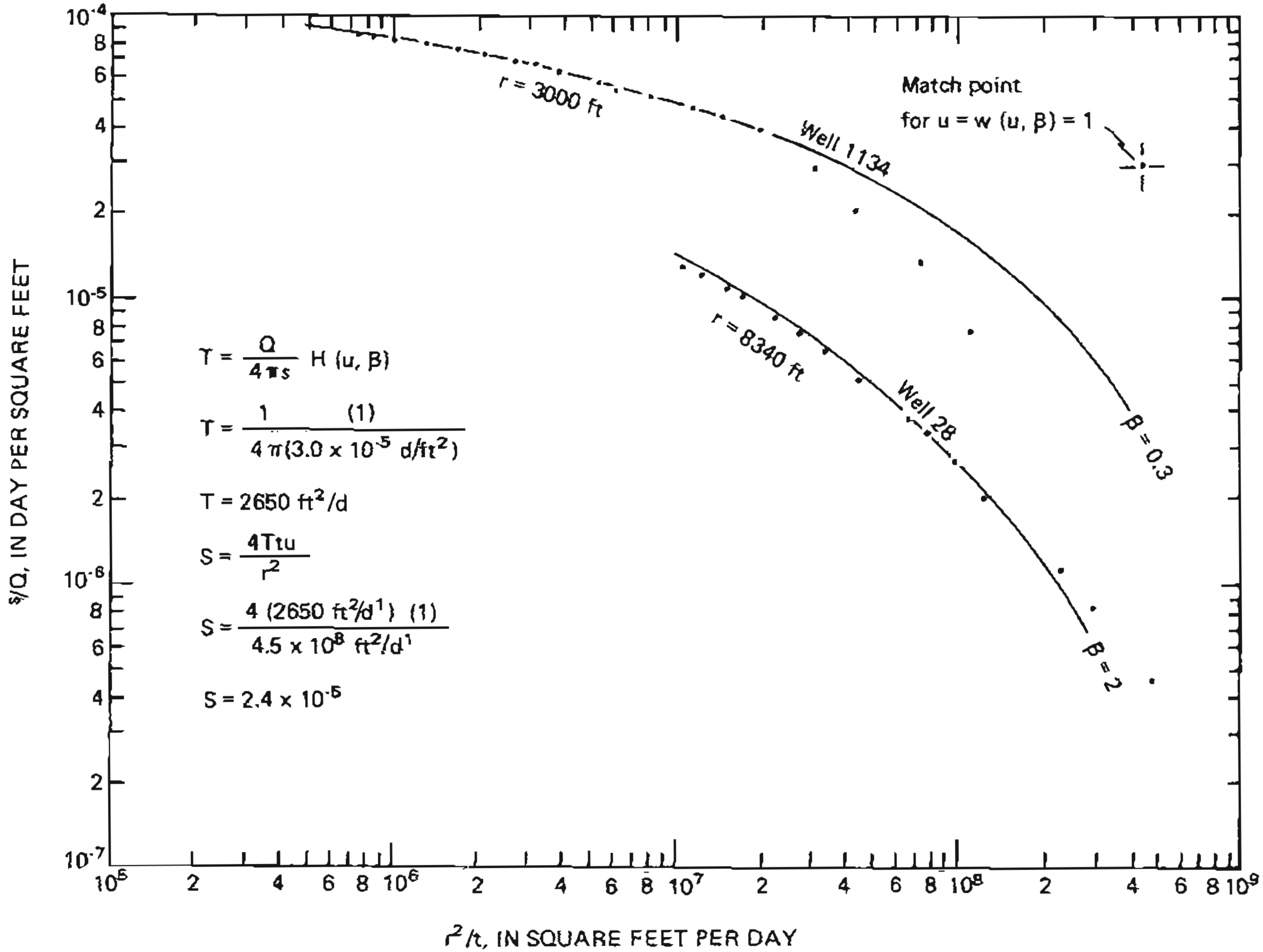

Figure 6.--Composite graph and calculations for tests of lower confined aquifer. Water-level data are from observation wells; well 163 was pumped .

Then for $\beta = 0.3$ and $r = 3{,}000$

$$0.3 = \frac{3000}{4(30)} \sqrt{\frac{K'(2.3 \times 10^{-6})}{(2650/30)\ (2.4 \times 10^{-5}/30)}}$$

and $K' = 4.4. \times 10^{-3}$ ft/d.

Similarly for $\beta = 2$ and $r = 8{,}340$

$$2 = \frac{8340}{4(30)} \sqrt{\frac{K'(2.3 \times 10^{-6})}{(2650/30)\ (2.4 \times 10^{-5}/30)}}$$

and $K' = 2.5 \times 10^{-2}$ ft/d.

Either of these K' values is possible, but neither can be confirmed as a true solution.

In 1955, an aquifer test was performed on well 163 before it was deepened to the lower confined aquifer. Although the observation well for the test was monitored with a poorly operating pressure gage and air line, the water-level recovery in the pumped well was measured accurately using an electric tape.

Papadopulos and Cooper (1967) described a method for analyzing the drawdown in a pumped well in which casing storage is a significant part of the early pumpage. This method was used to determine the transmissivity of the upper confined aquifer based on the recovery of water levels in the pumped well (fig. 7). The transmissivity obtained by the analysis shown in figure 7, 1,320 ft²/d, was used as a constraint in matching the questionable drawdown data from observation well 171. The match shown in figure 8 is the best fit for the data indicated and for the condition that $T = 1{,}320$ ft²/d.

Units III and V appear to be compositionally very similar. If the specific-storage values and the hydraulic conductivities of Units III and V are assumed to be identical, then equation 2 becomes:

$$\beta = \frac{r2}{4b} \sqrt{\frac{K'S'_s}{KS_s}} \quad . \tag{3}$$

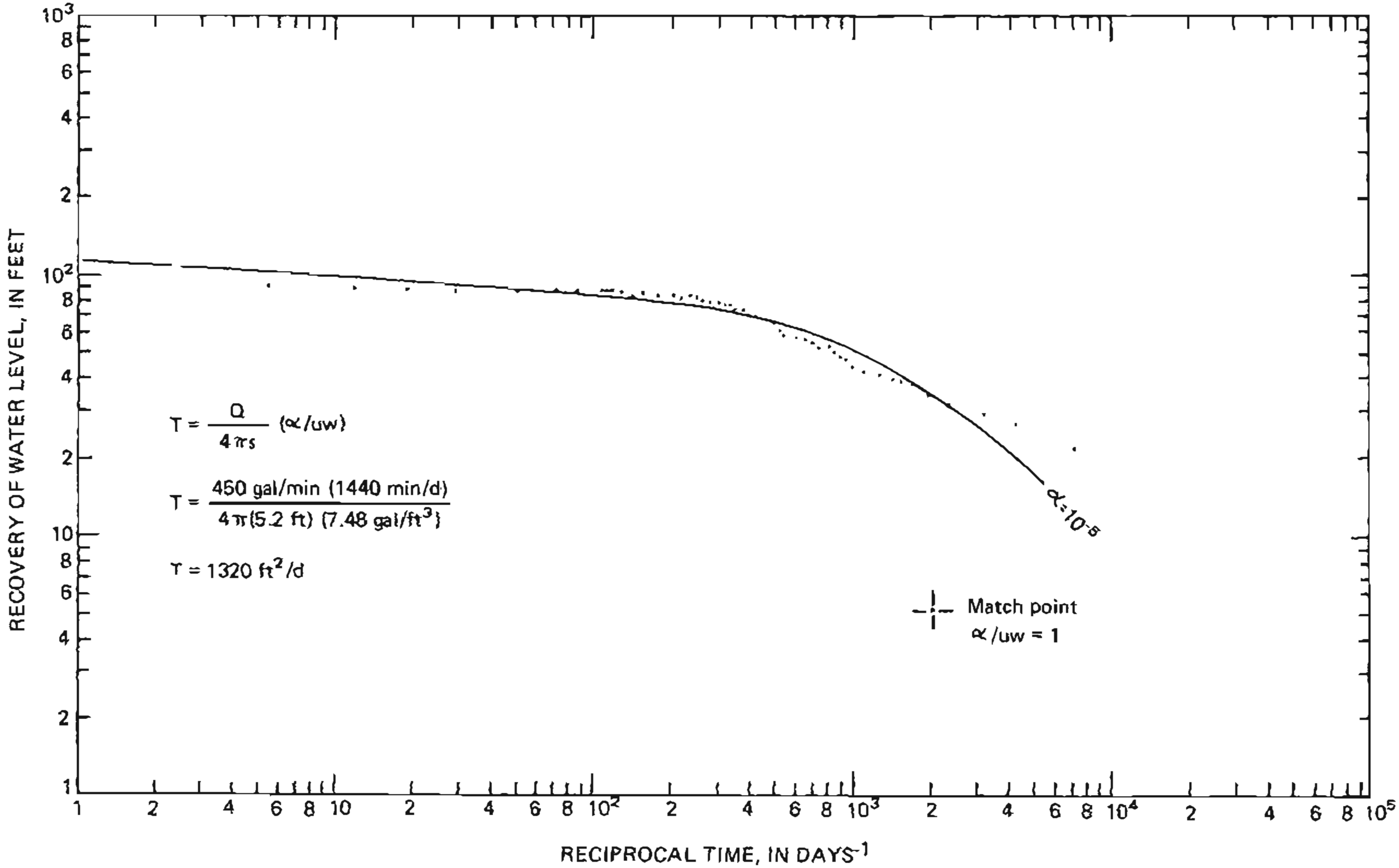

Figure 7.--Graph and calculations for recovery phase of test of upper confined aquifer. Data are from pumped well 163.

If the assumption that $K''S''_s = K'S'_s$ is not true, the error can be no greater than a factor of four for the larger of $K''S''_s$ or $K'S'_s$. However, the smaller of the values can be anything between the larger and zero.

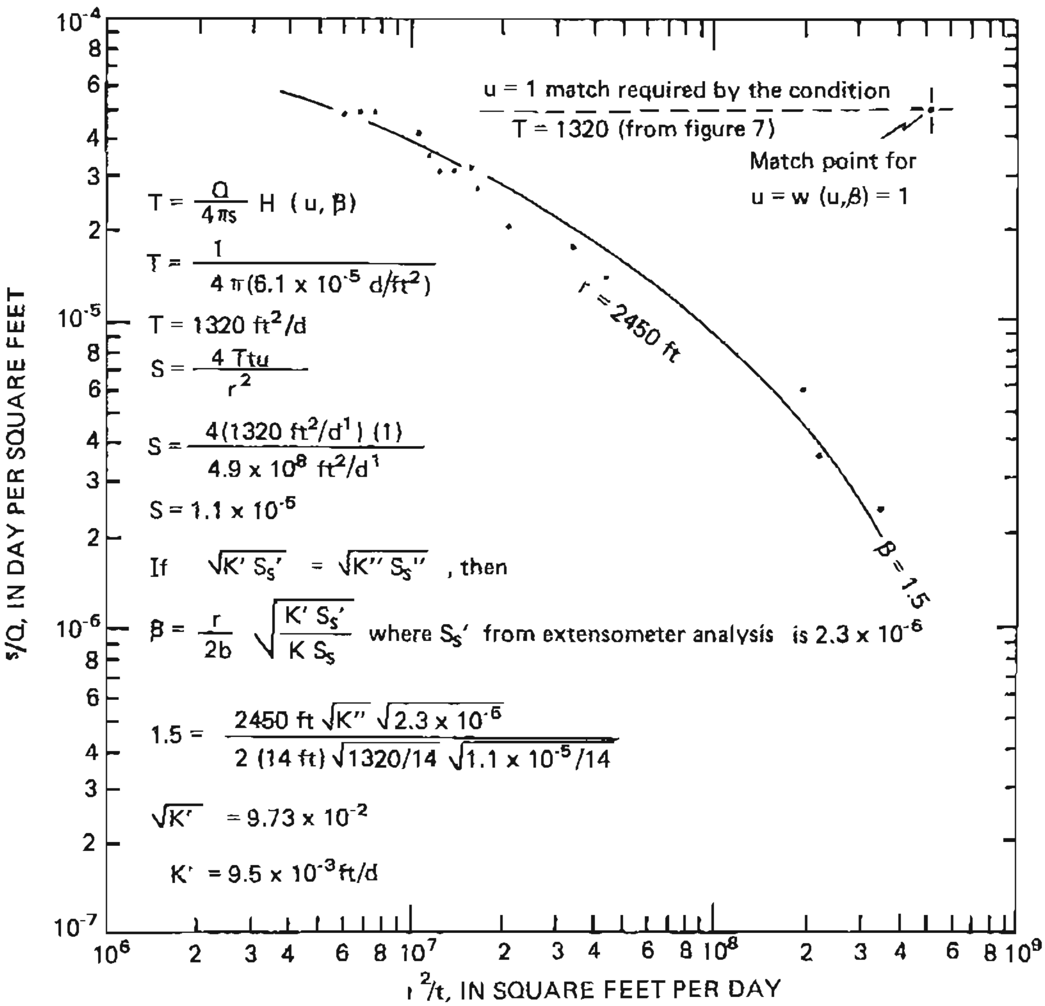

Figure 8.--Graph and calculations for drawdown phase of test of upper confined aquifer.

Solving equation 3 for the conditions

$r = 2{,}450$ ft

$b = 14$ ft

$K = T/b = 1{,}320/14$ ft/d

$S_s = S/b = 1.1 \times 10^{-5}/14\ ft^{-1}$

$\beta = 1.5$

$S'_s = 2.3 \times 10^{-6}\ ft^{-1}$ (from extensometer analysis)

yields $1.5 = \dfrac{(2450)\ 2\sqrt{K'}\ \sqrt{2.3 \times 10^{-6}}}{4(14)\ \sqrt{1320/14}\ \sqrt{1.1 \times 10^{-5}/14}}$.

Then $\sqrt{K'} = 9.73 \times 10^{-2}$

and $K' = 9.5 \times 10^{-3}$ ft/d $\approx$ 0.01 ft/d.

The first aquifer test gave values of the vertical hydraulic conductivity of 4.4×10^{-3} ft/d and 2.5×10^{-2} ft/d. The second test indicates a value of about 0.01 ft/d. The average of the two values from the first test (to one significant figure) is also 0.01 ft/d. Thus the assumption that the vertical hydraulic conductivity of Unit III equals that of Unit V is not disproved. A laboratory analysis of a drive-core sample from Unit III (see sample D in table below) indicated a vertical hydraulic conductivity of .031 ft/d. The laboratory value differs by only a factor of three from our estimate of 0.01 ft/d. This is considered adequate agreement. Laboratory results commonly differ from field results because cores are disturbed during drilling and when placed in experimental devices. The value of 0.01 ft/d is considered the best estimate of vertical hydraulic conductivity of Units III and V.

Geohydrologic properties of drive-core samples.

	CORE			
	A	B	C	D
Well number	2372	2371	2371	2372
Depth below bottom of refuse (feet)	14	18	20	44
Altitude above sea level (feet)	76	72	70	46
Unit cored	II	II	II	III
Median grain size (millimeters)	0.031	0.015	0.024	0.050
Clay-size particles (percent 0.004 millimeters)	52	30	57	10
Moisture content (percent by volume)	37	44	41	30
Vertical hydraulic conductivity (feet/day at 4°C and unit gradient).	1.3×10^{-4}	0.9×10^{-4}	0.1×10^{-4}	3.1×10^{-2}

Vertical Seepage

The downward flux of ground water per unit area is defined by

$$Q_{III} = K'_{III} \left(\frac{dh}{dl}\right)_{III} ,$$

where Q_{III} is flux per unit area of Unit III (in ft/d)

K'_{III} is vertical hydraulic conductivity of Unit III (in ft/d)

$\left(\frac{dh}{dl}\right)_{III}$ is hydraulic gradient through Unit III, as defined in figure 4.

Then $Q_{III} = 1 \times 10^{-2} (0.033) = 3.3 \times 10^{-4}$.

At steady-state conditions, the downward flux through Unit III is equal to the downward flux through Unit II.

Thus $Q_{II} = Q_{III} = K'_{II} \left(\frac{dh}{dl}\right)_{II}$,

and substituting values for Q_{III} and $\left(\frac{dh}{dl}\right)_{II}$

yields $3.3 \times 10^{-4} = K'_{II} (1.6)$.

Then $K'_{II} = 2 \times 10^{-4}$ ft/d.

This calculated value for the vertical hydraulic conductivity of Unit II differs by a factor of less than three from the average of the laboratory values determined from cores A, B, and C (see table). This is an acceptable agreement and the author considers the field value of hydraulic conductivity, 2×10^{-4} ft/d to be a "best estimate" for K'_{II}.

Theoretical velocity calculations, based on the equation (Lohman, 1972):

$$V_s = \frac{K}{\theta} \frac{dh}{dl}$$

are useful in providing approximate flow rates of water, which may contain pollutants, in various materials. In non-uniform sediments, small amounts of pollutants commonly travel faster than the theoretical rates by following preferred high-permeability paths. The theoretical rate of migration of the pollutants is also greatly affected by adsorption and dispersion. These factors may greatly retard the rate at which the concentrations of pollutants in the water reaching an aquifer increase to the "original" concentration of the leachate at its point of origin (base of a landfill). The dispersion and adsorption characteristics of the sediments beneath the Merrill Field landfill have not been determined.

The following calculations should be construed as giving broad ranges of travel times without regard to the lyophilic or lyophobic nature of the soil-water-pollutant solutions. They should be viewed in the context of the question, "Will breakthrough of pollutants to the upper confined aquifer be in terms of days, years, or centuries?"

The seepage velocity through Unit II is defined by:

$$V_s = \frac{K_{II}''}{\theta} \left(\frac{dh}{dl}\right)_{II}$$

where θ is porosity of Unit II.

θ is assumed to be equal to 0.35,

and $$V_s = \frac{2 \times 10^{-4}}{0.35} \quad (1.6) = 9.1 \times 10^{-4} \text{ ft/d.}$$

Then, if a molecule of water moves at that seepage velocity through Unit II, the transit time through the 25-foot thick unit would be:

$$\text{Time} = \frac{\text{Distance}}{\text{Velocity}} = \frac{25 \text{ ft}}{9.1 \times 10^{-4} \text{ ft/d}} = 27{,}473 \text{ d, or } 75 \text{ yr (rounded).}$$

The time it takes for the same molecule of water to migrate downward through Unit III to the upper confined aquifer can be calculated similarly. Assuming the porosity of Unit III is 0.25, the seepage velocity is:

$$V_s = \frac{1 \times 10^{-2}}{0.25} (.033) = 1.3 \times 10^{-3} \text{ ft/d.}$$

The travel time through Unit III, which is 42 ft thick, is then:

$$\text{Time} = \frac{42 \text{ ft}}{1.3 \times 10^{-3} \text{ ft/d}} = 32{,}308 \text{ d, or } 89 \text{ yr (rounded).}$$

The total travel time through Units II and III to the upper confined aquifer is 75 years + 89 years = 164 years.

The initial breakthrough of landfill-derived pollutants to the upper confined aquifer would presumably be much earlier than this, in the range of many tens of years. However, experiments conducted on the movement of solutes through clay-rich soils (Nielsen and Biggar, 1962) indicate that the leakage entering the aquifer may not reach the full concentration of the leachate until after twice the travel time, or more than 300 years.

Discussion of Test Results

From the extensometer analysis it was determined that the average specific storage of the sediments is 2.3×10^{-6} ft^{-1}. The part of this value that results from compaction of the aquifer skeleton is (Lohman, 1972):

$$S_{as} = \frac{\gamma \Delta b}{b \Delta p}$$

where γ is specific weight of water

Δb is the measured compaction (change in thickness, b)

b is the aquifer thickness

Δp is the change of head,

provided that the aquifer skeleton behaves elastically. Then:

$$S_{as} = \frac{(62.4)\ (.022)}{(50\text{ ft})\ (62.4)\ (229)} = 1.9 \times 10^{-6}\ \text{ft}^{-1}.$$

The specific storage due to compressibility of water is: $S_w = \theta \gamma / E_w$

where θ is porosity of sediments

E_w is the bulk modulus of elasticity of water.

Then $S_w = (0.3)\ (62.4)/4.32 \times 10^7 = 0.43 \times 10^{-6}$.

The specific storage due to elastic compaction of the aquifer is therefore 4.4 times that due to expansion of water. However, in both aquifer tests, the storage coefficients were about equally attributable to compression of the aquifer and expansion of water.

To evaluate the effect of underestimating specific storage an assumption was made that all sediments have a specific storage equal to that determined from the extensometer, 2.3×10^{-6}. If all sediments have the same specific storage, then equation 2 becomes:

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

Recalculating K' for the test of the upper confined aquifer and the match $\beta = 0.3$ at $r = 3{,}000$ ft, yields

$$0.3 = \frac{3000}{4\ (30)} \sqrt{\frac{K'}{2650/30}}$$

or

$$K' = .0127.$$

This is about three times the previously determined value. Similarly for $\beta = 2$ and $r = 8{,}340$ ft, the value is about three times the previously determined value. For the test of the lower confined aquifer, equation 2 becomes:

$$\beta = \frac{r}{2b} \sqrt{\frac{K'}{K}}.$$

Then

$$1.5 = \frac{2450}{2\ (14)} \sqrt{\frac{K'}{1320/14}}$$

or $K' = .028$ ft/d.

Again, this is nearly three times as great as the value of 0.01 ft/d, determined previously from the test of the upper confined aquifer. Although calculations based on a uniform specific storage are useful in assessing possible errors in the vertical hydraulic conductivities, these new values were not used. If it were concluded that the aquifer tests gave erroneous values for the storage coefficients, then it must also be concluded that the values for β are in error. However, there is no basis for selecting alternate values for β. Furthermore, the new K' values reduce the travel-time calculation by a factor of only three. After dividing previous times by a factor of three, the time of first breakthrough remains in the order of tens of years and the full breakthrough remains in the order of hundreds of years.

Throughout this report data from aquifer tests dating back almost 30 years were used. It appears that these data can not be used to analyze vertical hydraulic conductivities accurately. The maximum field value for hydraulic conductivity of Units III and V was 2.5×10^{-2} ft/d, which is more than twice as great as the author's best estimate. Similarly, the minimum field value was 4.4×10^{-3} ft/d, which is less than half of our best estimate. Carrying these maximum and minimum values through the calculations suggest travel times from as little as 65 years to as great as 368 years. Thus, the margin of error is likely to be a factor of two to three. If more accurate values are needed, new experiments must be designed.

Only two aspects of vertical migration of pollutants were considered--vertical gradients and vertical hydraulic conductivities. The author has no data on dispersion and adsorption characteristics of the aquifers and confining beds. These characteristics must also be determined before migration of pollutants can be analyzed fully.

It is also recognized that there is flow toward the production wells from areas not overlain by the landfill. This flow would dilute any leachate seeping through the confining layers to the aquifers. Again, a more detailed analysis would require that new experiments be designed and new models be developed.

CONCLUSIONS

1. Vertical gradients under the Merrill Field landfill vary with changes in pumping. However, using the conditions of July 1974 as typical of normal pumping, the gradients through the confining Units II and III are 1.6 and 0.033, respectively.
2. The "best estimate" of the vertical hydraulic conductivity of the gravelly sand and silt of Units III and V is 1×10^{-2} ft/d.
3. The "best estimate" of the vertical hydraulic conductivity of Unit II is 2×10^{-4} ft/d.
4. The data do not support a unique and accurate value of vertical hydraulic conductivity for any of the units. Values may be in error by plus or minus a factor of two or three.

5. Theoretical seepage calculations indicate that minor amounts of pollutants may reach the upper confined aquifer after many tens of years, but that water of the composition of the leachate may not reach the upper confined aquifer for more than three centuries.

6. Although there have been a number of aquifer tests performed since the 1950's, they have not been designed to obtain the type(s) of data necessary to predict migration of pollutants accurately. If such predictions are needed, new experiments and models must be designed.

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