THEORETICAL DRAWDOWN DUE TO SIMULATED PUMPAGE FROM THE OHIO RIVER ALLUVIAL AQUIFER NEAR SILOAM, KENTUCKY

U.S. GEOLOGICAL SURVEY Water-Resources Investigations 77-24

Prepared in cooperation with the
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UNITED STATES DEPARTMENT OF THE INTERIOR

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CONTENTS

ILLUSTRATIONS

TABLES

FACTORS FOR CONVERTING ENGLISH UNITS TO METRIC UNITS

MULTIPLY	BY	TO OBTAIN
Feet (ft)	0.3048	Meters (m)
Miles (mi)	1.609	Kilometers (km)
Gallons per minute (gal/min)	.06309	Liters per second (L/s)
Gallons per day per square foot [(gal/day)/ft^2]	1.547×10^{-6} 4.717×10^{-7}	Feet per second (ft/s) Meters per second (m/s)

Theoretical Drawdown due to Simulated Pumpage from the Ohio River Alluvial Aquifer near Siloam, Kentucky

By John Michael Kernodle

ABSTRACT

Theoretical drawdown due to simulated pumpage was determined for a site near Siloam, Kentucky by using a digital ground-water-flow model. The maximum sustained yield of water from a single well was shown to be less than 900 gallons per minute, and, for a simulated pumping rate of 450 gallons per minute from each of several wells, the optimum placement for either minimum water-level drawdown or minimum spacing of multiple wells was found to be in a line parallel to and at a distance of about 370 to 600 feet from the Ohio River. An analytical method was used to verify the optimum placement of wells determined by the digital model.

Calibration of the digital model was achieved by matching calculated water-level changes with observed changes in two wells during the passage of a flood-generated wave through the alluvial aquifer. The calibrated model was found to be insensitive to changes in values for the hydraulic conductivity and specific yield of the aquifer. Therefore, values were assigned to these parameters to obtain the theoretical drawdown due to the simulated pumping rate. However, the calibrated model was sensitive to variations in modeled aquifer diffusivity and the coefficient of retardation of the semi-pervious river bank. The analytical solution showed optimum well placement to be independent of aquifer hydraulic conductivity and dependent on the coefficient of retardation. This supports the validity of model results as to configuration and location of a well field.

INTRODUCTION

Increasing development of the ground-water resources of the Ohio River alluvial aquifer demands that great care be taken in well-field design and construction to ensure the most efficient use of the aquifer. Well location and spacing should always be determined as a function of the aquifer's local hydrologic characteristics. Width and saturated thickness of the aquifer as well as its hydraulic conductivity and storage coefficient, and the hydraulic conductivity of the river bed and bank govern the maximum rate of withdrawal of water from the aquifer.

Purpose and Scope

The purpose of this report is to present the results of hydrologic investigations at the Columbia Hydrocarbons Corporation site approximately 2 miles (mi) west northwest of Siloam, Ky., and to illustrate the importance of digital modeling as a planning aid in the design of well fields in the Ohio River alluvial valley. This site, and others which have been selected for similar investigations, were chosen according to the need for hydrologic information in areas where few data are available and development is anticipated, or in areas where overdevelopment could potentially occur. Once a digital flow model has been compiled and verified for each site the model can be used as a management tool to aid in the design of new well fields, or in the addition of new wells to existing well fields.

Problem and Method of Investigation

Investigations by Rorabaugh (1956) and Grubb (1974) have shown that large sustained rates of withdrawal of water from the alluvial aquifer are possible only because of induced infiltration from the Ohio River. Without this source of recharge, water in storage in the aquifer would soon be depleted. Most conventional aquifer tests, because of their short duration, or small applied stress on the hydrologic system, are of limited value in quantifying the effect of induced recharge even though they may, with usable accuracy, evaluate the characteristics of the aquifer itself.

Digital modeling of ground-water flow may, under the proper circumstances, provide accurate estimates of the hydraulic conductivity of the aquifer and of the riverbed and bank. The technique employed in this report uses a flood on the Ohio River as the applied stress and the observed water-level changes in the adjacent aquifer as the resultant hydrologic strain. Changes in water level in the aquifer are proportional to the hydraulic characteristics of the aquifer and the river bank. Two observation wells installed approximately 100 and 1,000 feet (ft) from the river are used to record the observed water-level response. Calibration of the digital model is achieved by varying hydrologic parameters within commonly accepted limits until the calculated water-level response at the observation points matches the observed response.

Theoretical aquifer response at the Columbia Hydrocarbons Corporation site near Siloam, Ky. (figs. 1 and 2) was obtained by using a digital ground-water-flow model developed and described by Pinder and Bredehoeft (1968), Bredehoeft and Pinder (1970), and modified by Trescott (1973). Additional modifications were made to the model by the author in order to simulate an aquifer which is truncated by and in partial hydraulic connection with a river whose stage changes with time. After the digital model had been calibrated, theoretical drawdown due to simulated pumpage was determined for withdrawal rates of 450 and 900 gallons per minute (gal/min) and at well locations from approximately 100 to 1,000 ft from the Ohio River.

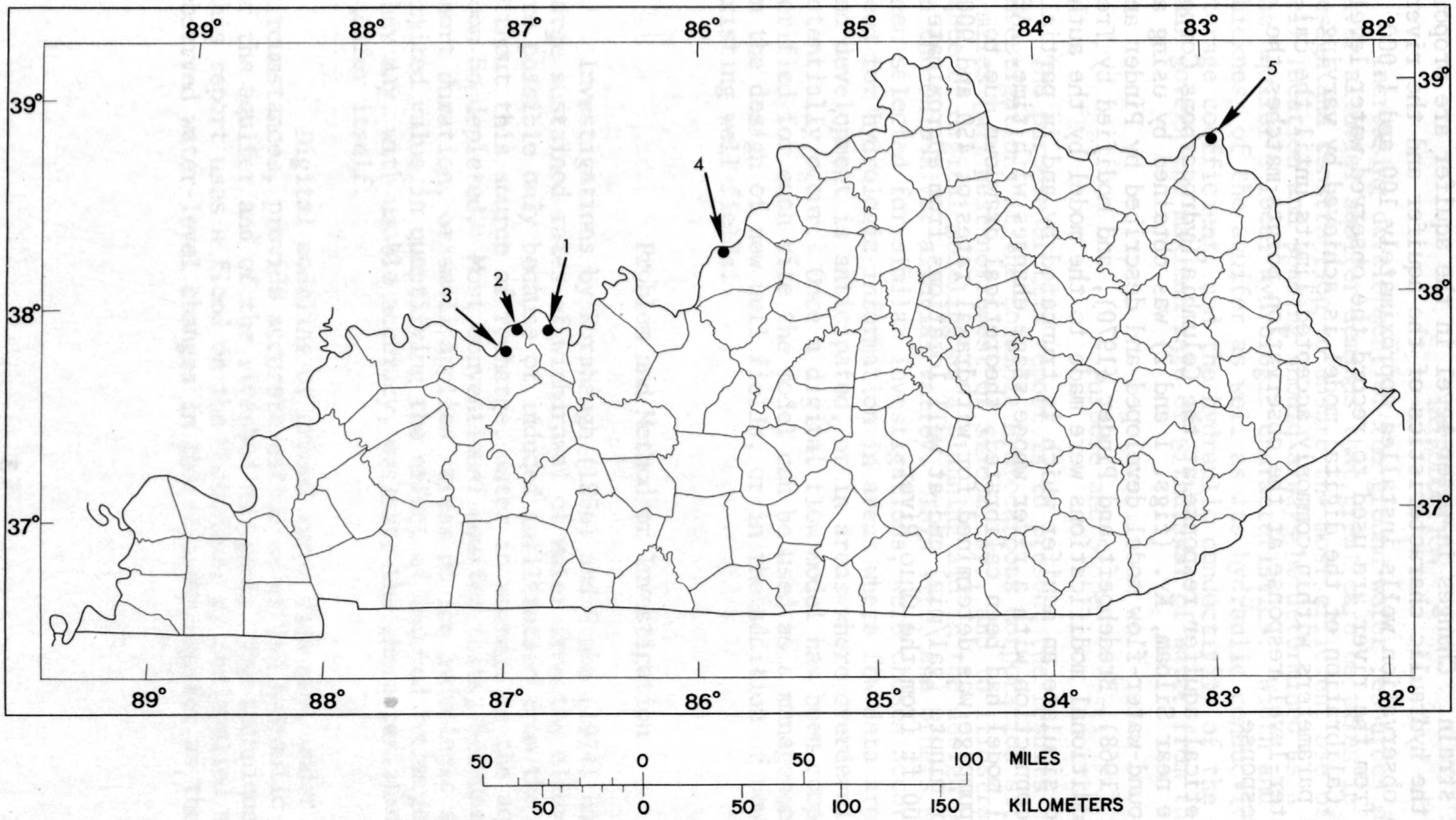

Figure 1-- Location of the Columbia Hydrocarbons Corporation site (5) and previous test sites near Hawesville (1), Lewisport (2), Owensboro (3) and Louisville, (4), Kentucky.

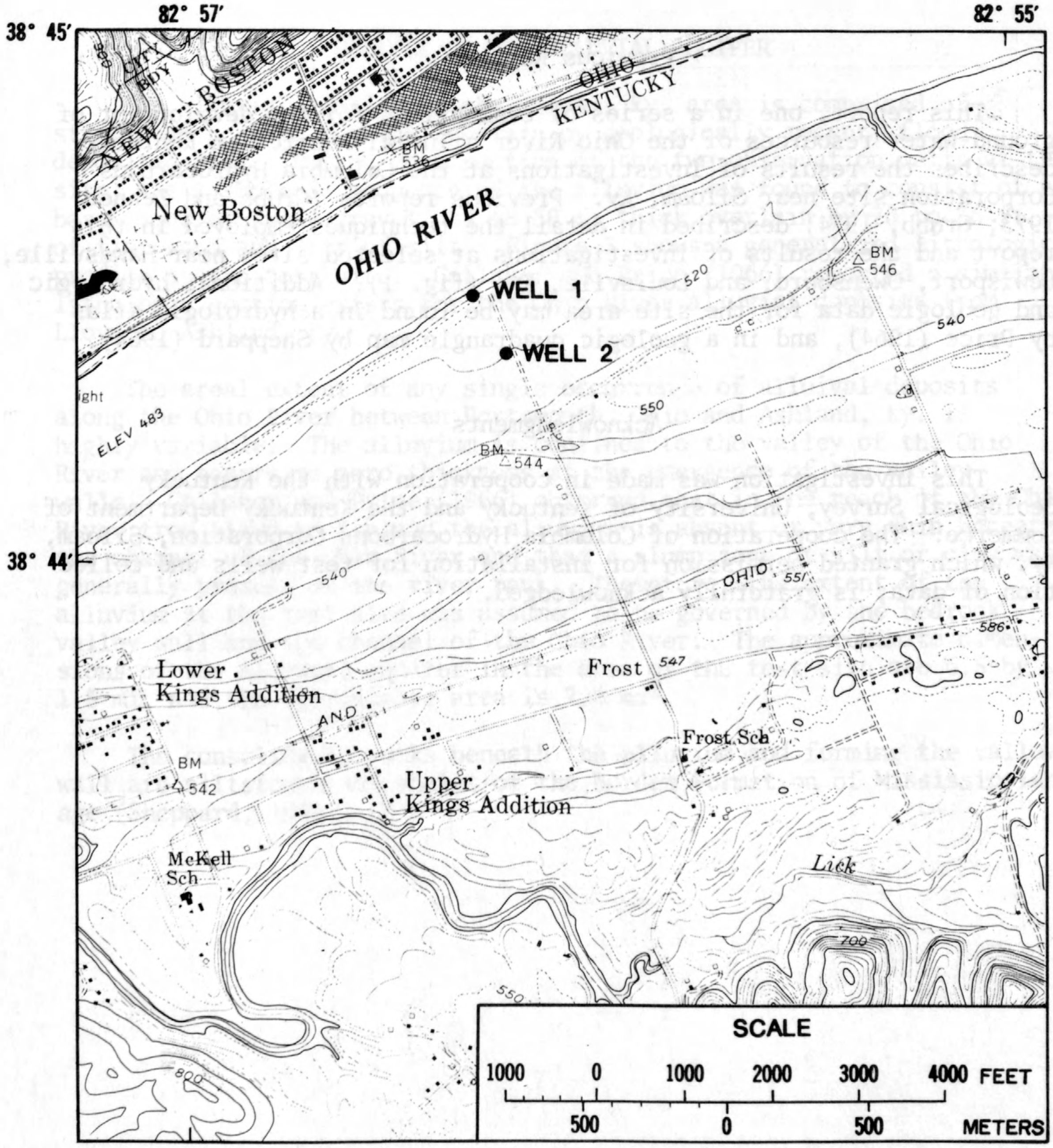

Figure 2-- Location of test wells at the Columbia Hydrocarbons Corporation site near Siloam, Kentucky.

Previous Reports

This report, one in a series of evaluations for the development of ground-water resources of the Ohio River alluvial aquifer in Kentucky, describes the results of investigations at the Columbia Hydrocarbons Corporation site near Siloam, Ky. Previous reports (Grubb and Zehner, 1973; Grubb, 1974) described in detail the techniques employed in this report and the results of investigations at selected sites near Hawesville, Lewisport, Owensboro, and Louisvlle, Ky. (fig. 1). Additional hydrologic and geologic data for the site area may be found in a hydrologic atlas by Price (1964), and in a geologic quadrangle map by Sheppard (1964).

Acknowledgments

This investigation was made in cooperation with the Kentucky Geological Survey, University of Kentucky and the Kentucky Department of Commerce. The cooperation of Columbia Hydrocarbons Corporation, Siloam, Ky. which granted permission for installation for test wells and collection of data, is gratefully acknowledged.

GEOLOGY OF THE ALLUVIAL AQUIFER

The alluvial aquifer in the Siloam, Ky. area is comprised of stratified glacial outwash overlain by geologically recent, flood-deposited silt. During construction of the two observation wells at the site, the lithologic sequence of the alluvium was found to consist of a basal zone of sandy gravel, 25 to 30 ft thick overlain by 20 to 36 ft of fine sand and 8 ft of silt. Figure 3 shows a generalized lithologic profile at the test site. Gallaher and Price (1966) reported a similar lithologic section exists for the Ohio River alluvial deposits from Lloyd to Ashland, Ky.

The areal extent of any single occurrence of alluival deposits along the Ohio River between Portsmouth, Ohio and Ashland, Ky. is highly variable. The alluvium is confined to the valley of the Ohio River and tapers to zero thickness at the emergence of the valley walls. Gallaher and Price (1966) observed that in the reach of the Ohio River from Lloyd to Ashland the alluvium is absent or very thin beneath the channel of the Ohio River and that a slump zone of silt or clay was generally present at the river bank. Therefore, the extent of the alluvium at the test site was assumed to be governed by the bedrock valley wall and the channel of the Ohio River. The approximate dimensions of the alluvial aquifer in the area of the test site are 6.5 by 1.5 mi, and the approximate area is 7.6 mi^2.

The consolidated rocks beneath the alluvium and forming the valley wall are siltstones and shales of the Borden Formation of Mississippian age (Sheppard, 1964).

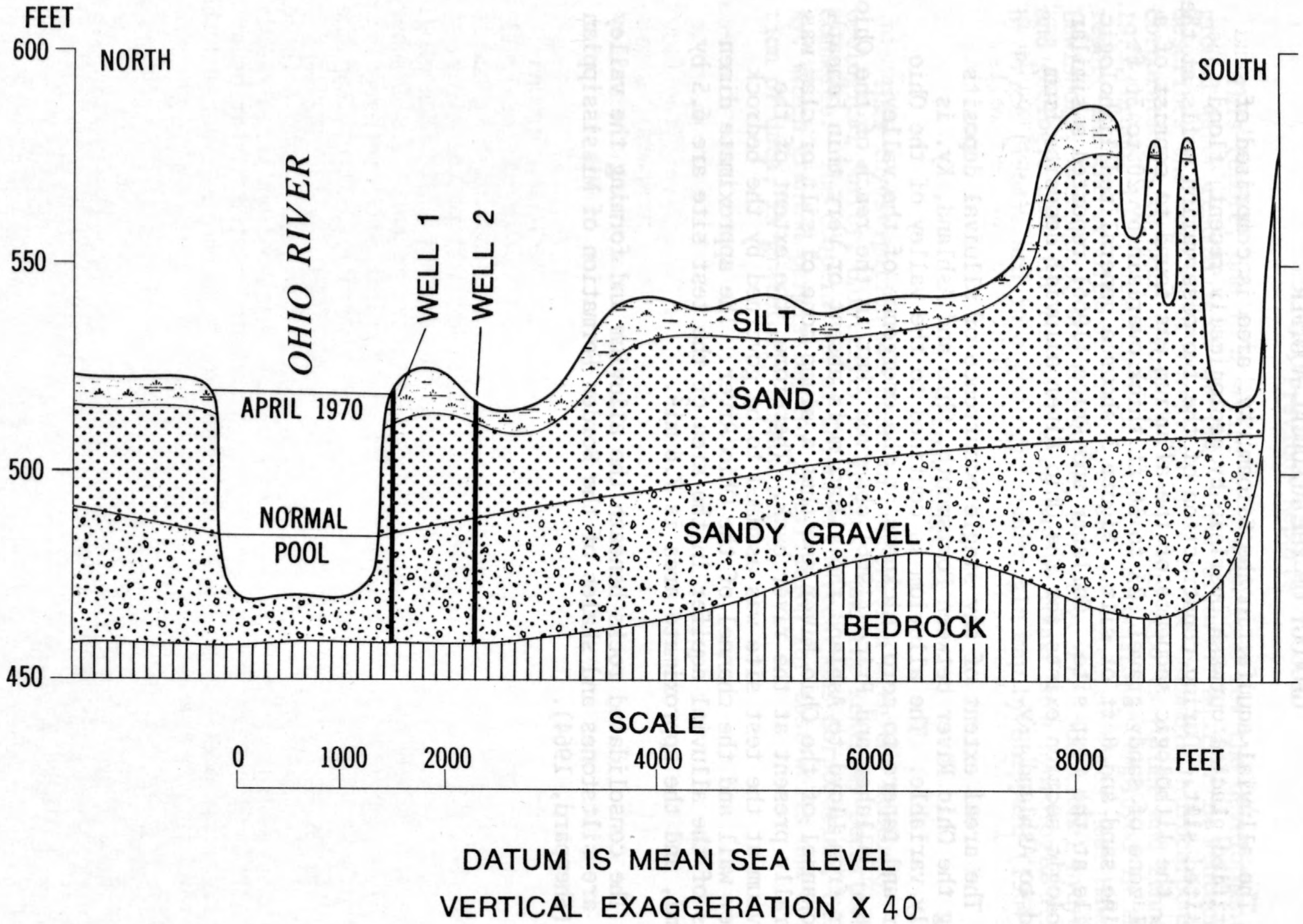

Figure 3-- Generalized lithologic section at the Columbia Hydrocarbons Corporation site. The section line is perpendicular to the river and through wells 1 and 2.

HYDROLOGY OF THE ALLUVIAL AQUIFER

The alluvial aquifer consists of interbedded and lenticular sand and gravel that are variable in vertical and areal distribution. The aquifer material is coarsest at the base and grades upward into fine sand and ultimately into silt or silty clay near the land surface. The hydraulic conductivity of the aquifer is therefore also vertically variable, with zones of high conductivity corresponding to lenses of coarse gravel and to the generally coarser material in the basal 25 ft of the alluvium. Water levels at the site remained within the sand and gravel during the period of observation so the aquifer is assumed to be unconfined. The saturated thickness observed at the two test wells during periods in which the Ohio River is at or near normal pool is approximately 26 ft.

Natural recharge to the aquifer occurs by vertical percolation of precipitation, outflow from the valley wall to the alluvium, vertical flow from the valley bottom, and by flood-induced infiltration from the river. Discharge from the aquifer is by base flow to the Ohio River, evapotranspiration, and pumped withdrawals.

In the natural system, water enters the ground-water reservoir from the several sources of recharge and is discharged to the Ohio River or lost by evapotranspiration. During floods of the Ohio River, the flow of ground water may reverse and the river may become a temporary source of recharge to the aquifer. Unusually large floods may raise normal ground-water levels for a considerable length of time.

A water-level profile of the aquifer during a period of ground-water discharge to the river would generally show that the water-level elevation is greatest near the bedrock valley wall, and least at the river bank with a gentle gradient from the valley wall to the river. The relief on the water-level surface is a function of 1) the hydrologic properties of the aquifer 2) the rates of natural recharge and discharge and 3) man's activities: rates of pumpage and artificial recharge. However, the Ohio River bank and channel are generally siltier than the adjacent aquifer and the river and aquifer are not in full hydraulic connection. Although water-level relief is governed by the above three factors, the minimum water level (at the river bank unless large-yield production wells are nearby) is largely controlled by the elevation of the river surface and the degree of connection between the river and the aquifer.

THE DIGITAL MODEL

The Numerical Solution and Restrictive Assumptions

The digital model which was used to simulate aquifer response at the Columbia Hydrocarbons Corporation site is a two-dimensional, finite-difference model written and documented by Trescott (1973). The model employs a row and column discretization of the aquifer area to be simulated to solve by the Taylor algorithm the two-dimensional flow equation:

$$\frac{\partial}{\partial x}\left(K_{xx} b \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy} b \frac{\partial h}{\partial y}\right) = S_y \frac{\partial h}{\partial t} + W(x,y,t)$$

in which K_{xx}, K_{yy} are the principal components of the hydraulic conductivity tensor (L/t);
S_y is the specific yield of the aquifer (dimensionless);
b is the saturated thickness of the aquifer (L).

Assumed conditions inherent in the model are:

(1) All flow in the aquifer system is horizontal. (Upper and lower aquifer boundaries are also horizontal).

(2) Aquifer anisotropy (if any) is aligned with the row and column axes of the discretization.

(3) No spatial or temporal variations in fluid viscosity exist.

(4) Changes in aquifer storage due to variations in the free water surface are instantaneous and complete.

(5) Aquifer conductivity is vertically uniform. (A composite hydraulic conductivity may possibly be used to approximate complex sections).

(6) Head changes in sub- or superadjacent aquifers do not occur in response to changing heads in the aquifer being simulated.

(7) Water lost to evaporation may be incorporated in the source-sink term, W(x,y,t), as a linear function of the depth to the potentiometric surface below land surface.

(8) Aquifer hydraulic characteristics and simulated response may be approximated by node-centered average values.

Capabilities and Characteristics of the Model

The digital flow model, written and documented by Trescott (1973), can simulate a wide variety of two-dimensional ground-water flow problems. Simulations may be of a water-table, artesian, or a combined water-table-artesian aquifer. Boundary conditions may include no flow, constant head or constant flux. The model can simulate aquifer response to recharging or discharging wells, uniform areal recharge (precipitation), evapotranspiration, and flow through a confining layer to or from a sub- or superjacent aquifer or a stream. Finally, the model can determine steady-state or non steady-state (time dependent) solutions. The numerical solution employed by the model allows variable node dimensions, multiple-pumping periods, and geometrically increasing time-step length within pumping periods.

Modifications of the Digital Model

In order to simulate a fully penetrating stream whose stage is variable and which is partly isolated from the aquifer by a vertical zone of reduced hydraulic conductivity, two changes were required in the digital model. The first change was to enable a stage value to be read into the matrix which defines the head in the adjacent aquifer (the river in this instance) at the initiation of each new time step. The second change was to define the aquifer beneath the river as having a constant head value equal to the river elevation for the duration of the time step. Programming to accomplish these two changes was simple and need not be documented in this report. The logic supporting these changes is discussed in the next section of this report.

CALIBRATION OF THE MODEL OF THE ALLUVIAL AQUIFER

The first step in developing a model to simulate aquifer response is to devise a conceptual model of the real flow system in terms which are compatible with the assumptions and numerical solution procedure of the digital model. Depending on the availability of data, some simplifying assumptions in the conceptual model may be necessary. The basic conceptual model has been described in the section on hydrology. Because of the limited amount of lithologic and boundary condition information available from the two test wells at the site, several simplifications were made in the construction of the digital model.

Flow boundaries were determined from land surface and bedrock topographic maps. The bedrock valley wall was assumed to be a no-flow boundary, and the river was modeled as fully penetrating and partly isolated from the aquifer by a vertical zone of low hydraulic conductivity (K_c). The upstream and downstream limits of the modeled area were modeled as no-flow boundaries. Hydraulic conductivity of the aquifer (K_a) was assumed to be isotropic and both K_a and the specific yield (S_y) of the aquifer were assumed to be horizontally and vertically uniform. Because of the technique used to calibrate the model no source of aquifer recharge or discharge other than the Ohio River was considered to be significant. Figure 4 shows a section through the aquifer from the Ohio River, through the two test wells and to the valley wall, as it would be constructed based on the above conditions (compare with fig. 3).

The hydraulic conductivity (K_c) of the vertical semi-pervious zone at the river bank was modeled as being proportional to the hydraulic conductivity of the alluvial aquifer. The thickness of this zone (b') was assumed to be 50 ft. Where variations in node dimension dictated that the thickness of this zone be increased, the value of the ratio $K_a b'/K_c$ (the coefficient of retardation, designated $\underline{a}$ by Hantush, 1965) was held constant. The actual value of K_c could be varied by changing K_a, b', or the constant of proportionality.

Figure 5 shows the row and column spatial discretization used in this model to locate observation points and flow boundaries. The same discretization was later used in simulations of drawdown.

The model was calibrated by matching simulated to observed aquifer response which was change in water level at the two observation points due to a known applied stress, two floods on the Ohio River. The two floods occurred during the 48-day period from March 20 to May 9, 1970. Each modeled time step was one day in length for a total of 48 time steps. For each new time increment the river elevation recorded at noon

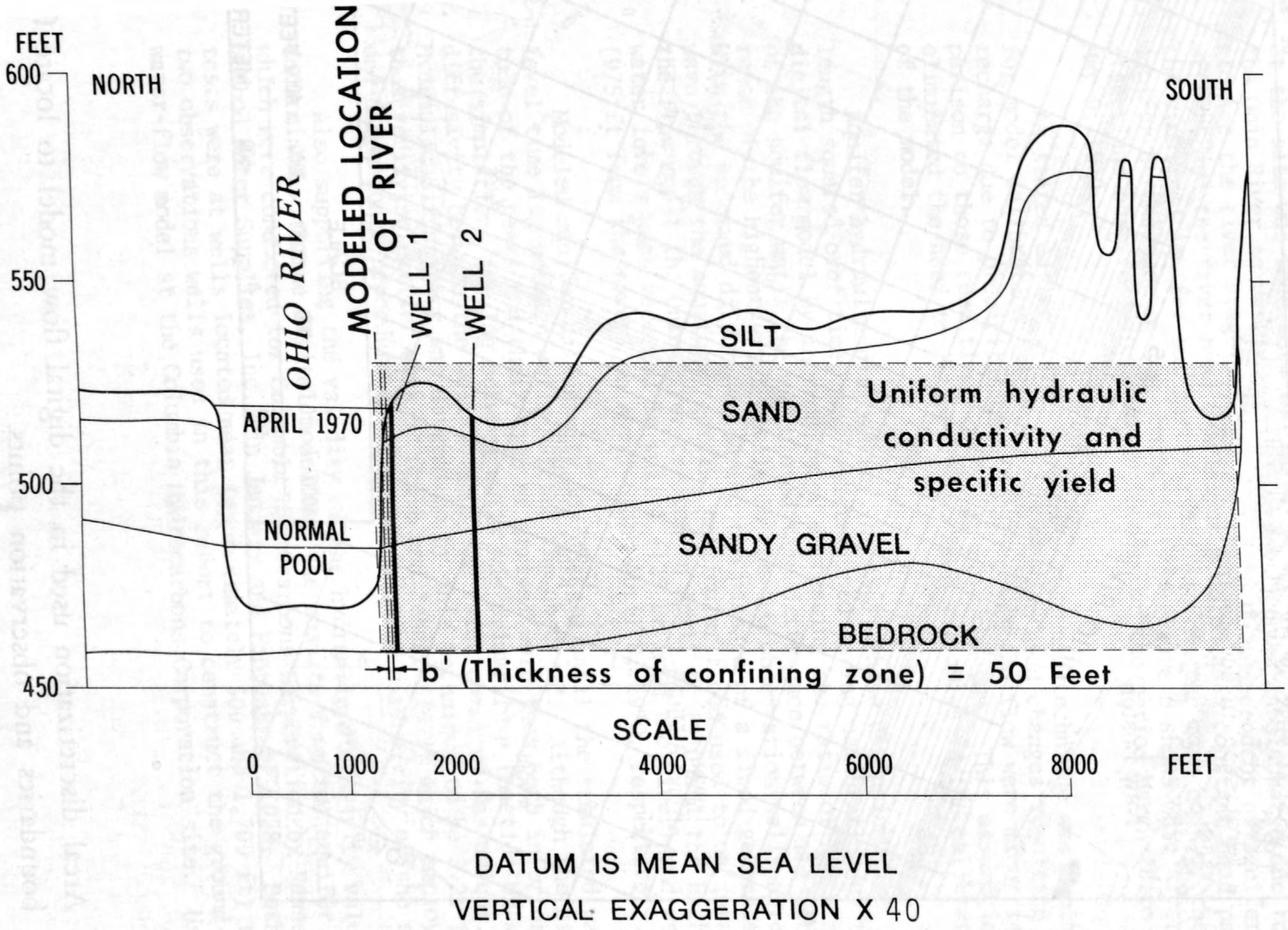

Figure 4-- The hydrologic system as simulated for the model of the Columbia Hydrocarbons Corporation site.

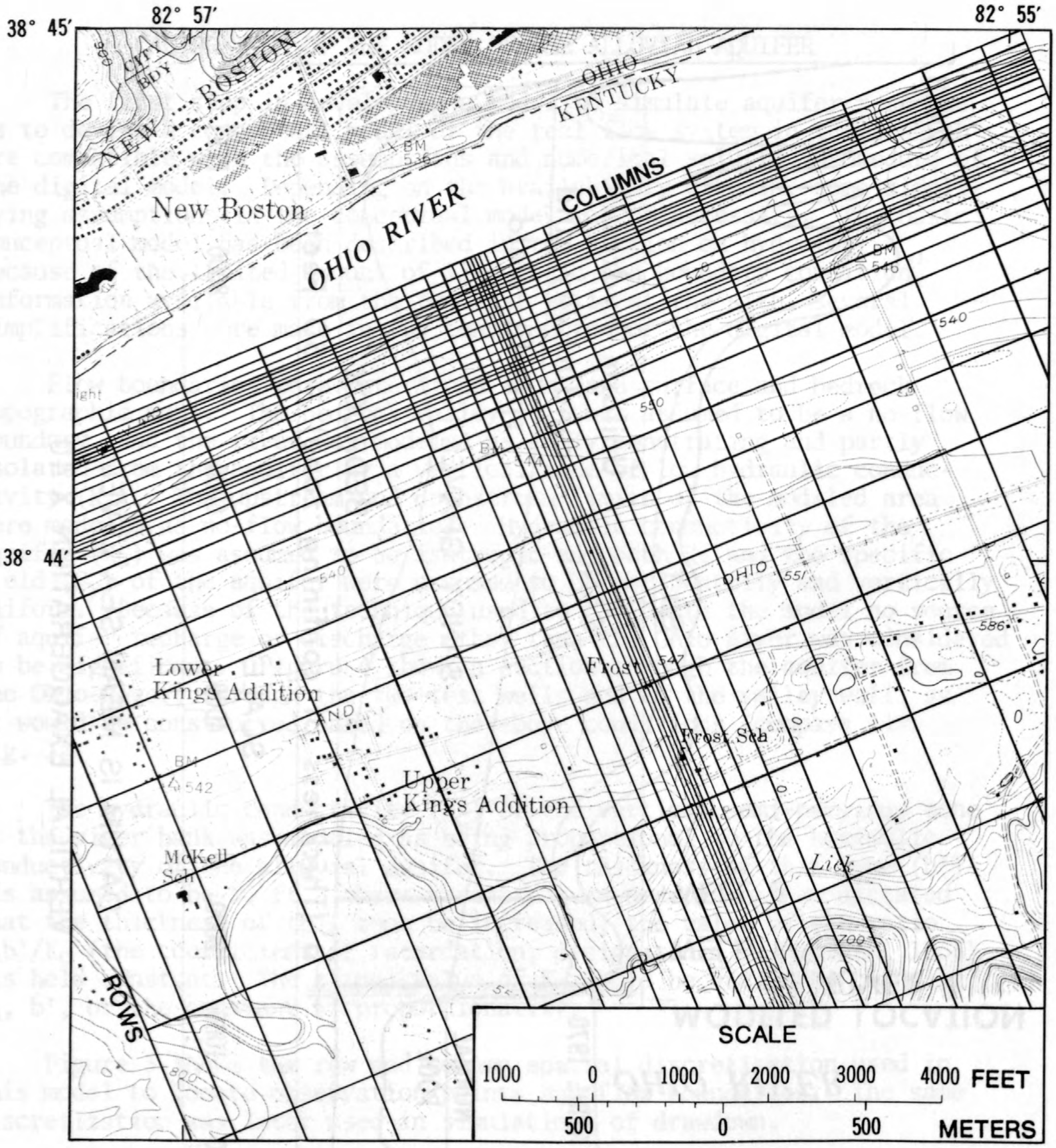

Figure 5-- Areal discretization used in the digital flow model to locate flow boundaries and observation points.

at the site was read into the river head matrix. Because the flow of the Ohio River so vastly overwhelms any changes in discharge due to bank storage the river stage was considered to be independent of aquifer effects and the river head matrix was modeled as a constant head part of the simulated aquifer but partly isolated from the main body of the aquifer by the zone of low conductivity. Figure 6 shows Ohio River stages and observed ground-water levels for the period March 20 to May 9, 1970.

As stated earlier, the flood-wave-response technique was employed for model calibration. It was assumed that head changes resulting from recharge due to precipitation and valley wall outflow were minor in comparison to those resulting from river head changes. This assumption eliminated the need to introduce two more variables in the calibration of the model.

Aquifer hydraulic diffusivity ($K_a b/S_y$), whose dimensions are length squared over time, may be determined during calibration of the digital flow model by increasing or decreasing the ratio of K_a to S_y of the aquifer until the calculated time difference between the occurrence of the highest water levels in the two observation wells agrees with the observed difference during the passage of a flood-generated wave through the aquifer. For the particular flood used to calibrate the flow model the Ohio River crested on April 5, 1970 and the highest water levels were recorded in well 1 (125 ft from the river) and well 2 (975 ft from the river) on April 6 and April 8, 1970, respectively.

Modeled aquifer diffusivity was changed until the simulated water-level time lag between wells matched the observed. Although the magnitude of the peaks and their time of arrival were then both in error for the simulation, the lag between wells was primarily a function of aquifer diffusivity between the wells. Thus the value obtained was unique. Hydraulic diffusivity was determined by this technique to be 2.21 ft^2/s thus verifying earlier work by Grubb and Zehner (1973) which employed a one-dimensional procedure to determine aquifer diffusivity at the site.

Also supporting the validity of the hydraulic diffusivity value determined with the digital model are the results from two aquifer tests which were conducted for Columbia Hydrocarbons Corporation by Ranney Method Water Supplies, Inc., in January and February of 1958. Both tests were at wells located near (approximately 500 and 1,700 ft) the two observations wells used in this report to construct the ground-water-flow model at the Columbia Hydrocarbons Corporation site. Using

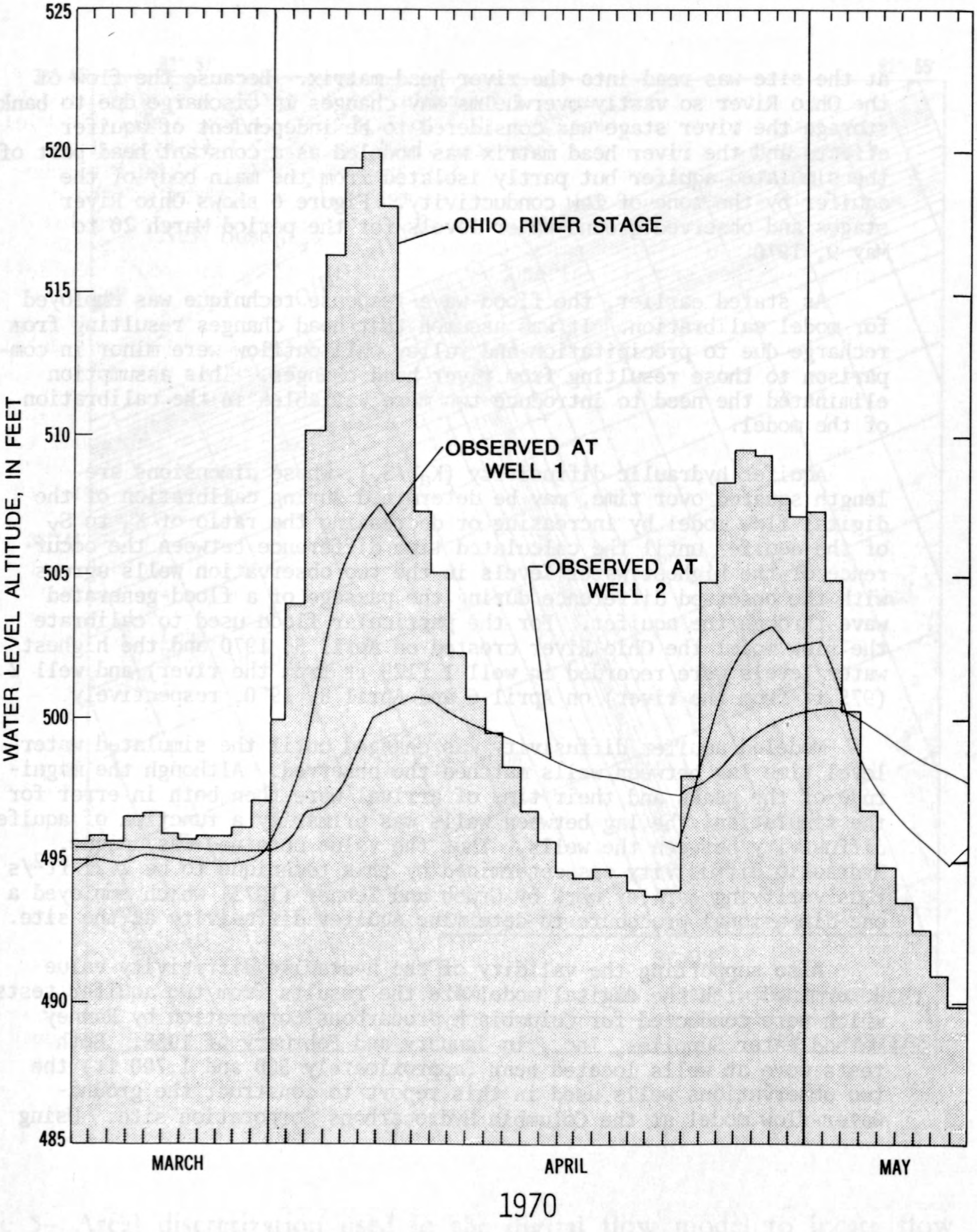

Figure 6-- Ohio River stages and water levels in wells 1 and 2 at the test site for the period of March 20 to May 9, 1970.

data from these tests, aquifer hydraulic conductivity was determined by the author to range from 2.3 X 10^{-3} to 3.2 X 10^{-3}ft/s and specific yield from 0.06 to 0.20. For a saturated thickness of 47.5 ft hydraulic diffusivity therefore ranged from 1.86 to 0.76 ft^2/s. Because temperature data were not available for the tests, no adjustment of these diffusivities to correct for different fluid viscosities is possible. The comparison betweeen model derived diffusivity and the values obtained by the conventional aquifer-test method is considered acceptable.

The next step in model calibration was to vary K_a, S_y, and the coefficient of retardation from one calibration attempt to the next until the simulated aquifer response at both wells matched the observed. Values of the three aquifer parameters were areally uniform for each calibration attempt. Because the hydraulic diffusivity had been previously defined, the ratio of $K_a b/S_y$ was held constant (b was equal to 47.5 ft, which was the maximum saturated thickness of the aquifer during the passage of the flood-generated wave) hence a change in S_y required a compensating change in K_a.

Changes in the coefficient of retardation of the river bank were accomplished by changing the values for K_c in the conductivity matrix. Because b' was not areally uniform, K_c for each node dimension was determined from the simulated value for $\underline{a}$ (which was areally uniform) by the relationship

$$K_c = K_a b'/\underline{a}$$

No attempt was made to vary $\underline{a}$ by altering the assumed dimension of the semipervious river bank. Such changes between calibration attempts would have required repeated redimensioning of the model matrices. Therefore, no evaluation was made of the sensitivity of the model to variations in b' while maintaining a constant and uniform value for $\underline{a}$.

Best overall calibration of the model was achieved using a value of 1,250 ft for $\underline{a}$. Subsequent sensitivity analysis of the parameters K_a and S_y showed that simulated water-level response was virtually independent of these parameters over the entire range of their commonly accepted limits. The simulated responses were also found to be only slightly sensitive to the specific yield of the low conductivity zone. Figures 7 and 8 show observed and calculated water level response at wells 1 and 2 respectively.

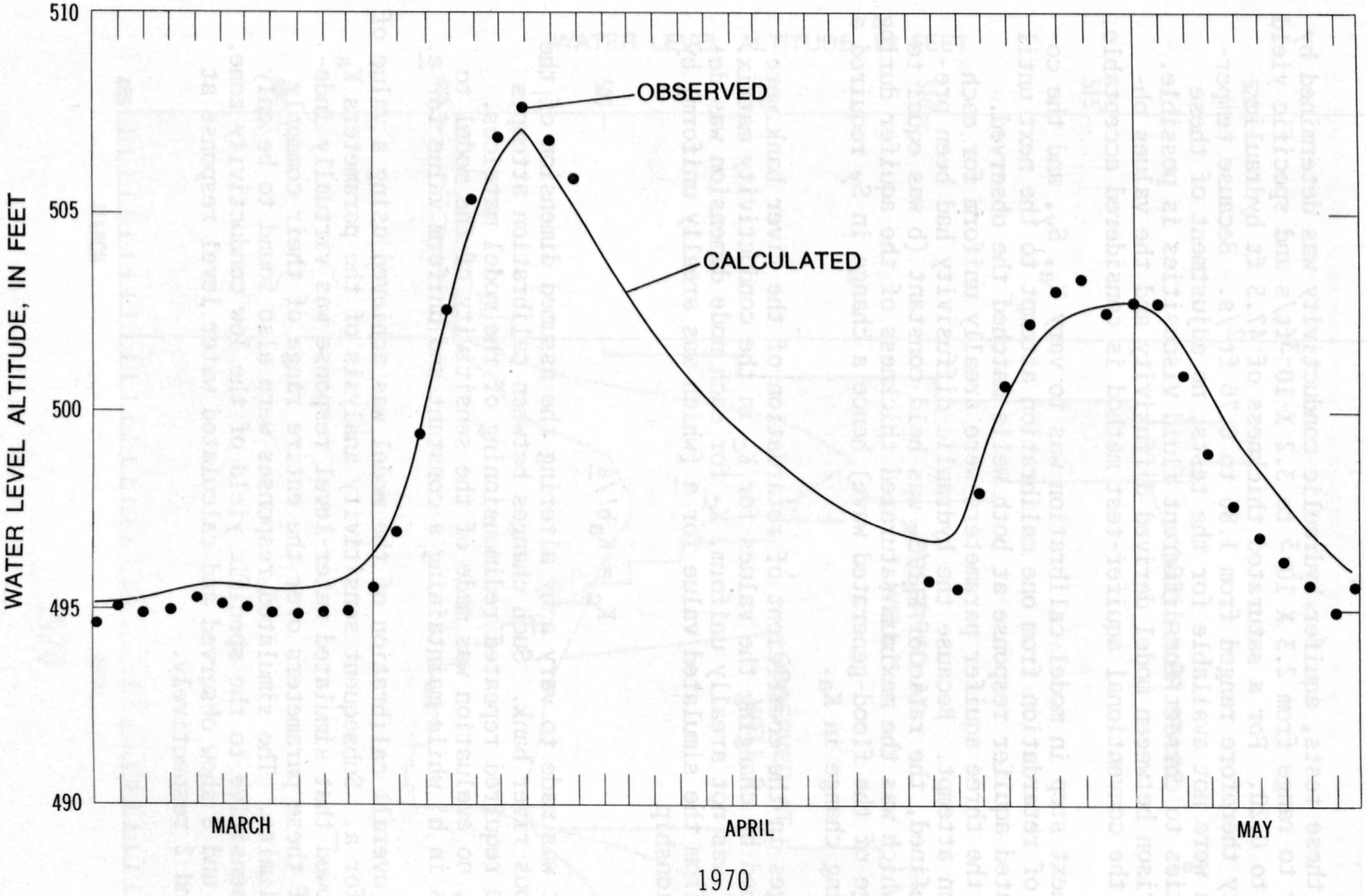

Figure 7-- Observed and calculated response of water level in well 1 to the floods of March 20 to May 9, 1970 on the Ohio River.

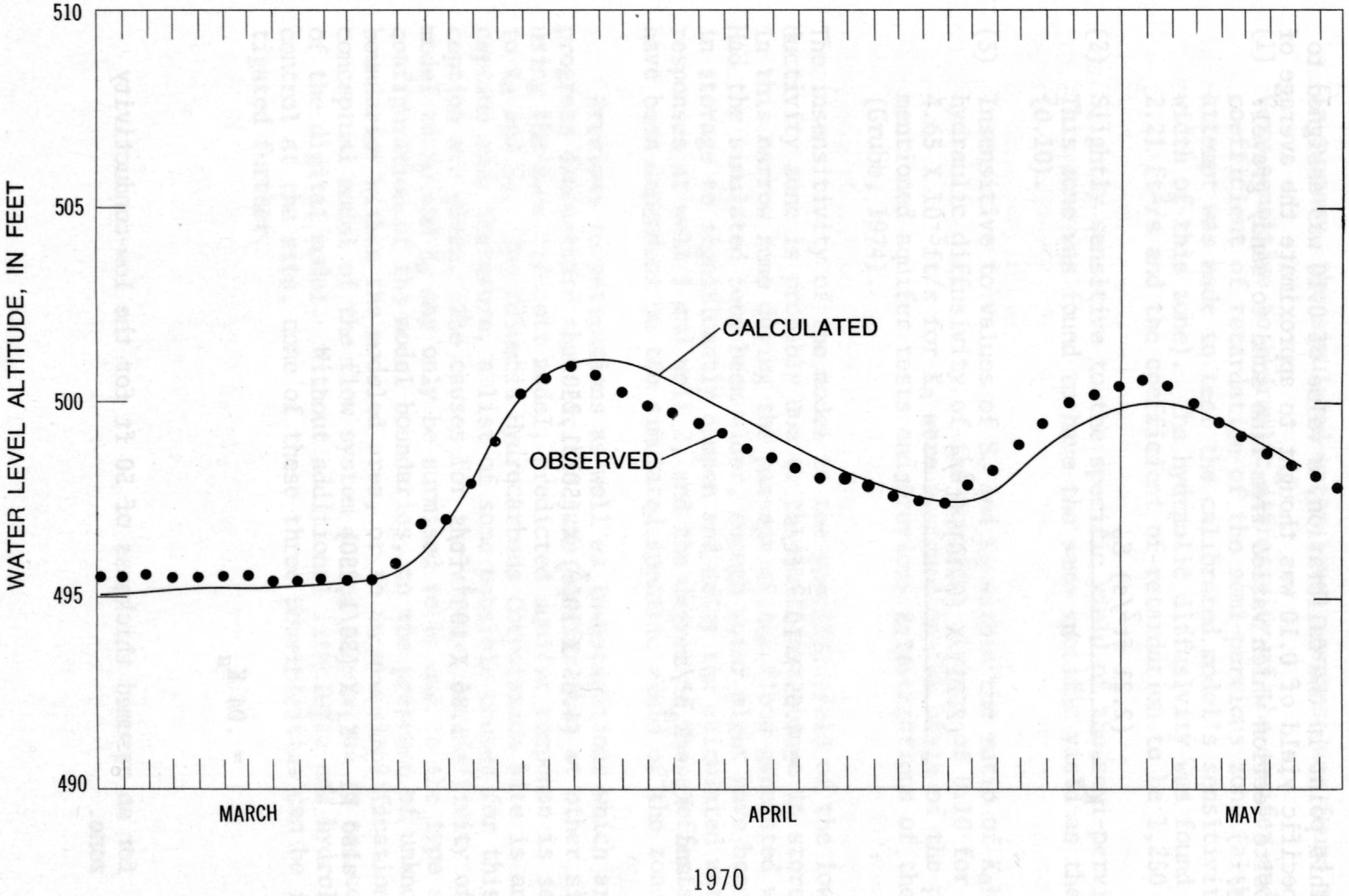

Figure 8-- Observed and calculated response of water level in well 2 to the floods of March 20 to May 9, 1970 on the Ohio River.

At this point in the calibration, a value of 0.10 was assigned to S_y. A specific yield of 0.10 was thought to approximate the average of the composite section which varied from fine sand to sandy gravel. Therefore:

$$K_a = \frac{(2.21\ ft^2/s)\ S_y}{b}$$

$$= \frac{(2.21) \times (0.10)}{47.5} \frac{ft^2/s}{ft}$$

$$= 4.65 \times 10^{-3}\ ft/s$$

and $K_c = K_a b'/a$

$$= (4.65 \times 10^{-3}) \times (50)/1{,}250$$

$$= 1.86 \times 10^{-4}\ ft/s$$

also $K_c = K_a \times (50/1{,}250)$

$$= .04\ K_a$$

for an assumed thickness of 50 ft for the low-conductivity zone.

In summary, the simulated water-level response was found to be:

(1) Very sensitive to hydraulic diffusivity of the aquifer and the coefficient of retardation of the semi-pervious zone (although no attempt was made to test the calibrated model's sensitivity to the width of this zone). The hydraulic diffusivity was found to be 2.21 ft^2/s and the coefficient of retardation to be 1,250 ft.

(2) Slightly sensitive to the specific yield of the semi-pervious zone. This zone was found to have the same specific yield as the aquifer (0.10).

(3) Insensitive to values of S_y and K_a within the ratio of $K_a b/S_y$, the hydraulic diffusivity of the aquifer. Values of 0.10 for S_y and 4.65 X 10^{-3}ft/s for K_a were assigned on the basis of the previously-mentioned aquifer tests and previous investigations of the aquifer (Grubb, 1974).

The insensitivity of the model to the specific yield of the low conductivity zone is probably due to the small net change in stored water in this narrow zone during the passage of the flood-generated waves. Had the simulated zone been wider, enough water might have been retained in storage to significantly dampen and delay the calculated water-level responses at well 1 and well 2, and the degree of these effects would have been dependent on the simulated specific yield of the zone.

Previous investigations as well as investigations which are in progress demonstrate that for the same aquifer (but at other sites), and using the same Trescott model, predicted aquifer response is sensitive to K_a and S_y. The Columbia Hydrocarbons Corporation site is an exception and, therefore, a list of some possible causes for this exception are given. The causes for the virtual insensitivity of the model to S_y and K_a may only be surmised to be due to the type and configuration of the model boundaries, to the presence of unknown boundaries within the modeled area, or to an oversimplification of the conceptual model of the flow system during adaptation to the constraints of the digital model. Without additional lithologic and hydrologic control at the site, none of these three possibilities can be investigated further.

THEORETICAL DRAWDOWN DUE TO SIMULATED PUMPAGE

The calibrated two-dimensional model, with the assumed values for S_y and K_a, was used to determine theoretical drawdown due to simulated pumpage for selected well locations at the Columbia Hydrocarbons Corporation site. Only one well field design was simulated (a line of wells parallel to the Ohio River), but the distance from the river and spacing of wells in the well field were varied. The results of these simulated responses are valid only in a qualitative sense. The absolute extent and magnitude of the theoretical drawdown depend on true values of the individual parameters S_y, K_a, K_c and b', which could not be determined with confidence during calibration. However, drawdown patterns and relative development for various pumping rates depend on aquifer diffusivity and the coefficient of retardation of the low conductivity zone both of which were determined with confidence.

In addition to the assumptions stated earlier, the following conditions were assumed for determination of drawdown due to simulated pumpage:

1. No recharge to the aquifer from precipitation or seepage from the bedrock valley wall.

2. The Ohio River remains at normal pool stage (an altitude of 485 ft) for the duration of the simulation (one year).

3. The wells are one foot in diameter, 100 percent efficient and are open to the entire saturated thickness of aquifer.

4. Aquifer hydraulic conductivity does not change in response to seasonal temperature variations of the ground water or variations resulting from induced infiltration from the river. Temperature conditions are assumed to be identical to those during the passage of the flood wave used to calibrate the model.

5. Maximum drawdown of water levels in the aquifer is limited to 20 ft.

The above assumptions regarding recharge represent unrealistically severe climatic conditions. These natural sources of recharge were not included in pumpage simulations for several reasons: (1) because the time and magnitude of occurrence of the recharge events are unpredictable; (2) because an attempt to include these sources of recharge might obscure simulated responses to pumpage due to aquifer characteristics;

and (3) because of the variability of these additional sources of recharge, the portrayal of these events as normal or average might lead to overdevelopment of the aquifer and possibly to a water-supply crisis in an abnormally dry year. These sources of recharge do exist and will certainly maintain a greater saturated thickness of aquifer than the simulation criteria specify, but they are not dependable.

Although the above sources of recharge were not included in the pumpage simulations their annual occurrence cycle was recognized; therefore, pumpage simulations were for response to one year of pumpage rather than being extended to steady-state solutions.

Figure 9 and 10 illustrate drawdown in a single pumped well as a function of time. If 20 ft is considered to be the maximum allowable drawdown (leaving a saturated thickness of 6 ft) it is apparent from figure 9 that 900 gal/min is an excessive pumpage rate for this aquifer; neither well location can sustain this pumpage rate for more than a day. On the other hand, for wells located 125 ft or 975 ft from the river, figure 10 shows that a pumpage rate of 450 gal/min is clearly sustainable for as much as one full year. Figure 11 shows areal drawdown at the end of one year of pumping 450 gal/min from a single well located 125 ft from the river.

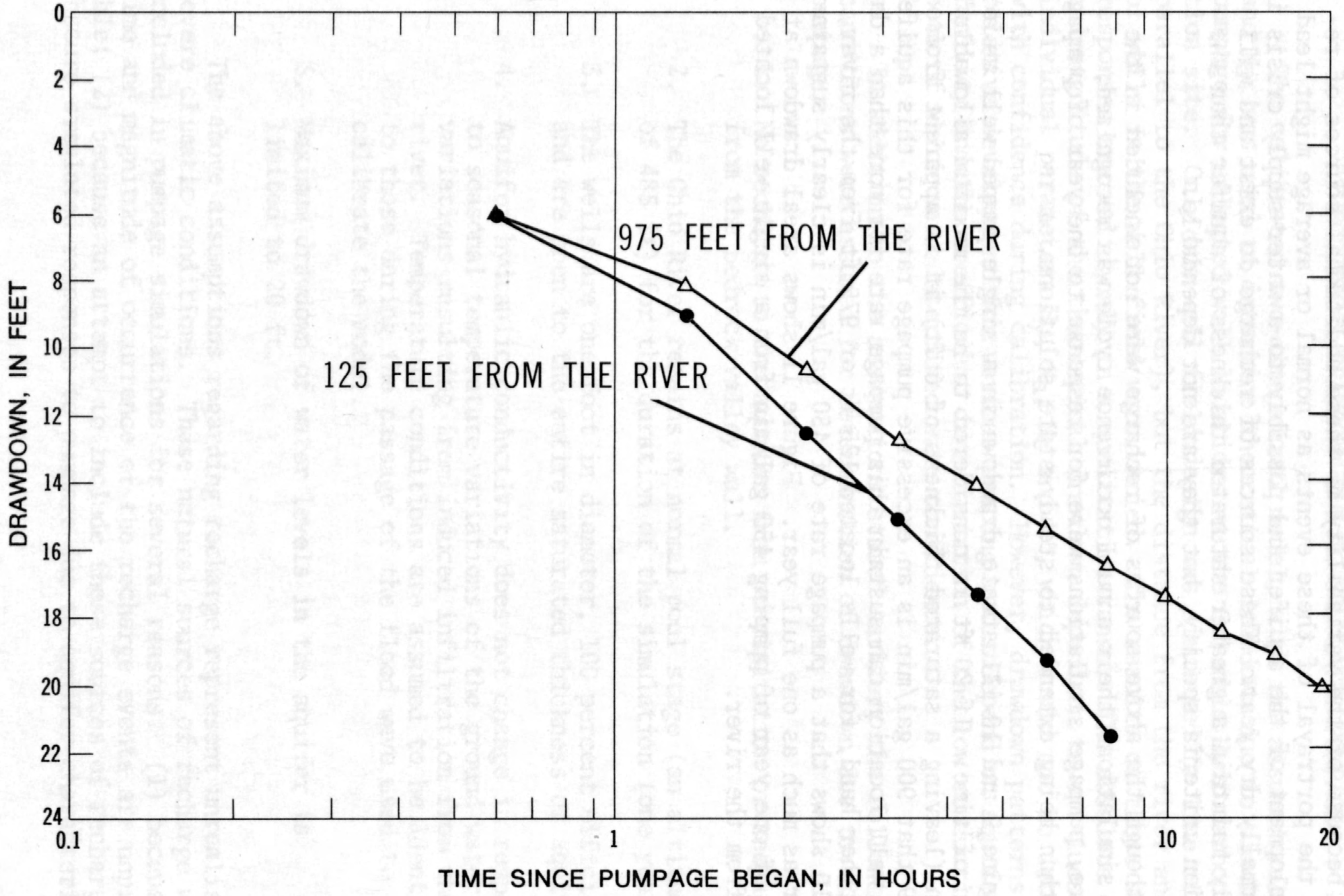

Figure 9-- Theoretical drawdown in pumping wells located 125 ft and 975 ft from the Ohio River, due to a simulated pumping rate of 900 gal/min.

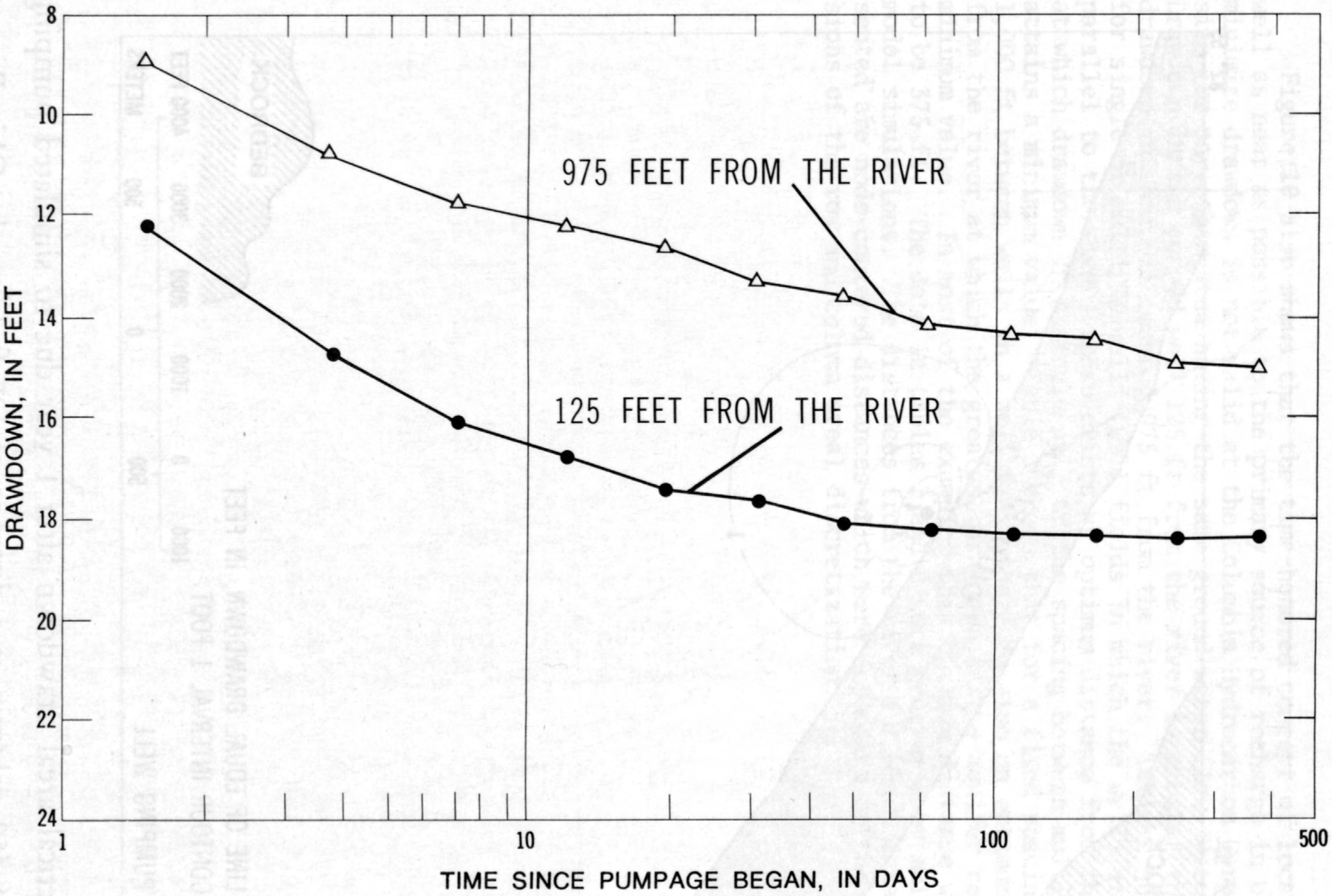

Figure 10-- Theoretical drawdown in pumping wells located 125 ft and 975 ft from the Ohio River, due to a simulated pumping rate of 450 gal/min.

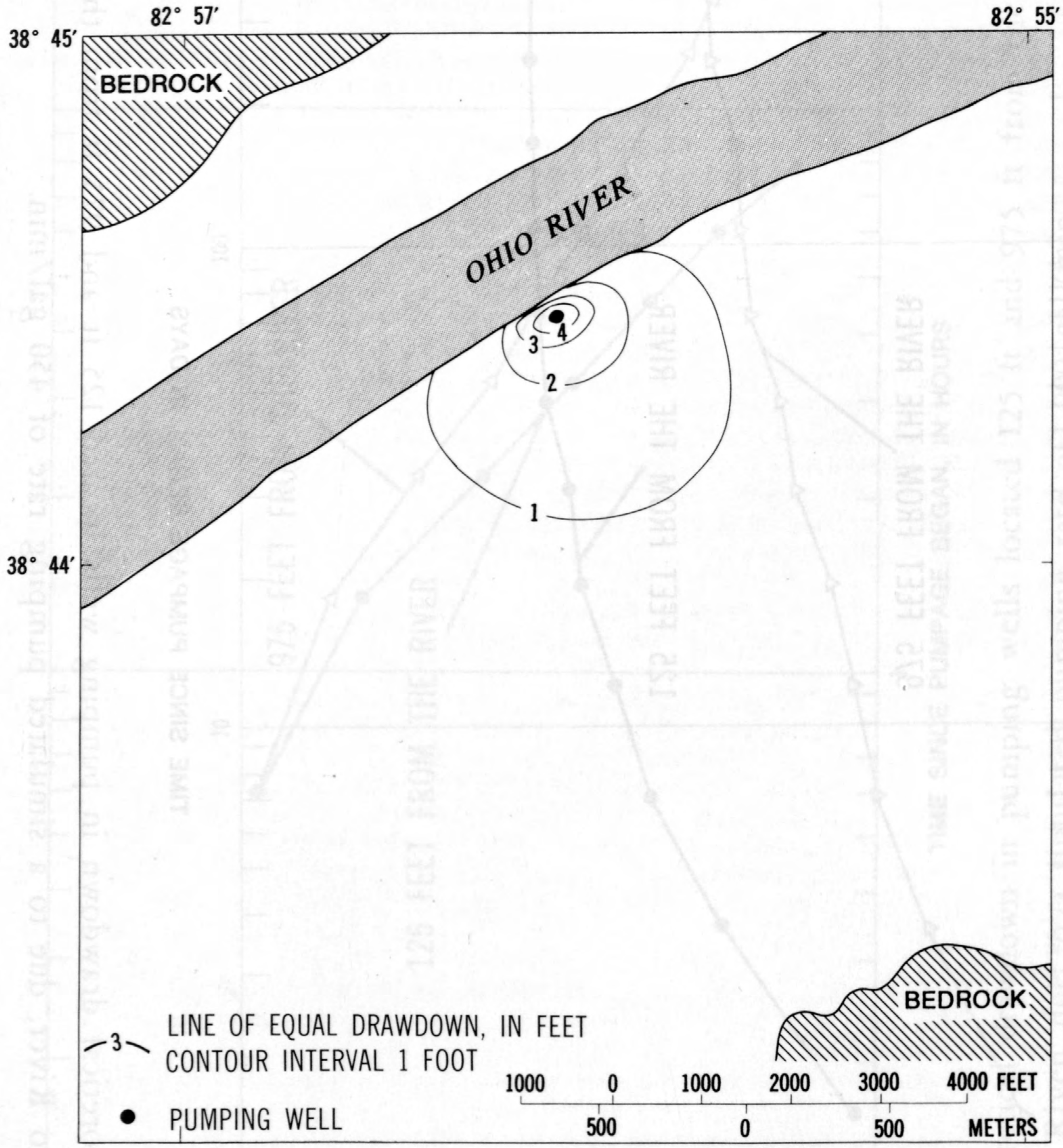

Figure 11-- Theoretical areal drawdown after 1 year due to simulated pumping rate of 450 gal/min at a distance of 125 ft from the Ohio River. Drawdown in the pumped well is 18.5 ft.

Figure 10 also shows that the time-honored concept of locating a well as near as possible to the primary source of recharge in order to minimize drawdown is not valid at the Columbia Hydrocarbon Corporation site (or any other area having the same ground-water flow system). Drawdown in the pumped well 125 ft from the river is greater than the drawdown in the well located 975 ft from the river. Table 1 shows that for single or multiple wells (well fields in which the wells are located parallel to the river) there exists an optimum distance from the river at which drawdown in a single well or the spacing between multiple wells attains a minimum value. Table 2 shows that for a fixed spacing of 1,000 ft between wells in a well field there is also an optimum distance from the river at which the greatest drawdown in the aquifer reaches a minimum value. In most of the examples this optimum distance was found to be 375 ft. The data in tables 1 and 2 were generated by multiple model simulations. The distances from the river for which data are presented are node-centered distances which were dependent on the dimensions of the row and column areal discretization.

Table 1.--Minimum well spacing in well fields parallel to the Ohio River for selected distances from the river. The pumping rate for all wells is 450 gal/min for one year. Well spacing, in feet, was determined for a maximum drawdown of 20 ft, and were determined based on assumed values of aquifer hydraulic conductivity and specific yield.

Number of wells	Distance from the Ohio River, in feet						
	125	275	375	488	613	725	975
1*	18.5*	14.6*	14.5*	13.9*	13.9*	14.6*	15.1*
2	699	141	127	88	89	141	191
3	944	590	545	515	529	590	668
4	944	742	690	724	746	742	792
5	944	842	787	870	898	842	873

*Drawdown, in feet, for a single pumped well after 1 year of pumpage.

Table 2.--Maximum drawdown, in feet, in the aquifer for a spacing of 1,000 ft between wells each pumping 450 gal/min for one year in well fields located at selected distances from the Ohio River. Drawdowns are computed based on assumed values of aquifer hydraulic conductivity and specific yield.

Number of wells	Distance from the Ohio River, in feet						
	125	275	375	488	613	725	975
3	19.7	18.0	17.7	17.7	17.8	18.0	18.4
4	19.7	18.4	18.0	18.4	18.6	18.4	18.7
5	19.7	18.8	18.3	19.1	19.4	18.8	19.0

The optimum location of wells in a well field is a complex function of many existing hydrologic conditions, rates of pumpage from the wells, and anticipated future water needs. When pumpage is begun in an undisturbed aquifer the water which is first withdrawn is derived entirely from storage. The storage coefficient of the aquifer in the immediate vicinity of a pumped well governs the initial drawdown in the well. Drawdown will continue and the cone of water-level depression will expand until recharge is intercepted which equals, in volume, the discharge from the well or wells. The only source of recharge included in the pumpage simulations was induced recharge from the river through the semi-pervious zone at the river bank.

Well location with regard to the river and river bank may be considered in three categories for the purpose of discussion: Too near, too far, and optimum.

A pumped well located near the river bank is close to the source of recharge, but because the river bank is only 4 percent as conductive as the aquifer, head loss through the bank will be large and the bank will appear to function as a local barrier to ground-water flow (at the same time the head loss between the bank and the well will be small). The water-level depression in the aquifer will extend along this semi-barrier until sufficient recharge is induced to balance the discharge from the pumped well. Lines of equal drawdown in the aquifer (fig. 11) bear a close similarity in configuration to those displayed when a pumped well is adjacent to a linear flow barrier. When a well is located near the river bank the low conductivity of the bank becomes dominant and drawdown in the well and aquifer indicates that the bank functions more as a flow barrier than as a source of recharge.

As the distance between the pumped well and the river bank increases, the semi-pervious nature of the bank becomes less significant. Most of head loss in the system occurs in the aquifer; very little loss occurs across the bank. The linear distance of river bank intercepted by the cone of depression is very large and the flux per unit area of bank material is small. When a well is located a great distance from the river bank, the remoteness of the sole source of recharge becomes dominant and drawdown in the pumped well is correspondingly large.

The digital model indicates that between the two extremes there is an optimum distance from the river at which a pumped well (or well field) can best induce recharge through the bank and simultaneously minimize the partial barrier effect of the low conductivity of the bank.

While table 1 lists the minimum well spacing for as many as five wells in a well field, table 2 shows that a spacing of 1,000 ft provides a safeguard against overdevelopment of the aquifer for well fields of as many as 5 wells.

The minimum well spacings given in table 1 do not allow any future enlargement of the well field. The addition of but one more well (pumping rate 450 gal/min) anywhere within the modeled area would theoretically cause at least one existing well to exceed the 20 ft maximum allowable drawdown. Whether this would be true in reality would depend on precipitation and on flood recharge, which were not considered in the pumpage model, and on refinement of the parameters K_a and S_y.

Figure 12 illustrates areal drawdown at the end of 1 year of pumping 450 gal/min from each of three wells located 375 ft from the river and at a spacing of 1,100 ft between wells. The lines of equal drawdown were plotted by the computer and then manually refined to eliminate distortions. Figure 13 is a water-level cross section through the three-well system illustrated in figure 12.

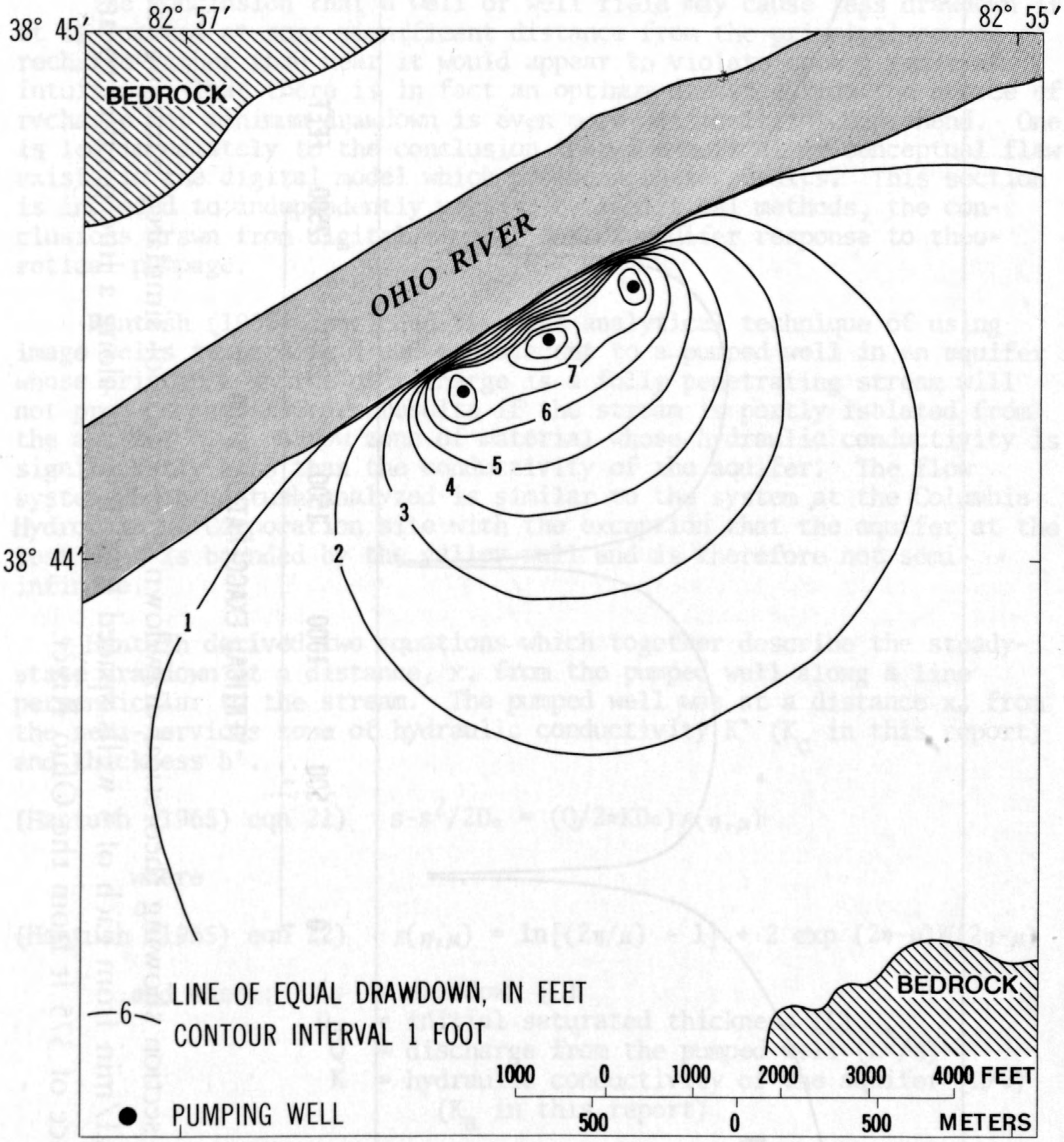

Figure 12-- Theoretical areal drawdown after 1 year due to simulated pumping rate of 450 gal/min from each of 3 wells spaced 1,100 ft apart along a line parallel to and at a distance of 375 ft from the Ohio River.

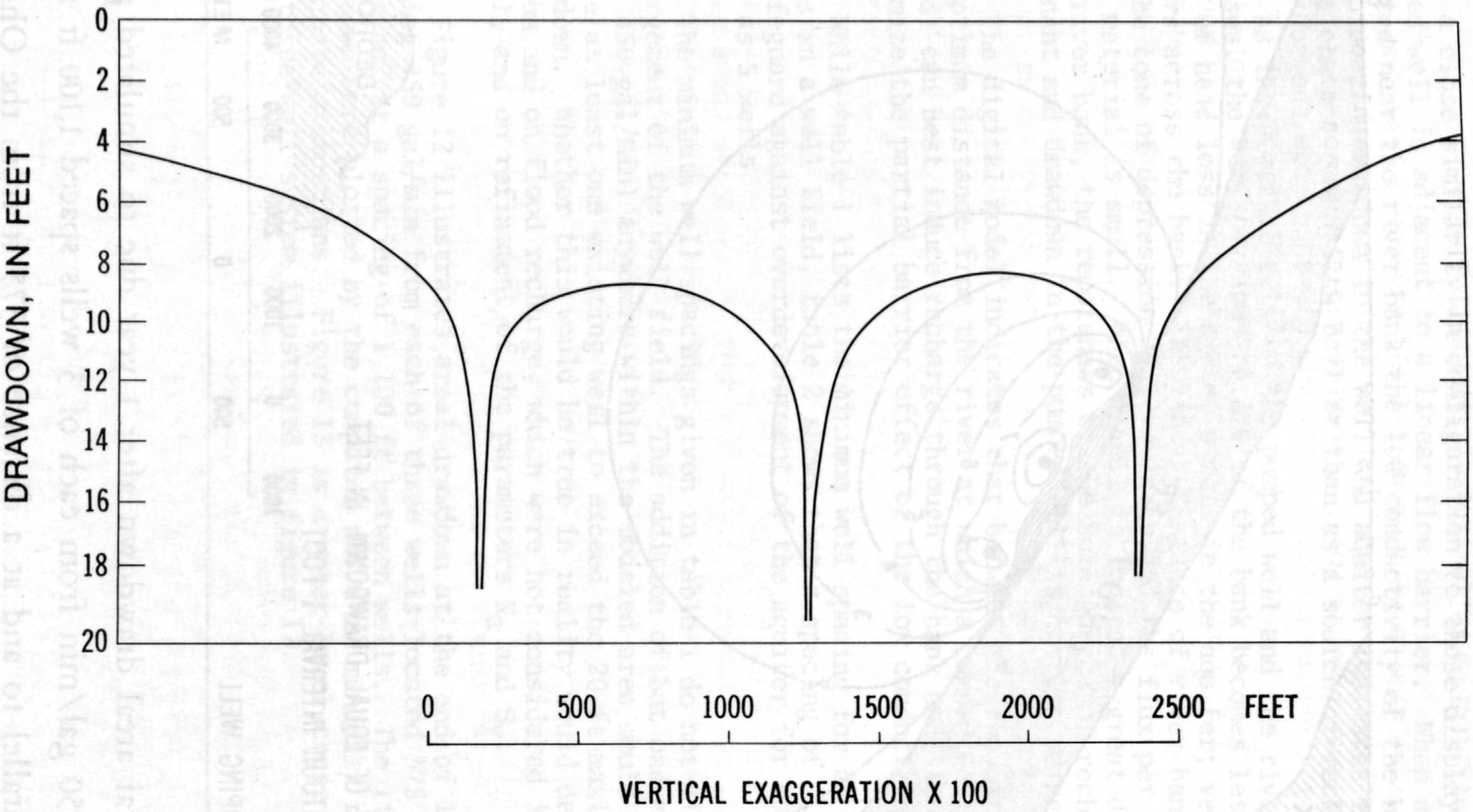

Figure 13-- Cross section showing theoretical drawdown after 1 year of simulated pumping rate of 450 gal/min from each of 3 wells spaced 1,100 ft apart along a line parallel to and at a distance of 375 ft from the Ohio River.

ANALYTICAL SOLUTION FOR OPTIMUM WELL PLACEMENT

The conclusion that a well or well field may cause less drawdown if it is located at some significant distance from the principal source of recharge rather than near it would appear to violate common sense and intuition. That there is in fact an optimum distance from the source of recharge for minimum drawdown is even more difficult to comprehend. One is led immediately to the conclusion that a numerical or conceptual flaw exists in the digital model which produced these results. This section is included to independently verify, by analytical methods, the conclusions drawn from digital simulations of aquifer response to theoretical pumpage.

Hantush (1965) concluded that the analytical technique of using image wells to predict drawdown adjacent to a pumped well in an aquifer whose principal source of recharge is a fully penetrating stream will not produce satisfactory results if the stream is partly isolated from the aquifer by a narrow zone of material whose hydraulic conductivity is significantly less than the conductivity of the aquifer. The flow system which Hantush analyzed is similar to the system at the Columbia Hydrocarbons Corporation site with the exception that the aquifer at the test site is bounded by the valley wall and is therefore not semi-infinite.

Hantush derived two equations which together describe the steady-state drawdown at a distance, x, from the pumped well along a line perpendicular to the stream. The pumped well was at a distance x_o from the semi-pervious zone of hydraulic conductivity K' (K_c in this report) and thickness b'.

(Hantush (1965) eqn 21) $$s - s^2/2D_o = (Q/2\pi K D_o) f(\eta,\mu)$$

where

(Hantush (1965) eqn 22) $$f(\eta,\mu) = \ln[(2\eta/\mu) - 1] + 2 \exp(2\eta-\mu) W(2\eta-\mu)$$

and where:
- s = drawdown (L)
- D_o = initial saturated thickness (L)
- Q = discharge from the pumped well (L^3/t)
- K = hydraulic conductivity of the aquifer (L/t) (K_a in this report)
- $W(z)$ = well function of $z = \int_z^\infty \frac{\exp(-z)}{z} dz$
- $\eta = x_o/a$
- $\mu = x/a$
- $a = Kb'/K'$ the coefficient of retardation (L) ($K_a b'/K_c$ in this report)

Combining equations 21 and 22 from Hantush (1965) yields:

$$(s-s^2/2D_o)/(Q/2\pi KD_o) = \ln\left(\frac{2x_o}{x} - 1\right) + 2\exp\left(\frac{2x_o}{a} - \frac{x}{a}\right) W\left(\frac{2x_o}{a} - \frac{x}{a}\right)$$

The left-hand term of the equation is dimensionless drawdown which is independent of the discharge of the well and the initial saturated thickness and hydraulic conductivity of the aquifer. Figure 14 was constructed by repeatedly solving the above equation for dimensionless drawdown by increments of x_o = 0.5 ft for x equals 1 ft and 0.5 ft and, for a = (K/K') b' = (1/0.04) 50 ft = 1,250 ft.

The optimum distance from the river for location of a well or well field is shown in figure 14 to be 382 ft. Locating a well at the optimum distance does not ensure that the well will not go dry, rather that the well is at the location where it will go dry least rapidly, if at all (compare figures 9 and 10).

Figure 15 shows that as a approaches zero (when the river and aquifer are in full hydraulic connection) the solution to Hantush's equations 21 and 22 converges to the more intuitive distribution of drawdown. As the coefficient of retardation decreases, the optimum distance from the river decreases until minimum drawdown occurs at the river bank when a, the coefficient of retardation, equals zero.

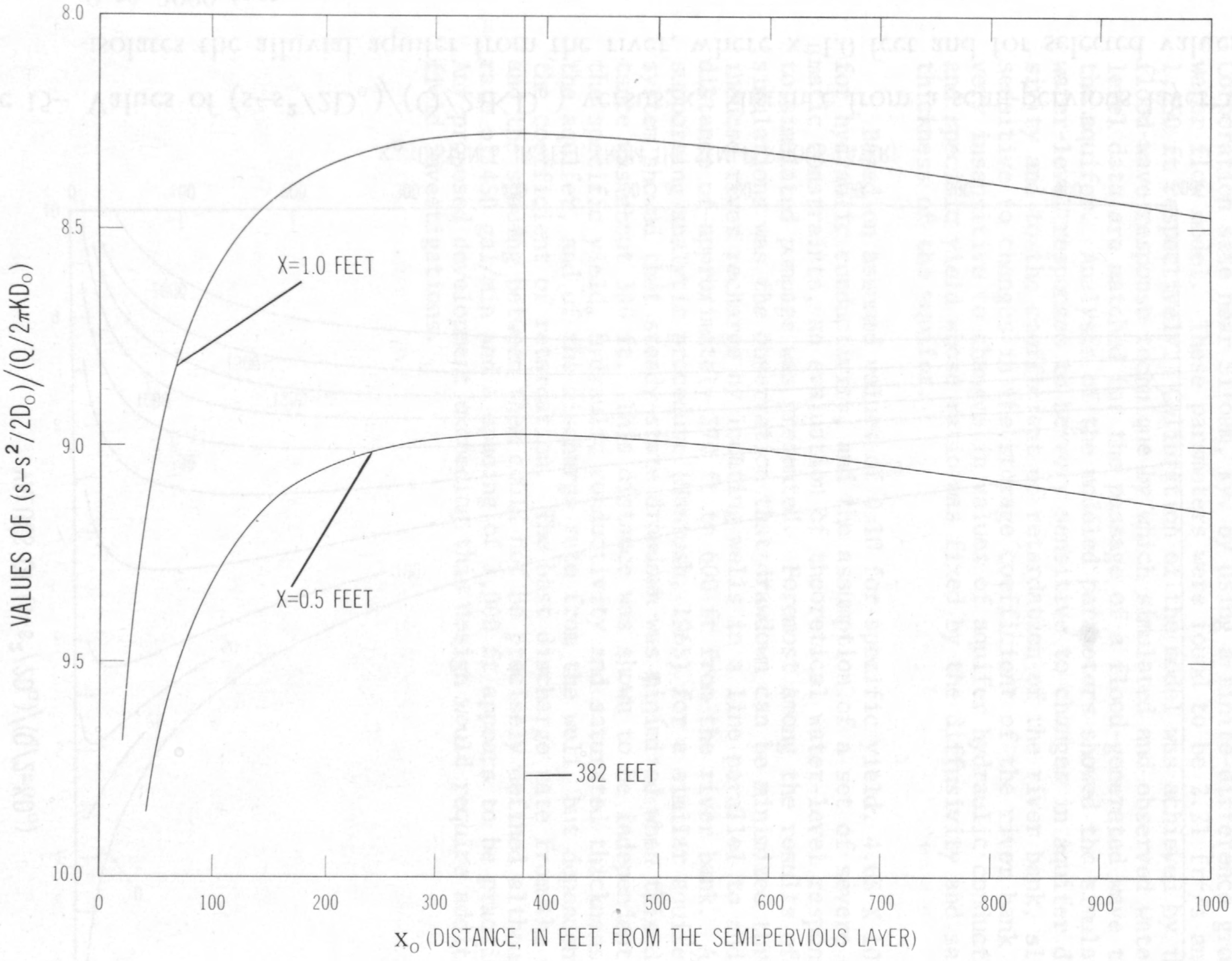

Figure 14-- Values of $(s - s^2/2D_o)/(Q/2\pi KD_o)$ versus x_o, distance from a semi-pervious layer which partly isolates the alluvial aquifer from the river, where a=1250 feet and x=1.0 and 0.5 feet.

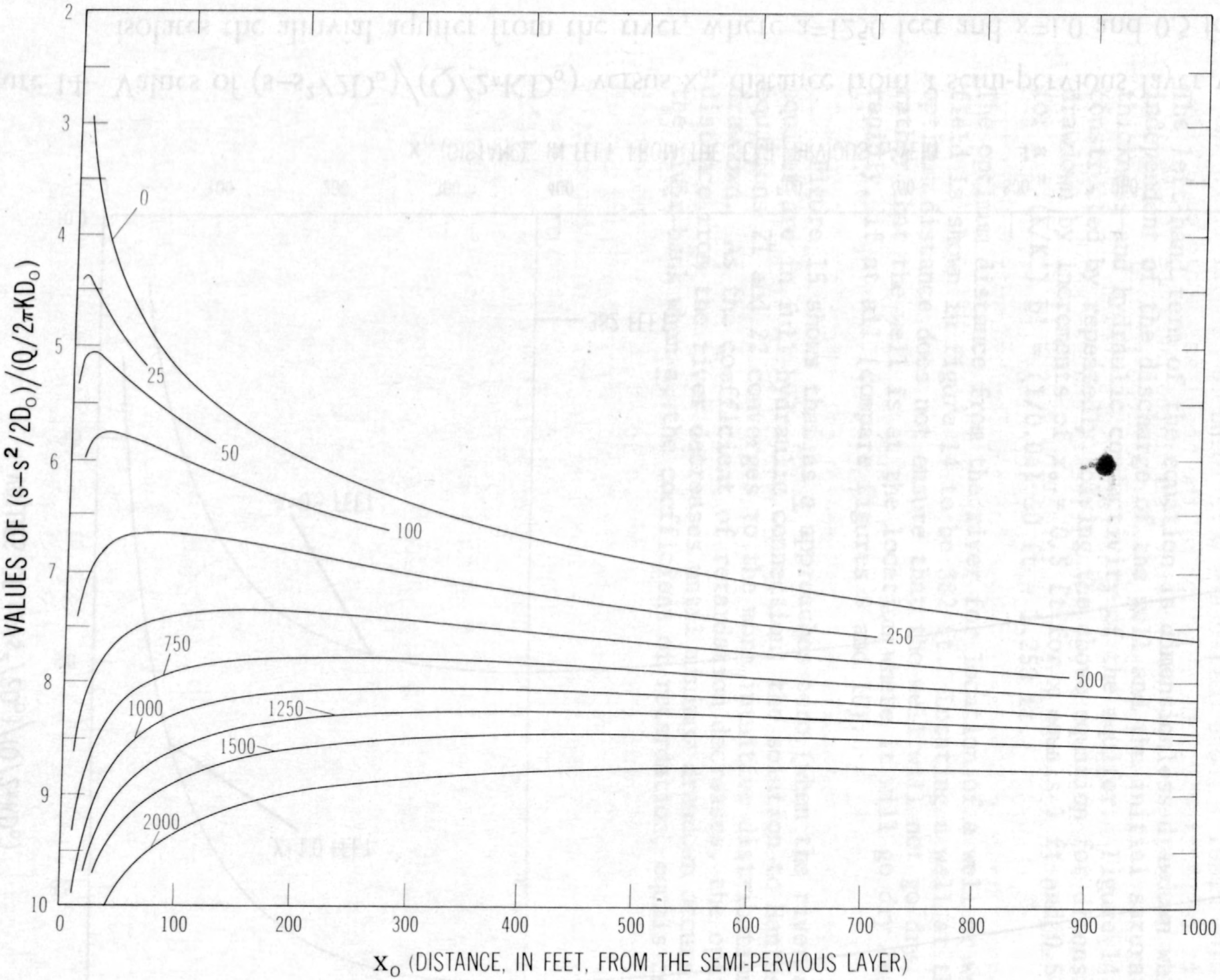

Figure 15-- Values of $(s-s^2/2D_o)/(Q/2\pi KD_o)$ versus x_o, distance from a semi-pervious layer which partly isolates the alluvial aquifer from the river, where x=1.0 feet and for selected values of a from 0 to 2000 feet.

SUMMARY AND CONCLUSIONS

Aquifer diffusivity and the coefficient of retardation of the river bank were determined for the alluvial aquifer at the Columbia Hydrocarbons Corporation site near Siloam, Ky. by using a finite-difference ground-water flow model. These parameters were found to be 2.21 ft^2/s and 1,250 ft respectively. Calibration of the model was achieved by the flood-wave response technique by which simulated and observed water-level data are matched for the passage of a flood-generated wave through the aquifer. Analysis of the modeled parameters showed the simulated water-level responses to be very sensitive to changes in aquifer diffusivity and to the coefficient of retardation of the river bank, slightly sensitive to changes in the storage coefficient of the river bank and very insensitive to changes in values of aquifer hydraulic conductivity and specific yield whose ratio was fixed by the diffusivity and saturated thickness of the aquifer.

Based on assumed values of 0.10 for specific yield, 4.65×10^{-3} ft/s for hydraulic conductivity, and the assumption of a set of severe climatic constraints, an evaluation of theoretical water-level response to simulated pumpage was presented. Foremost among the results of these simulations was the observation that drawdown can be minimized through induced river recharge by locating wells in a line parallel to and at a distance of approximately 370 ft to 600 ft from the river bank. A supporting analytic procedure (Hantush, 1965) for a similar aquifer system showed that steady-state drawdown was minimized when this distance was about 380 ft. This distance was shown to be independent of the specific yield, hydraulic conductivity and saturated thickness of the aquifer, and of the discharge rate from the well, but dependent on the coefficient of retardation. The best discharge rate from the wells and the spacing between them could not be precisely defined although a rate of 450 gal/min and a spacing of 1,000 ft appears to be practical Any proposed development exceeding this design would require additional field investigations.

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