Techniques of Water-Resources Investigations of the United States Geological Survey

Chapter B1

AQUIFER-TEST DESIGN, OBSERVATION AND DATA ANALYSIS

By Robert W. Stallman

Book 3

APPLICATIONS OF HYDRAULICS

CONTENTS

FIGURES

TABLE

NOMENCLATURE

Symbol	*Definition*	*Units*
α	A constant, or a delay index.	Dimensionless, or inverse time.
b	Thickness of saturated part of aquifer.	Length.
b'', b'''	Thickness of confining beds.	Length.
B	$(T/\alpha S)^{1/2}$, a constant.	Dimensionless.
β	A constant.	Dimensionless.
C	A constant.	Dimensionless.
K'', K'''	Hydraulic conductivity of confining beds.	Length per unit time.
K_z, K_r	Hydraulic conductivity of aquifer to vertical (z) and horizontal (r) flow, respectively.	Length per unit time.
Q	Discharge from a well.	Volume per unit time.
r	Distance from control well to observation point.	Length.
s	Change in head, or drawdown.	Length.
S	Storage coefficient of the aquifer.	Dimensionless.
S'	Apparent storage coefficient, observed in aquifers dewatered significantly in proportion to saturated thickness.	Dimensionless.
S'', S'''	Storage coefficient of confining beds.	Dimensionless.
t	Time.	Time.
T	Transmissivity of aquifer.	Area per unit time.
u	$r^2S/4Tt$.	Dimensionless.

AQUIFER-TEST DESIGN, OBSERVATION, AND DATA ANALYSIS

By Robert W. Stallman

Introduction

An aquifer test is a controlled field experiment made to determine the hydraulic properties of water-bearing and associated rocks. The test is made by observing ground-water flow that is produced by known hydraulic boundary conditions such as variations of head along a connected stream, pumping wells, changes in weight imposed on the land surface, or changes in recharge. The hydraulic boundary conditions may be imposed as part of the natural hydrologic system or by an act of man. Ground-water flow varies in space and time and is dependent on the hydraulic properties of the rocks and the boundary conditions imposed on the ground-water system.

The most common procedure for determining the hydraulic properties of aquifers is to (1) identify the hydraulic and physical boundary conditions affecting flow in the aquifer, (2) calculate the expected response in the aquifer by letting the hydraulic properties be variable and taking into account the hydraulic and physical boundaries, (3) compare the observed response with the computed response, and (4) adopt those hydraulic properties that were assumed in the computed response which matches the observed response.

Response can be computed in a variety of ways. Most commonly, the following methods are used:

Analytical solutions.—These are algebraic or integral equations that relate flow at boundaries, hydraulic properties, space, and time. Each solution generally applies to only one set of assumed conditions.

Flow nets.—Flow nets apply only to steady flow. Flow paths and head contours, at right angles to each other everywhere in the flow field of an isotropic, are drawn by trial and error. The resulting net may be used for estimating transmissivity.

Electric analogs.—Electric analogs can be used for generating response predictions effectively by simulating the aquifer and its boundary conditions that are highly variable in time and space.

Digital computers.—Where analytical solutions are expressed in integral form, the digital computer is effective for numerically evaluating the integral, thereby affording useful response information. Also, the digital computer can be used for simulating the flow system in much the same way as the electric analog.

Much has been written describing the development of response curves and their use for determining the hydraulic properties of aquifers. However, little attention has been given the mechanical aspects of test organization and data collection. Furthermore, the basic framework common to all aquifer tests is not apparent in the available literature. The purpose of this report is to (1) describe the common framework on which all test analysis is based, (2) detail the procedures for designing a test, (3) emphasize the importance of criteria to be met in the design and field execution of aquifer tests, and (4) call attention to required data.

It makes no difference in performing a test whether response curves are obtained as analytical solutions or by other methods, such as flow nets, electric analogs, or digital computers. Data requirements and methods of analysis are almost independent of the methods used for computing response curves. Also, in most situations aquifer response has the same magnitude regardless of whether boundary controls are positive or negative. The only difference between response in a drawdown test and in a recovery test is the direction of change in head. In this report, most illustrative material is therefore selected from analytical solutions for those situations in which drawdowns are imposed on the aquifer. Furthermore, discussion is limited to aquifer tests involving pumping wells. The general techniques described are equally effective for analyzing flows that are regulated by changing stream stages, for example, except that time and space scales need to be modified. Reader familiarity with the general concept of aquifer testing is presumed.

Acknowledgments

In this report, the writer acts merely as a recorder of information gleaned from association with hundreds of workers in hydrology. The report was made possible only through the fundamental contributions of C. V. Theis, M. S. Hantush, C. E. Jacob, J. G. Ferris, S. W. Lohman, and many others. I am indebted to about 75 members of the U.S. Geological Survey Water Resources Division who reviewed the original manuscript of this report and provided many helpful suggestions.

General procedure

Aquifer tests require three phases as follows:

DESIGN. This phase is probably the most important but least recognized aspect of aquifer tests. The cost of an aquifer test ranges from a few hundred dollars for the least complicated to several thousands of dollars for the most sophisticated, depending on the manpower and equipment allocated to the experiment. To improve the probability that a given test will be successful and therefore to avoid the fruitless expenditure of funds, it is especially important to exert considerable effort in experiment design. Careful and complete data collection must follow design. Information required in the design phase is as follows:

A. The geologic and hydraulic setting of the flow system is known in sufficient detail so that the boundary conditions controlling flow are known explicitly. Examples: aquifer thickness and extent, probable variations in transmissivity, values of transmissivity and storage coefficient, and magnitude of control to be imposed, such as change in discharge or head.

B. Response curves, based upon the control and boundary conditions to be imposed by the test, are available or can be developed at acceptable cost.

C. Equipment required for observing aquifer response is available or can be installed at acceptable cost. Examples: devices for measuring well discharge or stream stage, observation wells, and packers for isolating flow to a pumping well that is screened in several intervals of depth.

D. Available and (or) installed observation equipment will produce a definitive set of measurements, a set that will probably provide an accurate measure of the hydraulic properties of the aquifer.

Item D is a summary of the design phase—through careful examination of available information, decide whether a proposed test site and associated equipment are capable of accurately determining the hydraulic properties of the aquifer and whether the expected results are worth the effort.

FIELD OBSERVATION. The observations needed to measure boundary conditions and aquifer response adequately are decided largely in the design phase of the test. For example, by predicting response of the aquifer, the timing of measurements in the field is predetermined. If we expect drawdown at a given observation well to be measurable only after 4 days of pumping, there would be little use for water-level measurements taken at the observation well at half-minute intervals after a nearby well starts pumping. Adequate attention to design, wherein response is predicted, aids the efficient allocation of the observers' time.

DATA ANALYSIS. If the design and field-observation phases of the aquifer test are conducted successfully, data analysis is automatically routine and successful. Test-site and control conditions have been identified explicitly, the requirements for analytical methods have been specified, data collection has been completed, and there remains simply the chore of transforming field observations into estimates of the hydraulic properties.

At every test site the boundary conditions and hydraulic properties are unknown prior to testing; therefore, the analysis of the problem in the design phase contains uncertainties. The designer of the test must take these uncertainties into account to allow for latitude in so-called known conditions. For example, if the transmissivity can be estimated from other data within ±25 percent, the value of another field test is relatively small. If transmissivity is predictable within plus or minus an order of magnitude or more, placement of observation wells and timing of required measurements in the field will be affected strongly.

The working hypothesis

Field and laboratory studies have indicated that Darcy's law gives a satisfactory approximation of ground-water flow and that for many field situations water storage in the aquifer is proportional only to head. For these reasons and for a specific set of boundary conditions, it is feasible to predict the head distribution in an aquifer with respect to space and time.

A hydraulic stress may be applied by controlling either the flow or head along the well bore. Movement of water to or from the well affects head distribution in the aquifer; a complete specification of the hydraulic stress must include the well radius and the depth intervals at which the well is open to the aquifer. Head in the aquifer is related to the three-dimensional distribution of hydraulic conductivity and storage, which may change as a function of space and time.

To summarize, head distribution in the vicinity of a controlled well can be predicted if we know (1) the type (flow or head) and magnitude of control applied at the well, (2) the well radius, the depths at which the well bore is open to the aquifer, and the hydraulic conditions along the bore, (3) the three-dimensional distribution of hydraulic conductivity in the region affected by the well, and (4) the storage characteristics of the aquifer.

If any one of these four elements is unknown, accurate predictions of head distribution, or changes of head, are generally impossible.

Theis (1935) was the first to present a formula showing the nonsteady hydraulic response in an aquifer resulting from constant discharge from a well. Paralleling the requirements given in the preceding paragraph, Theis' assumptions may be stated as follows:

1. Discharge of the well is changed by the amount Q at an arbitrary reference time $t=0$.
2. The well is open to the aquifer throughout the aquifer thickness, the well radius is infinitesimal, and flow to the well per unit length open to the aquifer is uniform.
3. The aquifer is homogeneous and isotropic, is of uniform thickness, and remains filled with water. It extends to infinite radius from the pumped well and is bounded above and below by impermeable rocks.
4. Storage in the aquifer is proportional to head.

Assumption 3 is the basis for the concept of transmissivity—the capacity of the aquifer to transmit water along the principal plane of the aquifer. Transmissivity, T, is assumed to equal the average hydraulic conductivity times aquifer thickness. Assumption 4 is the basis for the concept of the storage coefficient, S—the quantity of water released from or taken into storage, per unit area of aquifer, when the head in the aquifer is changed by a unit length.

Theis (1935) equation showing the change in head, or drawdown, s, expected for the conditions assumed may be written as follows:

$$s=\frac{Q}{4\pi T}\int_{u}^{\infty}\frac{e^{-u}}{u}\,du=\frac{Q}{4\pi T}\,W(u), \qquad (1)$$

where $u=r^2S/4Tt$ (2)

and r is the distance from the center of the discharging well to the point at which s is observed. $W(u)$ is commonly called the "well function of u."

If we know from geologic and other information that the hydraulic conductivity, discharging well, and storage characteristics meet assumptions 1–4 at a particular site and if we observe s versus r^2/t for a known Q, only the values of T and S are unknown. T and S can be calculated by graphical manipulation of the data and equations 1 and 2, as described by Brown (1953).

Graphical analysis of drawdown is centered around a type curve—here a curve of u versus $W(u)$—plotted on log paper. The following is a brief description of the graphical method from Ferris, Knowles, Brown, and Stallman (1962, p. 94–98); only the equation and figure numbers and the constants have been changed to fit this text:

Rearranging equations 1 and 2 there follows:

$$s=\left[\frac{Q}{4\pi T}\right]W(u) \tag{3}$$

or

$$\log s=\left[\log\frac{Q}{4\pi T}\right]+\log W(u) \tag{3a}$$

and

$$\frac{r^2}{t}=\left[\frac{4T}{S}\right]u \tag{4}$$

or

$$\log\frac{r^2}{t}=\left[\log\frac{4T}{S}\right]+\log u. \tag{4a}$$

If the discharge, Q, is held constant, the bracketed parts of equations 3a and 4a are constant for a given pumping test, and $W(u)$ is related to u in the manner that s is related to r^2/t. This is shown graphically in figure 1. Therefore, if values of the drawdown s are plotted against r^2/t, or $1/t$ if only one observation well is used, on logarithmic tracing paper to the same scale as the type curve, the curve of observed data will be similar to the type curve. The data curve may then be superposed on the type curve, the coordinate axes of the two curves being held parallel, and translated to a position which represents the best fit of the field data to the type curve. An arbitrary point is selected anywhere on the overlapping [part] of the sheets and the coordinates of this common point * * * are recorded. It is often convenient to select a point whose coordinates [on the type curve graph] are both [either] 1 [or multiples of 10]. These data are then used with equations 3 and 4 to solve for T and S.

A type curve on logarithmic coordinate paper of $W(u)$ versus $1/u$, the reciprocal of the argument, could have been plotted. Values of the drawdown (or recovery), s, would then have been plotted versus t, or t/r^2, and superposed on the type curve in the manner outlined above. This method eliminates the necessity for computing $1/t$ values for the values of s.

Theis' type curve—u versus $W(u)$—is unique only because it pertains to a particular set of conditions at the pumped well and in the aquifer. The dimensionless group u is fundamental to all aquifer systems. Thus a general form of equation 1 may be written as follows:

$$s=\frac{Q}{CT}f\left(\frac{r^2S}{Tt},\alpha,\beta,\ldots\right), \tag{5}$$

where the constant C and the function f are dependent on the shape of the aquifer and on the distribution and nature of hydraulic coefficients of the rocks. α and β refer to other dimensionless groups required to define a particular aquifer, such as the ratio K_r/K_z indicating a form of anisotropy where K_r is hydraulic conductivity to radial flow and K_z is hydraulic conductivity to flow paralleling the axis of the well. Equation 5 may be written in terms of either s or Q, depending on which is controlled for testing purposes. Data analysis for all aquifer tests, using the appropriate type curve u versus $f(r^2S/Tt,\alpha,\beta\ldots)$ can be accomplished by the graphical method illustrated in figure 1. If s is controlled at the pumped well and Q is allowed to vary with time, the data curve of figure 1 will be s/Q versus $1/t$ at the pumped well, and at distance r from the pumped well, the data curve s/Q versus t or s/Q versus t/r^2 may be used for analysis.

Many descriptions of data analysis have stressed the use of special types of graph paper, intercepts, and asymptotic forms of each type curve. In the writer's experience, none of these techniques enhances the hydraulic data obtained. Rather, the analyst is confounded by the need to learn, remember, and apply the limiting conditions required of the data to permit use of each specialized approach. One must keep informed about the various type curves available and their limitations, as they each apply to individual site conditions. The luxury of learning specialized data treatment beyond the format of figure 1 can be extremely costly if obtained at the expense of understanding the various forms of equation 5 in the literature. It seems most efficient to rely entirely on the form of equation 5 as an algebraic model and the logarithmic curve-matching process illustrated in figure 1 for obtaining numerical values of hydraulic characteristics from aquifer-test data.

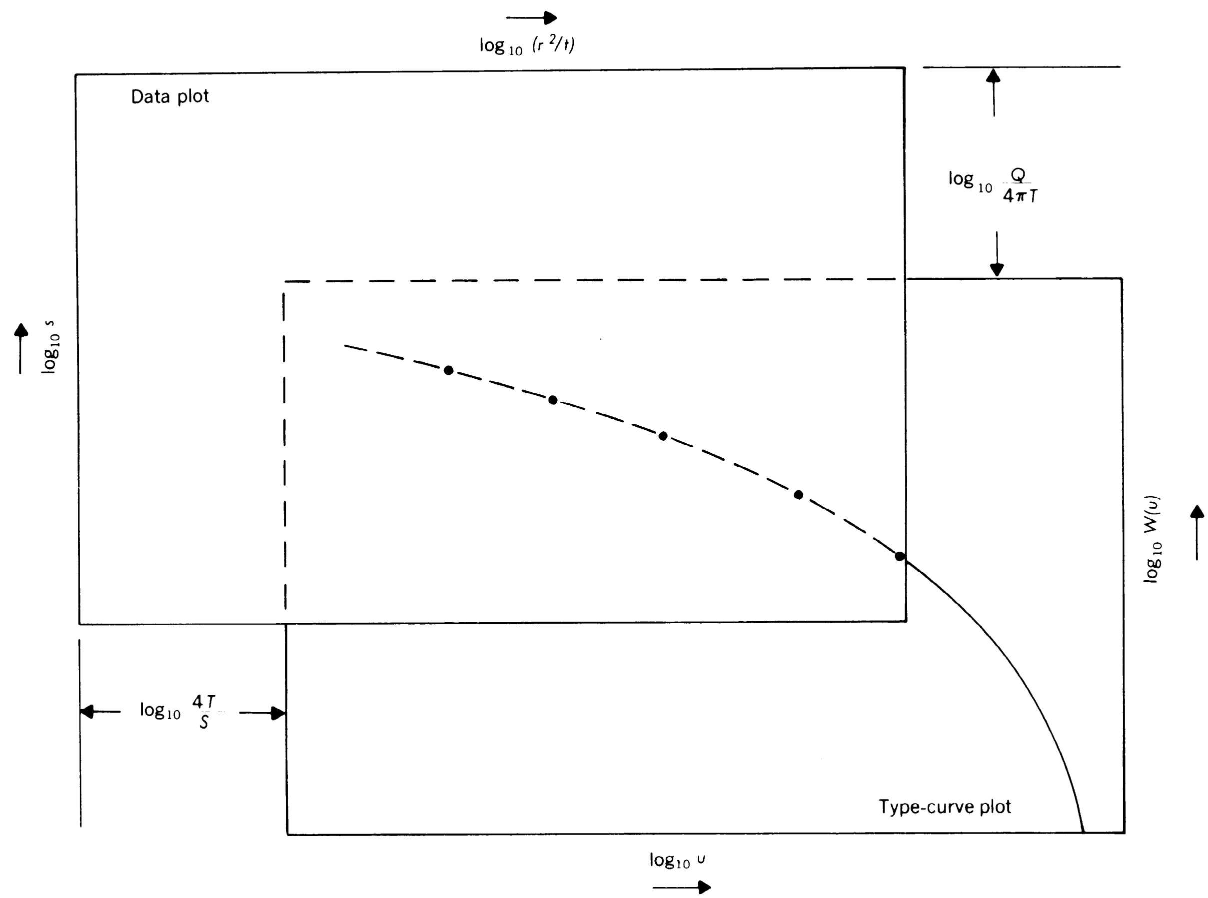

Figure 1.—Relation of W(u) and u to s and r^2/t and displacements of graph scales by amounts of constants shown. After Ferris, Knowles, Brown, and Stallman (1962, p. 98, fig. 24).

Type curves in the hydrologic literature

Early development of algebraic equations that describe aquifer response to discharge centered upon simplified models of the well and aquifer. As with all products, once the worth of aquifer tests became recognized by successful use in the field (through the efforts of pioneers such as C. V. Theis), the demand increased, the market was enlarged, and the product was improved by continuing refinement. It is interesting to chart the progress made with artesian situations, wherein the discharging well fully penetrates a homogeneous isotropic aquifer of infinite areal extent and the discharge is changed in amount Q at time $t=0$. Gunther Thiem is credited with first developing, in 1906, the response equation for steady radial flow (Ferris and others, 1962). The Thiem formula endured as the analytical equation for aquifer-test analysis for about 30 years. The Theis (1935) equation represents a highly significant improvement in the technology of ground-water flow because an effective mathematical description of nonsteady ground-water flow accounting for storage was introduced. Also, the Theis equation permitted making tests in much less time than required by the Thiem steady-state analysis. Prior to 1946, equations were based on the assumption that confining beds above and below the aquifer had zero hydraulic conductivity. Jacob (1946) changed that by introducing the steady-flow equation for a "leaky" aquifer, an aquifer in which one or both of the confining beds are significantly permeable. Hantush and Jacob (1955) advanced a solution for nonsteady flow in the leaky aquifer, assuming continuous steady flow in one of the contiguous confining beds. Hantush (1960) presented equations for nonsteady contributions from both the upper and lower confining beds of an artesian aquifer. However, the equations obtained fit only parts of the drawdown, and their applicability to a particular test must be investigated by comparison with time criteria.

The sequence of work cited in the previous paragraph spans 60 years and has resulted in stepwise improvement in test utility for artesian aquifers. An estimated 100 papers have been written to describe details of data analysis by the theory cited. However, the evolutionary aspects of the process largely rule out the present need for using any more than the latest paper (Hantush, 1960) on the subject for test analysis. This is true because each of the earlier papers describes a particular case of Hantush's (1960) solution. If time of discharge approaches infinity, Hantush's solution is identical with Jacob's (1946). If the confining beds are assumed to be impermeable, Hantush's solution and the Theis equation become one and the same. Thus, because of generality, Hantush's (1960) type curves supersede much earlier work and can be employed for analyzing a wide variety of artesian systems without a confusing array of special approaches.

Few other equations for aquifer-test analysis have the evolutionary character of Hantush's (1960). Recent work has emphasized unconfined flow and more realistic treatment of boundary conditions at the control well. More general solutions, each applicable to a wide range of test conditions, are needed and will probably be developed as theoretical studies gain scope.

The single most complete description of aquifer-test formulas now available was written by the master of radial-flow theory—Hantush (1964a).

The numerous conditions at the control well and in the aquifer which are commonly treated in analytical solutions are listed in table 1, which is indexed to examples of a few selected theoretical studies. Although not mathematically elegant, the list serves to emphasize that many physical conditions at test sites are important to aquifer response and that there is no single response equation capable of handling all test configurations. Categories A to D in table 1 parallel the requirements for fully specifying the test-site conditions as noted earlier in this paper. Category E has been added to show the availability of type curves in the literature and emphasis in each article. Adequate numerical values of type curves were not published in many of the original theoretical derivations. Useful sources of type-curve values are given in footnotes.

Study of table 1 reveals by induction that there is no justification, nor any need, for one to show favoritism toward a single type-curve formulation in aquifer-test analysis. Yet, such favoritism is frequent among hydrologists. A hidebound attitude toward selection and use of one type curve to the exclusion of all others may result in erroneous values of the hydraulic coefficients. Selection of the proper curve, or sets of curves, for data analysis can be made only if the characteristics of the control well and aquifer are known and are analogous to those assumed in derivation of an analytical equation. Should the literature fail to contain a type curve that fits site conditions, one may resort to modeling techniques as an alternate to mathematical analysis for finding the form of C and $f(r^2 s/Tt, \alpha, \beta \ldots)$ in equation 5 (Stallman, 1965).

Test design

The purpose of design is to improve the probability that a test will yield acceptably accurate values of the hydraulic coefficients.

Site evaluation

The cost of testing is frequently reduced by using combinations of production and abandoned wells rather than installing new wells. Few existing well configurations are suitable for test purposes, and most wells are ill equipped for observation. Evaluation of existing facilities in the area where tests are proposed to find those which are potentially usable is the first step in design. By recalling that an aquifer test involves applying a known stress to a known aquifer and observing the response, one may easily establish criteria for site evaluation, as follows:

Table 1.—Site conditions treated and subject emphasis in selected literature on pumping tests

[x, condition treated in this paper; o, artesian storage release assumed to be zero]

Category	Artesian aquifer			Horizontal plane anistropy				Partial penetration			Well characteristics						Vertical flow components, unconfined			Delayed yield, unconfined				Variable Q			Miscellaneous			
	Thiem (1906)	Theis (1935)	Hantush (1960)	Hantush (1966a)	Hantush (1966b)	Hantush and Thomas (1966)	Papadopulos (1967)	Hantush (1961a)	Hantush (1961d)	Weeks (1964)	Jacob (1947)	Rorabaugh (1953)	Cooper and others (1965)	Bredehoeft and others (1966)	Cooper and others (1967)	Papadopulos and Cooper (1967)	Boulton (1954a)	Stallman (1965)	Norris and Fidler (1966)	Boulton (1954b)	Boulton (1963)	Boulton (1964)	Prickett (1965)	Jacob and Lohman (1952)	Ferris and others (1962)	Hantush (1964b)	Hantush (1962)	Hantush and Papadopulos (1962)	Bixel and others (1963)	Boulton (1965)
A. Type of control imposed:																														
Step change Q	x	x	x	x	x	x	x	x	x	x	x	x	--	--	--	x	x	x	x	x	x	x	x	--	--	--	x	x	x	--
Step change s	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	x
Pulsed Q	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Pulsed s	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Variable Q	--	--	--	--	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	x	--	--	--	--
Variable s	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B. Control-well characteristics:																														
Full penetration	x	x	x	x	x	x	x	--	--	--	x	x	x	x	x	x	x	x	--	x	x	x	x	x	x	x	x	--	x	x
Partial penetration	--	--	--	--	x	--	--	x	x	x	--	--	--	--	--	--	--	x	x	--	--	--	--	--	--	--	--	x	--	--
Diameter infinitesimal	x	x	x	x	x	x	x	x	x	x	x	x	--	--	--	--	x	--	x	x	x	x	x	--	x	x	x	--	x	--
Diameter finite	--	--	--	--	--	--	--	--	--	--	--	--	x	x	x	x	--	x	--	--	--	--	--	--	--	x	x	--	--	x
Seepage face	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x
Well loss	--	--	--	--	--	--	--	--	--	--	x	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Radial screens	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--
C. Conductivity and flow conditions:																														
Homogeneous, isotropic	x	x	x	--	--	--	--	x	x	--	x	x	x	x	x	x	x	x	--	x	x	x	x	x	x	x	--	x	x	--
Homogeneous, anisotropic	--	--	--	x	x	x	x	--	--	x	--	--	--	--	--	--	--	x	x	--	--	--	--	--	--	--	--	--	--	x
Heterogeneous, isotropic	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--
Permeable confining beds (nonsteady)	--	--	x	x	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--
Sloping beds	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--
Areally infinite	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	--	x	x	x	x	--	x	x	x	x	--	x
Areally semi-infinite	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	x	x	--	--
Areally discontinuous	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--
Dewatering negligible	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	--	x	x	x	x	--	x	x	x	x	x	x
Dewatering significant	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	--	--	--	--	--
Flow radial	x	x	x	x	x	x	x	--	--	--	x	x	x	x	x	x	--	--	--	x	x	x	x	x	x	x	x	--	x	--
Flow radial and vertical	--	--	--	--	--	--	--	x	x	x	--	--	--	--	--	--	x	x	x	--	--	--	--	--	--	--	--	x	--	x
Nonsteady flow	--	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Steady flow	x	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
D. Storage relation:																														
Linear to head	--	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	--	--	--	--	x	x	x	x	x	x	--
Head and time	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	x	x	x	--	--	--	--	--	--	--
Artesian	--	x	x	x	x	x	x	x	x	--	x	x	x	x	x	x	o	o,x	o	o	o	--	--	x	x	x	x	x	x	o
Unconfined	--	--	--	--	--	--	--	--	--	x	--	--	--	--	--	--	x	x	x	x	x	x	x	--	--	--	--	x	--	x
E. Emphasis of paper:																														
Q versus time	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	x	x	x	--	--	--	x
s versus time and space	--	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	--	x	x	x	--	--	x	x	x	x	x	--
Analytical equation	x	x	x	x	x	x	x	x	--	--	x	x	x	x	x	x	x	--	--	x	x	x	--	--	--	x	x	x	x	x
Graphical type curve	--	--	x	--	--	--	--	x	--	--	--	--	x	x	x	x	x	x	--	--	x	--	--	x	--	x	--	--	x	x
Tables, type-curve	--	[1]	x[2,3]	--	--	--	--	x	--	--	--	--	--	--	x	x	x	--	--	--	x	x	--	--	--	x	--	--	--	x
Analog computer	--	--	--	--	--	--	--	--	--	--	--	--	--	x	--	x	--	x	--	--	--	--	--	--	--	--	--	--	--	--
Theory development	x	x	x	x	x	x	x	x	--	--	x	x	x	x	x	x	x	x	--	x	x	x	--	x	x	x	x	x	x	x
Application of data	--	--	--	x	--	x	x	--	x	x	x	x	--	--	x	--	--	--	x	x	x	x	x	--	--	x	--	--	x	--

[1] Ferris and others (1962).
[2] Hantush (1961b).
[3] Hantush (1961c).

Control well:

1. If it is to be pumped, the control well must be equipped with reliable power, pump, and discharge-control equipment.
2. The water discharged must be conducted away from the control well so it cannot return to the aquifer during the test. This point is of special importance in testing shallow unconfined aquifers.
3. The wellhead and discharge lines should be accessible for installing discharge regulating and monitoring equipment.
4. It should be possible to measure depth to water in the control well before, during, and after pumping.
5. The diameter, depth, and position of all intervals open to the aquifer in the control well should be known, as should total depth.

Observation wells:

1. Response of all wells to changing water stages should be tested by injecting a known volume of water into each well and measuring the subsequent decline of water level. The initial rise of water should be dissipated within a few minutes (to within about 0.01 of the initial rise) if the observation well is to reflect changes of head in the aquifer during the test satisfactorily. Long abandoned wells tend to become clogged, and consequently the response test is one of the most important prepumping examinations to be made if such wells are to be used for observation.
2. Total depth, diameter, and screened interval should be known for each observation well.
3. Radial distance from the control well to each of the observation wells must be determined.

Aquifer:

1. Depth to, thickness of, and areal limits of the aquifer to be tested should be known.
2. Nearby aquifer discontinuities caused by changes in lithology or by incised streams and lakes should be mapped.
3. Estimates of all pertinent hydraulic properties of the aquifer and adjacent rocks must be made by any means feasible. Estimates of transmissivity and storage coefficient should be made. Also, if leaky confining beds are suspected, leakage coefficients should be estimated. For unconfined aquifers, conductivity to vertical flow is important. In the absence of any data, assume $T=100{,}000$ gpd per ft (gallons per day per foot) $=13{,}000$ ft^2 per day; artesian storage coefficient $=1\times10^{-6}\times$ aquifer thickness, in feet; specific yield $=$ 0.20 in unconfined systems; conductivity to vertical flow $=1.3$ ft per day.

The importance of making the pretest site evaluation cannot be overstressed. To ignore such an evaluation is to invite failure of the test. Although in this report application of site evaluation to existing well configurations is emphasized, all elements apply equally if wells are to be specially installed for testing the aquifer.

Response prediction

The pretest evaluation serves two general purposes. It describes the aquifer and control well in sufficient detail so the appropriate type curve to be used for data analysis is evident (provided of course that one analogous to the site conditions exist). Also, it provides the basis for predicting the outcome of the test with available facilities and pinpoints deficiencies in observation-well locations. If site conditions deviate markedly from all known type-curve formulations, the site should be abandoned for testing purposes unless the cost of a special-purpose type-curve development is considered acceptable.

If a suitable type curve is available, estimates of the hydraulic properties are entered into the response equation, together with the magnitude and type of control, to find the expected head or drawdown distribution at the available observation wells. Test data, to be most effective, must sample a variety of positions on the type curve(s). If the observation wells are not located to accomplish such a sampling, calculated results will be in doubt. As an example, consider the nonleaky artesian

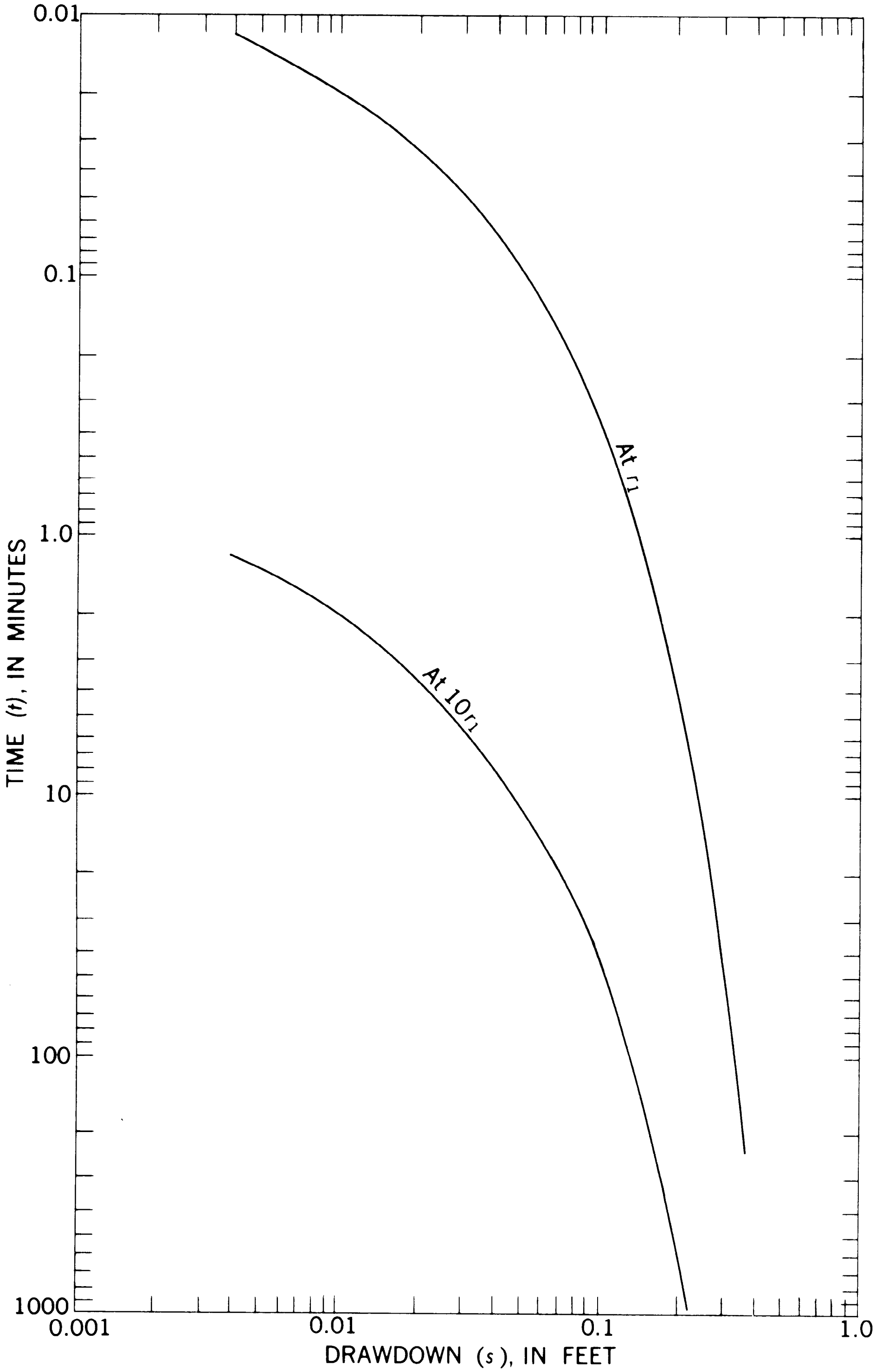

Figure 2.—Predicted response to pumping in observation wells.

model of Theis. Figure 2 shows schematically the drawdown versus time expected in a particular observation well located at distance r_1 from a control well. It is practically impossible to obtain meaningful measurements of drawdown during the first minute of the test. Thus only the flat part of the right-hand curve can be defined by measurements in the available observation well. Study of the matching process in figure 1 shows that such data will produce accurate measurement of transmissivity because the data curve can be located relatively accurately with respect to $W(u)$, the ordinate, disregarding the misfit along the abscissa. Finding the match point for t and u is another matter, however. For the estimates shown in figure 2, it is unlikely the storage coefficient could be obtained within an order of magnitude. If the pumping test is to provide accurate values of both T and S, drawdowns must be observed over both the steep and flat parts of the type curve.

If only one observation well is to be drilled for testing, where should it be located? From the Theis type curve, we find the time scale is proportional to r^2. To produce a detectable drawdown of 0.01 foot 2 minutes after pumping starts, the time scale should be shifted about two orders of magnitude to the right in figure 2. Thus, an observation well at a distance of about $10r_1$ from the control well would provide definitive data for both T and S, assuming that our evaluation of the test site is correct. Furthermore, test observations made over a 24-hour period will obviously afford all the curve definition needed to determine T and S accurately.

Where several wells are available, predicted response should be plotted as t/r^2 or r^2/t for all wells on one sheet of graph paper. The total of all information from the test site should afford complete definition of curve shape. It is not required that each well provide data that fits both the steep and flat parts of the type curve. Data from one or more of the wells having small r may plot only in the flat, while others may plot only in the steep part. The objective of design is to assure adequate response measurements over the whole of the test facility—not piecemeal.

Where site conditions are more complicated, such as those including partial penetration or unconfined flow, the prediction of observation-well response is of course less certain. However, for all proposed tests such a prediction should be made without fail; by using the theoretical response curves that are based on conditions comparable to those at the test site, one can guard against major deficiencies in the configuration of observation wells. Many test configurations are rescued from failure by the simple expedient of drilling only one observation well at a key point in the system.

Artesian or confined systems are generally more amenable to testing than unconfined systems. Artesian flow to wells is described by the simpler set of boundary conditions. In unconfined systems, mobility of the upper boundary, vertical flow components, and nonlinear release of water from storage are difficult to treat but have been successfully attacked recently. However, all these complicating features of unconfined flow affect the early response simultaneously. Present state of the art does not provide a means for quantitatively recognizing the magnitude of response due to each factor individually.

The most significant theoretical advances in testing unconfined systems have been made by N. S. Boulton, of the University of Sheffield, Sheffield, England. Boulton (1954a) presented type curves for nonsteady flow, assuming negligible dewatering of the aquifer, linear release from storage, and termination of flow lines on the water table. Extensive values of the type curves for water-table response from Boulton's (1954a) study were given by Stallman (1962), and curves applicable to response at depth were developed from Boulton's (1954a) model by electric analog (Stallman, 1965) and by digital computer (Dagan, 1967). Delayed yield from storage, expressed as an exponential function of time, was analyzed by Boulton (1954b, 1963).

Before analytical work described the effects of vertical flow and delayed yield in unconfined flow to wells, it was common practice to pump "long enough" that such effects became negligible and response approached that of the simple artesian model. However, no criteria existed for judging how long is "long enough."

With the analytical solutions now available, some criteria exist for judging the length of time required for effectively attaining an artesian response in an unconfined system.

According to Boulton (1954a) and Hantush (1964a, p. 366), vertical flow components in unconfined aquifers significantly affect response when

$$t<5bS/K_z, \text{ in the region } 0<r/b<0.2, \quad (6)$$

where b is the aquifer thickness and S is specific yield. Equation 6 is derived analytically and it is assumed that nonsteady radial and vertical flow components exist in the vicinity of a fully penetrating well, specific yield is constant in time and space, and drawdown is negligible compared with aquifer thickness. Equation 6 produces a rather startling revelation of the pumping time required for approaching artesian-type flow. For $b=100$ feet, $S=0.2$, and $K_z=10$ feet per day, we find response is affected by vertical flow for as long as $t=5\times100\times0.2/10=10$ days near the pumped well.

Electric-analog studies (Stallman, 1965) showed vertical flow components to be significant for

$$t<r^2S/T, \text{ or } t<9bS/K_z, \text{ in the region } (r/b)(K_z/K_r)^{1/2}<3, \quad (7)$$

where K_r and K_z are hydraulic conductivity to radial and vertical flow, respectively.

Criteria such as equations 6 and 7 are not now available from type curves for delayed yield. However, a dimensionless plot of column drainage (Stallman, 1967) has shown that about 70 percent of the ultimate drainage due to lowering the water table will be attained at

$$t=10Ss/K_z. \quad (8)$$

Thus, for $K_z=10$ feet per day, $s=10$ feet, and $S=0.2$, delayed yield will be pronounced for at least $t=10\times0.2\times10/10=2$ days.

If the pumping time in a test of an unconfined aquifer can be extended long enough to surpass the time requirements evident from equations 6–8, equations of radial artesian flow can be employed for data analysis, provided the control well fully penetrates the aquifer. If the control well partially penetrates the aquifer, equations accounting for partial penetration (Hantush, 1961a, d; 1964a, p. 355; Stallman, 1965) must be employed, or one is restricted further to using data in the region

$$r>1.5b(K_r/K_z)^{1/2}. \quad (9)$$

Use of type curves for predicting response at a test site and the liberal use of criteria like equations 6–9 for design purposes are necessary to reduce the prospect of failure to achieve the test objective, the accurate measurement of the hydraulic properties of the aquifer.

Field observation

The records required for analysis and the tolerance in measurement generally considered acceptable are as follows:

1. Control-well discharge (±10 percent).
2. Depth to water in wells below measuring point (±0.01 ft).
3. Distance from control well to each observation well (±0.5 percent).
4. Synchronous time (±1 percent of time since control effected).
5. Description of measuring points.
6. Elevations of measuring points (±0.01 ft).
7. Vertical distance between measuring point and land surface (±0.1 ft).
8. Total depths of all wells (±1 percent).
9. Depth and length of screened intervals of all wells (±1 percent).
10. Diameter, casing type, screen type, and method of construction of all wells (nominal).
11. Location of all wells in plan, relative to land-survey net or by latitude and longitude (accuracy dependent on individual need).

Items 1–4 are data collected specifically as part of the testing process. Many items are recorded on well-schedule and water-level forms by personnel taking part in the tests, and these forms become part of the permanent records on water resources maintained by the U.S. Geological Survey. Examples of two of the various types of forms in use are shown in figure 3. The well schedule (fig. 3*A*) is designed to make use of automatic data-storage and retrieval techniques. For recording water-level data from aquifer tests, the form shown in figure 3*B* is used. Mud, rain, bitter cold, intense

WRD Exp. (GW)
Aug. 1964

U. S. DEPARTMENT OF THE INTERIOR GEOLOGICAL SURVEY

Water Resources Division Well Schedule Form

MASTER CARD

Record by ______ Source of data ______ Date ______ Map ______

State ______ [1 2] County (or town) ______ [3 4]

Latitude: [5 deg 7 min 9 sec 11] N S Longitude: [12 degrees 15 min sec 18] Sequential number: [19]

Lat-long accuracy: [20] T ______ N S, R ______ E W, Sec ______, ______ ¼, ______ ¼, ______ ¼ ______ B & M

Local well number: [21 25 30 34] Other number: ______

Local use: [35 40 45 51] Owner or name: ______

Owner or name: [52 56 61 66] Address: ______

Ownership: (C) County, (F) Fed Gov't, (M) City, (N) Corp or Co, (P) Private, (S) State Agency, (W) Water Dist ______ [67]

Use of water: (A) Air cond, (C) Comm, (D) Dewatering, (F) Fire, (H) Dom, (I) Irr, (N) Ind, (P) P S, (S) Stock, (T) Instit, (U) Unused ______ [68]

Use of well: (A) Anode, (D) Drain, (G) Seismic, (O) Obs, (P) Oil-gas, (R) Recharge, (S) Spring, (T) Test, (U) Unused, (W) Withdraw, (X) Waste, (Z) Destroyed [69]

DATA AVAILABLE: Well data [70] Freq. W/L meas.: ______ [71] Field aquifer char. [72]

Hyd. lab. data: ______ [73]

Qual. water data; type: ______ [74]

Freq. sampling: ______ [75] Pumpage inventory: yes no, period: ______ [76]

Aperture cards: ______ yes [77]

Log data: ______ [78 79]

WELL-DESCRIPTION CARD

SAME AS ON MASTER CARD [1 19] Depth well: ______ ft [20 23] Meas. rept ______ accuracy [24]

Depth cased: (first perf.) ______ ft [25 28] Casing type: ______; Diam. ______ in [29 30]

Finish: (C) porous concrete, (F) gravel w. (perf.), (G) gravel w. (screen), (H) horiz. gallery, (O) open end, (P) perf., (S) screen, (T) sd. pt., (W) shored, (X) open hole, (Z) ______ other [31]

Method Drilled: (A) air rot, (B) bored, (C) cable, (D) dug, (H) hyd rot., (J) jetted, (P) air percussion, (R) reverse rotary, (T) trenching, (V) driven, (W) drive wash, (Z) ______ other [32]

Date Drilled: ______ [33 35] Pump intake setting: ______ ft [36 38]

Driller: ______ name, ______ address

Lift (type): (A) air, (B) bucket, (C) cent, (J) jet, (L) multiple, (cent.) (M) multiple, (turb.) (N) none, (P) piston, (R) rot, (S) submerg, (T) turb, (Z) other [39] Deep Shallow [40]

Power (type): diesel, elec, nat gas, gasoline, hand, LPG gas, wind; H.P. ______ [41] Trans. or meter no. ______

Descrip. MP ______ ft above below lsd, Alt. MP ______

Alt. LSD: ______ [42 45] Accuracy: (source) ______ [47]

Water Level ______ ft above below MP; Ft above below lsd [48 51] Accuracy: ______ [52]

Date meas: ______ [53 55] Yield: ______ gpm [56 60] Method determined [61]

Drawdown: ______ ft [62 64] Accuracy: ______ [65] Pumping period ______ hrs [66 68]

QUALITY OF WATER DATA: Iron ______ ppm [69] Sulfate ______ ppm [70] Chloride ______ ppm [71] Hard. ______ ppm [72]

Sp. Conduct ______ K x 10^6 [73] Temp. ______ °F [74 76] Date sampled ______ [77 79]

Taste, color, etc. ______

Figure 3.—Sample forms for recording data.

County :__________ Observation well no.__________

Location:__________ Pumped well no.__________

Average Q__________ gpm r =__________ft. r^2 =__________

Date	Hour	t (min)	t′ (min)	t/t′	Depth to water	s (unadjusted)	Adjustment Δs	s′ (adjusted)	Q (gpm)	Remarks

Figure 3.—Continued.

heat, insects, barbed wire, balky engines, and a host of other environmental distractions associated with aquifer tests compete for the observer's attention against the mudane measurements. Simple but complete forms for all data recording, prepared in a noncompetitive environment, serve as convenient reminders to the observer so he will not inadvertently omit a key measurement while in the field, provided, of course, that the observer takes the forms with him.

Lithology and the construction features of the control and observation wells are generally determined by interviewing the well owners and (or) the well drillers. Field measurements of well depths and casing diameters afford a qualitative check on the accuracy of information obtained by interview. Detailed lithologic logs and construction features should be noted during drilling of all wells installed expressly for testing purposes. It is commonly advisable to make geophysical logs in the test wells, such as resistivity, self-potential, gamma, and neutron logs, to provide a rather complete understanding of subsurface conditions. Accurate location in plan of observation wells, referenced to the position of the control well, is especially important in a test of a heterogeneous or anisotropic aquifer. Position of the test site with respect to a regional land-survey net or by latitude and longitude is also required so that the data obtained can be effectively used to portray the regional characteristics of the aquifer.

Water levels

Depths to water in observation wells are commonly measured with a steel tape. The lower few feet of the tape is coated with blue carpenter's chalk and lowered a foot or so below the water level in the well. A convenient even-foot marker on the tape is held opposite the measuring point, at or near the top of the casing, for a fraction of a second. The tape is reeled out of the hole, and the length wetted is noted. (The chalk coating provides a distinctive contrast between the wet and dry parts of the tape.) The wetted length is subtracted from the value of the even-foot marker held on the measuring point to find the depth to water. Float-actuated mechanical recorders, electrical pressure transducers with electrical recorders, and tapes having an electrically operated device for signaling contact with the water surface in the observation well are especially useful under some circumstances. The electrical tape and pressure transducer are most useful if the water level is hundreds of feet below the land surface, where perched water enters and wets the side of the well, and in crooked holes or obstructed holes difficult to enter rapidly. Float-operated recorders are convenient if the test is to be of long duration and it is not practical to have the observer at the site throughout the test.

Many pumps are so tightly sealed to the casing that access to the water level in the control well is not possible. In some, the pump occupies such a large part of the casing cross section that it is virtually impossible to lower a tape or any other equipment to the water surface. Although many methods for measuring water levels are used in control wells, the writer prefers the air line. A small-diameter tube is lowered to a known depth below the deepest pumping level expected. Pressure in the tube is either recorded or indicated on a pressure gage attached to the upper end of the line, while an excess of gas is slowly and continuously bubbled out the bottom of the line. The pressure recorded is equivalent to the height of water column above the base of the air line. Fortunately, many well owners install air lines at the time the pump is placed.

Very narrow tapes that can enter the well through small-diameter air lines are available for measuring depth to water as an alternative to the air-line pressure measurement. Usually weights are attached to the lower end of the tape so that the tape is stretched straight and is easily lowered down the well. The weights displace water and can produce erratic measurements of depth to water if they are of large volume. If practical, the tape should be used without weights. Of course, if the tape has been severely kinked, twisted, or otherwise distorted, it should be discarded. All tapes should be usable interchangeably in all wells at a test site without the need for correcting measured depths to water for tape calibration. The cost of such corrections generally exceeds the cost of new equipment.

Pressure gages on air lines become inoperable owing to physical damage. Therefore it is good practice to install a reliable gage, one that has recently been calibrated, prior to the test.

The measuring point at each observation well is a reference point, and as such, should be stable. More than one observer is generally employed on pumping tests, and unless measuring points are clearly marked on the casing or other firm structure, each observer is likely to find his own. A sharp edge formed by the intersections of a vertical and horizontal plane, fixed firmly in place, should be used for the measuring point at each well. The edge should be sharp enough so that a tape marker can be held on the edge with no more than 0.003-foot uncertainty in marker elevation. A paint mark should be used to indicate the position for the tape marker so that successive observers can make measurements at the same place. Jagged casings that have been cut by torch and worn edges of a concrete pump base are definitely not suitable references. Inadequate attention to proper construction of measuring points causes loss of accuracy in computed head or drawdown, which leads to erratic response curves.

Should it be necessary to change the measuring point during the test, care should be taken to note the time when the change was made and the altitude of the new point and to strike the old and paint the new location. The importance of making detailed remarks about measuring points cannot be overstressed because the writer has observed that lack of attention to such detail sometimes tips the scale from success to failure when the data are analyzed.

Most type curves relate control to change in head, or drawdown. The changes in depth to water observed during the test may include components due to other variables such as recharge or barometric response. Flow in most aquifers is not steady. It is therefore necessary to observe depths to water for a time before testing to determine the trend of the water level for use in determining drawdowns. A graphical definition of drawdown is illustrated in figure 4. Accurate drawdowns can be observed only if water-level trends are accurately forecast, or the response due to testing is large compared with other effects. As a general rule the period of observation before $t=0$ should be at least twice the length of the pumping test.

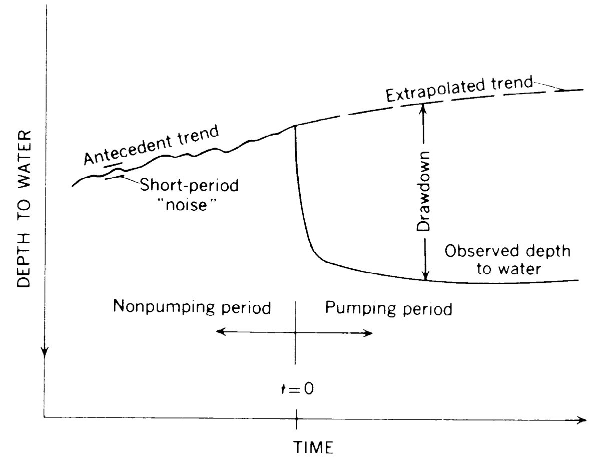

Figure 4.—Hydrograph for hypothetical observation well showing definition of drawdown.

In many artesian wells, water levels fluctuate in response to changes of barometric pressure. Such fluctuations may be as large as a foot or more in a few days (Boettcher, 1964, p. N15). If response to pumping is relatively small, test observations should be corrected to remove the barometric effects before comparison with type curves. At all test sites for artesian aquifers, continuous recordings of barometric pressure (to a sensitivity of ± 0.01 inch of mercury) should be made throughout the trend-identification and testing periods. Graphs showing the correlation between the magnitudes of short-term fluctuations in barometric pressure and depth to water observed before the test can be used to correct the water depths measured during the test. Such fluctuations may be seen in the depths to water used for constructing the trend line shown in figure 4. Conditions at very few test sites are so stable that the hydrologist need not be concerned about antecedent trends. Therefore, he must always have sufficient background information on water-level fluctuations at the test site to evaluate accuracy of the drawdowns inferred from measurements of depth to water.

Water levels in wells are also affected by other "noise," such as the operation of nearby wells, recharge, earth and ocean tides, and loading of the aquifer by trains and earthquakes. Measurements made before the test will identify the extent to which such effects control the depth to water at the test site. Significant effects due to nearby wells can ordinarily be removed from the data with accuracy if the on-off times of the wells are monitored before and during the test.

Depth to water in all wells should be measured with sufficient frequency that each logarithmic cycle in time on the data plots contains at least 10 data points spread through the cycle. Thus, after $t=0$, depth to water should be measured in each well, as possible, at $t=1$, 1.2, 1.5, 2, 2.5, 3, 4, 5, 6, 7, and 8 minutes, approximately and at all succeeding decimal multiples of these numbers to the end of the test. If the test design indicates that measurable drawdown is not expected at a given observation well for several hours after the test starts, the early measurements may of course not be necessary.

During the first 2–3 hours after control is initiated, ideally an observer should be stationed at the control well and each nearby observation well. After $t=300$ minutes, measurements are nearly 2 hours apart, and it becomes a relatively simple task for one observer to "make the rounds." It is not necessary to measure all wells simultaneously, but it is highly desirable to achieve nearly uniform separation of the plotted drawdowns on a logarithmic scale. All watches used should be synchronized before the test is started, and provision should be made to notify all observers at the instant control is initiated.

Discharge

Discharge from the control well ordinarily is measured by passing the flow through a shaped restriction for which the relation between discharge and head loss is known. A few weirs and flumes are described in a U.S. Bureau of Reclamation (1967) manual that contains calibration tables. Other weirs and flumes are described in standard hydraulics handbooks, such as by Chow (1964); also, see Buchanan and Somers (1969) for information on the use of weirs, flumes, floats, and volumetric tanks for measuring discharge. The relation between head loss and free discharge for thin circular orifices is described by Jorgensen (1969). Small discharge rates, say not more than 50 gpm (gallons per minute), can be measured conveniently by noting the time needed to fill a container of known volume. Discharge rates in excess of 1,000 gpm, if carried from the pump in an open channel, are sometimes measured by stream-gaging methods or by weirs temporarily installed in the channel.

For those tests in which discharge is to be held constant throughout the test, it is important to observe the discharge periodically and adjust as needed. Pumps powered by electric motors produce the most constant discharge. Engines, even though equipped with automatic speed control, produce discharge varying as much as 25–50 percent between day and night. A continuous vigil on most engine-powered pumps must be maintained if reasonably constant discharge is to be realized, unless automatic control of discharge is effected by a costly electronic discharge monitor coupled with an electrically driven valve near the wellhead.

If the test requires a step change in discharge, a valve in the discharge line is partly closed before starting the pump. Ideally, discharge (pretest $Q=0$) is measured for the first time within a minute or two after the pump is started. As the water level in the pumped well declines, discharge tends to decrease. Discharge is maintained constant by progressively opening the valve, a procedure tending to keep the total head against the pump constant.

How frequently the discharge needs to be measured and adjusted for a test depends on the pump, well, aquifer, and power characteristics. Output from electrically driven equipment nominally requires measurement, and possibly adjustment, at 5, 10, 20, 30, 60, 120, 240, 480, 480, 720, and 1,440 minutes after the pump is started and daily thereafter. All other pumping equipment requires more frequent attention. No rules can be set because there is a wide variation in equipment response. When there is doubt about control of discharge, the observer should continuously monitor the test until experience indicates drift rates; then measurement frequency can be established accordingly. The discharge rate should never be allowed to vary more than ± 10 percent, if practicable, because such variations produce aberrations in drawdowns that are difficult to treat in data analysis.

Tests can be made on shut-in artesian wells by suddenly opening the well to flow and thereby creating a nearly constant drawdown. Such tests require measurements of discharge at about the same frequency as indicated for drawdown, about 10 measurements per logarithmic cycle in time. The variations in discharge with time can be analyzed by using the type curve of Jacob and Lohman (1952).

Data Analysis

Drawdown corrections

Data analysis involves chiefly the transformation of raw field data into calculated values of hydraulic coefficients. The first step is to plot observed depths to water and discharge as a function of time for each observation point. Notes are made on the resulting hydrographs to indicate sources of aberrations in depth to water, and those which might be adjusted are identified. Pretest observations are compared with records of barometric pressure to find whether corrections are needed. The observations are also examined to determine whether tide-induced or other interference is likely to inject components that affect depths to water, have relatively short duration, and are not related to the test control. The effects of all extraneous factors should be removed from all the data by applicable correlation techniques. Hydrographs are then prepared showing corrected depth to water versus time. Extrapolation of the corrected pretest trend line is then used to determine what depth to water would have occurred at the observation well if there had been no test control (fig. 4). For each time of measurement after control is initiated, drawdown is obtained by subtracting the trend value of depth to water from the corrected observed depth to water.

Few type curves account for dewatering of unconfined aquifers. Jacob (1963) showed that if drawdowns observed in thin unconfined aquifers are adjusted by subtracting $s^2/2b$, equations based on assumed negligible dewatering and radial flow can be used for analysis. Where the dewatering adjustment is significant, the data plot used is $s - (s^2/2b)$ versus t or t/r^2 rather than s versus t. The apparent storage coefficient, S', is computed by matching the graph of $s - (s^2/2b)$ and type curves of artesian flow. The value of S is determined by adjusting S' as follows:

$$S=(b-s)S'/b, \tag{10}$$

where s is the approximate drawdown at the geometric mean radius of all observation wells at the end of pumping. Jacob's corrections for dewatering apply only if flow is predominantly radial. Therefore they should not be relied on to produce exact adjustment where vertical flow components are dominant, such as near a partially penetrating pumping well or at early times in unconfined systems.

Application of type curves

After correction of the raw data to eliminate or reduce the amount of extraneous interference, and to adjust for dewatering, the resulting drawdowns are plotted on log paper versus t or t/r^2. If the design and observation phases of the test have been conducted successfully, the adjusted data curve(s) is fitted to the type curve already selected and the hydraulic coefficients are calculated by the matching process shown in figure 1.

One of the minor benefits the hydrologist derives from the design and observation phases of aquifer tests is a strong sense of complacency. After all that care, how is it possible the results might be in error? Large errors in interpretation are commonplace where there is uncertainty about the characteristics of the aquifer. Note the type curve (Hantush, 1960) for early times (toward right side of graph) in figure 5. The curve $\beta=0$ is for nonleaky artesian aquifers and is identical with the Theis type curve.

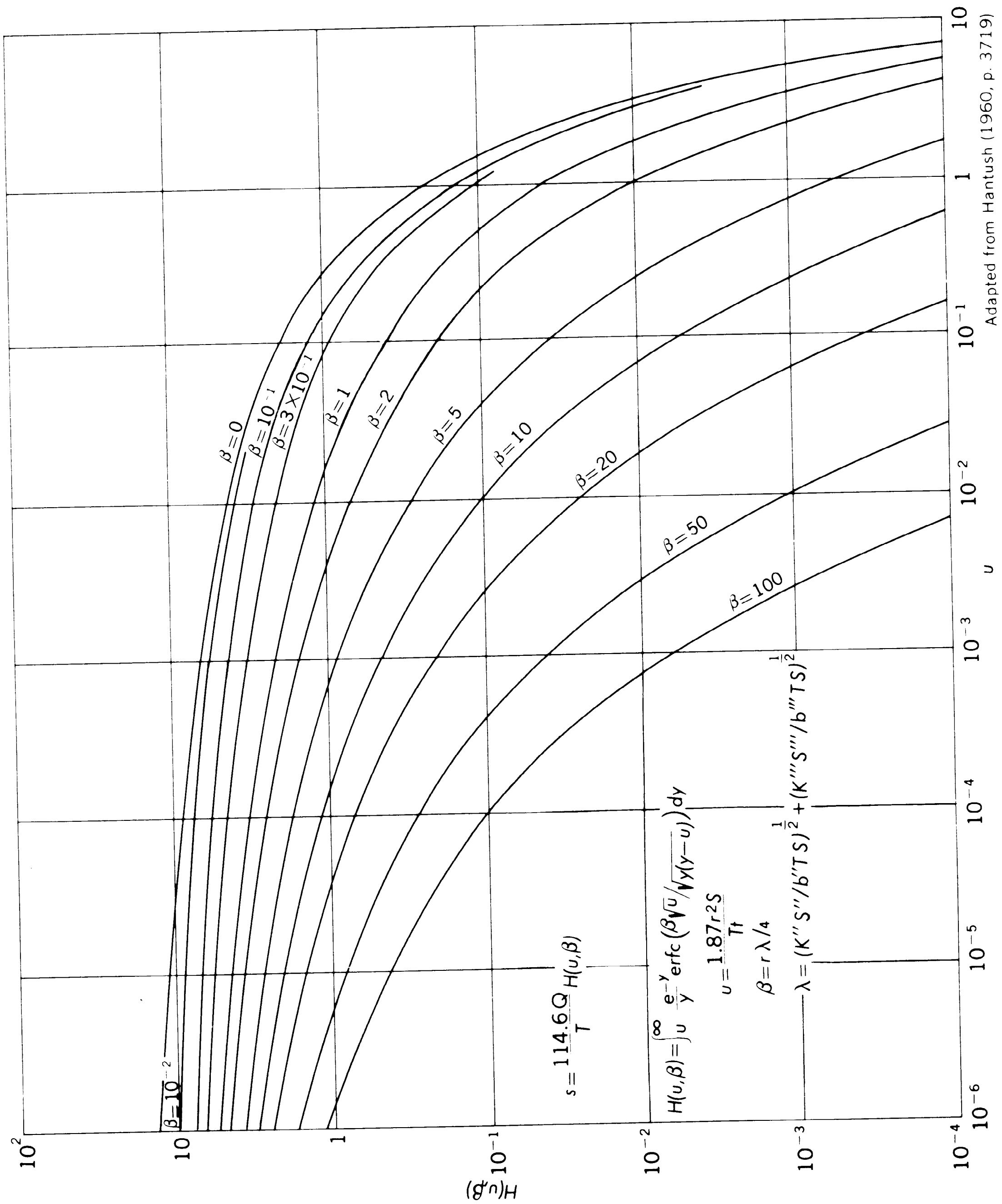

Figure 5.—Logarithmic graph of the function H (u, β).

Increasing β is associated with increasing contribution to flow from permeable confining beds. It is easily seen that if the Theis type curve is applied to data without discrimination, errors in T may easily be as much as an order of magnitude, while the computed value of S can be in greater error if the confining beds are permeable.

An example is shown in figure 6. The data plot on the left shows two values of T and S, obtained by matching early and late measurements to the Theis type curve. Before Hantush's (1960) type curves were available, it was common to emphasize results from either early or late data, depending on the individual hydrologist's rationale. On the one hand, early data seem better because a smaller area is affected, and therefore heterogeneity and boundaries of other types are not reflected in the drawdowns. Also, extrapolation of antecedent conditions is most accurate during the first part of the test. Alternatively, later data seem to yield more accurate results because time lag in response of observation wells is then negligible, pumping rate is stabilized (not a problem in recovery tests), and drawdowns are large compared with interference effects. The correspondence between data and the leaky-aquifer type curve indicated in the right-hand side of figure 6 offers a rather improved sense of complacency. But improved accuracy in result is not indicated unless there is available supporting information showing that the leakage inferred by the β value is realistic.

Unconfined aquifers

Tests of unconfined aquifers, involving a more complex set of boundary conditions, offer greater prospect for complacency than do tests on artesian systems. To illustrate, we will consider a set of data from a pumping test made on an unconfined aquifer near Lamar, Colo. Boulton's (1954a) model considers vertical flow components to be significant in controlling drawdown if storage releases are related only to drawdown. Boulton's (1954b) model considers delayed yield to be important and vertical flow components to be inconsequential. Neither model accounts for dewatering (table 1). Both models are here applied to data collected in April 1964 on the farm of Harry R. and Kenneth H. Nevius, T. 22 S., R. 45 E., sec. 30, east of Lamar, Colo., and about 1,500 feet north of the Arkansas River.

The control well fully penetrated a thin alluvial fill and had a total depth of 49 feet below the land surface; the casing diameter was 16 inches. Observation wells of 1¼-inch steel pipe were driven to selected depths at selected radii (fig. 7). The water table was about 11 feet below the land surface at the test site.

Depths to water were measured by tape, and discharge was disposed of in a ditch, where it was observed by stream-gaging methods. The control well was pumped at the rate of 2,000 gpm for about 3 days beginning at 0900 hours, April 14, 1964. The pump was powered by an electric motor, and discharge was constant within ± 5 percent throughout the test.

Graphs of s versus t/r^2 from wells 1, 3, 5, and 6 are shown in figure 8. Because observed drawdowns (9 ft) were large compared with saturated thickness (38 ft), field data were corrected for dewatering (Jacob, 1963). The data were matched to type curves (solid lines) for the middepth of the aquifer (Stallman, 1965, p. 305, fig. 10D). Note that data to the right in figure 8 form a flat slope, difficult to match with the Theis type curve for an accurate indication of S. Visual fitting of the type curves produces values of S lying between 0.15 and 0.25. Nuclear-meter measurements of specific yield (Stallman, 1967, fig. 11) show that $S=0.184$ at a nearby location. The match point was selected in figure 8 by trial and error so that S' would equal 0.22. This value of S' was determined by using equation 10 so as to adjust for dewatering, as follows:

$$S'=0.184\times 38/(38-6)=0.22$$

At the final match point, $Tt/r^2S=10$, $t/r^2=1.07 \times 10^{-1}\text{min/ft}^2$, $sT/Q=0.01$, and $s=0.13$ ft, from which $T=3\times 10^4$ ft$^2/d$, $S'=0.22$, and $S=0.185$.

Rather than attempt to determine K_z/K_r directly from figure 8, it is convenient to draw an auxiliary type curve and a recast data curve showing the relations sT/Q versus r/b from the data and sT/Q versus $(r/b)(K_z/K_r)^{1/2}$ on log scales. The matching process using the auxiliary type curve should provide an estimate of K_z/K_r. Values of sT/Q taken from the type

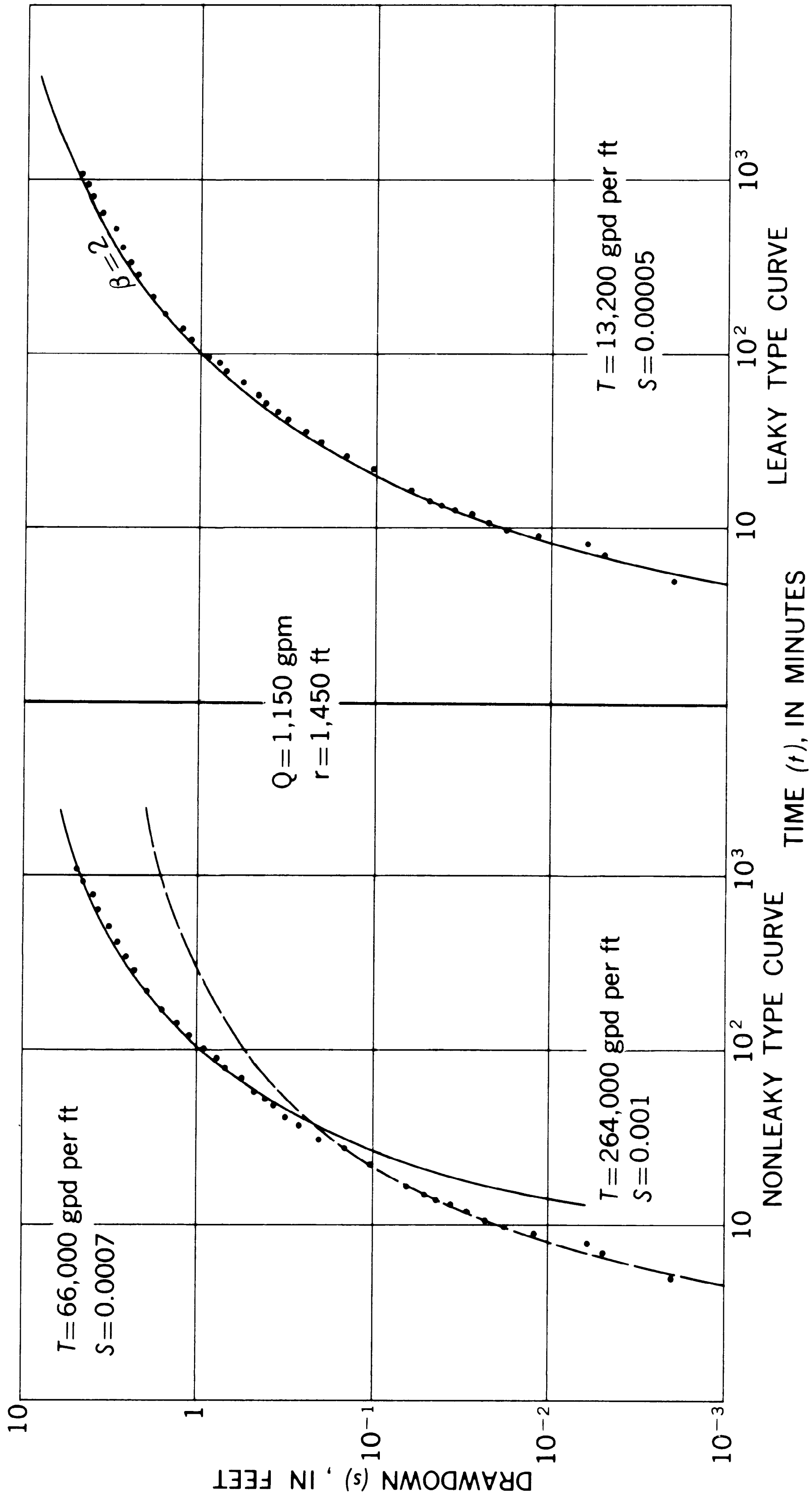

Figure 6.—Graph showing matching of recovery data to type curve for observation well, Pixley, Calif. Courtesy of F. S. Riley, U.S. Geol. Survey, Sacramento, Calif.

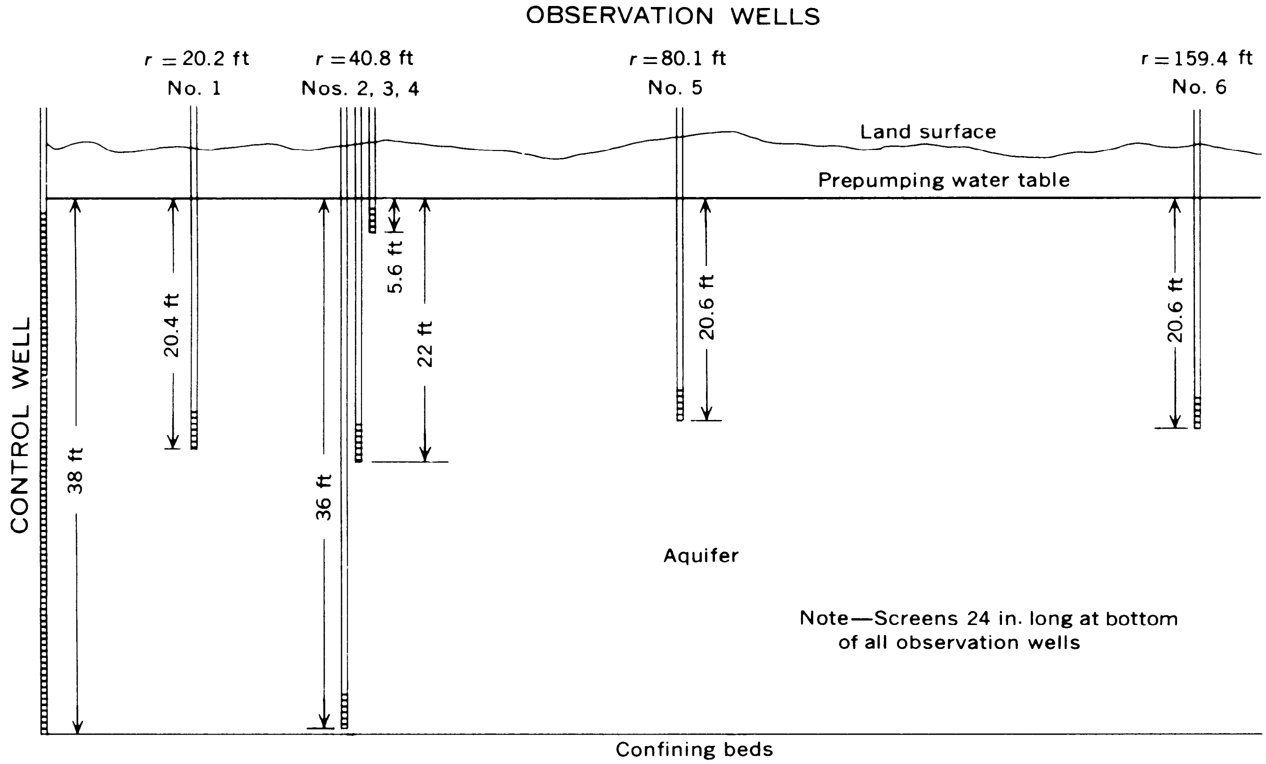

Figure 7.—Section through Nevius test site, near Lamar, Colo., showing radii to and depths of observation wells (not to scale).

curves (Stallman, 1965, p. 305, fig. 10D) where $Tt/r^2S=1.0$ are shown versus r/b in figure 9. Values of sT/Q at $Tt/r^2S=1.0$ are shown as a function of $(r/b)(K_z/K_r)^{1/2}$ in figure 10, marked as "vertical flow components." Match point A, where $r/b=1.0$ and $(r/b)(K_z/K_r)^{1/2}=0.08$, in figure 10 was obtained with the data of figure 9 by assuming vertical flow components are the signficant boundary condition. This indicates

$$(r/b)(K_z/K_r)^{1/2}=0.08=1.0(K_z/K_r)^{1/2},$$

and therefore

$$(K_z/K_r)=0.0064.$$

Note, however, the curve marked "Delayed yield" in figure 10. It is derived as an auxiliary type curve from Boulton (1963, p. 480) by assuming that there is radial flow only and that release from storage does not occur instantaneously with a lowering of the water table. Interestingly the two auxiliary type curves are nearly parallel over about two orders of magnitude in r/B. The data of figure 9 were also matched to the auxiliary type curve "Delayed yield" of figure 10, with the following results:

$$r/B=0.175 \text{ at } r/b=1.0;$$

thus,

$$r/0.175B=r/1.0b, \text{ and } B=5.72\times 38=217.$$

From Boulton (1963, p. 473, eq. 7),

$$Tt/SB^2=\alpha t, \tag{11}$$

where α is a delay index defined by Boulton (1963). Where αt is greater than 3, the effective specific yield is constant (Boulton, 1963, p. 477, fig. 3) at small values of r/B. If

$$Tt/SB^2>3 \text{ at } r/B<0.5, \tag{12}$$

effects on drawdown due to delayed yield are negligible. From equation 12, $t=1.2$ days for the Nevius pumping test site, and one expects the equations of artesian flow to apply after that time of pumping.

The results from figure 10 are completely anomalous. The assumption that specific yield is constant throughout the test yields the esti-

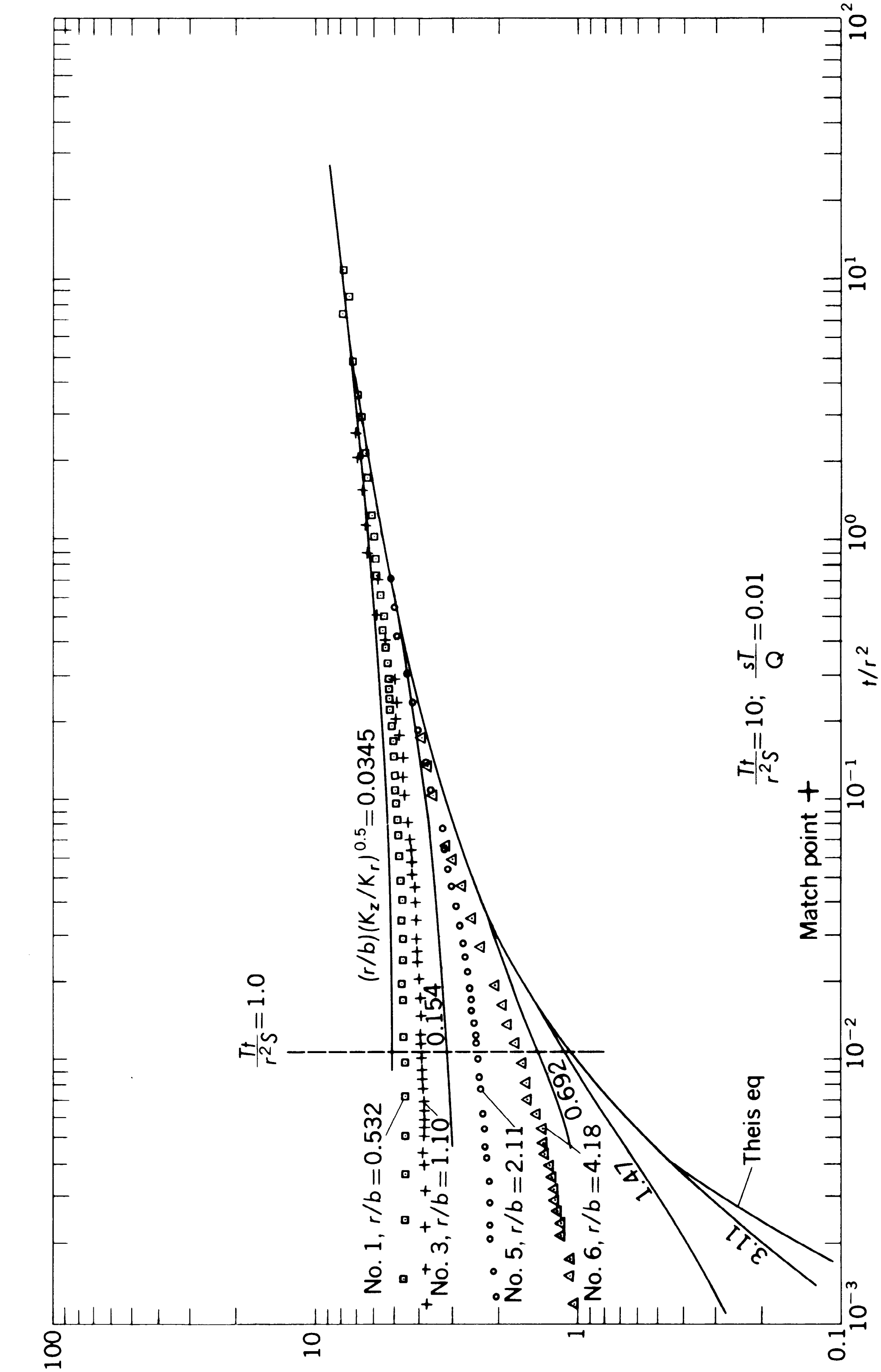

Figure 8.—Data from wells screened in middepths of aquifer at Nevius test site, showing type-curve match. Type curves from Stallman (1965, p. 305, fig. 100).

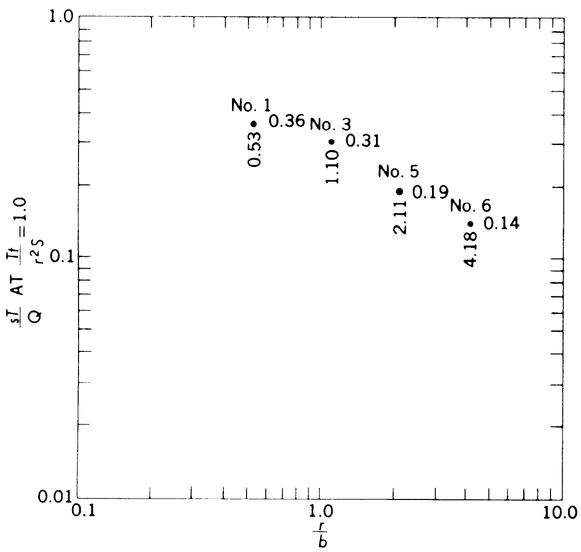

Figure 9.—Dimensionless drawdowns from Nevius test data at $Tt/r^2S = 1.0$ as a function of r/b.

mate that the vertical-to-horizontal conductivity ratio is 0.0064. The other assumption—that delayed yield is most important—also results in a reasonable fit between the data and auxiliary type curve. Which analysis is correct?

$$\text{If } T = 3\times10^4 \text{ft}^2/d, \text{ then } K_r = 790 \text{ ft}/d \text{ and } K = 5.1 \text{ ft}/d,$$

according to the vertical flow model. For this value of K_z and $s = 6$ ft, equation 8 predicts delayed yield is small after

$$t = 10\times0.184\times6/5.1 = 2.2 \text{ days},$$

which is in reasonable agreement with Boulton's delayed yield model. Analysis of data from the unsaturated zone near the Nevius test site shows that K_z is between 15 and 40ft/d (Stallman and Reed, 1968), which is considerably larger than obtained from match point A of figure 10.

Results from the Nevius pumping test illustrate the uncertainty surrounding calculated coefficients for unconfined aquifers. Both models—the radial-flow–delayed-yield and the constant-yield–vertical-flow systems—are meritorious in their own right. In truth, delayed yield and vertical flow both probably contributed significantly to the drawdown configuration observed during the first day of pumping at the Nevius site.

At the present state of the art, it is not academically feasible to relate the vertical to the horizontal conductivities from the Nevius data, nor positively to assign a numerical value to the role of delayed yield. Rather, such information must be produced by added measurements in the pumping test. Monitoring variations of moisture content above the water table by nuclear meter during the test would better define specific yield and its variation with time. Graphs of drawdown versus radius observed at the top and bottom of the aquifer in the vicinity of a partially penetrating well should indicate the value of K_z/K_r when the system approaches artesian flow, as indicated by Norris and Fidler (1966). If the hydraulic properties of aquifers are to be determined accurately from pumping tests, site conditions require evaluation by a variety of measurements transcending the scope available from the standard observations of drawdown, discharge, radius, and depth.

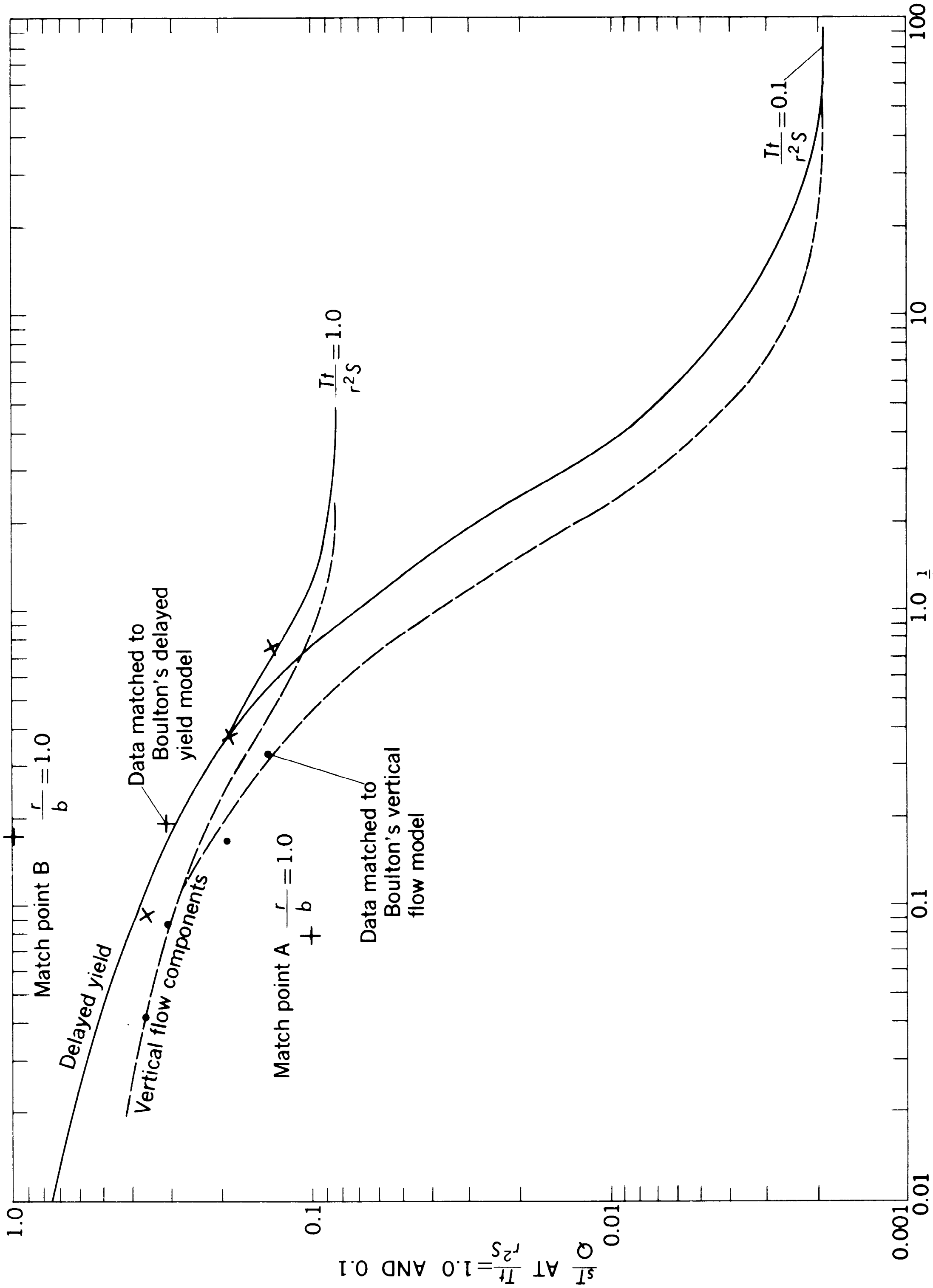

Figure 10.—Auxiliary type curves from Boulton's (1963) models, showing fit to Nevius data.

References

Bixel, H. C., Larkin, B. K., and Van Poollen, H. K., 1963, Effect of linear discontinuities on pressure build-up and drawdown behavior: Jour. Petroleum Technology, v. 15, no. 8, p. 885–895.

Boettcher, A. J., 1964, Geology and ground-water resources in eastern Cheyenne and Kiowa Counties, Colorado: U.S. Geol. Survey Water-Supply Paper 1779–N, 32 p.

Boulton, N. S., 1954a, The drawdown of the water table under non-steady conditions near a pumped well in an unconfined formation: Inst. Civil Engineers Proc., v. 3, pt. 3, p. 564–579.

______ 1954b, Unsteady radial flow to a pumped well allowing for delayed yield from storage: Internat. Assoc. Sci. Hydrology, Pub. 37, p. 472–477.

______ 1963, Analysis of data from non-equilibrium pumping tests allowing for delayed yield from storage: Inst. Civil Engineers Proc., v. 26, p. 469–482.

______ 1964, Discussion of "Analysis of data from non-equilibrium pumping tests allowing for delayed yield from storage," by N. S. Boulton, Inst. Civil Engineers, Proc. v. 26, 1963, p. 469–482: Inst. Civil Engineers Proc., v. 28, p. 603–610.

______ 1965, The discharge to a well in an extensive unconfined aquifer with constant pumping level: Jour. Hydrology, v. 3, p. 124–130.

Bredehoeft, J. D., Cooper, H. H., Jr., and Papadopulos. I. S., 1966, Inertial and storage effects in well-aquifer systems—An analog investigation: Water Resources Research, v. 2, p. 697–707.

Brown, R. H., 1953, Selected procedures for analyzing aquifer test data: Am. Water Works Assoc. Jour., v. 45, no. 8, p. 844–866.

Buchanan, T. J., and Somers, W. P., 1969, Discharge measurements at gaging stations: U.S. Geol. Survey Techniques Water-Resources Inv., book 3, chap. A8, 65 p.

Chow, V. T., ed., 1964, Handbook of applied hydrology: New York, McGraw-Hill Book Co., 1,418 p.

Cooper, H. H., Jr., Bredehoeft, J. D., and Papadopulos, I. S., 1967, Response of a finite-diameter well to an instananeous charge of water: Water Resources Research, v. 3, p. 263–269.

Cooper, H. H., Jr., Bredehoeft, J. D., Papadopulos, I. S., and Bennett, R. R., 1965, The response of well-aquifer systems to seismic waves: Jour. Geophys. Research, v. 70, p. 3915–3926.

Dagan, G., 1967, A method of determining the permeability and effective porosity of unconfined anisotropic aquifers: Water Resources Research, v. 3, p. 1059–1071.

Ferris, J. G., Knowles, D. B., Brown, R. H., and Stallman, R. W., 1962, Theory of aquifer tests: U.S. Geol. Survey Water-Supply Paper 1536–E, p. 69–174.

Hantush, M. S., 1960, Modification of the theory of leaky aquifers: Jour. Geophys. Research, v. 65, no. 11, p. 3713–3725.

______ 1961a, Drawdowns around a partially penetrating well: Am. Soc. Civil Engineers Proc., v. 87, no. HY4, p. 83–98.

______ 1961b, Tables of the function $H(u, \beta)$: Socorro, N. Mex., New Mexico Inst. Mining and Technology Prof. Paper 103, 13 p.

______ 1961c, Tables of the function $W(u, \beta)$: Socorro, N. Mex., New Mexico Inst. Mining and Technology Prof. Paper 104, 13 p.

______ 1961d, Aquifer tests on partially penetrating wells: Am. Soc. Civil Engineers Proc., v. 87, no. HY5, p. 171–195.

______ 1962, Flow of ground water in sands of nonuniform thickness—[Part] 3, Flow to wells: Jour. Geophys. Research, v. 67, no. 4, p. 1527–1534.

______ 1964a, Hydraulics of wells, *in* Advances in Hydroscience, Volume 1: New York, Academic Press, Inc., p. 281–442.

______ 1964b, Drawdown around wells of variable discharge: Jour. Geophys. Research, v. 69, no. 20, p. 4221–4235.

______ 1966a, Analysis of data from pumping tests in anisotropic aquifers: Jour. Geophys. Research, v. 71, no. 2, p. 421–426.

______ 1966b, Wells in homogeneous anisotropic aquifers: Water Resources Research, v. 2, no. 2, p. 273–279.

Hantush, M. S., and Jacob, C. E., 1955, Non-steady radial flow in an infinite leaky aquifer: Am. Geophys. Union Trans., v. 36, no. 1, p. 95–100.

Hantush, M. S., and Papadopulos, I. S., 1962, Flow of ground water to collector wells: Am. Soc. Civil Engineers Proc., v. 88, no. HY5, p. 221–244.

Hantush, M. S., and Thomas, R. G., 1966, A method for analyzing a drawdown test in anisotropic aquifers: Water Resources Research, v. 2, no. 2, p. 281–285.

Jacob, C. E., 1946, Radial flow in a leaky artesian aquifer: Am. Geophys. Union Trans., v. 27, p. 198-205.

______ 1947, Drawdown test to determine effective radius of artesian wells: Am. Soc. Civil Engineers Trans., v. 112, p. 1047–1070.

______ 1963, Determining the permeability of water-table aquifers: U.S. Geol. Survey Water-Supply Paper 1536–I, p. 245–271.

Jacob, C. E., and Lohman, S. W., 1952, Nonsteady flow to a well of constant drawdown in an extensive aquifer: Am. Geophys. Union Trans., v. 33, no. 4, p. 559–569.

Jorgensen, Donald G., 1969, Field use of orifice meters: Ground Water, v. 7, no. 4, p. 8–11.

Norris, S. E., and Fidler, R. E., 1966, Use of type curves developed from electrical analog studies of unconfined flow to determine the vertical permeability of an aquifer at Piketon, Ohio: Ground Water, v. 4, no. 3, p. 43–48.

Papadopulos, I. S., 1967, Nonsteady flow to a well in an infinite anisotropic aquifer: Internat. Assoc. Sci. Hydrology Pub. 73, p. 21–31.

Papadopulos, I. S., and Cooper, H. H., Jr., 1967, Drawdown in a well of large diameter: Water Resources Research, v. 3, p. 241–244.

Prickett, T. A., 1965, Type-curve solutions to aquifer tests under water table conditions: Ground Water, v. 3, p. 5–14.

Rorabaugh, M. I., 1953, Graphical and theoretical analysis of step-drawdown test of artesian wells: Am. Soc. Civil Engineers Trans., v. 79, Separate 362, 23 p.

Stallman, R. W., 1962, Boulton's integral for pumping test analysis: U.S. Geol. Survey Prof. Paper 424–C, p. C24–C29.

——— 1965, Effects of water table conditions on water level changes near pumping wells: Water Resources Research, v. 1, p. 295–312.

——— 1967, Flow in the zone of aeration, *in* Chow, Ven T., ed., Advances in Hydroscience, Volume 4—1967: New York, Academic Press, p. 151–195.

Stallman, R. W., and Reed, J. E., 1968, Steady flow in the zone of aeration, water in the unsaturated zone: Wageningen Symposium, Internat. Assoc. Sci. Hydrology Pub. 83, v. 2, p. 565–575.

Theis, C. V., 1935, The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using ground-water storage: Am. Geophys. Union Trans., v. 14, pt. 2, p. 519–524.

Thiem, Gunther, 1906, Hydrologische Methoden (Hydrologic methods): Leipzig, J. M. Gebhardt, 56 p.

U.S. Bureau of Reclamation, 1967, Water measurement manual [2d ed.]: Washington, D.C., U.S. Gov. Printing Office, 327 p.

Weeks, E. P., 1964, Field methods for determining vertical permeability and aquifer anisotropy: U.S. Geol. Survey Prof. Paper 501–D, p. D193–D198.

☆U.S. GOVERNMENT PRINTING OFFICE: 1983-381-614/551