Methods and Applications of Electrical Simulation in Ground-Water Studies in the Lower Arkansas and Verdigris River Valleys, Arkansas and Oklahoma

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1971

Prepared in cooperation with the U.S. Army Corps of Engineers

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By M. S. BEDINGER, J. E. REED, C. J. WELLS, *and* B. F. SWAFFORD

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To Leo Emmett —
with best wishes,
thanks and fond memories
of the Ark Valley.
Doug

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BY M. S. BEDINGER,[1] J. E. REED,[1] C. J. WELLS,[2] and B. F. SWAFFORD [2]

ABSTRACT

The Arkansas River Multiple-Purpose Plan will provide year-round navigation on the Arkansas River from near its mouth to Muskogee, Okla., and on the Verdigris River from Muskogee to Catoosa, Okla. The altered regimen in the Arkansas and Verdigris Rivers will affect ground-water conditions in the adjacent alluvial aquifers. In 1957 the U.S. Geological Survey and U.S. Army Corps of Engineers entered into a cooperative agreement for a comprehensive ground-water study of the lower Arkansas and Verdigris River valleys. At the request of the Corps of Engineers, the Geological Survey agreed to provide (1) basic ground-water data before, during, and after construction of the Multiple-Purpose Plan and (2) interpretation and projections of postconstruction ground-water conditions.

The data collected were used by the Corps of Engineers in preliminary foundation and excavation estimates and by the Geological Survey as the basis for defining the hydrologic properties of, and the ground-water conditions in, the aquifer. The projections of postconstruction ground-water conditions were used by the Corps of Engineers in the planning, design, construction, and operation of the Multiple-Purpose Plan.

Analysis and projections of ground-water conditions were made by use of electrical analog models. These models use the analogy between the flow of electricity in a resistance-capacitance circuit and the flow of a liquid in a porous and permeable medium.

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Verification provides a test of the validity of the analog to perform as the aquifer would, within the range of historic forces. The verification process consists of simulating the action of historic forces which have acted upon the aquifer and of duplicating the aquifer response with the analog.

The areal distribution of accretion can be treated as an unknown and can be determined by analog simulation of the piezometric surface in an aquifer. Comparison of accretion with depth to piezometric surface below land surface shows that accretion decreases with decreasing depth to water level. The decrease in accretion is attributed mostly to the increase in evapotranspiration from the aquifer, and where water levels are very near the land surface, to the rejection of recharge. The maximum accretion and the decrease in accretion with the decrease in depth to water are dependent upon the climate and the thickness and lithology of the fine-grained material overlying the aquifer.

Dams on the Arkansas and Verdigris Rivers will impose a direct change in water levels in the aquifers adjacent to the rivers. This change will be attenuated by the resultant change in accretion to the aquifer. The analogs of aquifers in the valleys were used to determine the change in ground-water level from preconstruction to postconstruction conditions.

INTRODUCTION

ARKANSAS RIVER MULTIPLE-PURPOSE PLAN

The Arkansas River Multiple-Purpose Plan provides for development of the Arkansas River and tributaries in Arkansas and Oklahoma. The plan, authorized by the River and Harbor Act of July 24, 1946, includes construction of coordinated developments to serve navigation, produce hydroelectric power, afford additional flood control, and provide benefits for such activities as recreation, wildlife propagation, and soil and water conservation. The river in its natural state presents problems in making it suitable for navigation. The principal problems are floods, low flows, which do not permit commercial river traffic, and the heavy sediment load, which creates an extremely unstable channel. Channel rectification and stabilization works have been constructed to provide a stable channel. Upstream reservoirs, shown on plate 1, have been constructed to prevent flooding, augment low flows, and help control sedimentation. Eighteen navigation structures that have been completed or are under construction on the main stem of the Verdigris and Arkansas Rivers will provide channel depths needed for river traffic. The navigation channel follows the Verdigris River from Catoosa, Okla., to its confluence with the Arkansas River at Muskogee, Okla., thence along the Arkansas River to the Arkansas Post Canal, down stream from Pine Bluff, Ark., thence along the canal to the

White River, and thence along the White to the Mississippi River. By providing river traffic this access to the Mississippi River, the navigation features of the Arkansas River Multiple-Purpose Plan will add a large area to that already served by inland waterways.

COOPERATIVE GROUND-WATER STUDY

In the early phases of the Multiple-Purpose Plan, the U.S. Army Corps of Engineers recognized that the natural ground-water flow system in the alluvium of the Arkansas and Verdigris River valleys would be altered by the navigation features of the plan. The U.S. Geological Survey and the Corps of Engineers entered into an agreement on April 15, 1957, which provided for a comprehensive study of the preconstruction and postconstruction ground-water conditions.

The purpose of the ground-water study was to establish on a continuing basis, a ground-water data-collection program throughout the lower Arkansas and Verdigris River valleys—the length of the Arkansas River Multiple-Purpose Plan. This study would provide comparative ground-water data for conditions prior to and after completion of the project, and projections of postconstruction water levels for use in design, construction, and operation of the plan.

The ground-water study had two phases—a data-collection phase, and an interpretive phase. The data-collection phase included geologic mapping of the principal hydrologic boundaries, alluvium and terrace deposits; a complete inventory of existing wells suitable for periodic water-level measurement; test-hole drilling and construction of observation wells to delineate the nature and extent of the alluvial aquifer and to supplement water-level data from existing wells; laboratory analyses of test-hole samples to determine grain size, porosity, specific yield, and permeability; chemical analyses of ground-water samples from selected observation wells in the alluvium and terrace deposits; and collection of water-level data.

The spacings of test holes and observation wells were planned to provide areal coverage compatible with the degree of detail needed in projections of postconstruction conditions. The spacing requirements for test holes and wells in the area downstream from Little Rock differed from the spacing requirements in the area upstream from Little Rock. Between Little Rock and the Mississippi River, the study area essentially includes one large continuous aquifer. Upstream from Little Rock, the study area comprises many small separate aquifers. The large aquifer downstream from Little Rock is less variable in hydrologic properties than the aquifers upstream from Little Rock. Therefore, fewer test holes for definition of aquifer properties are required. The piezometric surface downstream from Little Rock is regional in nature and requires fewer control points for definition than the many small aquifers upstream from Little Rock.

The density of test holes drilled to obtain lithologic information on the aquifer averaged about one test hole per 2 square miles. The average density ranged from about five test holes per square mile for the area from Little Rock to the Mississippi River to about one test hole per one-third square mile for the alluvial area upstream from Little Rock.

The density of wells used for collecting water-level data averaged about one well per 1½ square miles. The density ranged from about one well per 2½ square miles for the area from Little Rock to the Mississippi River to about one test hole per one-third square mile for alluvial areas upstream from Little Rock.

The basic geohydrologic data not only directly served the purpose of the ground-water study, but also provided benefits to the Corps of Engineers in the form of supplemental-project geologic data. Lithologic descriptions, top-of-rock elevations, and water-level data from the Geological Survey test-hole and piezometer installations were used in preliminary studies for the design of bridge pier footings, drainage structures, pile and rock dikes, channel cutoff excavations, and bank revetments and in estimating rock excavation requirements in the navigation channel. Water-quality data were used in the design of water-supply-treatment systems and as a guide to sources of concrete-curing water.

The interpretive phase consisted principally of projecting the anticipated postconstruction ground-water conditions. The projections made by the Geological Survey were used as one criterion in selecting dam locations, pool elevations, and areas to be protected by levees and were used in the design and plans for the operation of drainage systems.

During the early part of the study, the Geological Survey selected and developed the analytical methods used in projecting the postconstruction ground-water conditions. The problem required that the methods be adaptable to wide variations in size, shape, and hydrologic characteristics of the aquifer and that they be capable of portraying all boundary forces acting upon the aquifer. Further, the methods had to be accurate, had to provide the required areal detail, and had to be operational to meet a predetermined schedule. The electrical analog was chosen as the principal tool used in making ground-water projections.

The analog is an excellent tool for projecting the effects of changes in forces on the aquifer, such as changes in stream stage produced by navigation pools. Electrical analogs of the alluvial aquifers in the navigation reach were constructed and used in analysis of preconstruction ground-water conditions and in making projections of postconstruction ground-water conditions. The analogs represent an area of about 2,350 square miles in the Arkansas and Verdigris River valleys of Arkansas and Oklahoma. The results of the analog studies were presented to the Corps of Engineers in more than 25 administrative reports, dating from June 1960 to June 1967, for use in project design and operation.

In addition to the administrative reports to the Corps of Engineers, a series of reports were prepared for public release. These reports include open-file compilations of basic data, such as logs of test holes, records of wells, and water-level measurements, a series of U.S. Geological Survey publications describing the geohydrology of the Arkansas and Verdigris River valleys, and numerous papers on the hydrologic methods developed and adapted for use in the cooperative ground-water study. A list of the open-file and published reports is given in the bibliography.

Many of the analog techniques and methods developed for this study have not been described. These methods and projections and their application by the Corps of Engineers can be useful in similar projects in other areas. The purpose of this report is to describe the electrical analog methods used in the study and to describe their application in the Arkansas River Multiple-Purpose Plan.

THE ARKANSAS AND VERDIGRIS RIVER VALLEYS

PHYSIOGRAPHY

Upstream from Little Rock, the Arkansas and Verdigris Rivers are confined to narrow valleys by hard, resistant rocks of Paleozoic age. The flood plain of the Verdigris River ranges in width from ½ to 4 miles and averages about 2 miles. The Verdigris River valley is in the Osage Plains section of the Central Lowland province (fig. 1). From its confluence with the Verdigris River, the Arkansas River traverses the Interior High-

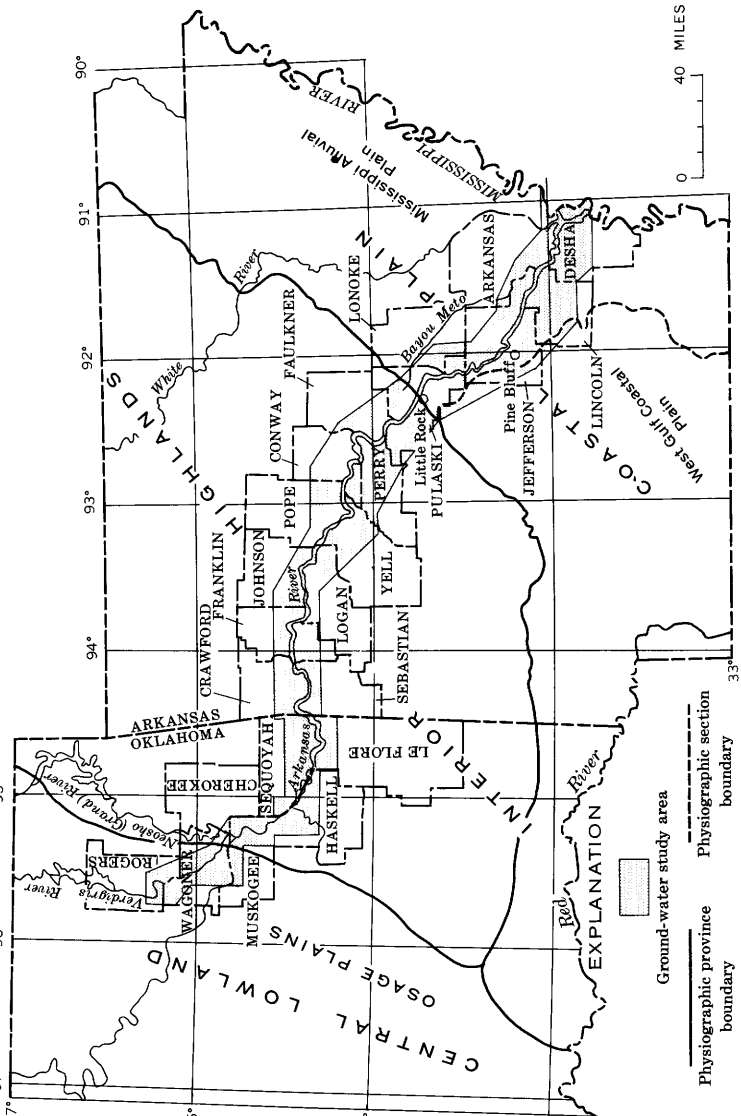


Figure 1.—Location and physiography of the ground-water study area.

lands. In the Interior Highlands, the Arkansas River is confined to a flood plain ranging from 1 to 3 miles in width. The bedrock, which in places directly borders the river, separates the flood plain into a series of disconnected segments ranging in length from 3 to 11 miles. The valley walls rise abruptly from 50 to several hundred feet above the flood plain.

Downstream from Little Rock, the Arkansas River lies in the Mississippi Alluvial Plain of the Coastal Plain province. The alluvial plain is relatively flat, with flood-plain features—point bars, natural levees, abandoned channels and backswamps, and low terraces. Terraces, rising about 20 feet above the flood plain, border the valley on the south from Little Rock to Pine Bluff. The Grand Prairie region, a terrace about 10 feet above the flood plain, lies north of the Arkansas River, between the White River and Bayou Meto.

ALLUVIAL GEOLOGY AND HYDROLOGY

The alluvium filling the valley can be divided into two parts: a substratum of coarse sand and gravel, which generally grades upward into medium and fine sands, and a top stratum composed of fine-grained material—silts, clays, and very fine sands.

The top stratum forms distinct deposits in the alluvial plain. These deposits are classed according to their mode of deposition and include natural-levee, point-bar, accretion, swale, channel-fill, and backswamp deposits.

The alluvium in the Verdigris and Arkansas River valleys in Oklahoma ranges in thickness from 25 to 59 feet and averages about 40 feet. From Fort Smith to Little Rock, Ark., the average thickness of the alluvium increases from 40 to 80 feet. Downstream from Little Rock, the alluviium ranges in thickness from 75 to 200 feet and averages 200 feet.

The aquifer in the Verdigris River valley is composed of fine-grained poorly sorted material. The transmissibility is low and ranges from about 2,000 to 8,000 gpd per ft (gallons of water per day per foot). In the Arkansas River valley, average permeability of aquifer materials decreases slightly downstream, whereas the transmissibility increases downstream because of the increase in aquifer thickness. The transmissibility ranges from about 40,000 gpd per ft to more than 300,000 gpd per ft.

The Arkansas and Verdigris Rivers are deeply incised into the substratum. In many places the Verdigris River penetrates the full thickness of the alluvium. The channel thalweg of the Arkansas

River upstream from Little Rock is scored to bedrock at many places during floods.

The natural flow of ground water in the alluvium is toward the Arkansas and Verdigris Rivers. Recharge is chiefly by infiltration of rainfall. During periods of high river stage, the discharge of ground water to the rivers is prevented, and water is stored in the aquifer at the riverbanks until the river recedes. Locally, the natural ground-water gradient has been reversed by pumping, such as at Ozark, Johnson County, Ark., and in several localities in the Coastal Plain, such as in the Grand Prairie region, Arkansas County, Ark. In these localities, water is induced from the river into the aquifer.

ELECTRICAL ANALOG DESIGN AND CONSTRUCTION

Electrical analogs of ground-water systems utilize the analogy between the flow of water in a porous medium and the flow of electricity in an electrical circuit. The analogy has been described by Skibitzke (1960) and Brown (1962). This analogy permits the construction of a model of a ground-water flow system having convenient laboratory dimensions. Hydraulic head, resistance to flow, and rate of water flow in the aquifer are analogous to electrical potential, electrical resistance, and rate of flow of electricity in an electrical circuit, respectively.

Analog simulation of an aquifer requires the definition of two significant parameters—the internal state of the aquifer and the shape of the external boundary and the force field around it (Skibitzke and da Costa, 1962). The first parameter is measured in terms of the spatial distribution of transmissibility and storage characteristics of the aquifier. These characteristics are modeled electrically by a network of resistors and capacitors, respectively. The second parameter concerns the shape of the aquifer and the external forces acting upon the boundaries of the aquifer. Boundaries of flow are modeled using sources of electrical potential, direct-current voltage for steady-state simulation, and arbitrary-function generators, or step-function generators for simulating variations of potential with time. Boundaries across which there is no flow, such as those formed by impermeable rocks, are modeled by terminating the electrical network.

The analog is made up of three components (fig. 2). Two of these, the resistance-capacitance network and the electrical

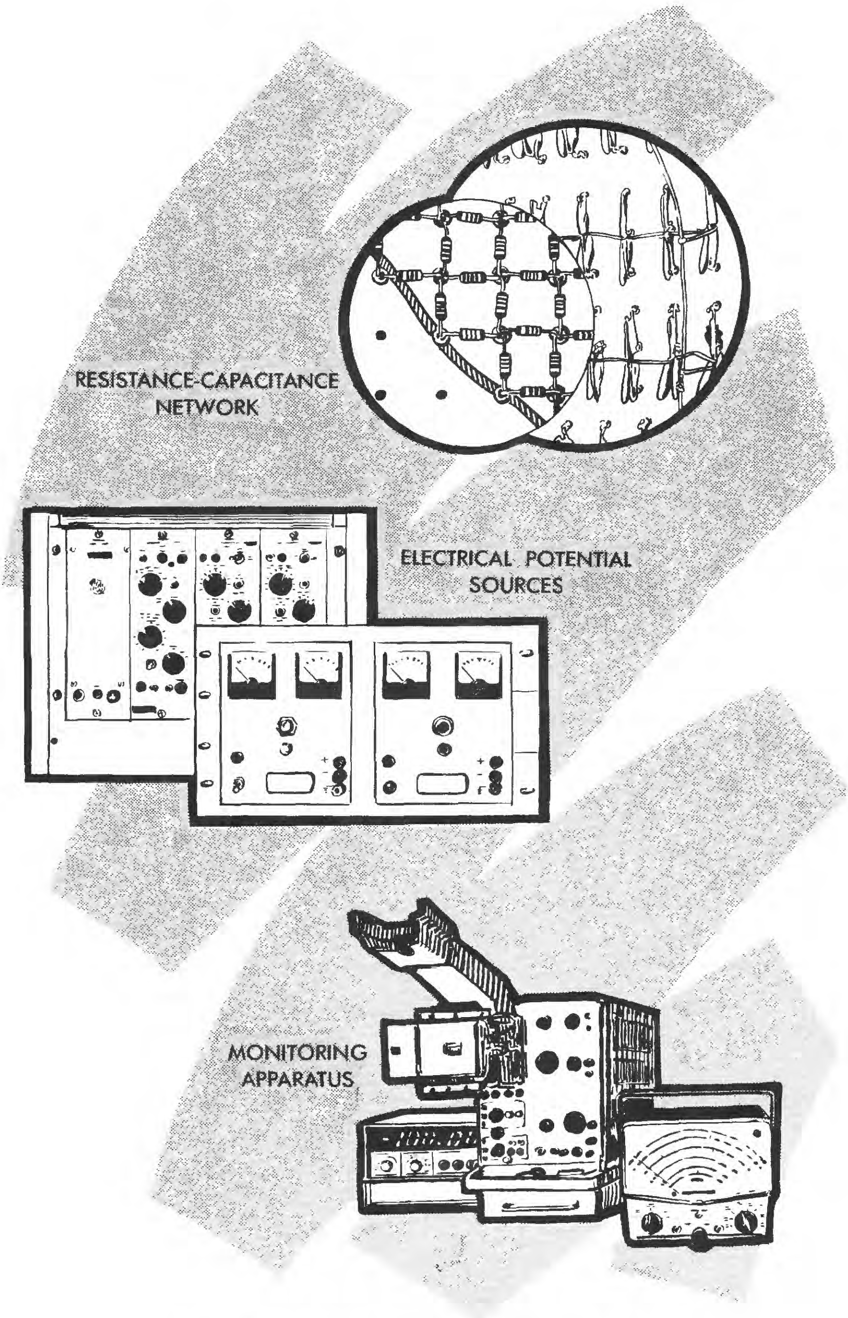


FIGURE 2.—Components of an electrical analog.

potential sources, are described in the preceding paragraph. The third component is the monitoring apparatus used to measure the imput to the analog from the electrical potential sources and to measure the response of the analog to the potentials acting upon the boundaries.

SCALE FACTORS

The hydraulic parameters of the ground-water flow system are related to an electrical model by means of scale factors K_ϕ, K_ξ and K_T. K_ϕ is expressed in feet per volt; K_ξ, in gallons per day per ampere; and K_T, in days per second. The scale factors are defined as follows:

$$K_\phi = \frac{h}{V}, \tag{1}$$

$$K_\xi = \frac{Q}{I}, \text{ and} \tag{2}$$

$$K_T = \frac{t_d}{t_s}, \tag{3}$$

where h is hydraulic head, in feet; V is electrical potential, in volts; Q is flow of water, in gallons per day; I is flow of electricity, in amperes; t_d is real time, in days; and t_s is analog time, in seconds.

Initial selection of scale factors should be made with knowledge of the capabilities and limitations of the components used in the analog, the power supplies, and the readout equipment. However, adjustments in scale factors can be made after design of the analog has been made. For example, to adjust for limitations in power supplies or readout equipment, in steady-state analyses the scale factors K_ϕ, relating feet to volts, and K_ξ relating flow of water to flow of electricity, can be changed without physical modification of component parts of the analog.

Another use of scale-factor modification is in verification. For example, it may be desirable during verification of an analog to change the design diffusivity. If K_T is changed by a given factor, a, then the diffusivity, the ratio of transmissibility to storage, of the design analog is changed by the inverse of the factor, or $1/a$. Also, the transmissibility and storage of the analog can be modified separately by changing scale factors. The transmissibility can be changed without altering the storage coefficient by changing K_ξ and K_T, and maintaining K_ϕ, unchanged. Equation 5 must be satisfied in making this change. Also, the storage coefficient can be changed without altering the transmissibility by changing K_τ and maintaining K_ζ and K_ϕ, unchanged.

The ranges in scale factors used in models of the Arkansas valley are as follows: K_ϕ = 1 to 10 feet per volt; $K_\zeta = 1\times10^9$ to 1×10^{10} gpd per amp; and $K_\tau = 5\times10^4$ to 1×10^6 days per second. The use of these scale factors results in analogs that have a relatively short time constant. The short time constant requires that electronic apparatus for simulating nonsteady boundaries must be fast acting and repetitive; equipment monitoring the analog response must have an equally fast response and have a high internal impedance.

MODELING HYDROLOGIC PROPERTIES OF THE AQUIFER

Utilizing the scale factors, the quantitative description of the internal state of the aquifer (spatial variations of transmissibility and storage) can be modeled by electrical components. The electrical analog of an aquifer is composed of a large number of analog elements. Each element represents a finite part of the flow system. The electrical analog consists of a network of resistors and capacitors (fig. 3) constructed on a rectangular grid.

The resistors, R in figure 3*A*, are scaled to represent the transmissibility of the aquifer, utilizing the following relation between scale factors K_ϕ and K_ζ; resistance, R, in ohms; and transmissibility, T, in gallons per day per foot:

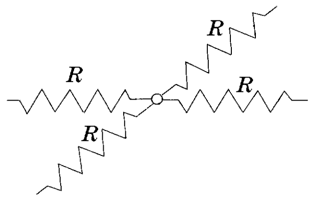
$$R = \frac{K_\zeta}{K_\phi T}. \tag{4}$$

Capacitor C in figure 3*B* represents the storage characteristics of the aquifer and is included in the analog for analyzing changes in hydraulic head with time. Capacitors are connected to the nodes of the resistor grid and are grounded. The following equation can be used to determine the capacitance:

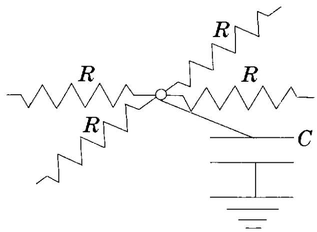
$$C = 7.48n^2S\frac{K_\phi}{K_\zeta K_\tau}, \tag{5}$$

where C is capacitance, in farads; n is the capacitor grid spacing in the scale of the aquifer, in feet; and S is the coefficient of storage, expressed as a decimal fraction.

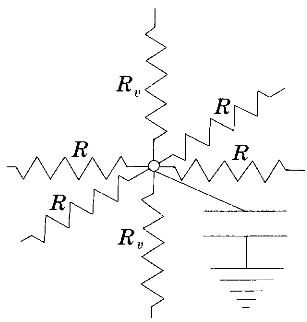
Vertical flow between distinct layers of the aquifer can be simulated through resistors, R_v in figure 3*C*, connecting two or more resistance networks. Values of R_v can be determined from the following equation:



A. TRANSMISSIBILITY
Resistors (R) represent resistance of aquifer to ground-water flow



B. WATER–STORAGE CAPACITY
Capacitor (C) simulates storage characteristic



C. VERTICAL PERMEABILITY
Resistors (R_v) connecting two or more layers of aquifer resistors model resistance of aquifer to vertical flow

FIGURE 3.—Diagrams illustrating electrical analogs of elemental parts of an aquifer.

$$R_v = \frac{K_t l}{K_\phi P_v n^2}, \tag{6}$$

in which R_v is the resistance, in ohms; l is the vertical node spacing, or the distance between layers, in feet; P_v is the coefficient of vertical permeability, in gallons per day per foot squared; and n is the horizontal grid spacing, in feet.

Analogs of the aquifer in the Arkansas River valley were constructed on ⅛-inch-thick masonite, perforated with holes, on a 1-inch rectangular grid. A map of the area to be modeled was painted on the masonite board. Solder-plated eyelets were inserted in each hole to provide terminals for the resistors, which represent the transmissibility of the aquifer. The resistors are ½-watt 10-percent-tolerance carbon resistors. Grid spacings of the Arkansas valley analogs ranged from one resistor per 150 feet to one resistor per one-half mile. A photograph showing the rectangular resistor network and transmissibility contour lines on the map side of the masonite board is shown in figure 4.

Capacitors, representing the storage characteristics of the aquifer, were soldered to the resistor network on the reverse side of the masonite board. Figure 5 is a photograph of the ceramic-disk capacitors used in an Arkansas River valley analog.

MODELING AQUIFER BOUNDARIES AND FORCES ACTING UPON THE AQUIFER

Aquifer boundaries are of two types: boundaries across which there is no flow, and boundaries across which there is flow. Boundaries across which there is no flow are commonly effected where the aquifer abuts impermeable rock. In large parts of the Arkansas River valley, relatively impermeable rocks underlie and laterally join the alluvial aquifer and form effective no-flow boundaries. No-flow boundaries are modeled electrically by terminating the resistance-capacitance network.

All energy within the system is gained and lost through the boundaries. Boundaries across which there is flow include stream boundaries; pumping wells; leaky, confining beds; and the water table. To model boundaries of flow, it is necessary to know either the head along the boundary or the rate of flow across the boundary.

The forces that act upon the ground-water flow system are many, varied, and complex. Simultaneous treatment of all the boundary forces may be impractical. Fortunately, for linear-flow systems, the principle of superposition can be employed to treat

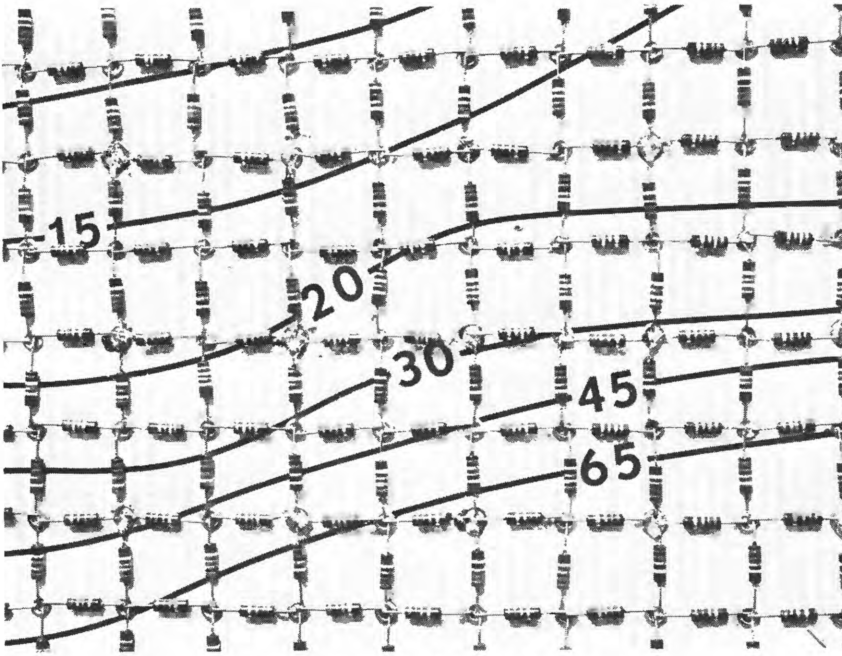


FIGURE 4.—Resistor network.

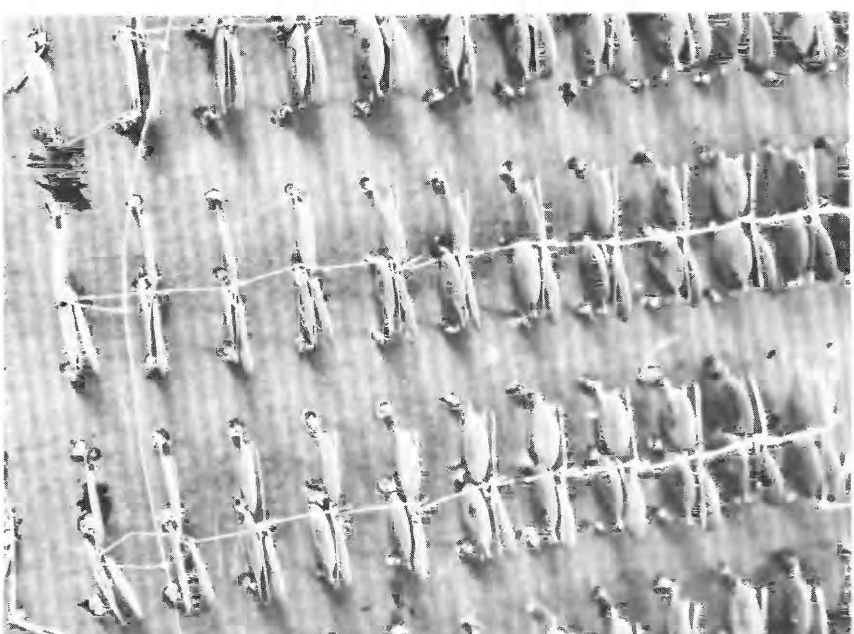


FIGURE 5.—Capacitor network.

separately the distinct components of a flow field. The principle of superposition permits simplification of the analog analysis by individual analysis of the head distributions throughout the component flow fields. The component head distributions are combined by algebraic summation, yielding finally the head distribution throughout the resultant field (Brown, 1963). The alternative to superposition is simultaneous duplication of the complete suite of boundary forces acting upon an aquifer.

One of the chief uses of superposition is in analysis of head-change components. For example, we can analyze by analog methods the component of head in the aquifer caused by pumping from a well field, commonly referred to as drawdown. The head-change component (the drawdown) can be superimposed on the natural head distribution in the aquifer to obtain a projection of the head distribution.

Energy for modeling boundaries of flow is derived from electrical power supplies—direct-current voltage supplies for simulating steady flow, and pulse generators and waveform synthesizers for analyses of nonsteady flow. Where flow rates across the boundaries are known, resistors are commonly used as metering devices to control flow to or from the analog. The response of the analog to equilibrium conditions is monitored by electronic voltmeters (fig. 6). Oscilloscopes are used to monitor rapidly changing voltage (fig. 7). A permanent record of time-voltage response can be obtained by photographing the oscilloscope trace. The boundary conditions described below are common ones but are only a few of many that are used in modeling. They are selected as examples because they will be referred to in later sections of this report.

STREAMS

Stream boundaries are commonly modeled as lines along which head is known. A part of an analog simulating an aquifer bounded by the Arkansas River is shown in figure 8. Schematic diagrams of analog elements for steady modeling and nonsteady-head conditions on the stream boundary are shown in figure 9. Figure 9 shows configurations of resistors used for steady-state stream boundaries. In figure 9*A*, the stream boundary is shorted to ground and, therefore, simulates a constant stage. In this configuration, the stage on the stream boundary is independent of forces acting upon other boundaries of the aquifer, such as pumping from wells. The stream acts as a recharge boundary when the cone of depression reaches the stream. In figure 9*B*,

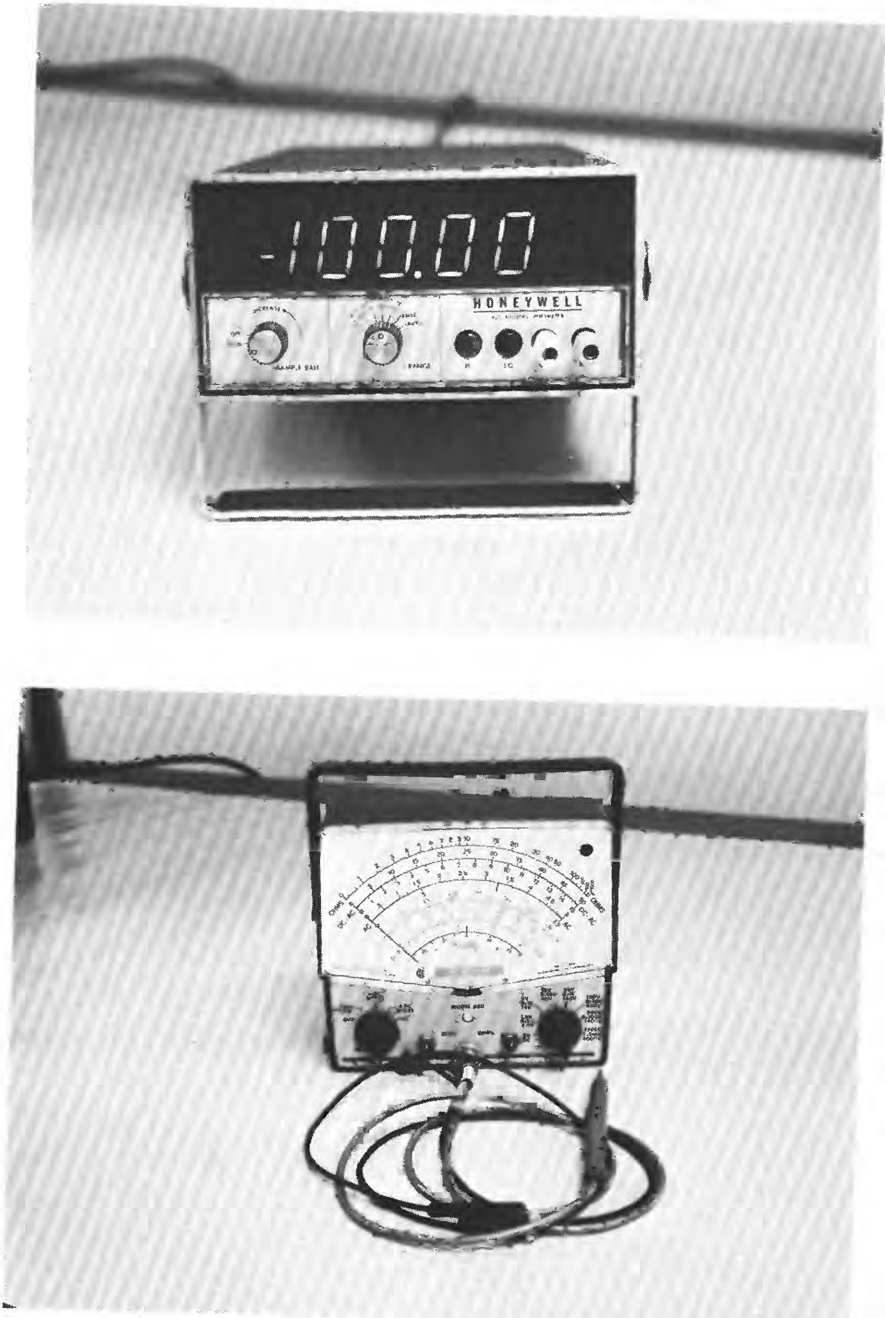


FIGURE 6.—Electronic voltmeters.

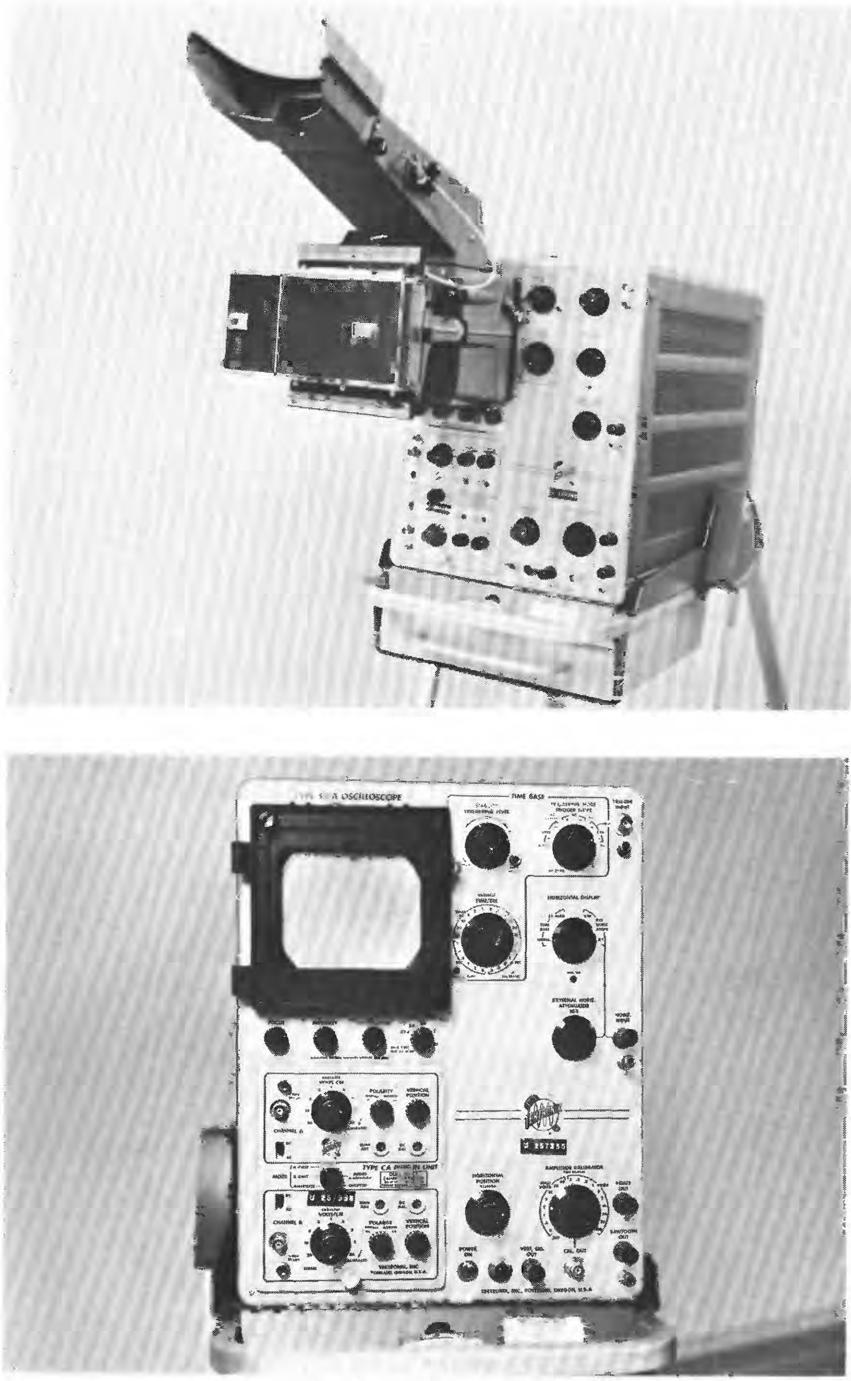


FIGURE 7.—Oscilloscope and camera.

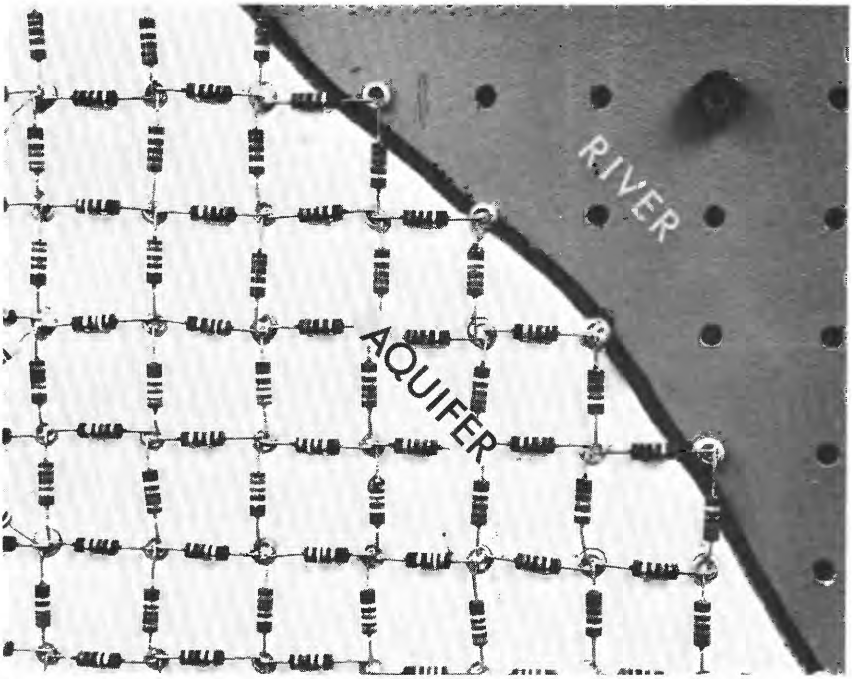


FIGURE 8.—Analog of river boundary.

steady voltages V_1, V_2, and V_3 are applied at nodes on the river boundary. This configuration can be used to simulate a sloping stream or a change in stream stage. A photograph of a direct-current voltage generator used in modeling steady-state stream boundaries is shown in figure 10.

Figure 9*C* illustrates the setup simulating a fluctuating stream stage. Because the problem involves changes in head with time, capacitors are added to the analog. This boundary configuration is used to study river-induced fluctuations of ground-water levels. A waveform synthesizer, used to simulate nonsteady stream boundaries in Arkansas valley analogs, is shown in figure 11.

In modeling potentials along boundaries, such as streams, voltage is related to hydraulic head by the scale factor K_ϕ, and the change in voltage with time is related to the change in head with time by scale factor K_T.

The streams, modeled as illustrated in figure 9, are assumed to fully penetrate the aquifer and to be hydraulically perfectly connected with it. Small streams, however, seldom meet these assumptions. Commonly, small streams penetrate only a fraction

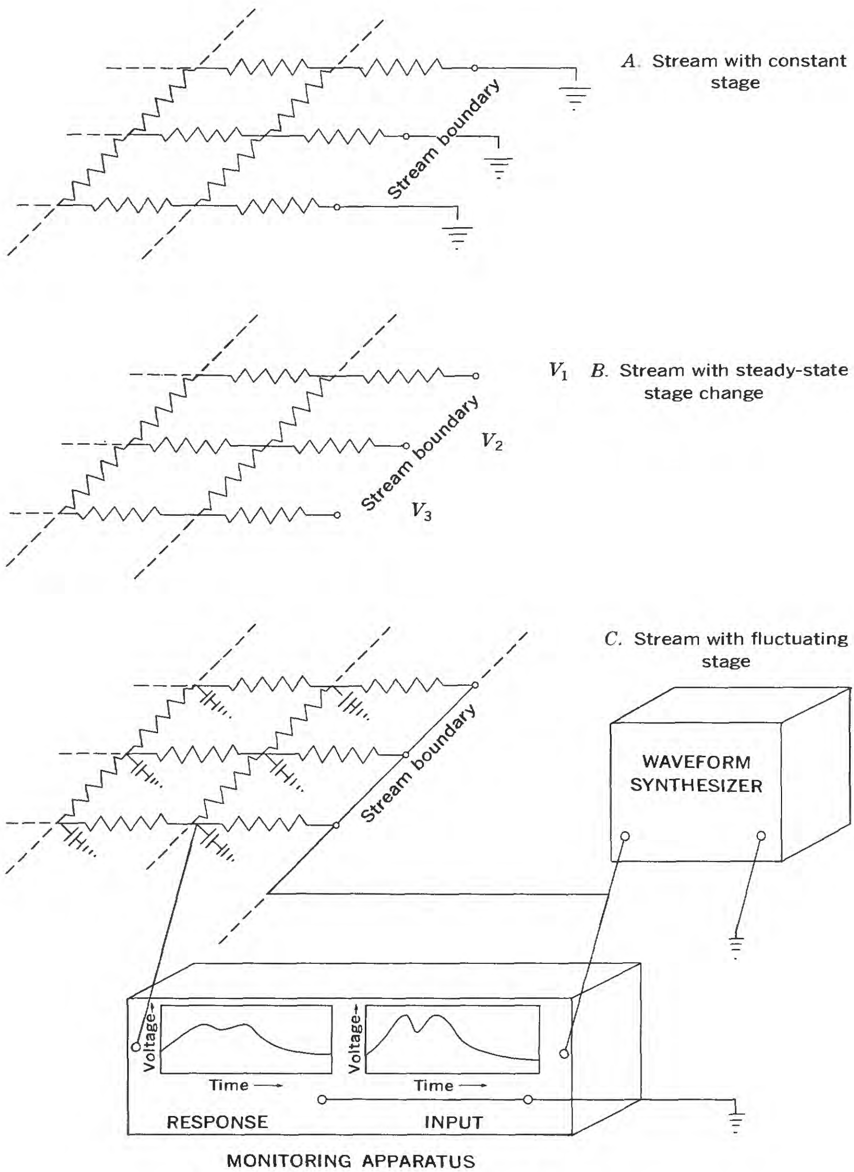


FIGURE 9.—Analogs of stream boundaries.

of the aquifer, or lie above it, separated from the aquifer by less permeable, fine-grained deposits. A method has been developed by J. E. Reed (written commun., 1968) for modeling such streams. Diagrams of the stream-aquifer flow system and its electrical analog are shown in figure 12. The loss of potential across resis-

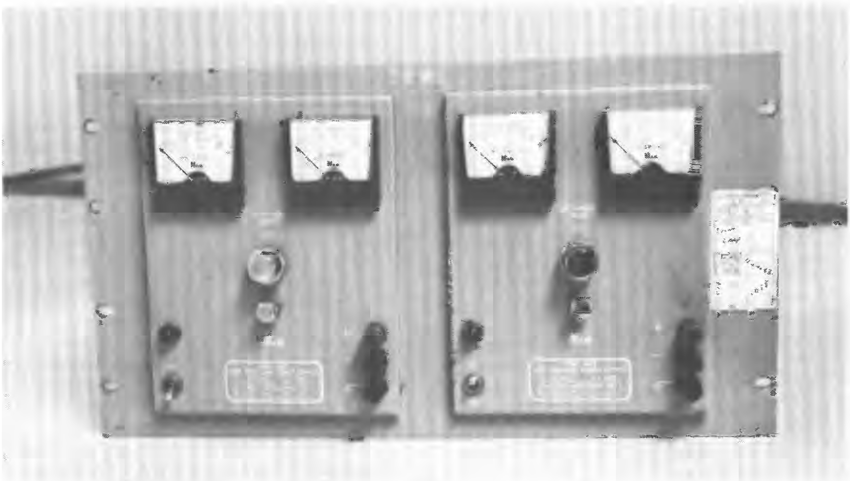


FIGURE 10.—Direct-current voltage generators.

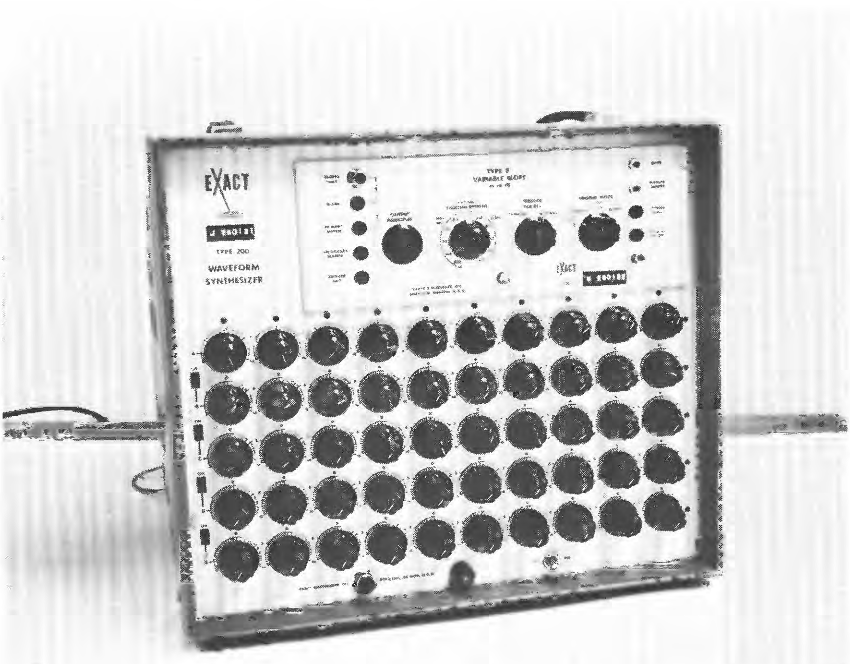
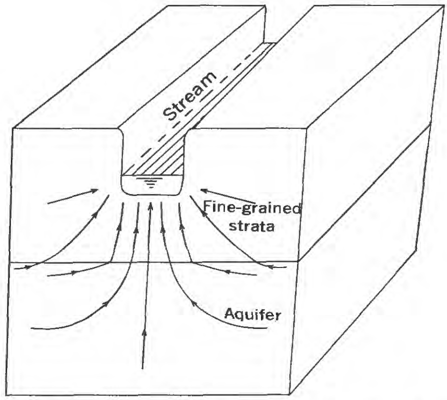
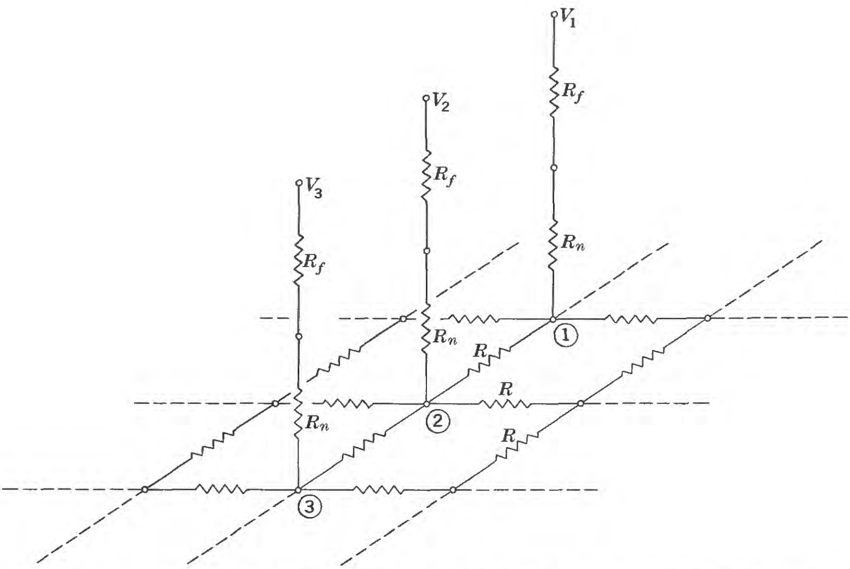


FIGURE 11.—Waveform synthesizer.



A. Block diagram of flow system showing relation of aquifer, fine-grained strata, and nonpenetrating stream



B. Schematic diagram of analog utilizing components to represent head loss near a nonpenetrating stream.
R, aquifer resistors; (1), (2), and (3), analog nodes coinciding with the position of the stream; R_n, resistor models head loss in aquifer due to nonpenetration of stream; R_f, resistor models head loss in the fine-grained strata above aquifer; and V_1, V_2, and V_3, electrical potentials representing the head in the stream

FIGURE 12.—Block diagram of flow system near a nonpenetrating stream, and schematic diagram of analog.

tor R_n is analogous to the loss of head in the acquifer due to non-penetration of the stream. The resistance R_n, in ohms, is given by

$$R_n = \tfrac{1}{2}\frac{RmA}{n},$$

where R is the resistance of the aquifer resistor in the analog at the stream; m is the thickness of the aquifer, in feet; n is the horizontal node spacing, in feet; and A is a factor that depends upon the ratio of the stream width to the aquifer thickness and the square root of the ratio of vertical to horizontal permeability of the aquifer. A can be obtained from the graph in figure 13.

The loss of head through the fine-grained strata above the stream is simulated by the loss of potential across R_f. The value of R_f, in ohms, is given by

$$R_f = \frac{K_\xi m'}{K_\phi P_f wn},$$

where K_ϕ and K_ξ are scaling constants relating feet to volts and gallons per day to amperes, respectively; m' is the thickness, in

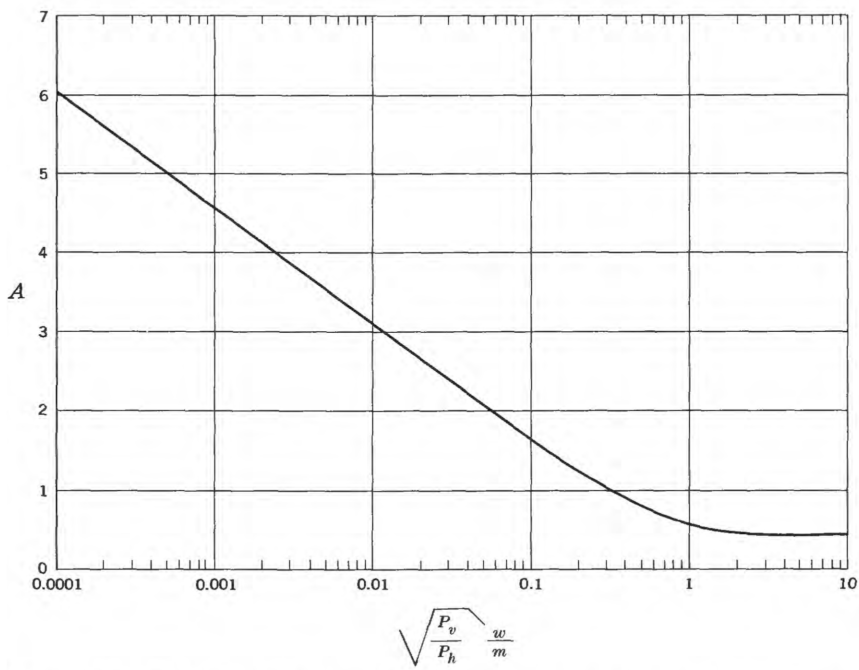


FIGURE 13.—Relation of A to horizontal permeability, P_h; vertical permeability, P_v; thickness of aquifer, m; and width of the stream, w.

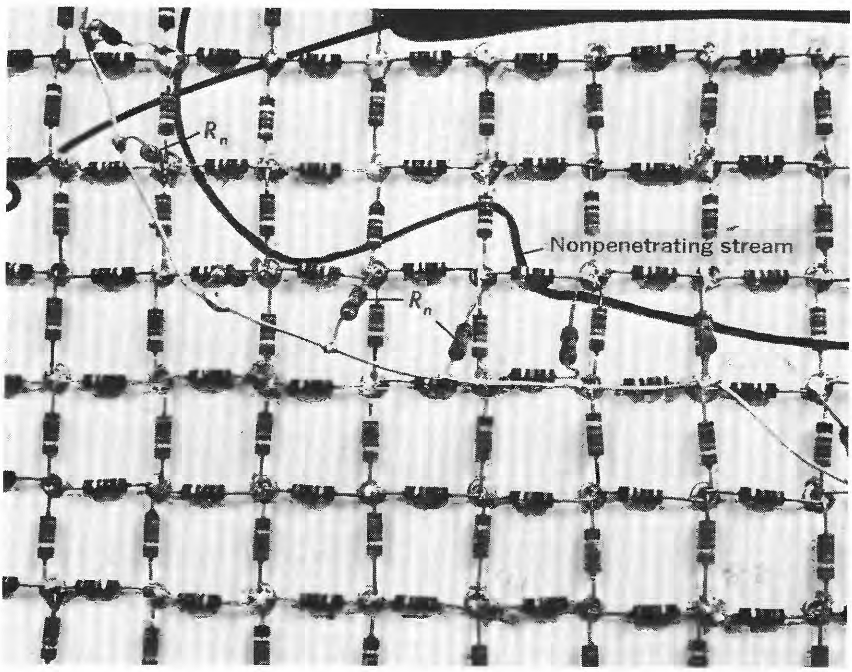


FIGURE 14.—Analog of a nonpenetrating stream boundary. The resistors (R_n) model the head loss at the nonpenetrating stream.

feet, of the fine-grained strata below the streambed; P_f is the permeability, in gallons per day per foot squared, of the fine-grained strata above the stream; w is the width, in feet, of the stream; and n is the node spacing, in feet.

Part of a nonpenetrating stream in an Arkansas River valley analog is shown in figure 14.

PUMPING WELLS

The analog configuration depicting a pumping well is shown in figure 15. The resistance, R_p, is found by the following equation:

$$R_p = \frac{V_r}{Q} K_\xi,$$

Where V_r is the potential drop across resistor R_p, in volts; and Q is the rate of pumping, in gallons per day.

A photograph of pulse generators and associated electronic components used in simulating pumping wells is shown in figure 16.

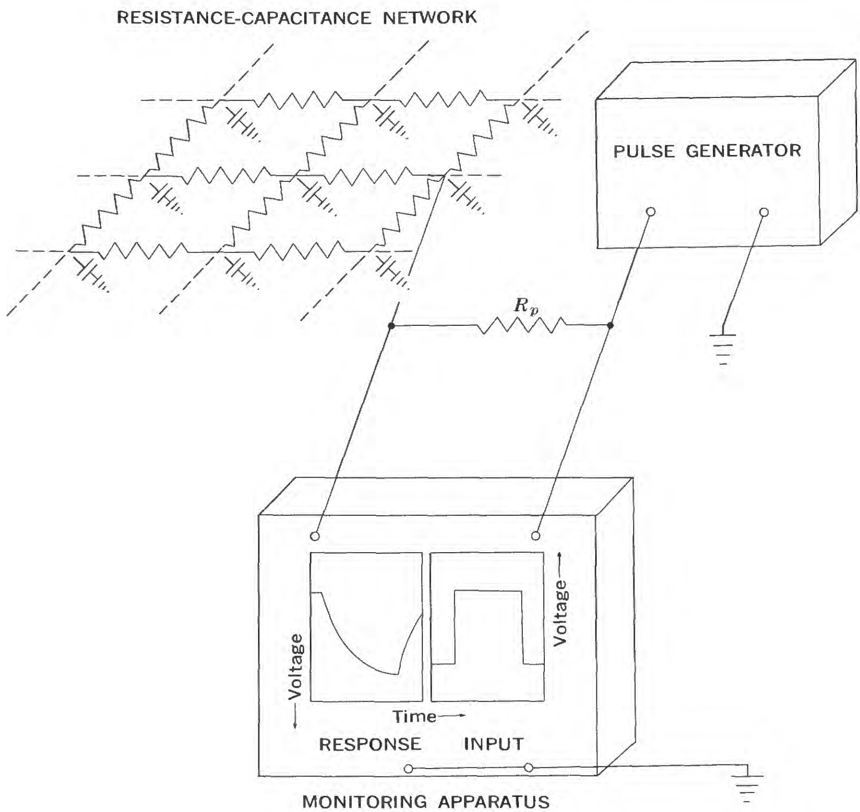


FIGURE 15.—Analog configuration depicting a pumping well.

ACCRETION

The piezometric-head distribution in an aquifer due to accretion is determined by the shape and the internal properties of the aquifer and the nature of its boundaries. Accretion is the net gain or loss of water vertically through the external surface of the aquifer in response to external forces; and in the Arkansas and Verdigris River valleys, is largely recharge from precipitation and evapotranspiration. Accretion can be treated as an unknown in analog simulation of the aquifer, and the areal distribution of accretion can be determined by appropriate analog measurements. Steady accretion is modeled by a network composed of elements shown in figure 17*B*. Accretion is simulated by electrical flow to or from the analog in resistor R_a. The resistance of R_a is chosen arbitrarily, and the rate of flow is determined by measuring voltages V_1 and V_2 (fig. 17*B*) at each R_a.

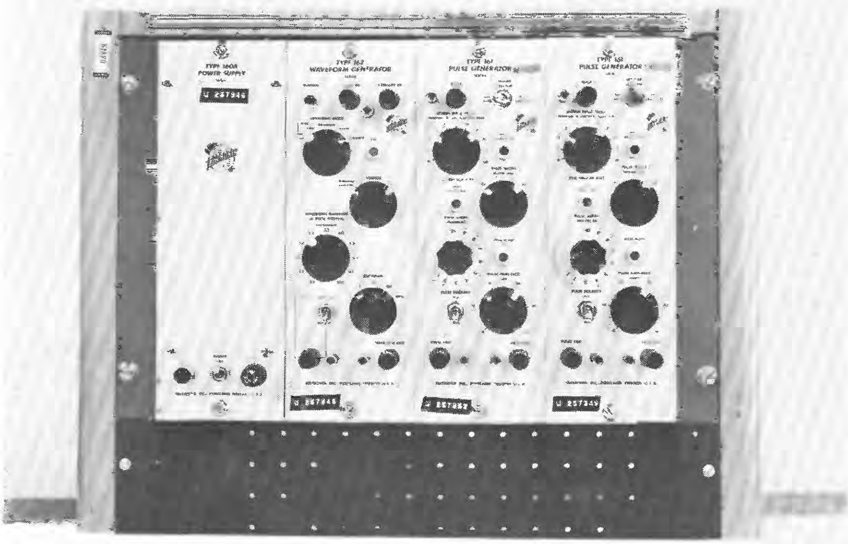
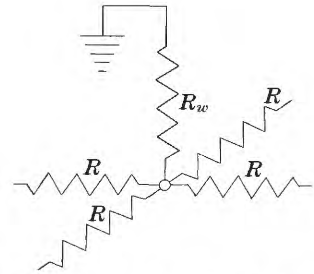
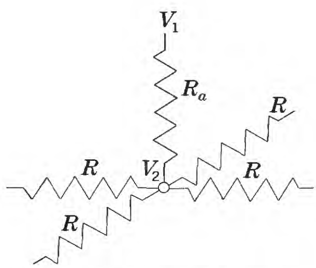


FIGURE 16.—Pulse generators and associated electronic components.

The basic analog element used for modeling the areally distributed relation between (1) change in accretion with (2) the change in piezometric head is shown in figure 17*A*. Evapotranspiration is treated as though it varies inversely to the depth to the piezometric surface. However, there are two limits to the relationship between evapotranspiration and depth to water. At depths to water near the surface, the maximum rate of evapotranspiration may be equal to, but not greater than, the capacity of the atmosphere to evaporate water. At increasing depth to water, the evapotranspiration decreases to zero. The maximum depth to water at which water is evapotranspired and the depth to water near the land surface where evapotranspiration reaches the maximum rate vary, depending upon the nature of the materials above the aquifer. The change in evapotranspiration is simulated by the flow of electricity through resistor R_w, as shown in figure 17, inasmuch as current is proportional to the voltage drop across the resistor. The values of these resistors may be determined from the following equation:



A. Flow through resistor R_w connected to aquifer network (resistors *R*) simulates change in accretion with change in water level



B. Flow through resistor R_a to aquifer network (resistors *R*) simulates accretion to the aquifer

FIGURE 17.—Analog elements modeling accretion to the aquifer.

$$R_w = \frac{K_\xi}{7.48K_\phi b^2 \Delta W/\Delta h},$$

where R_w is the resistance, in ohms; and $\Delta W/\Delta h$ is the change in evapotranspiration, in feet per day per change in hydraulic head, in feet; and b is the spacing between points in the aquifer at which R_w is connected in the model. A photograph of evapotranspiration resistors in an analog resistance network is shown in figure 18.

MODEL-NODE SPACING

The major considerations in selecting a node spacing in the aquifer are (1) the accuracy and detail of control needed to solve a hydrologic problem; (2) compatibility with the density of data available for design and verification of the analog; and (3) tolerable errors due to representing the flow field by a system of discrete parts.

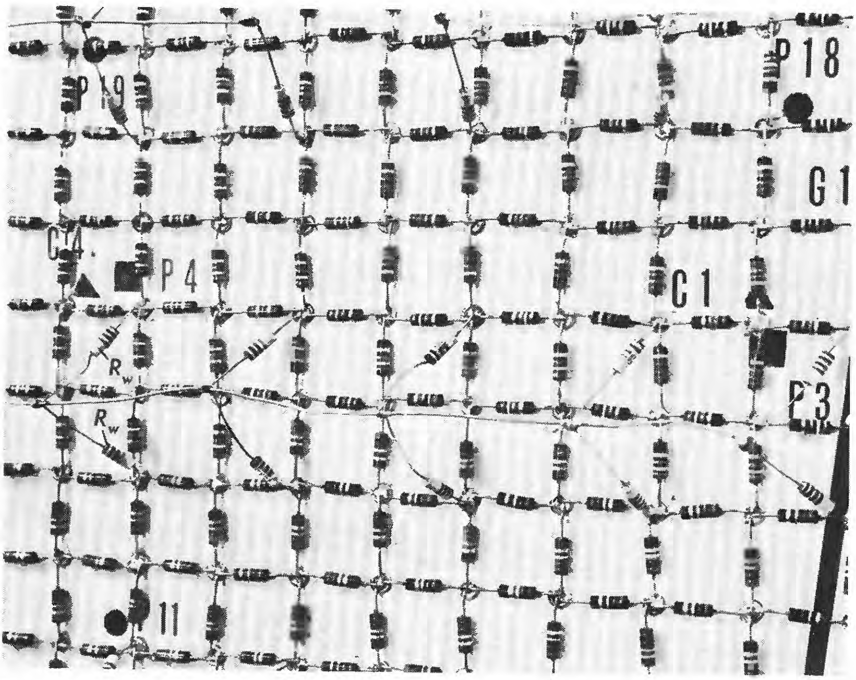


FIGURE 18.—Evapotranspiration resistor network in an analog. Resistors, R_w, model the change in accretion with the change in piezometric surface.

To be useful, the first criterion must be fulfilled in planning and collecting data in the area to be modeled because the detail and accuracy achieved from analysis of the model cannot exceed that of the data available for design and verification. The data available for design and verification establish an optimum node spacing for utilization of data or hydrologic properties of the aquifer. A node spacing larger than the optimum does not take advantage of all knowledge of the aquifer; however, a node spacing smaller than the optimum may be required to minimize discretization errors (errors due to finite-difference approximation).

Three sources of error arise in representing the aquifer by a system of discrete resistance- capacitance elements: (1) errors in representing a continuous flow field by flow paths in two directions; (2) errors made by representing continuous irregular boundaries by discrete points that may not coincide with the field position of the boundary; and (3) errors in interpolating between analog measurements to obtain potentials at field positions that do not coincide with node points.

Discretization errors cannot be avoided, but their effects can be minimized by recognizing where they occur and where they are likely to be large. In most analog analyses, discretization errors associated with boundaries are probably the most significant. These errors are greatest at the boundary and diminish with distance from the boundary. Generally, these errors are small at distances of from two to five nodes from the boundary.

Unfortunately, the criteria for node spacing are not readily reduced to a pat formula. Experience with analogs, however, provides a guide for design of node spacing. The node spacings used on analogs of the Arkansas River valley range from 150 feet to half a mile. Experience suggests that a favorable node spacing is achieved when the number of resistors per mile is equal to the number of control points per square mile times a factor ranging from one to three. Where the factor was less than one, the node spacing was too large for the detail of control; where the constant was greater than three, the node spacing was too fine for the degree of control.

Only those errors due to modeling the aquifer by a system of discrete parts were considered above. These errors can be minimized in design of the analog. Other errors evident in analogs include those in the component parts (resistors and capacitors) and in the monitoring equipment used to meter inputs and outputs of the analog. Component and equipment errors can be controlled by selection of components with desired tolerance ratings and by periodic calibration of equipment.

DETERMINATION OF AQUIFER CHARACTERISTICS

SIMULATION OF AQUIFER STIMULI AND RESPONSE

Analogs provide a means of portraying complex boundary conditions and nonhomogeneous aquifers. The response of an aquifer to a specified set of boundary conditions is determined by the hydrologic properties of the aquifer. Therefore, analog simulation of aquifer response to known forces provides a means of defining the aquifer properties.

Analog methods are suitable for determining aquifer properties in large areas. The boundary forces suited for analog simulation are those that affect a large number of analog nodal points—fluctuating stream stages, accretion, and pumping wells. Analysis of aquifer response to forces having small areas of influence offers no advantage over methods employing mathematical models.

Table 1 illustrates analog analyses that may be made to define aquifer properties. The data required for each analysis include information on the force acting upon the aquifer and the water-level response of the aquifer to the force. The method used for each analysis involves electrical simulation of the force and duplication of the water-level response in the aquifer. Analyses of steady states—those in which the force acting upon the aquifer is constant—yield information on the transmissibility of the aquifer. Nonsteady conditions—those in which the force acting upon the aquifer varies with time—yield information on the transmissibility and storage of the aquifer. Because the quantities of water exchanged between the aquifer and stream in response to streamstage fluctuations are not known, transmissibility and storage cannot be separated and are determined as a ratio. This ratio is called the coefficient of diffusivity.

SIMULATION OF INDUCED RIVER FLUCTUATIONS

Analog simulation of stream-induced fiuctuations provides for refinement of field data on the ratio of transmissibility to storage. The procedure consists of (1) duplicating the river stage on the analog; (2) observing the induced fiuctuation in the analog; (3) comparing the analog and aquifer response; and (4) modifying the analog. These steps are repeated until the desired match is achieved between analog and aquifer response.

A diagram of the analog hookup used in simulating river-induced water levels is shown in figure 7. The variation of stream stage with time is represented as a voltage varying with time by the waveform synthesizer. This voltage function is imposed upon the stream boundary of the aquifer. In so doing, it is assumed that the hydrograph is identical in time and in head change at each point on the stream boundary. These assumptions are permissible where the length of the river involved is relatively short, and where the stage versus discharge relationship does not vary significantly. Where these conditions are not met, the stream hydrograph is simulated in parts for short lengths of the river that have similar hydrographs. This is accomplished by simultaneous use of several waveform synthesizers, or by successive simulation of the hydrograph along parts of the stream boundary and recomposing the aquifer response by superposition of the effects of individual stream segments.

A systematic process is followed in comparing analog and aquifer response. The comparison is expressed quantitatively as the ratio of the design diffusivity to the effective diffusivity of the

TABLE 1.—*Outline of procedures for analog definition of aquifer properties.*

Force	Data required	Method of analysis	Aquifer property determined or verified	
			Steady-state analysis	Nonsteady analysis
Pumping wells	Pumping rate, water-level response	Simulation of pumping, duplication of water-level response.	Transmissibility	Transmissibility and storage.
Recharge pits or wells.	Recharge rate, water-level response	Simulation of recharge, duplication of water-level response.	do	Do.
Stream-stage fluctuations.	Stream-stage hydrograph, ground-water level hydrograph.	Simulation of stream hydrograph, duplication of water-level response.		Ratio of transmissibility to storage.
Accretion	Accretion rate, water-level fluctuation	Simulation of accretion, duplication of water-level response.		Transmissibility and storage.
Do	Ground-water and base-flow recession curves.	Simulation of ground-water and base-flow recession curves.		Do.
Do	Steady-state piezometric map, average accretion or discharge rate.	Simulation of accretion, duplication of piezometric map.	Transmissibility.	

analog. Design diffusivity of the analog refers to the ratio of transmissibility to storage used in construction of the analog. The analog, however, is not limited to the design diffusivity. By altering the time constant, K_T, the analog diffusivity is also changed in an inversely proportionate ratio. For example, if K_T is changed from 5×10^{5} to 6×10^{5}, diffusivity of the analog is decreased by $5\times10^5/6\times10^5=0.83$. The effective diffusivity is the diffusivity of the analog, obtained by changing the time constant, K_T, which best matches the aquifer response. The ratio of design diffusivity to effective diffusivity is equal to the ratio of K_T used in design, to K_T used in obtaining the best analog match to aquifer response. In the preceding example, where K_T is changed from 5×10^{5} to 6×10^{5} to obtain an analog match to observed water-level fluctuations, the ratio of design to effective diffusivity is $1/0.83=1.2$. Altering the time constant, K_T, changes the diffusivity of the entire analog by the same factor. Generally, the effective diffusivity differs from one control point to another, because of relative errors in design from place to place within the analog. Recording the ratios of design to effective diffusivity (D_d/D_e) for each control point provides a basis for refining the design of the analog. The design diffusivity is decreased where D_d/D_e is less than one and is increased where D_d/D_e is greater than one.

SIMULATION OF ACCRETION TO THE AQUIFER

Simulation of steady-state water-level contours yields the ratio of accretion to transmissibility. Estimates of accretion made from base flow, or from rainfall, evapotranspiration, and soil type, afford a basis for establishing limits on transmissibility. The principal use of duplicating the piezometric contours on the analog is for determining areal variations in accretion, as discussed later in this report. This is a useful technique where the transmissibility has been defined by other methods.

The elemental analog configuration for simulating the piezometric surface is shown in figure 15. Accretion to the aquifer is simulated by electrical flow through resistors, R_a, connected in a rectangular pattern to nodes in the resistor network representing the aquifer. Voltage, V_1, is applied to R_a, so that the voltage, V_2, is proportional to the piezometric head at that point. The accretion rate, W, in feet per year, is determined by the following equation:

$$W = \frac{48.8K_\xi(V_2 - V_1)}{n^2 R_a},$$

where K_ξ is the scale factor relating gallons per day to amperes; n is the horizontal node spacing, in feet; and R_a is the resistance of the accretion resistor, in ohms.

The ratio of the known accretion to the modeled accretion is a measure of possible error in assumed transmissibility; a ratio greater than one, indicates the model transmissibilty may be too small, and a ratio less than one, indicates that the model transmissibility may be too large.

SIMULATION OF PUMPING WELLS

Simulation of pumping wells is probably one of the more common methods of refining data on the hydraulic characteristics of the aquifer. The procedure consists of (1) duplicating the pumping history; (2) observing the induced drawdown in the analog; (3) comparing the analog response with the aquifer response; and (4) modifying the analog to achieve a match between the analog and the aquifer responses.

The analog hookup in simulating nonsteady well pumping is shown in figure 15. The well-field discharge is simulated by the current flow through resistor R_p, induced by the pulse generator. The output of the pulse generator is a step function, shown graphically in figure 15, which simulates one cycle of pumping at a constant rate.

Two or more pulse generators connected in series to pulse sequentially can be used to represent a series of steps in the pumping rate. A waveform synthesizer can be used to simulate very complex pumping histories. A single nodal point in an analog may represent the position of one well or of several wells, depending upon the scale of the analog and the locations of the wells. Where a node represents more than one well, the pumpage from the wells is composited to make a single pumping history.

A separate waveform source (pulse generator(s) or waveform synthesizer) is required for each pumping history that is to be simulated. A pumping history common to two or more points can be generated by the waveform source and fed to each point through separate well resistors, R_p, (eq. 7). If complex pumping histories require more waveform sources than are available for simultaneous modeling of all pumping centers, the analysis can be made by imposing the pumping history node by node and determining the total response by superposition.

Where steady-state conditions are created by pumping, the pulse generator can be replaced by a direct-current voltage source, and the aquifer response can be observed with a direct-current voltmeter.

The process of comparing the analog response with that of the aquifer and modifying the analog components is similar to that used in analyses involving stream-induced fluctuations. The verification procedure of duplicating the response of the aquifer to pumping wells allows for separate evaluation of transmissibility and of storage of the aquifer.

A systematic process should be followed in comparing analog response with aquifer response. The comparisons can be expressed quantitatively as the ratio of design transmissibility to effective transmissibility, and the ratio of design storage to effective storage. The analog response is matched to the aquifer response by two adjustments: First, changing K_ξ by adjusting the voltage on the well resistor, so that the curves coincide at points of greatest drawdown; second, changing K_T, the scale factor relating time in the analog to time in the aquifer, so that the response curves coincide at early times during the pumping cycle. This process is repeated until the best match is obtained between the analog and aquifer-response curves. The ratio of design transmissibility to effective transmissibility, T_d/T_e, s equal to the design K_ξ divided by the effective K_ξ. The ratio of the design storage to effective storage, S_d/S_e, is equal to the ratio of the design $K_\xi K_T$ to the effective $K_\xi K_T$. Recording these ratios for each control point provides a basis for refining the design of the analog. The design transmissibilty should be decreased where T_d/T_e is greater than one; and the design storage should be decreased where S_d/S_e is greater than one.

VERIFICATION OF ARKANSAS AND VERDIGRIS RIVER VALLEY ANALOGS

Combinations of the procedures shown in table 1 were used for verifying analogs of the aquifers in the Arkansas River valley. The analog of the Ozark area is used for illustration.

The Arkansas River alluvium at Ozark comprises about 1½ square miles of bottom land south of the Arkansas River in Franklin County, Ark. (pl. 2). The alluvial section ranges from about 50 to 60 feet in thickness. Depths to water, where unaffected by pumping, range from 38 feet on the crest of the natural levee to 25 feet in the backswamp near the valley walls. Ground water is unconfined in most of the area. The water table is generally in the sand below the fine-grained top stratum, except near the valley wall, where the fine-grained material attains its greatest thickness.

The aquifer in the area is tapped by six wells, which individually yield from 30 to 240 gallons per minute. These wells are pumped intermittently to supply about 300,000 gpd used by Ozark.

The design of the analog was based on transmissibility, calculated from pumping tests at five of the city wells, and on estimates of transmissibility from test-hole logs. Laboratory tests of test-hole samples were utilized in defining a relation between grain size and permeability. The use of this relation in estimating transmissibility from test-hole logs is described by Bedinger (1961).

As indicated on plate 2, the piezometric surface in most of the area is in sand, and the aquifer is unconfined. Laboratory analyses of samples from the area showed that specific yield averages about 0.30. Pumping tests of a few hours' duration yielded storage coefficients ranging from 2×10^{-3} to 7×10^{-2}. Experience has shown that the specific yield determined in the laboratory is generally high and that the coefficient of storage observed in short-term field tests is generally lower than is appropriate for long-term changes in head. The type and size of the material at the upper boundary of the aquifer suggest that a storage coefficient within the range commonly observed in water-table aquifers (0.05 to 0.30) would be appropriate for the analog design. Therefore, a design storage coefficient of 0.2 was assigned to the analog.

The analog was constructed at a scale of one resistor per 150 feet.

Verification of the Ozark analog comprised three analyses. The first analysis consisted of duplicating the river-induced fluctuations of ground-water levels. The second analysis consisted of simulating nonsteady response of water levels due to pumping wells. The third analysis consisted of simulating steady-state piezometric levels due to average pumping rate, recharge rate, and river stage.

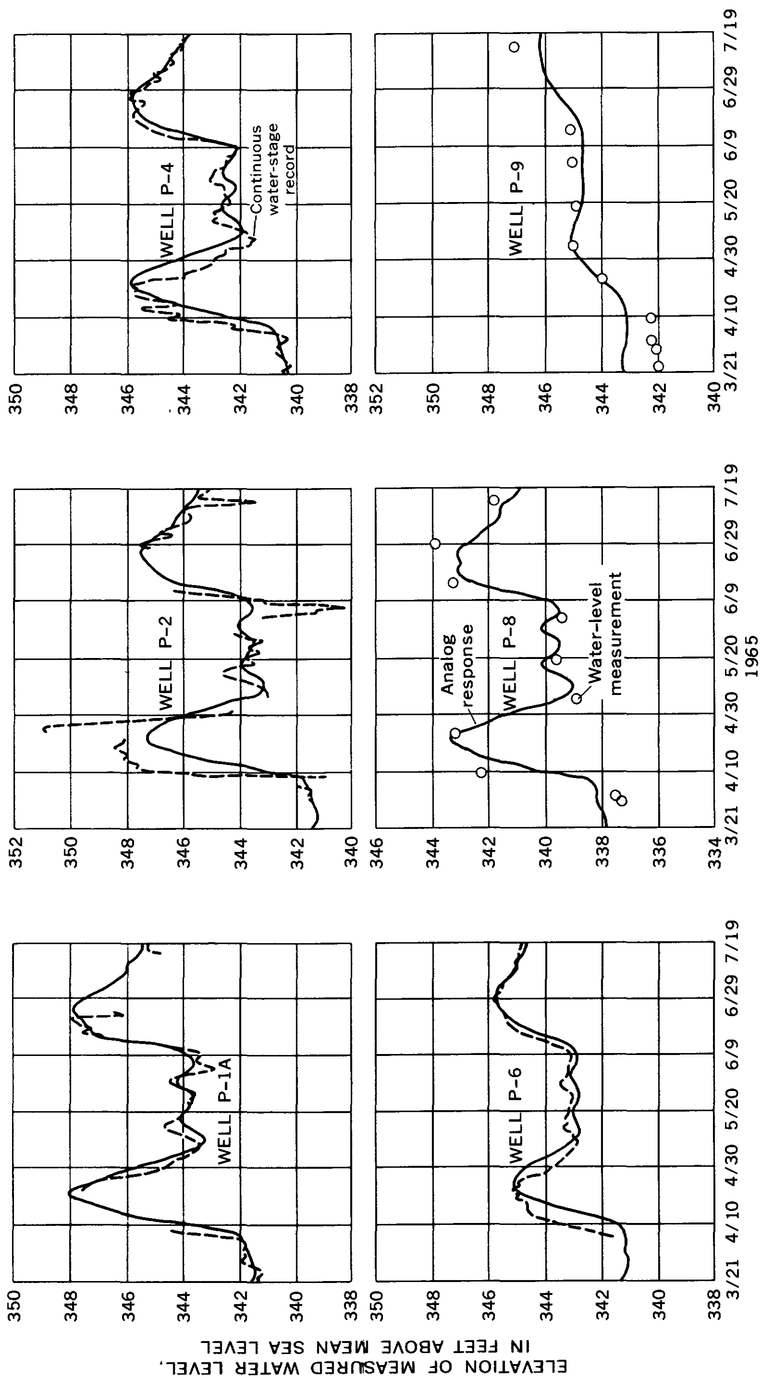
The river hydrograph was simulated using a waveform synthesizer (fig. 11). The river hydrograph of 250 days' duration was represented by 50 segments, each of which is the average stage for 5 consecutive days. Beginning with the design transmissibility and storage, the analog response was recorded at points corresponding to the location of observation wells in the aquifer. At points where the analog response and observed water-level fluctuation were at variance, the design diffusivity of the analog was changed to obtain an effective diffusivity. The ratios of design to effective diffusivities guided refinement of the design properties of the analog, as discussed previously in this report.

The analog response to river-stage fluctuations from the verified analog and the water levels measured in the aquifer are shown in figure 19.

The second step in verifying the Ozark analog was simulation of nonsteady conditions near pumping wells. This type of analysis provides for independent appraisal of the design coefficients of transmissibility and storage. As a result of this analysis, among other modifications of the analog, the storage coefficient of the aquifer was changed from 0.20 to 0.15.

The third step of analog verification consisted of simulating the average piezometric surface, considered to represent the steady state observed from March through November 1965. The forces acting upon the aquifer included principally: (1) The stage of the river; (2) pumpage from the city wells; and (3) accretion to the aquifer. These forces were modeled. The average accretion imposed on the analog was 3 inches per year—a rate which is probably reasonable, considering the lithology of the alluvium above the water table and the experience gained in determining recharge in other parts of the Arkansas River valley.

A verified model is valid only for a given set of conditions or for the given ranges of forces. In few instances, however, will analysis of the problem be restricted to the range of boundary conditions for which the analog was verified. Problem solving may include projection of drawdown for pumping rates and perhaps for placement of wells other than those used in analog verifications. Stream-stage changes may be altered to fluctuate in a different range, or a fluctuating stream stage may be changed by a dam to a relatively stable, higher than average stage. The flow system may function differently in response to stresses acting in ranges beyond those observed during verification. These are examples of problem solving for which the model may not be completely verified. If observed conditions include small temporary fluctuations of water level, evapotranspiration may be relatively constant for the range and time experienced. However, a permanent large change in water level may significantly affect evapotranspiration. Likewise, drawdown from pumping in excess of that observed and used in verification may cause com-


WELL P-4
Continuous water-stage record
WELL P-2
WELL P-1A
WELL P-9
WELL P-8
Analog response
Water-level measurement
WELL P-6
3/21 4/10 4/30 5/20 6/9 6/29 7/19
1965
ELEVATION OF MEASURED WATER LEVEL, IN FEET ABOVE MEAN SEA LEVEL

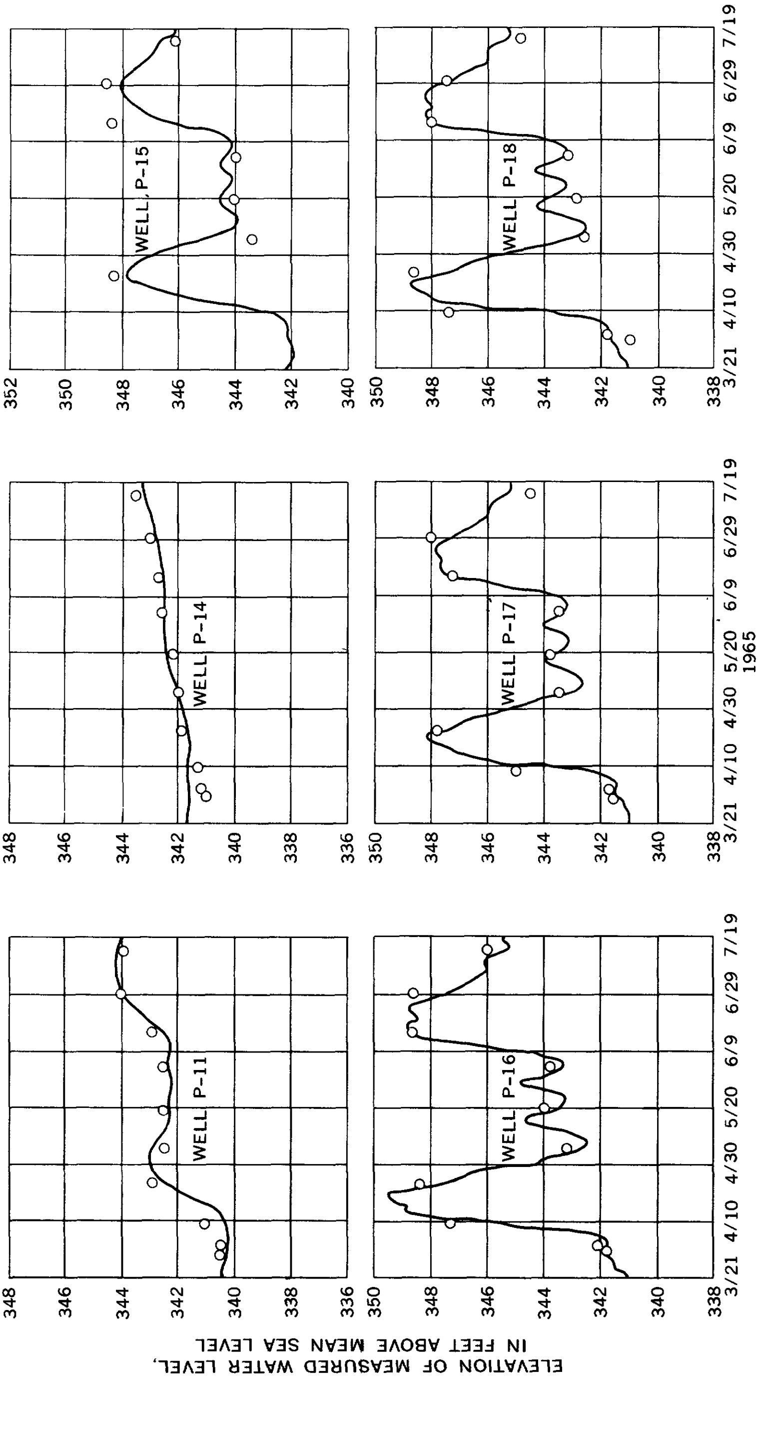


FIGURE 19.—Measured water levels in the Ozark area, Franklin County, Ark., and analog response to fluctuating river stage. Location of wells shown on plate 2.

paction of earth materials and drainage of water from clays, or may cause dewatering of the aquifer and a reduction in transmissibility. A verified model is valid only for the range of boundary conditions imposed during verification. Thus, the hydrologist cannot be content with a verified analog; rather, in designing the analog, he must consider the operation of the hydrologic system beyond the range of verified conditions.

PLANNING DATA COLLECTION FOR DESIGN AND VERIFICATION OF ANALOGS

Data-collection programs must provide for quantitative description of the aquifer and its boundaries, definition of forces acting upon the system, and measurement of the response of the aquifer to these forces. Where possible, methods independent of model analysis should be used to design the analog. The use of independent methods, such as pumping tests, laboratory tests, and test drilling, to determine the hydrologic properties of the system will facilitate analog verification.

The data required for all methods of verification are the definition of forces acting upon the aquifer (accretion, pumping, fluctuating river stage), and the response of water levels in the aquifer to the forces. The frequency of data needed on each of these elements depends upon (1) the magnitude, duration, and variability of the force acting upon the system, (2) the distance of the water-level observation point from the boundary, and (3) the diffusivity of the aquifer. Because the aquifer response is a function of antecedent river stages, a longer history is required of the forces acting upon the boundaries than of the aquifer response. The areal density of data should be compatible with the scale of the analog.

ANALOG ANALYSIS OF STEADY GROUND-WATER FLOW

EQUILIBRIUM PIEZOMETRIC SURFACE

The aquifer is acted upon by continually varying forces such as recharge, evapotranspiration, and fluctuating stream stages. The action that these natural and any manmade forces impose upon the system preclude establishment of equilibrium of the piezometric surface. However, in many instances the equilibrium condition is represented by the average position of the piezometric surface taken from observations made during a long period of time.

In selecting the time period for which the average water table is determined, the following should be considered: (1) The characteristics of the force acting upon the boundaries; that is, whether the force is repetitive, or cyclic, within a given range of magnitude or whether it continually increases or decreases; and (2) the timelag between ground-water boundary and fluctuations.

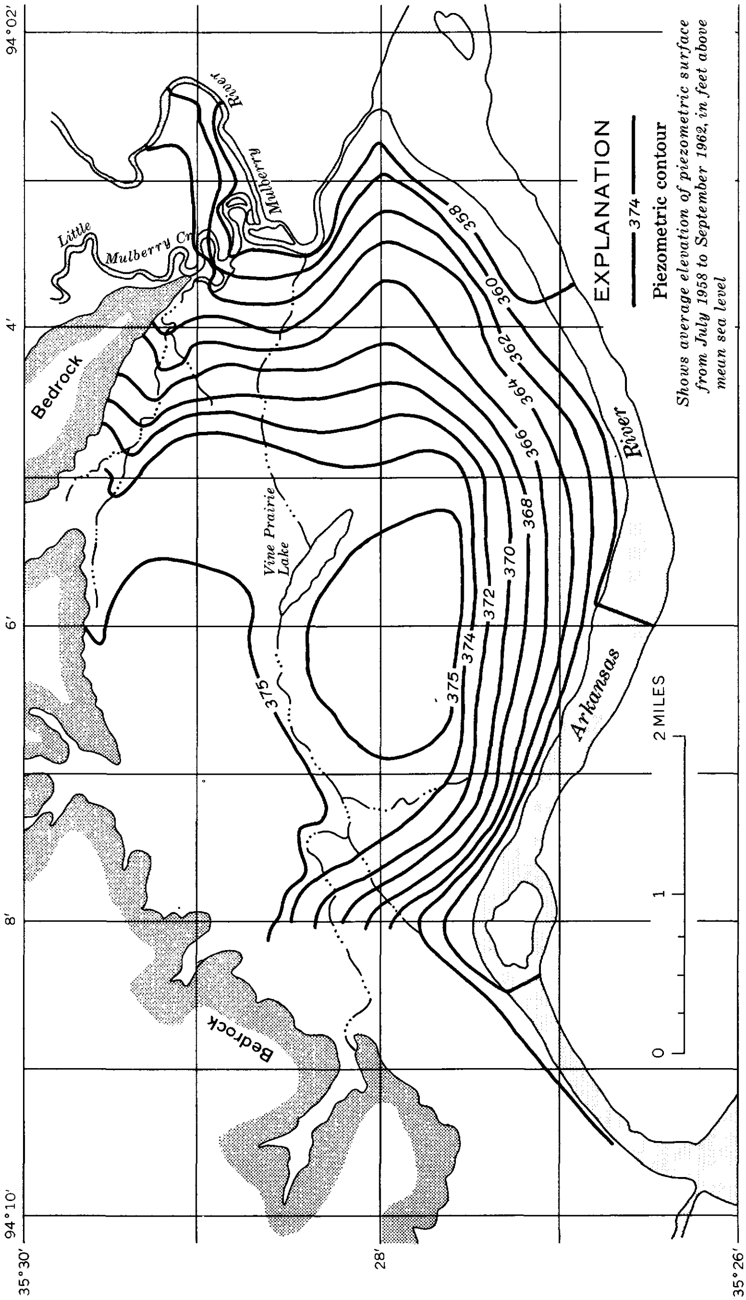
In the alluvium of the Arkansas River valley, the boundary forces include river-stage fluctuations, accretion, and pumping wells. Each of these forces is repetitive in nature; however, the magnitude and frequency of individual fluctuations vary. Empirical guidelines used in preparing maps for steady-state analysis in the Arkansas River valley are as follows: (1) The time period should include several cycles; and (2) the timelag between the aquifer response and boundary forces should be small, compared with the length of the period chosen for determining the average position of the piezometric surface.

DETERMINATION OF ACCRETION

Accretion can be determined by analog simulation of the average piezometric surface by use of the analog configuration shown in figure 17. The steady-state piezometric surfaces and simulated piezometric surfaces in two alluvial areas are shown in figures 20 and 21. Analog simulation of the piezometric surfaces in these areas yields average accretion rates of 0.34 foot per year for the Mulberry-Dyer Bottoms and 0.32 foot per year for the Morrilton East Bottom. Accretion rates at individual points range from −0.4 to 2.0 feet per year.

RELATION OF AREAL VARIATIONS IN ACCRETION TO HYDROGEOLOGIC FACTORS

Several factors are responsible for variations in accretion. Among those having the greatest effect are the lithology of surficial materials overlying the aquifer and the depth to ground water. Undoubtedly, land use and vegetal cover also affect accretion. In the areas studied 70 to 80 percent of the land is under cultivation, the rest is occupied by woods or grass, or is bare. The principal crops are soybeans, cotton, and alfalfa. Locally, truck crops are grown. Because the cropping pattern changes from year to year and the land-use grid is small compared with the grid used in the analog, the effects of land use on accretion cannot be determined from available data.


94°02′
4′
6′
8′
94°10′
35°30′
28′
35°26′
Mulberry River
Little Mulberry Cr
Bedrock
Bedrock
Vine Prairie Lake
Arkansas River
358
360
362
364
366
368
370
372
374
375
375
EXPLANATION
374
Piezometric contour
Shows average elevation of piezometric surface from July 1958 to September 1962, in feet above mean sea level
0
1
2 MILES

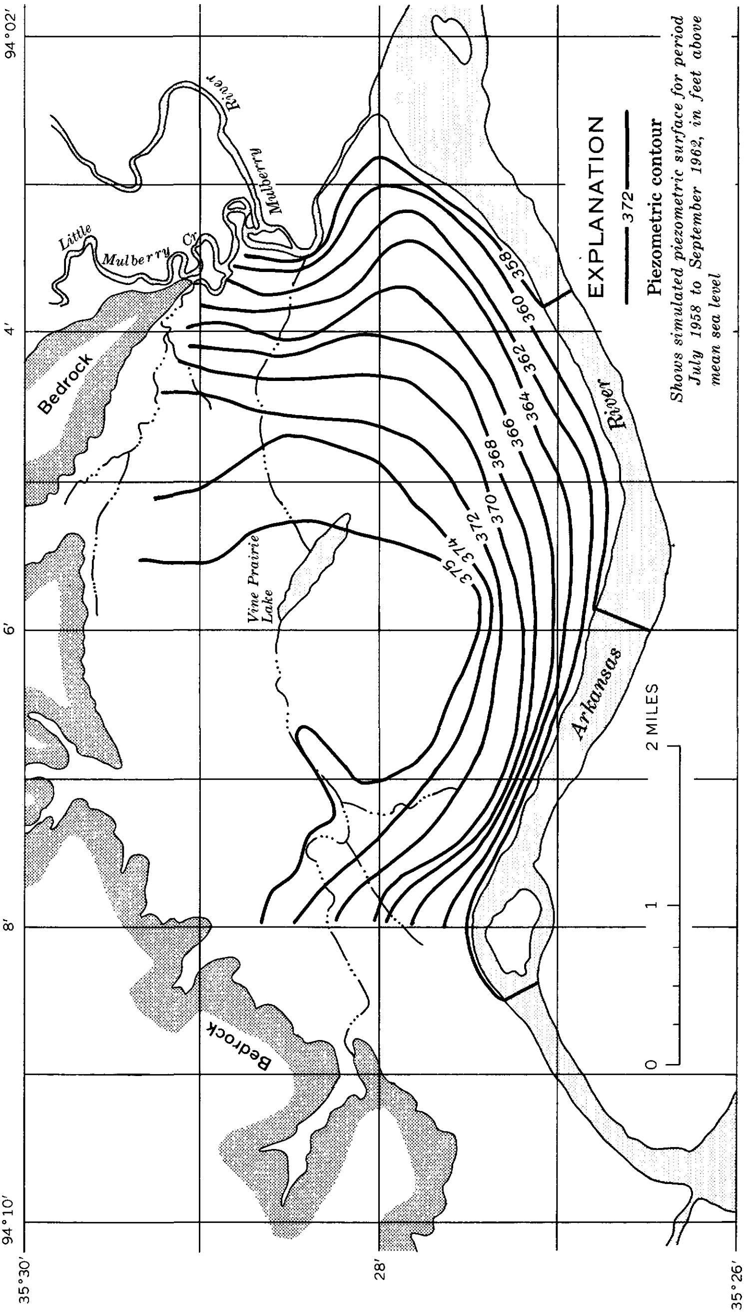
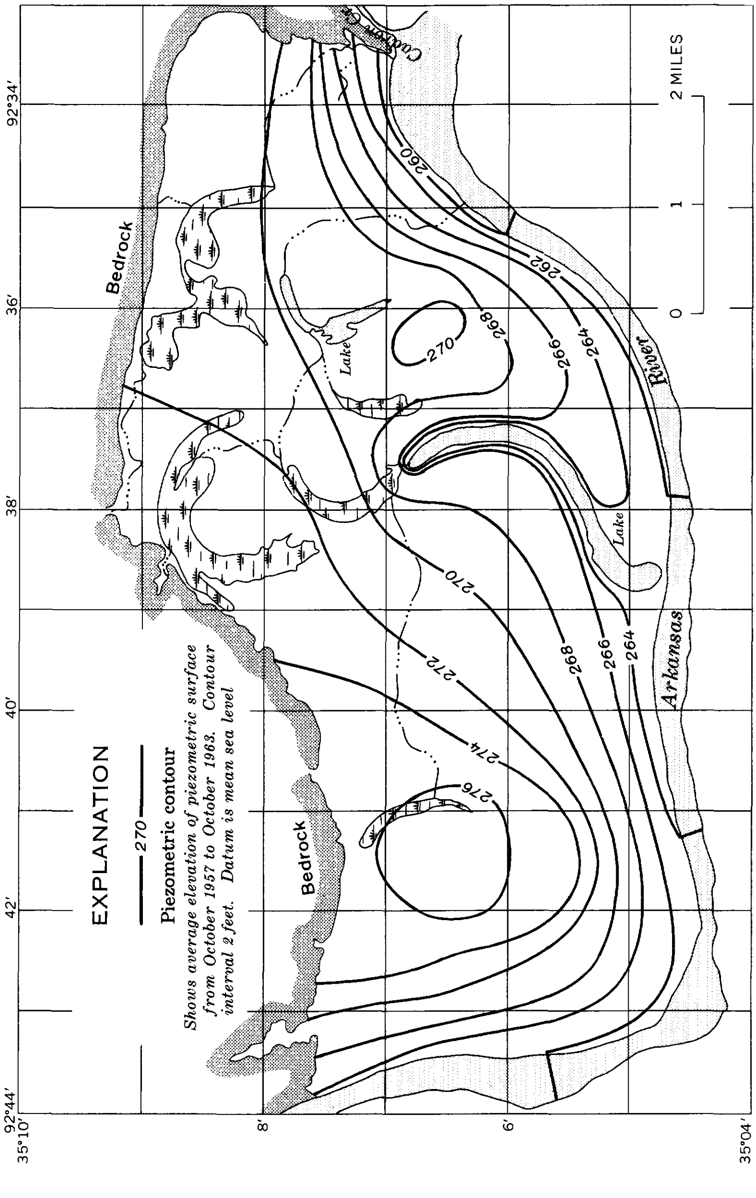


FIGURE 20.—Measured and simulated piezometric surfaces in the Mulberry-Dyer Bottoms, Crawford County, Ark.


EXPLANATION
270
Piezometric contour
Shows average elevation of piezometric surface from October 1957 to October 1963. Contour interval 2 feet. Datum is mean sea level
Bedrock
Bedrock
Lake
Lake
Arkansas
River
Cadron Cr
0 1 2 MILES
260
262
264
266
268
270
272
274
276
92°44′
35°10′
42′
40′
38′
36′
92°34′
8′
6′
35°04′

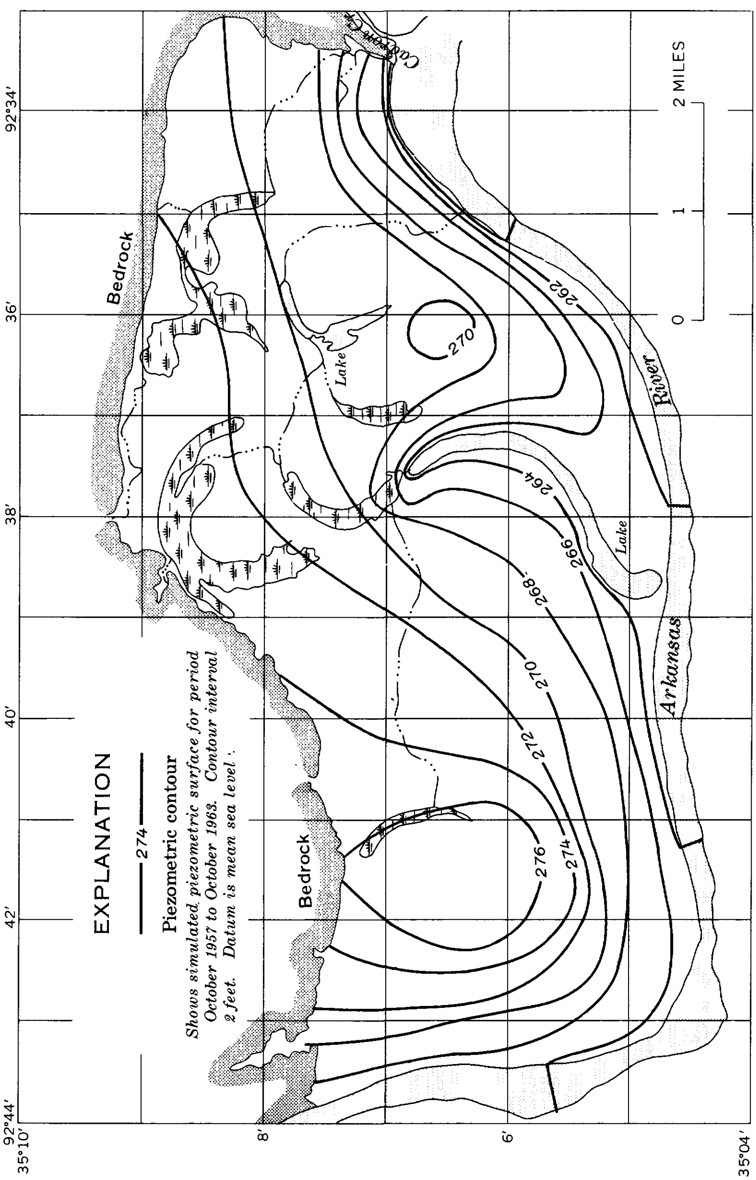


FIGURE 21.—Measured and simulated piezometric surfaces in the Morrilton East Bottom, Conway County, Ark.

The surficial material in the Arkansas River valley can be differentiated into several geologic types. Definition of these types is based upon mode of deposition (Bedinger and Reed, 1961). The types include point-bar, natural-levee, backswamp, channel-fill, and swale deposits. Of these, point-bar, natural-levee, and backswamp deposits are areally extensive, and accretion rates in these deposits have been computed. The point-bar deposits are the coarsest grained of the alluvial types. Accretion to point-bar deposits ranges from -0.6 to 2.5 feet per year. Natural-levee deposits are laid down near the stream and generally comprise the coarsest particle sizes deposited by overbank flows. Backswamp deposits are laid down at greater distance from the stream and comprise the finest particle sizes of the sediment in the overbank flow. Accretion in natural-levee deposits range from −0.2 to 1.4 feet per year; accretion rates in backswamp deposits range from −0.3 to 0.5 foot per year.

Plate 3 shows the surficial geology, average piezometric surface, and accretion rates in the McLean Bottom.

Graphs of the relation between accretion and depth to water are shown in figures 22 and 23. The scatter in the plots is probably caused largely by the lithologic variations within the respective geologic types. Lithologic variations in the geologic types can be compared by using equivalent thicknesses of silty clay. Silty clay equivalents were determined from test-hole logs by the following relationships based on permeability of silty clay and the indicated lithologic type.

Material	*Conversion factor* [1]
Silty sand	0.5
Sandy silt	.2
Clay silt	.3
Silty clay	1
Clay	3

[1] Factor to convert 1 ft of the indicated material to an equivalent thickness of silty clay.

Expressed in silty clay equivalents, the point-bar deposits range in thickness from 1 to 17 feet and average about 2.5 feet; the natural-levee deposits range in thickness from 1 to 36 feet and average 9.5 feet; and the backswamp deposits range in thickness from 1 to 89 feet and average 24 feet.

The change in accretion with change in water levels is particularly significant in the analysis of regional water levels.

The decrease in accretion with decreasing depth to water is attributed largely to greater evapotranspiration from the aqui-

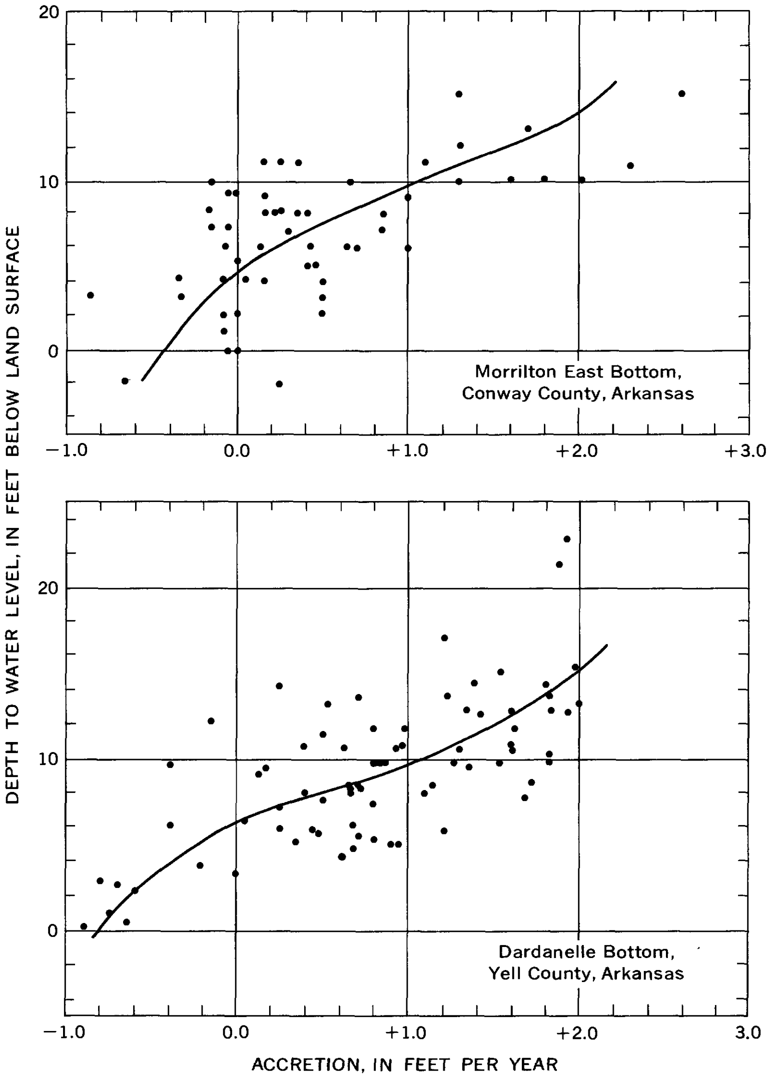


FIGURE 22.—Relation between depth to water and accretion in point-bar deposits.

fer where water levels are nearer the land surface. Using Darcy's law, Stallman and Reed (1966) developed a method for determining the relation between depth to the piezometric surface and evapotranspiration from the aquifer through an aquitard. This relation is shown schematically in figure 24. Evapotranspiration computed by this method should be considered potential

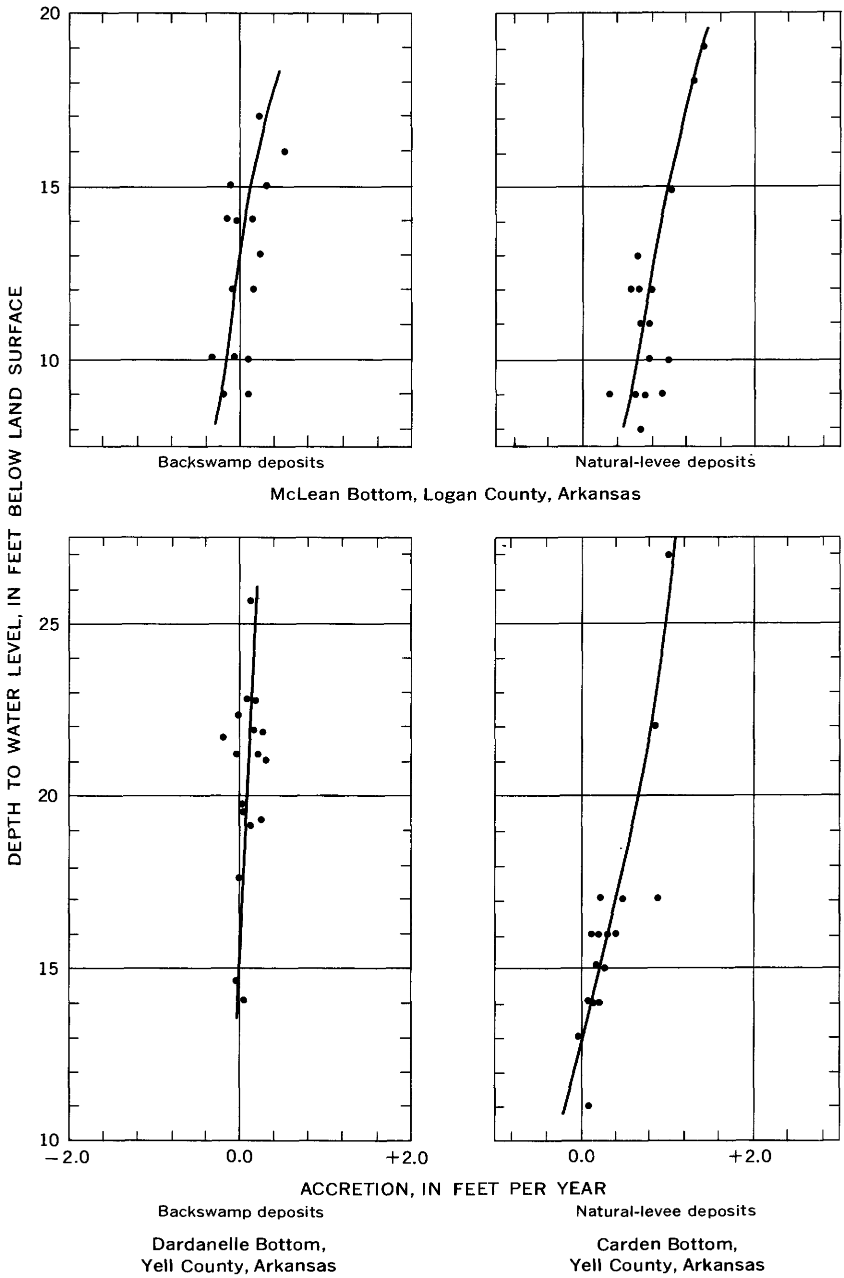


FIGURE 23.—Relation between depth to water and accretion in natural-levee and backswamp deposits.

evapotranspiration because it represents the maximum rate of movement under conditions of maximum hydraulic potential and steady flow.

Recharge to the water table is generally seasonal, occurring largely in the winter and spring. Evapotranspiration occurs mostly during the summer and fall. However, recharge and evapotranspiration are by no means limited to these seasons. Movement of water in the zone above the water table varies constantly, depending upon the weather, depth to water, and the lithology of the surficial material overlying the aquifer. Evapotranspiration from ground water is less than the potential evapotranspiration. However, curves of potential evapotranspiration, as shown in figure 24, illustrate that for a given material, evapotranspiration decreases with increasing depth to water. Evapotranspiration ranges from a maximum when the water level is near

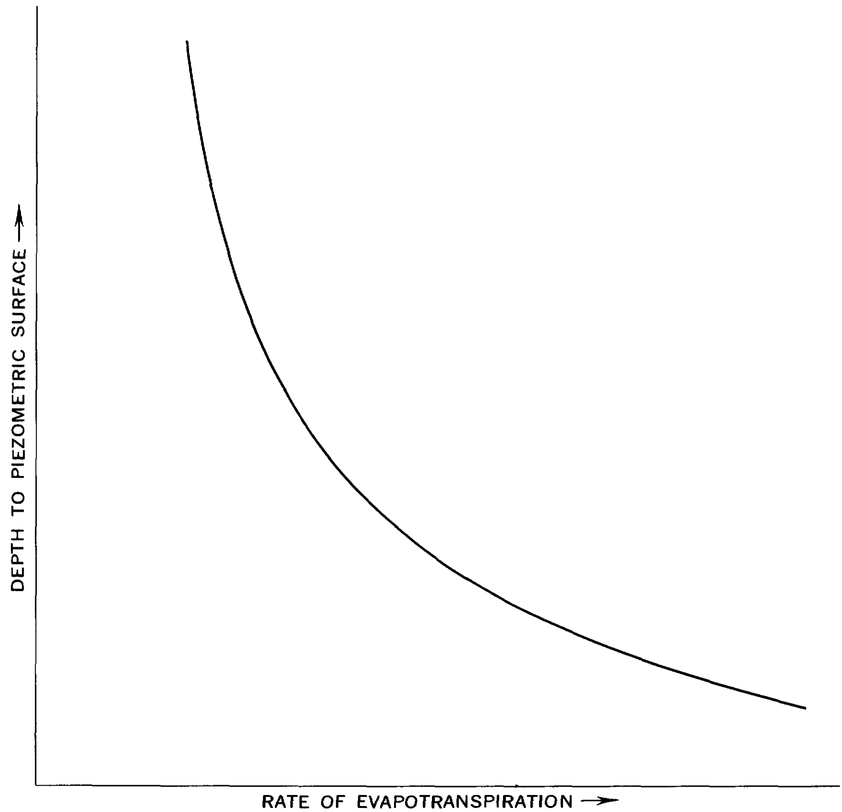


FIGURE 24.—Change in evapotranspiration with change in piezometric surface.

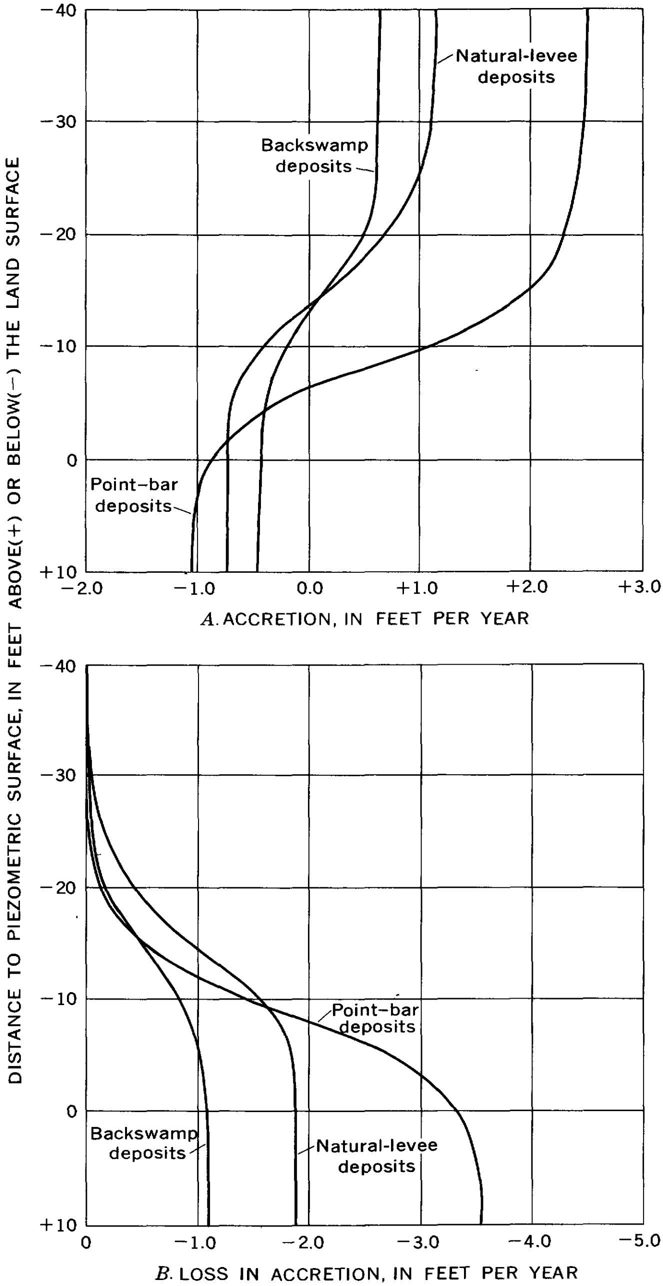


FIGURE 25.—Relation between accretion and depth to water.

the surface, to a negligible rate when the water level is at great depth. This relationship is shown also by observations of accretion made at various depths to water.

Generalized graphs taken from the observed accretion versus depth to the piezometric surface in figures 22 and 23 are shown in figure 25*A*. The upper ends of the curves shown in this figure approach a vertical line approximating the maximum rate of accretion. The maximum rate is limited by the lithology of the materials and the climate. The middle parts of the curves show a decrease in accretion with a decrease in depth to water. This decrease in accretion is attributed largely to the increase in evapotranspiration from the aquifer with decreasing depth to water, although other factors, such as rejection of recharge, where the zone of saturation is at the land surface, contribute to the decrease in accretion. The lower ends of these curves approach the vertical at a point near the maximum rate of negative-accretion factors. Following this reasoning, the curves in figure 25*A* can be transposed to the coordinates of figure 25*B* to yield curves of decrease in accretion versus depth to water.

EFFECT OF STREAM IMPOUNDMENT ON THE GROUND-WATER FLOW SYSTEM

CHANGES IN FLOW SYSTEM CAUSED BY STREAM IMPOUNDMENT

Dams on the Arkansas River will alter the natural river-discharge level of ground water. In response to the change in discharge level, water levels in the aquifer will adjust to provide the gradient necessary for discharging the natural ground-water recharge. The change in water levels will, in turn, affect the accretion to, and the transmissibility of, the aquifer. Accretion will be reduced in areas where water levels are raised and will be increased where water levels are lowered. The rise or fall in ground-water levels resulting from direct changes in stream stage, thus will be attenuated by the changes in accretion. The alluvial aquifers in the lower Arkansas River valley generally grade upward from coarse sand and gravel near the base of the aquifers to very fine sand, silt, or clay at the water table. The changes in saturated thickness of the aquifer, resulting from impoundment of the river, will take place in the fine-grained material of low permeability at the top of the aquifer. The resulting change in transmissibility will be a negligible part of the preimpoundment transmissibility. Therefore, no adjustment in transmissibility of the aquifer is required in the analog study of the effects of stream impoundment.

The changes in the aquifer due to a permanent change in stream stage can be illustrated by considering the component forces which determine the configuration of the piezometric surface. For example, the alluvial aquifer, shown in cross section in figure 26, is bounded by a stream. Impermeable rocks underlie the aquifer and form the valley walls. The upper surface of the aquifer is a boundary through which water is received by infiltration of rainfall and is lost by evapotranspiration. This gain and loss of water through the upper surface of the aquifer is included in the term "accretion." Discharge from the aquifer is by seepage to the stream.

The piezometric surface in the aquifer can be considered the sum of two components of head—the boundary and accretion components (Reed and Bedinger, 1961). The boundary component of head is determined by the type and configuration of the lateral boundaries of the aquifer and by the head distribution on these boundaries (fig. 26). Alone, this component would create a water-table configuration for zero accretion to the aquifer. The accretion component of head is determined by the rate of accretion, the transmissibility, and the shape and nature of, and head on, the lateral boundaries of the aquifer.

Consider a change in stream stage from level A to level B resulting from the construction of a dam on the stream (fig. 27*A*).

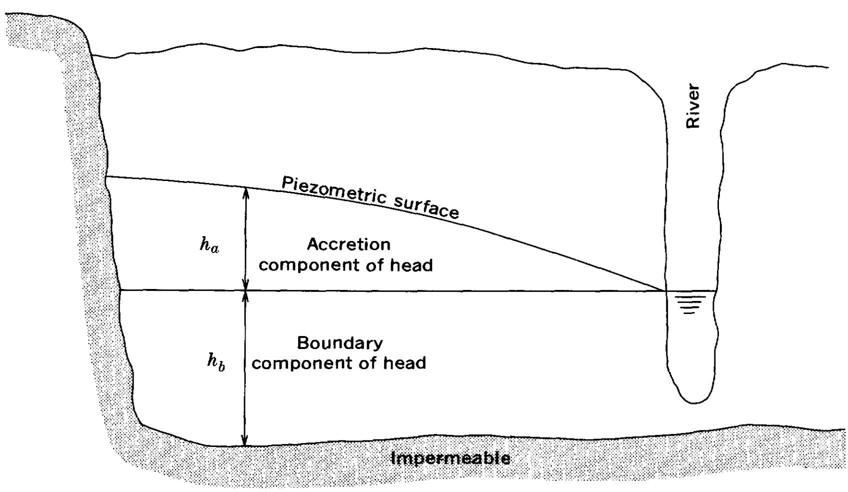
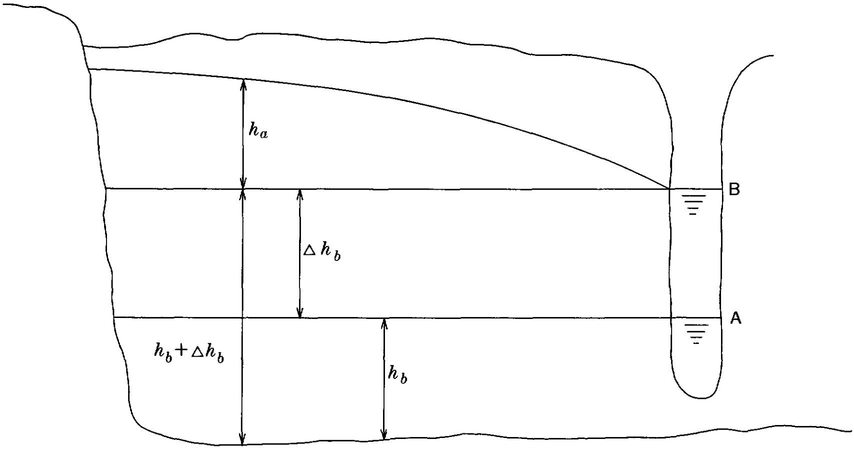
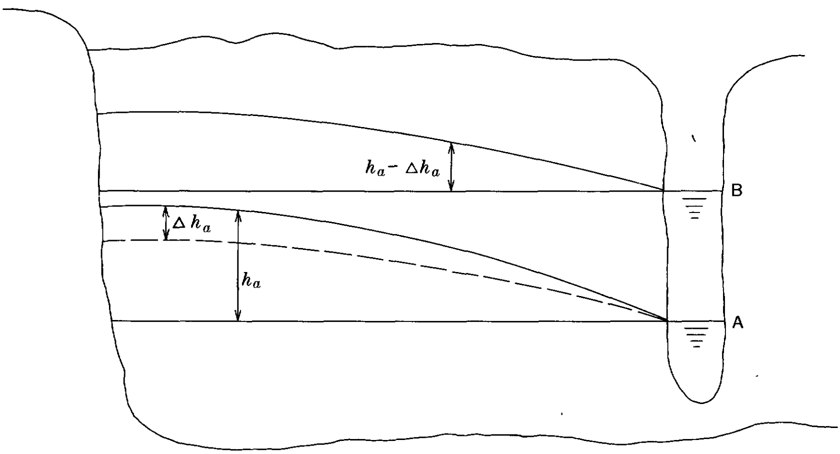


FIGURE 26.—Boundary and accretion components of head in an alluvial aquifer.



A. A permanent change in river stage imposes a direct change, Δh_b, in boundary componert



B. The rise in water level increases the evapotranspiration and reduces the accretion component by Δh_a

FIGURE 27.—Response of boundary and accretion components of head to a permanent change in stream stage.

This change effects a direct change in the boundary component of head. After equilibrium is reached, the boundary component in the aquiger is increased by Δh_b.

In many instances, the change in boundary component will be accompanied by a change in accretion component. The preconstruction accretion component is shown in h_a in figure 27*B*.

As the ground-water levels are raised in response to the change from A to B in stream stage, there is greater evapotranspiration and a consequent decrease in accretion to the aquifer. When equilibrium is reached with stream stage B, the accretion component is reduced by Δh_a. The postconstruction accretion component therefore is equal to $h_a - \Delta h_a$.

ANALOG ANALYSIS OF EFFECTS OF STREAM IMPOUNDMENT

Projection of the postconstruction piezometric surface involves analysis of the direct effect imposed by both the change in river stage and the resultant change in evapotranspiration and consequent change in rate of accretion. The analog configuration used to analyze the change in head in the aquifer is shown in figure 28.

The preconstruction base, or datum for the projection, is the average piezometric surface. The change in river stage imposed on the analog is the difference between the averages of preconstruction and postconstruction river stages. The voltage applied

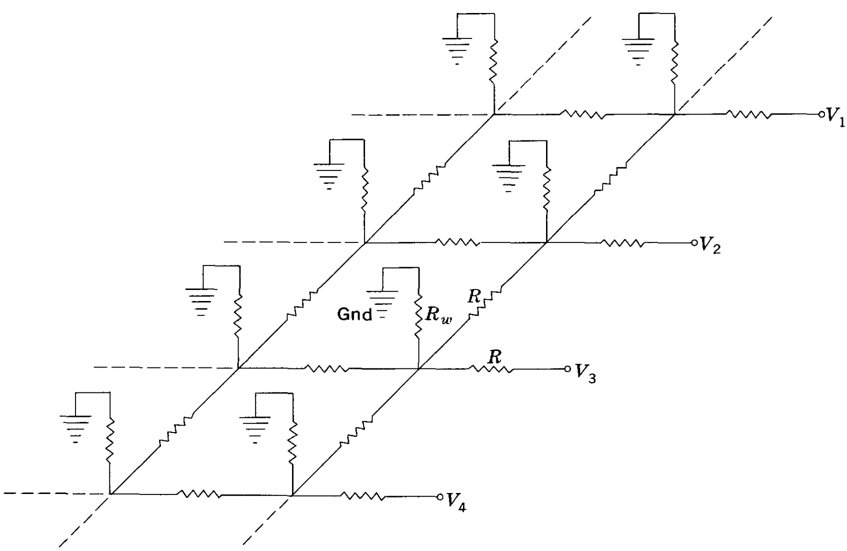


FIGURE 28.—Schematic wiring of analog configuration to analyze steady-state effect of change in stream stage. V_1 through V_4, voltage input, with reference to ground voltage on river boundary proportional to change in stream stage; R, aquifer-network resistor; R_w, accretion resistor; Gnd, ground (zero voltage) connection.

to the river boundary is scaled to the head change in the river by the equation:

$$V=\frac{h}{K_\phi}$$

where V is electrical potential, in volts; h is hydraulic head, in feet; and K_ϕ is the analog-scale factor relating feet to volts.

Graphs, such as those in figure 25, are used in designing analog components that simulate the relation between the change in accretion and depth to water. Two points are located on the graphs—the present depth to piezometric surface and an estimate of the projected depth to piezometric surface. The change in accretion per foot of water-level change is determined from these two points.

The ratio is analogous to the flow of electricity in a resistor, as the current is proportional to the voltage across the resistor. The resistors representing the accretion paths are connected by their proximal leads to the aquifer net and their distal leads are grounded. The resistance used for simulation is determined by equation 8.

An initial estimate of the projected piezometric water level is needed to determine $\Delta W/h$. This estimate is used to determine initial values of $\Delta W/\Delta h$ for the first model analysis. The head change from the first analysis is compared with the values of head change used for calculating the initial $\Delta W/\Delta h$. Values of $\Delta W/\Delta h$ are then revised as needed, and a second analog analysis is made. The above process is repeated until Δh used in calculating $\Delta W/\Delta h$ is sensibly equal to Δh from analog analysis.

PROJECTED PIEZOMETRIC SURFACE IN THE DARDANELLE POOL AREA

Projections of the average postconstruction ground-water levels have been made for the alluvial areas adjacent to the navigation pools in the lower Arkansas and Verdigris Rivers, Ark. and Okla. Dardanelle Dam, at Dardanelle, Ark., was completed in 1964, and the reservoir began filling on October 1, 1964. The nominal top of the power and navigation pool, 338 feet mean sea level, was reached in February 9, 1965. The alluvial areas affected by the Dardanelle Pool are the Hartman–Coal Hill Bottoms in Johnson and Franklin Counties, Ark., and the McLean Bottom in Logan County, Ark. (fig. 29).

Water-level changes in the vicinity of the Dardanelle Pool have been monitored closely since filling of the pool. Because of

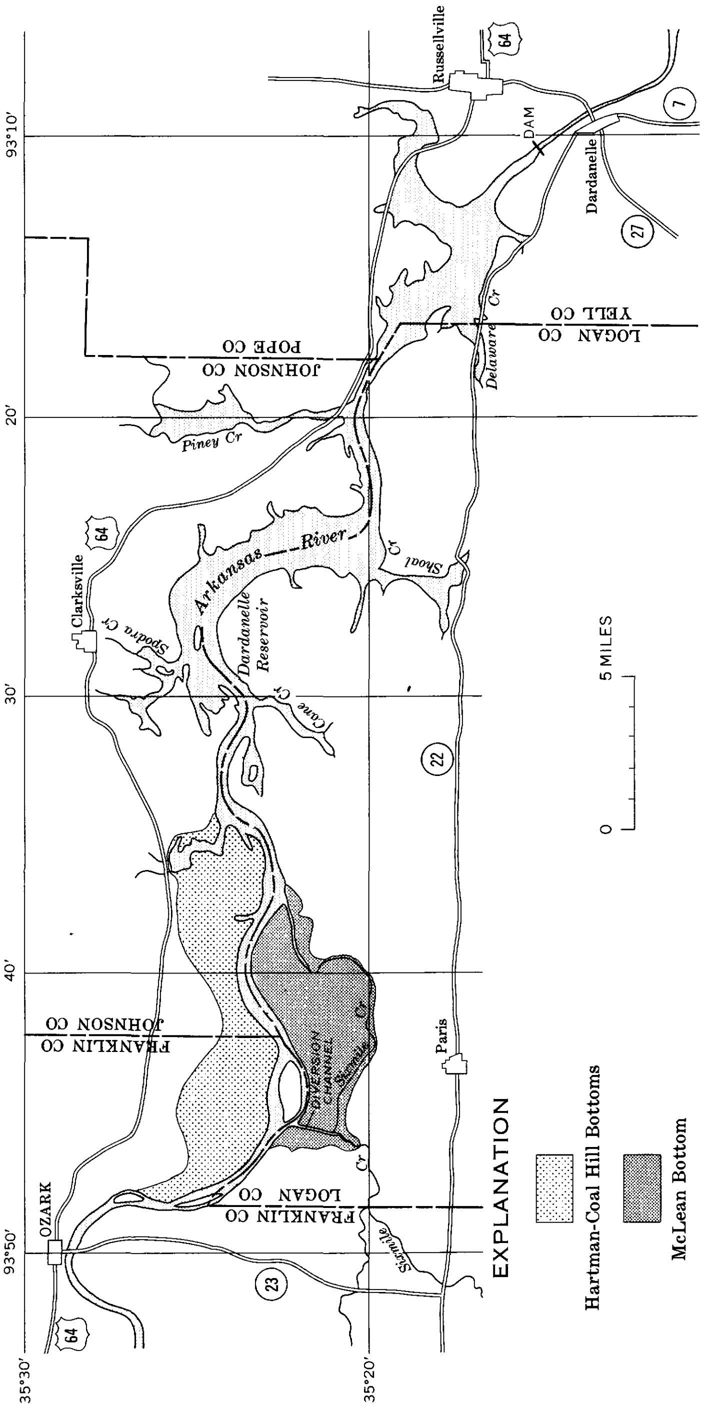


FIGURE 29.—Location of Dardanelle Dam and Reservoir, Ark.

the aquifer's resistance to flow of water and its water-storage characteristics, there is a pronounced timelag in the response of the aquifer to the changes in river stage. To date (1968) the water levels are not in equilibrium with the Dardanelle Pool. The analog analysis of the timelag response of the aquifer to the change in river stage at various distances from the river is shown in figure 30. The hydrographs in figure 5 indicate that between 30 and 95 percent of the ultimate river-induced change had taken place by the spring of 1966, and between 54 and 96 percent had taken place by the spring of 1967. Further study of analog response to the change in pool stage indicates that complete adjustment of the ground-water flow system to the Dardanelle Pool will take as long as 10 years after filling of the pool.

Water-level fluctuations in wells since closure of Dardanelle Dam are shown in figure 31. These wells record the aquifer response to the change in river stage and, also, to fluctuations

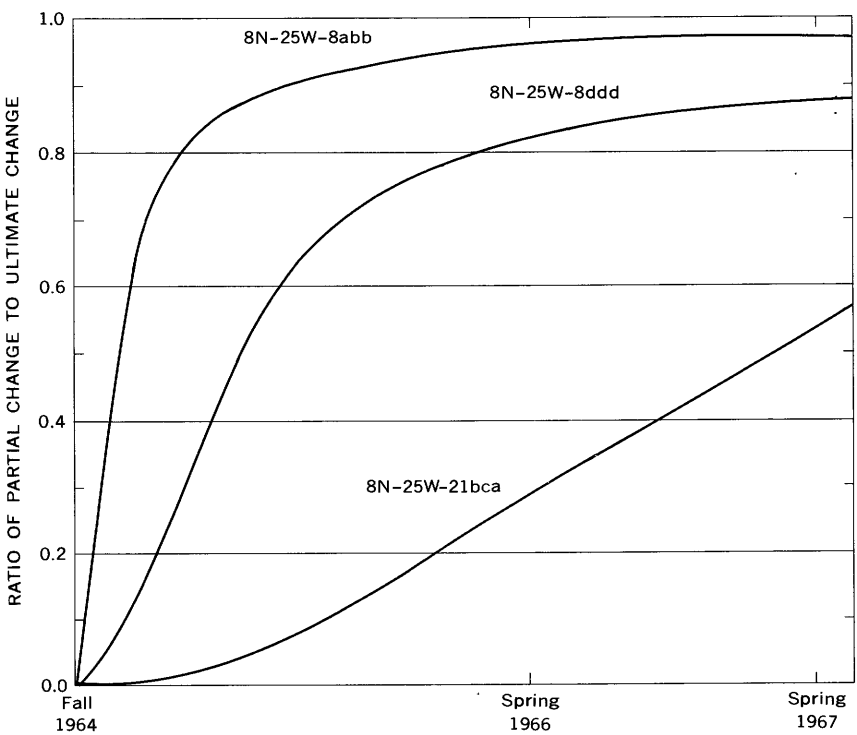


FIGURE 30.—Response of the aquifer to change in river stage in the McLean Bottom, Logan County, Ark. Well locations are shown on plate 3.

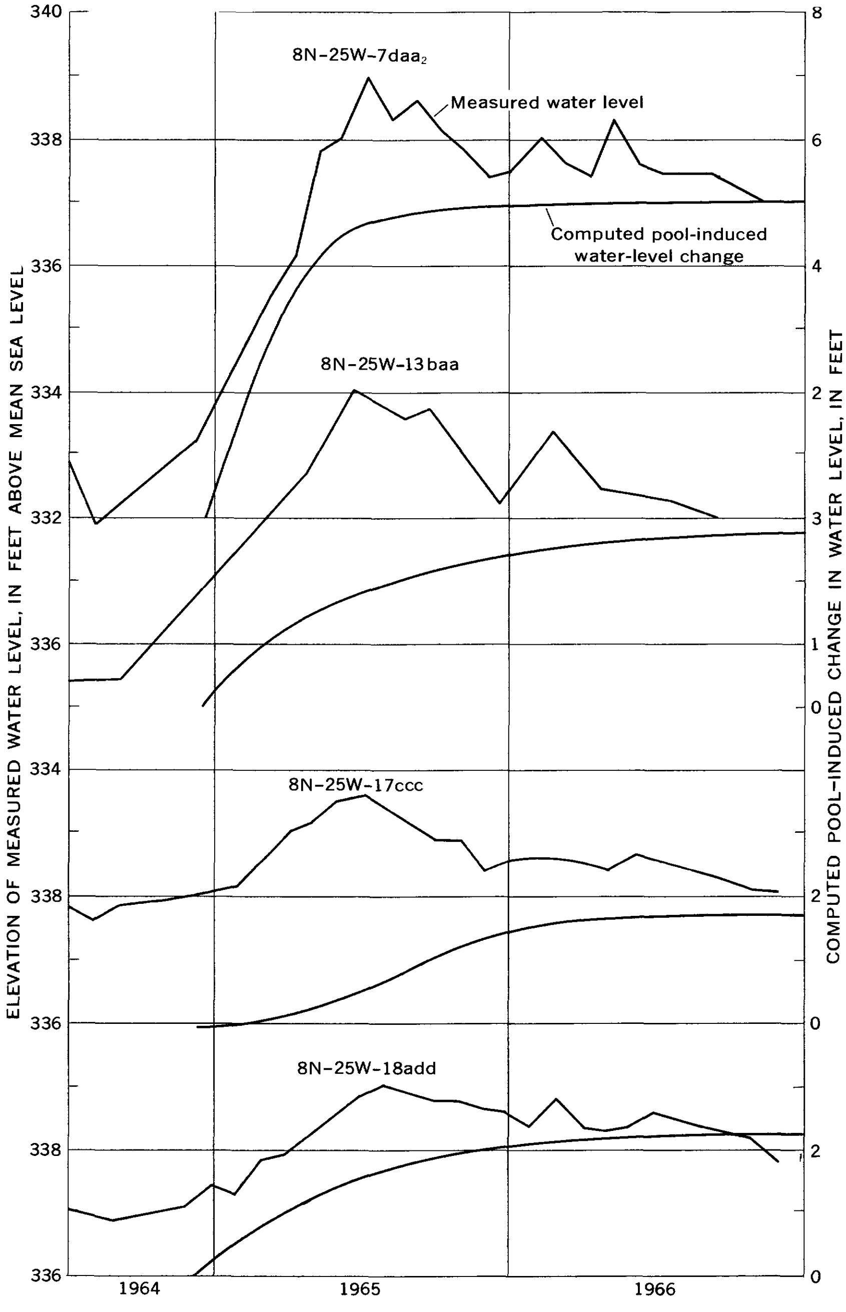


FIGURE 31.—Measured water levels and computed response to change in river stage for selected wells in the McLean Bottom, Logan County, Ark. Well locations are shown on plate 3.

in accretion. Comparison of the well hydrographs with the computed river-induced fluctuations shows a general decline in

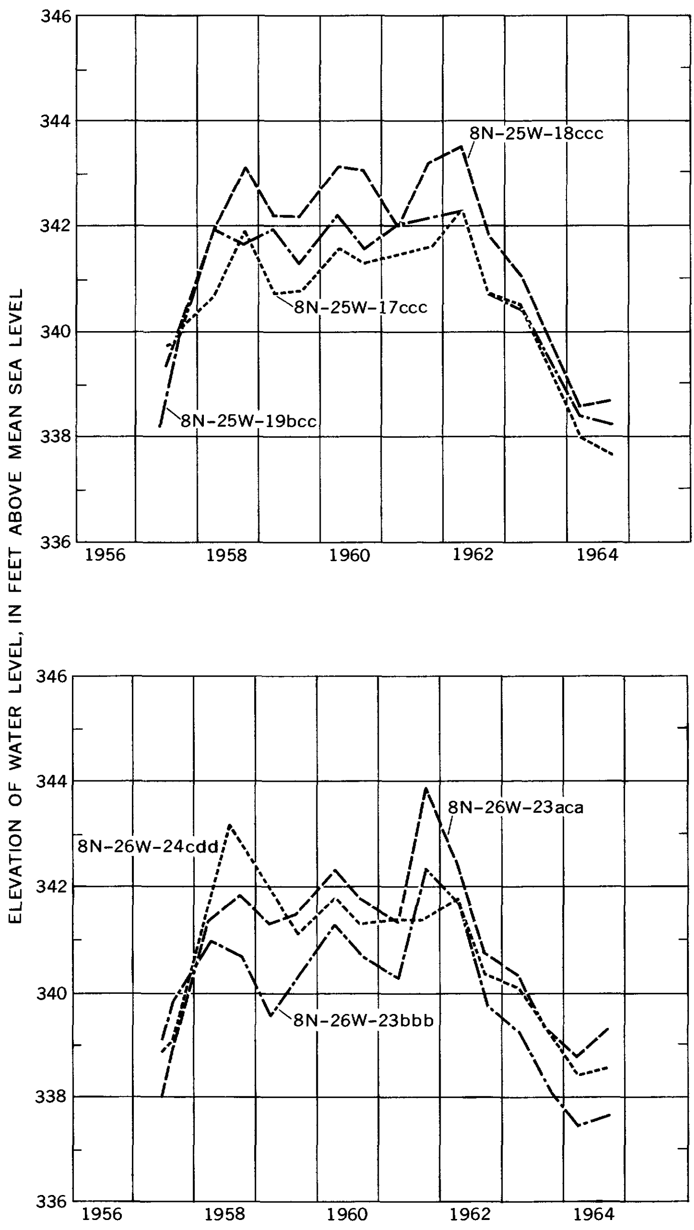


FIGURE 32.—Water levels in selected wells during the period of preconstruction observation in the McLean Bottom, Logan County, Ark. Well locations are shown on plate 3.

accretion since the high-water level in mid-1965. This decline corresponds to continued below-normal precipitation in the area since 1964.

Because average accretion was higher during the preconstruction period than during the postconstruction period, projections based on the average piezometric surface would not serve as a valid basis for comparison with measured water levels since the pool has filled. Water levels have been measured in the Dardanelle Pool area since 1957. The average piezometric surface for the period 1957 to the fall of 1964 reflects the average accretion rate during the preconstruction period. The variation in accretion is shown by comparing water levels (fig. 32) with precipitation (fig. 33). These illustrations indicate that accretion was high from 1957 through 1961 and that it declined during 1962 through 1966.

Tests of the accuracy of water-level projections made for the Dardanelle Pool area pose two problems. First, accretion has been lower since impoundment than it was during the preconstruction period. Second, the ground-water levels have not reached equilibrium with the pool stage.

The first problem was minimized by using measurements made in the fall of 1964—immediately preceding the closure of Dardanelle Dam—as the datum for the projection. Each of these

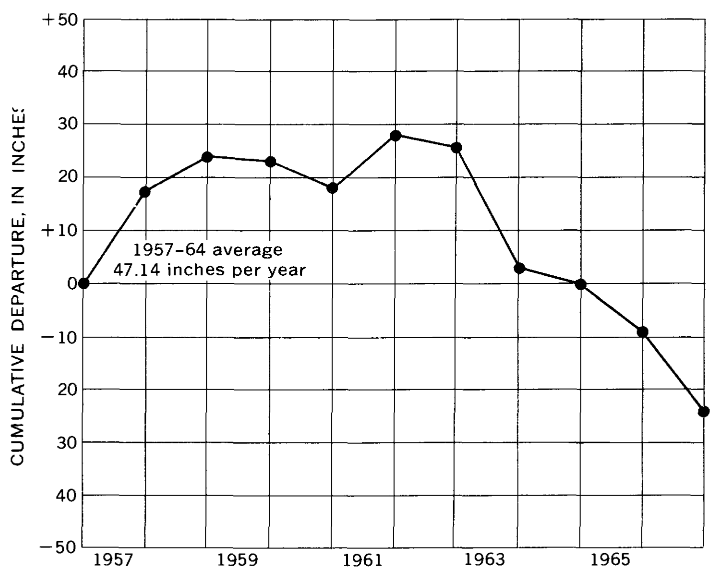


FIGURE 33.—Cumulative departure of annual precipitation from the 1957–64 average at Paris, Logan County, Ark.

measurements was adjusted to account for the nonsteady antecedent river stages.

By using the peizometric surface for the fall of 1964 adjusted for the antecedent river-stage changes as a datum, a nonsteady projection was made to the spring of 1966. The projected water levels in relation to the measured water levels in the spring of 1966 are shown on plate 4.

The projected and measured water levels agree closely, within 1 foot, in most comparisons. The differences reflect, in part, errors in definition of the aquifer properties and errors in modeling the change in accretion with the change in water level. However, the differences also reflect errors in the nonsteady analysis of the change in water level. The projection method described in the previous section is designed for steady-state conditions. Therefore, part of the error shown by the comparison would not be present in projections of steady-state conditions.

SUMMARY OF ANALOG METHODS

The sequence of analog work in a study could be categorized as (1) design and construction, (2) verification, and (3) problem analysis.

Design of the analog in a broad sense includes the planning of the data-collection programs that provide criteria for use in design and verification of the analog. The physical scale of an analog is a function of the nature of the hydrologic system and the detail of data available for design and verification of the analog. Design of the analog essentially comprises a definition of the internal characteristics of the aquifer, in terms of variations in transmissibility and storage characteristics, and a definition of the head, or flow, conditions on the external surfaces of the aquifer. These definitions of the aquifer are translated into analogous electrical components and forces through the use of scale factors.

Verification provides an opportunity for testing the analog response to such forces as pumping wells and fluctuating stream stages. The analog can be revised to respond the same as the aquifer. However, unique distribution of internal characteristics of the aquifer cannot be defined by duplication of observed head response. Therefore, it is essential that the first design of the analog be based on data independent of that used in the verification process.

A verified model is known to reproduce the responses of the aquifer for the range in boundary forces experienced during the period of observation. Problem analysis may extend boundary forces beyond the range experienced during the observation period. The response of the analog to forces beyond those observed must be defined before completing a valid problem analysis.

An analog analysis may be facilitated by use of the principle of superposition. Superposition permits the separate analysis of a single steady or nonsteady component of the flow field. For example, the aquifer response to a stream-stage fluctuation or to a pumping well can be analyzed separately from the other forces acting upon the flow field. The resultant effect in the aquifer can be determined by algebraically adding the head, or flow, change to the flow system. In analysis of a single force acting upon the aquifer, the integrity of all boundaries must be maintained—that is, all boundaries must be modeled as either dependent or independent with regard to the force being analyzed.

GROUND-WATER PROJECTIONS IN PLANNING AND DESIGN

In the Arkansas River Multiple-Purpose Plan, ground-water projections were considered in preliminary and final phases of program development and design.

The first studies on the navigation project were made to select a scheme for preliminary siting of locks and dams on the river from Dardanelle to Arkansas Post. Several plans were considered, including schemes for seven, eight, or 11 locks and dams in this reach.

The principal factors considered in locating the damsites and normal pools in these plans were economics, adequate navigation depth, satisfactory river reaches for approach conditions to the locks, surface and subsurface drainage in areas adjacent to the river, estimated riverbed elevations, and harbors at Pine Bluff and Little Rock, Ark.

The plan of seven dams was the minimum number of dams that would feasibly provide the necessary depths for navigation without creating major drainage problems. The 11-dam plan approached the maximum number of dams. Insofar as practical, the damsites were selected so that the principal tributaries entered near the heads of pools, rather than in the lower parts of the pools, to avoid interference with drainage. Both surface-water drainage and ground-water drainage were major limiting factors in selecting normal pool elevations.

Preliminary ground-water projections were made in alluvial areas adjoining each pool for the three plans studied. Projected ground-water contour lines were then superimposed on topographic maps, and areas where the projected water table occurred near ground surface were delineated. For estimating purposes, the appraised cost of acquisition of these areas was considered as a part of the drainiage costs in the cost estimate of each plan considered.

Time did not permit development of a nine- or 10-dam plan. However, it was evident from the preliminary studies that overall costs had a tendency to increase with the increase in number of dams. Total estimated costs for the seven-, eight-, and 11-dam plans were $220 million, $235 million, and $284 million, respectively. Estimated drainage costs for the three plans, including surface-water and ground-water drainage, were $11 million, $9 million, and $10 million, respectively. Although the total cost of the seven-dam plan was the lowest, this plan would have caused a major amount of drainage interference. The estimated cost of the 11-dam plan was the highest and would have produced more drainage interference than the eight-dam plan.

The eight-dam plan was recommended as the optimum plan that would meet navigation requirements and cause a minimum of drainage interference. The plan was recommended with the intent of fixing an approximate location for each lock and dam and an approximate pool elevation, with the view that further detailed studies would affirm, or provide nominal revisions in, the plan.

In the final planning stages, the postconstruction ground-water conditions were a major consideration in locating the optimum sites and pool elevations of the locks and dams and in planning and designing of levee-protected agricultural areas.

In evaluating alternate damsites, pool elevations, and levee systems, the ground-water projections were used to design drainage facilities or other ground-water-control measures that would prevent high ground-water levels from affecting land utilization. As a criterion in design of ground-water-control measures, it was assumed that land utilization would be affected in areas where the depth to ground water was 5 feet or less. All effects of shallow water levels would not be adverse. For example, recent studies showed that some crops may be benefited from water levels shallower than 5 feet below land surface.

The following discussion of lock and dam 13 is an example of the application of ground-water projections in final site-selection studies. The use of ground-water projections in siting lock and dam 13 is similar in many respects to use of projections in evaluating other damsites and pool elevations in the navigation reach. Lock and dam 13 will be a low-head navigation structure on the main stem of the Arkansas River, a few river miles downstream from Fort Smith, Ark. (fig. 6).

The dam will consist of a 1,000-foot-long tainter-gated concrete spillway section bearing on rock with a concrete training wall on the north end. The 110- by 600-foot lock adjoining the dam on the south side will be constructed with concrete walls bearing on rock and will provide a maximum lift of 22 feet. An overflow-embankment section will extend about 4,000 feet, from the north end of the concrete dam to the levee on the left bank. The normal upper and lower pools at lock and dam 13 will be 392.0 and 372.0 feet above mean sea level, respectively. All flows in excess of lockage requirement of as much as an upper pool elevation of 395.0 feet, or less than 245,000 cubic feet per second, will pass through the spillway section in the main channel. All greater discharges will be divided between the spillway and the overflow-embankment section.

Site-selection studies for lock and dam 13 initially included four alternate sites between river-miles 341.0 and 347.0. Two of the four sites were eliminated early in the study because of inadequate approach conditions to the lock. The remaining two sites, at river-mile 341.7 and river-mile 346.5, were studied in detail to analyze the relative costs for each location.

The area studied to determine the difference in cost between the two sites is shown in figure 34. The 30-square-mile area is underlain by alluvium of the Arkansas River, ranging in thickness from 35 to 45 feet. The area is bordered on the north by relatively impermeable rocks of Paleozoic age. The alluvial bottom land is intensively cultivated for cotton, soybeans, and vegetables. Irrigation wells in the alluvium supply supplemental water during dry years.

The average preconstruction piezometric surface is within 5 feet of ground surface on approximately 1,000 acres. These areas comprise largely swampy and wooded abandoned river channels. The projected average piezometric surfaces for a dam at river-mile 346.5 and at river-mile 341.7 are shown in figures 35 and 36, respectively. Approximate areas are delineated where the projected piezometric surface would be within 5 feet of land surface.

In addition to ground-water projections for the dam at river-mile 341.7 and at river-mile 346.5, other hydrologic data were

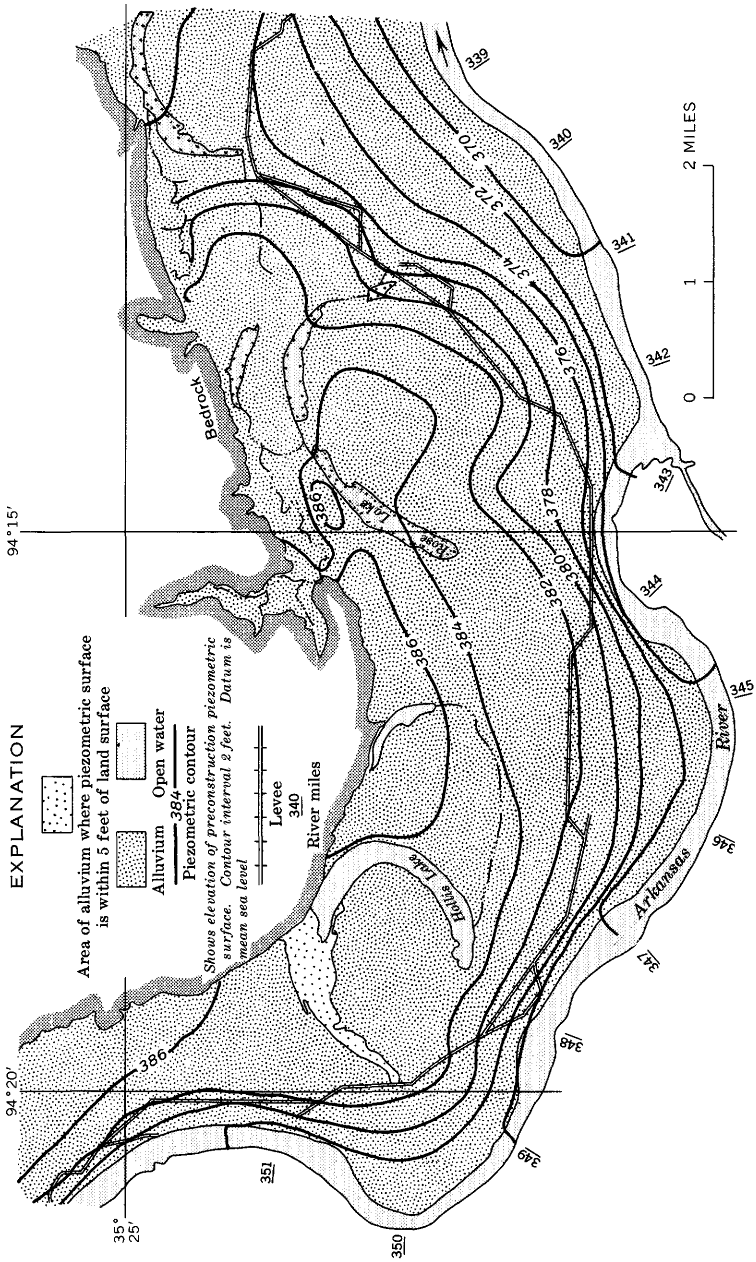


FIGURE 34.—Area studied in selection of a site for lock and dam 13, near Fort Smith, Ark.

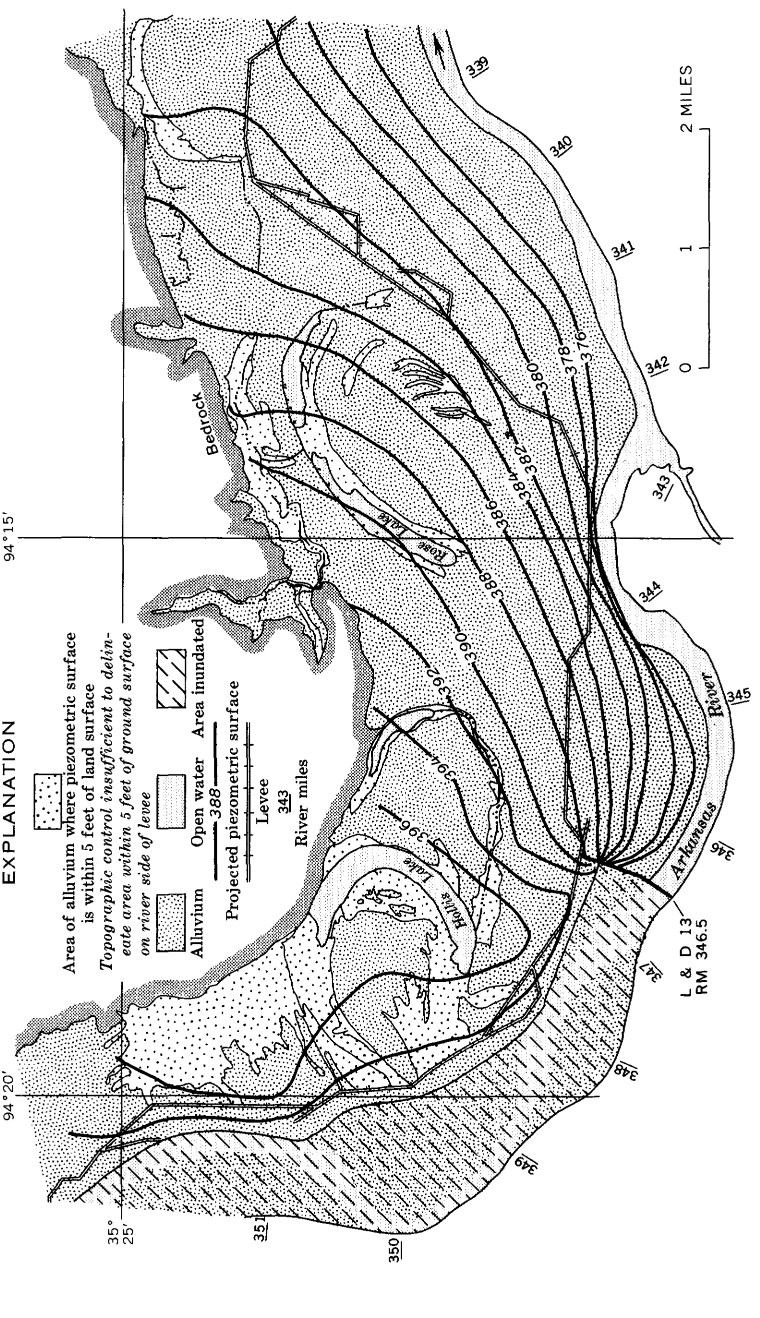


FIGURE 35.—Projected piezometric surface for damsite (lock and dam 13) at river-mile 346.5.

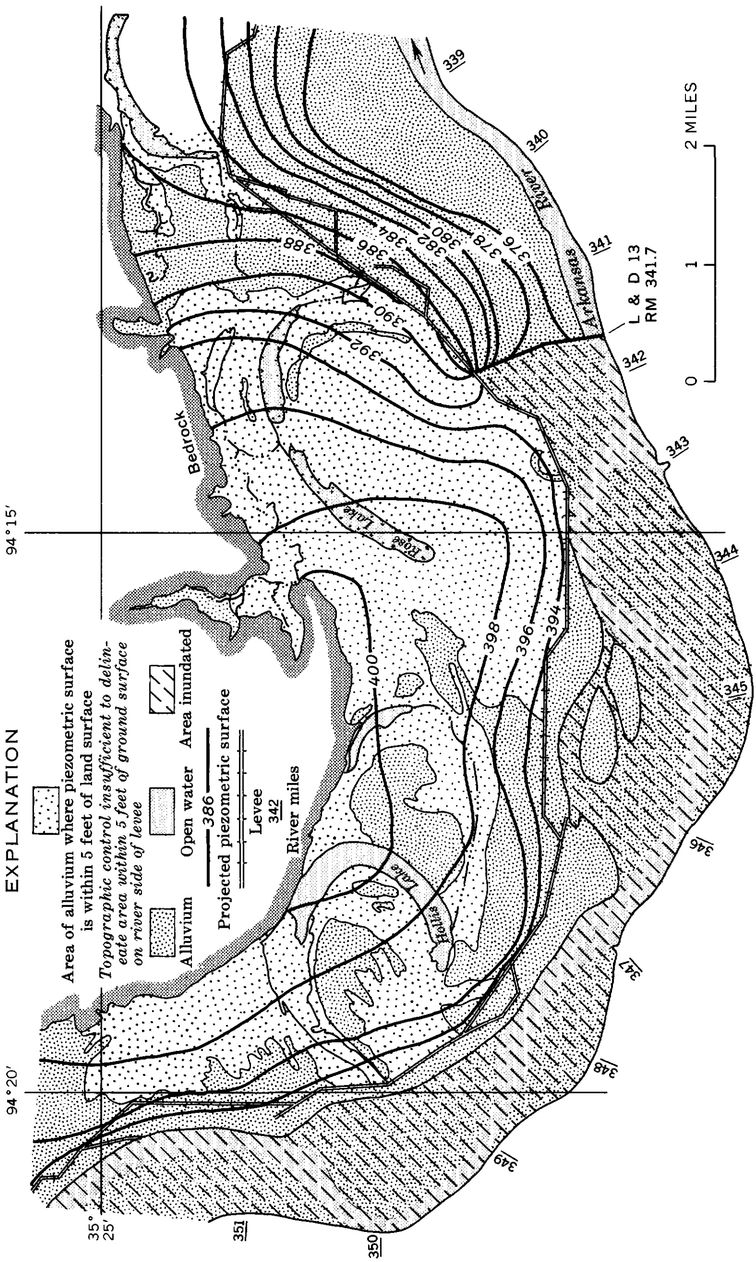


FIGURE 36.—Projected piezometric surface for damsite (lock and dam 13) at river-mile 341.7.

available for use in estimating costs of ground-water-control measures. Hydrologic properties of the alluvial aquifer were known from laboratory permeability tests on undisturbed samples, grain-size analyses, and visual classification of samples from approximately 40 test holes. These data were supplemented by a pumping test and alluvial geologic maps of the area. Water-level measurements made in observation wells, installed in each of the test holes and in privately owned wells, provided data on ground-water-level fluctuations for a 6-year period from August 1957 to the date of the study, May 1963. Daily river-stage readings at the upstream end of the study area were available for the period of record of ground-water measurements.

A cost comparison of ground-water drainage and of alternative remedial measures needed to reduce the elevation of the postconstruction piezometric surface was made for each of the lock and dam locations. The design and cost of five different control methods were considered to determine the most economical method.

The ground-water-control methods evaluated included (1) a fully penetrating drainage ditch constructed landside of the levee; (2) a bentonite slurry cutoff trench parallel to the river; (3) agricultural subdrains; (4) a system of open ditches; and (5) a system of drainage wells fully penetrating the aquifer. The comparative costs of these control measures are shown in table 2.

TABLE 2.—*Comparative cost estimates of ground-water control for the area near lock and dam 13*

[Operation and maintenance charges are not included in the estimates]

Item No.	Item	Estimated cost	
		River-mile 341.7	River-mile 346.5
1	Fully penetrating drainage ditch	$2,500.000	$1,000.000
2	Bentonite slurry cutoff trench	2,200,000	1,530,000
3	Agricultural subdrains	1,538,000	409,000
4	Open ditches	1,016,000	279,000
5	Drainage wells	283,900	71,700

The studies indicated that a drainage well system would provide a positive and flexible means of ground-water control. Of the plans considered, the cost of the well system was the lowest and included operation and maintenance charges for the life of the Multiple-Purpose Plan. The estimated cost of a drainage well system for each of the site locations was considered to be representative of the cost that could ultimately accrue to the sites from possible adverse ground-water effects.

Other factors considered in the cost comparison studies were land acquisition; relocation of powerlines, roads, pipelines, and so forth; dredging; and construction costs. The cost analysis shows the cost differential for items where a difference could be found, and the total cost differential between sites. These items and the analysis are given in table 3.

TABLE 3.—*Cost analysis for lock and dam 13 siting studies*

[Items shown are not complete, but reflect that part of work where a differential was found. Differential is the cost of site at river-mile 341.7 minus the cost of site at river-mile 346.5]

Item No.	Item	Estimated cost		
		River-mile 341.7	River-mile 346.5	Differential
1	Ground-water control	$283,000	$71,700	$211,300
2	Land acquisition	450,900	0	+450,900
3	Relocations	1,246,000	323,000	+923,000
4	Dam (with embankments)	7,476,195	5,884,700	+1,591,495
5	Lock	824,900	360,000	+464,900
6	Permanent roads	175,000	105,000	+70,000
7	Channel and canals	26,750	508,500	−481,750
	Total	10,482,745	7,252,900	3,229,845

As a result of the cost differential studies, in which the cost of ground-water control was considered, the upstream damsite, river-mile 346.5, was selected for construction as the most feasible and economic location.

Ground-water projections, as utilized in the lock and dam 13 studies and in other similar studies on the Arkansas River Multiple-Purpose Plan, provided a basis for sound engineering judgment on problems affecting the overall project costs. The ground water projections assisted the Corps of Engineers in developing the navigation project at a minimum of cost and drainage interference.

GLOSSARY

Accretion, net rate at which water is gained or lost vertically through the aquifer surface in response to external forces. Infiltration of precipitation and evapotranspiration are the principal examples of accretion in the Arkansas and Verdigris River valleys. Accretion in the study area ranges from a negative rate of a few tenths of a foot per year to a positive rate of about 2 feet per year.

Aquifer, a formation, group of formations, or part of a formation that is water bearing and that will yield water in sufficient quantities to be of consequence as a source of supply. The aquifer of the alluvium in the Arkansas and Verdigris River valleys is composed predominantly of sand and gravel. Permeability ranges from about 10 gallons per day per foot squared (1.34 ft per day) to more than 15,000 gallons per day per foot squared (2,000 ft per day).

Aquitard, a geologic unit (or group of units) that is porous and permeable, but capable of transmitting water only slowly. Head losses in the unit are high, even when saturated and at low rates of flow. Aquitards in the alluvium are composed principally of silt- and clay-sized particles; aquitard permeabilities generally are less than 10 gallons per day per foot squared (1.34 ft per day).

Cone of depression, a cone-shaped depression in the water table that develops in the vicinity of a well during pumping.

Diffusivity, coefficient of hydraulic, the ratio of the coefficient of transmissibility to the coefficient of storage of an aquifer.

Discharge, as applied to ground water, the removal of water from the aquifer —by evapotranspiration, pumping, and seepage—to surface-water bodies.

Drawdown, depression of the water level due to pumping.

Evapotranspiration, discharge of ground water by the combined processes of evaporation and plant transpiration.

Ground water, water in the zone of saturation in which the voids in the rock are filled with water under positive hydraulic head.

Ground-water movement, the movement of ground water within an aquifer or from one aquifer to another.

Head, the pressure head of water at a given point in an aquifer is its hydrostatic pressure, expressed as the height of a column of water that can be supported by the pressure.

Hydraulic gradient, gradient of the water table measured in direction of the greatest slope. Generally expressed in feet per mile. The hydraulic gradient of an aquifer is measured on the **piezometric surface.**

Hydrologic properties, the properties of an aquifer that control the occurrence of ground water.

Hydrologic system, the hydraulically interconnected system of aquifers and surface-water bodies.

Infiltration, the process whereby water enters the surface soil and moves downard toward the water table.

Nonsteady flow, the state of imbalance between the forces of recharge and discharge in which there is change in hydraulic head with time.

Permeability, the capacity of a porous material to transmit water. The permeability is the rate of flow of water through a given cross-sectional area and gradient. The Survey's field coefficient of permeability is expressed as flow of water, in gallons per day, under prevailing field conditions through a square-foot cross-sectional area for each foot per foot of hydraulic gradient (gallons per day per foot squared, or gpd per ft^2). Another common unit of permeability is flow of water, in cubic feet per day, through a cross-sectional area of 1 foot under a gradient of 1 foot per foot (cu ft per day per ft^2, or ft per day).

Piezometric surface, the surface to which the water from a given aquifer will rise under its full head.

Pressure head, the equivalent weight of liquid, P/W, representing a pressure, P, in a fluid of specific weight, W. The pressure head of water is expressed as the height of a column of water that can be supported by the pressure.

Recharge, the process by which water is absorbed and added to an aquifer. Used also to designate the quantity of water added to an aquifer.

Steady flow, the state of balance between the opposing forces of groundwater recharge and discharge, in which there is no change in hydraulic head with time.

Storage, water stored in openings in the zone of saturation. The **coefficient of storage** is the volume of water released from, or taken into, storage in an aquifer per unit surface area of aquifer per unit change in component of head normal to that surface.

Transmissibility, the capacity of an aquifer to transmit water under pressure. The **coefficient of transmissibility** of an aquifer is expressed as the flow of water, in gallons per day, under prevailing field conditions through a section of the aquifer 1 mile wide and under a gradient of 1 foot per mile. The coefficient of transmissibility is the weight-average permeability of the aquifer, in gallons per day per foot squared, multiplied by the thickness of the aquifer, in feet.

Unsaturated zone, zone above the water table in which water is under negative hydraulic pressure.

Water table, the horizon at which water is at zero hydraulic pressure. Water in the zone beneath the water table (zone of saturation) is under positive hydraulic pressure; water in the zone above the water table (zone of aeration), including the capillary fringes, is under negative hydraulic pressure (tension).

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