PREDICTIVE SIMULATION OF ALTERNATIVES FOR MANAGING THE WATER RESOURCES OF NORTH FORK SOLOMON RIVER VALLEY BETWEEN KIRWIN DAM AND WACONDA LAKE, NORTH-CENTRAL KANSAS

By Robert D. Burnett

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CONTENTS

ILLUSTRATIONS

ILLUSTRATIONS--Continued

Figure | Page

TABLES

Table | Page

CONVERSION FACTORS

The following table may be used to convert the inch-pound units of measurement used in this report to the International System (SI) of Units:

Multiply inch-pound unit	By	To obtain SI unit
Length		
inch	25.4	millimeter
foot	0.3048	meter
mile	1.609	kilometer
Area		
acre	0.4047	hectare
square mile	2.590	square kilometer
Volume		
acre-foot	1,233	cubic meter
acre-foot per year	1,233	cubic meter per year
Flow		
foot per second (ft/s)	0.3048	meter per second
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
cubic foot per day (ft^3/d)	0.02832	cubic meter per day
gallon per minute (gal/min)	0.06309	liter per second
Hydraulic conductivity		
foot per day (ft/d)	0.3048	meter per day
Transmissivity		
square foot per day (ft^2/d)	0.09290	square meter per day
Temperature		
degree Fahrenheit (°F)	(1)	degree Celsius (°C)

1 °C = (°F -32)/1.8.

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ABSTRACT

Since 1974, water levels in the alluvial aquifer of the North Fork Solomon River valley in north-central Kansas have decreased due to increases in ground-water pumpage, decreases in availability of surface water for irrigation, and below-average precipitation. A finite-element model was developed in cooperation with the U.S. Bureau of Reclamation to simulate changing conditions between 1970-79. Model results indicate that annual recharge to the aquifer due to precipitation, application of water for irrigation, and canal leakage averaged about 22,825 acre-feet and that annual groundwater discharge to the river averaged about 16,590 acre-feet.

Predictive simulations for 1980-2000 were made using management alternatives that involved clay-lining of irrigation ditches, reduction of surface-water availability with and without an increase in ground-water pumping, and continuation of 1979 pumping conditions. The simulations indicated that as much as 5.5 feet of additional average water-level drawdown in wells would occur by 2000 if the surface-water supply was reduced 100 percent and ground-water pumpage increased to make up the difference. The simulations also indicated that a rise in average water levels of 0.55 foot would occur by 2000 and that base flow to the river would increase to 12,300 acre-feet per year if 1979 pumping conditions remained constant, if a surface-water supply equal to the average supply from 1960 to 1979 was available, and if precipitation was equal to the normal precipitation from 1941 to 1970.

Results of predictive simulations indicated that the management alternative of projecting the 1979 pumpage conditions to the year 2000 (simulation 8), with the long-term average values of precipitation, surface-water availability, and river and tributary flow rates, had the least effect on water levels and base flow. Results also indicated that the management alternative of reducing the net supply of surface water by 100 percent, while increasing the ground-water-diversion rate to compensate (simulation 7) had the greatest effect on water levels and base flow.

INTRODUCTION

Purpose and Scope

The availability of surface water for irrigation in the North Fork Solomon River valley, north-central Kansas, has become less reliable in recent years. Consequently, irrigation wells are being used to supplement surface-water supplies. These events have prompted a study to apply a transient model of the stream-aquifer system to (1) gain a more complete understanding of the hydrology and ground-water hydraulics in the study area, and (2) to make predictive simulations of management alternatives based on proposals to line irrigation ditches with clay, reduce surface-water availability, and increase ground-water pumping.

This study was made by the U.S. Geological Survey as part of a cooperative program with the U.S. Bureau of Reclamation. The work presented in this report represents an extension of the work described in the report entitled "Hydrology and Model of North Fork Solomon River Valley, Kirwin Dam to Waconda Lake, North-Central Kansas" by Donald G. Jorgensen and Lloyd E. Stullken (1981).

Location and Description of Study Area

The study area encompasses the North Fork Solomon River valley between Kirwin Dam and Waconda Lake, as shown in figure 1. The valley within the study area is about 35 miles long and averages 2 to 3 miles in width. This area encompasses about 100 square miles in Phillips, Smith, and Osborne Counties in north-central Kansas. The upland adjoining the valley consists of gentle hills dissected by small valleys traversed by intermittent streams. Average annual precipitation is about 24 inches. Average annual potential evapotranspiration is about 40 inches. Irrigation is practiced extensively using both surface water and supplemental ground water. Releases from Kirwin Reservoir are diverted directly into the Kirwin Irrigation Canal and constitute the supply for surface-water irrigation.

The alluvial aquifer within the study area ranges in saturated thickness from zero towards the valley sides to as much as 55 feet near the middle of the valley. The alluvial aquifer is bounded on both sides and underneath by Cretaceous rocks consisting mostly of shale. The areal extent of the alluvial aquifer within the study area is shown in figure 2.

Well-Numbering System

Well and test-hole numbers used in this report identify the location of wells according to the U.S. Bureau of Land Management's system of land subdivision. The well number is composed of township, range, and section numbers, followed by letters that indicate the subdivision of the section in which the well is located. The first letter denotes the quarter section or 160-acre tract; the second letter denotes the quarter-quarter section or

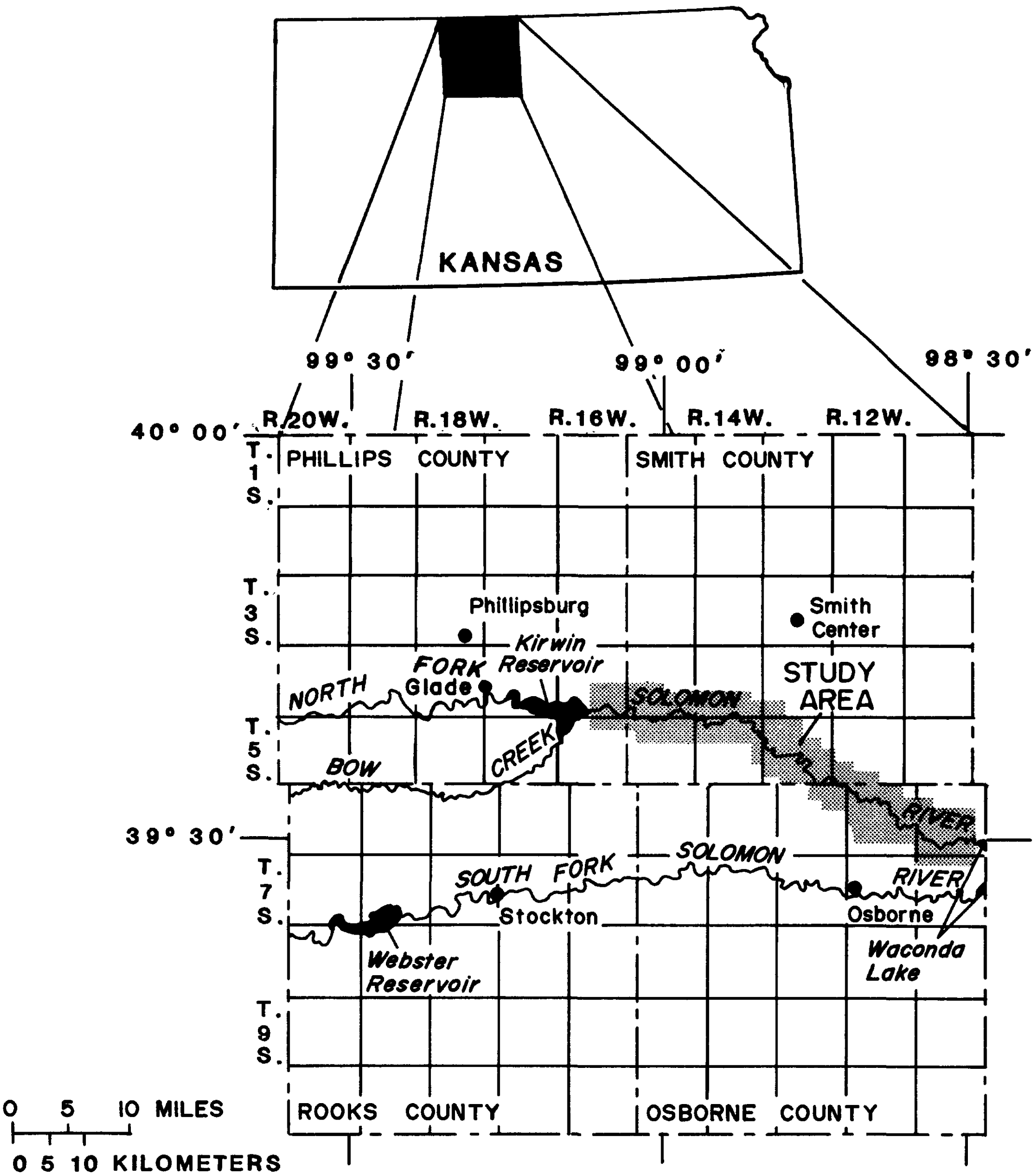


Figure 1.--Location of study area.

40-acre tract; and the third letter, when used, indicates the quarter-quarter-quarter section or 10-acre tract. The 160-acre, 40-acre, and 10-acre tracts are designated A, B, C, and D in a counterclockwise direction, beginning in the northeast quadrant (fig. 3). Any additional wells located within a 10-acre tract are numbered serially, according to the order in which they were inventoried. For example, well 6-12W-23CDC is in the SW1/4 SE1/4 SW1/4 of sec. 23, T. 6 S., R. 12 W., and is the first well inventoried in that tract.

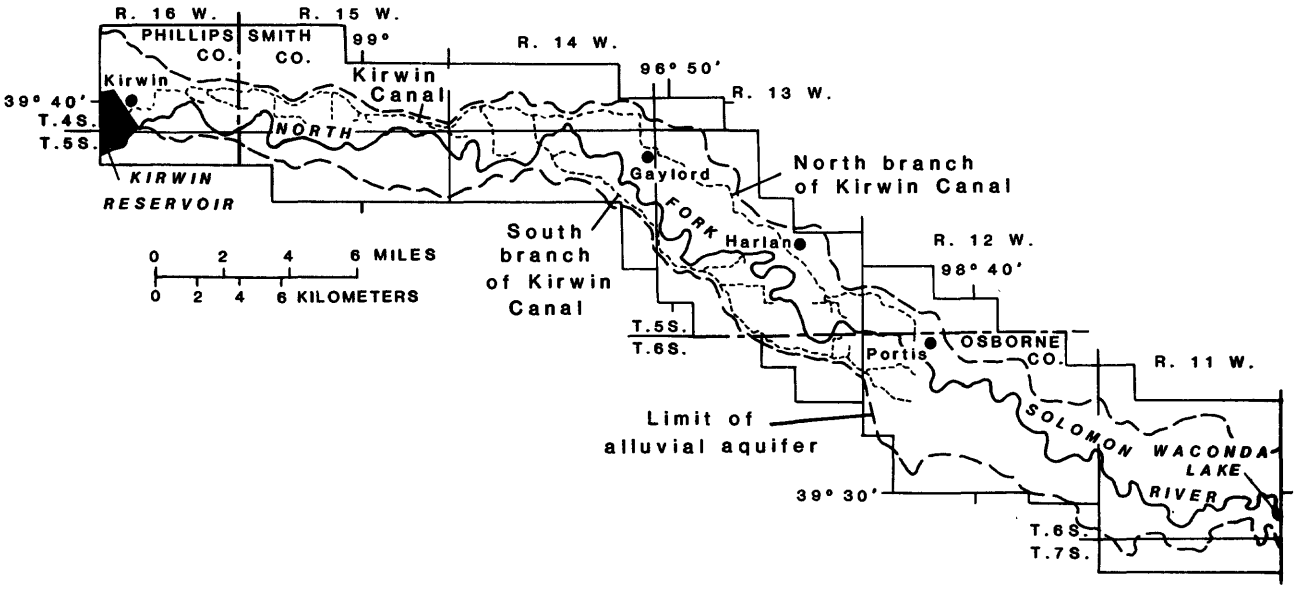


Figure 2.--Areal extent of alluvial aquifer and location of irrigation structures in Kirwin Irrigation Unit.

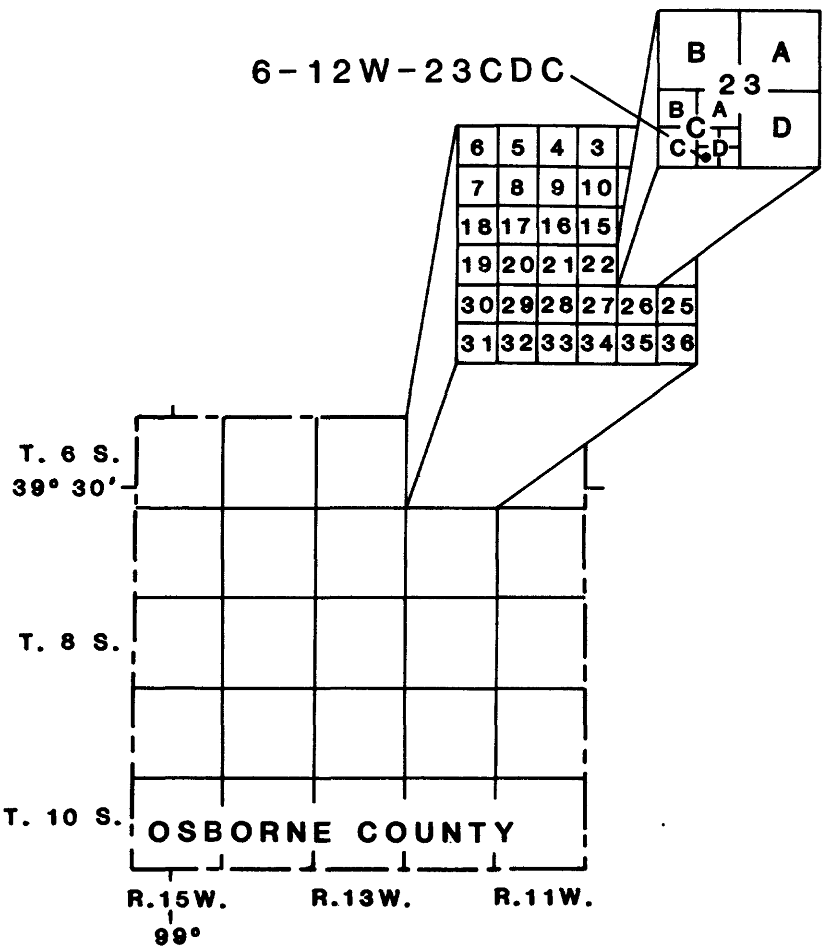


Figure 3.--Well-numbering system.

Acknowledgments

The author is grateful for information provided by the U.S. Bureau of Reclamation and the Kirwin Irrigation District. Appreciation is extended especially to Leland Stroup, manager of the Kirwin Irrigation District, who provided much time and information.

PRESENT STREAM-AQUIFER SYSTEM

Surface Water

During the 1950's, the U.S. Bureau of Reclamation began construction of a multipurpose water project consisting of a dam and reservoir and a canal and lateral system designed to serve 10,000 irrigable acres. These structures collectively were entitled the Kirwin Irrigation Unit. The Kirwin Unit consists of the main canal and the north and south branch segments, as well as the canal laterals (fig. 2). About 30 percent of the main canal and north and south branches are clay lined. The purpose of the water project was to provide flood protection and a dependable supply of water for irrigation, wildlife, and recreation. Filling of the reservoir was completed during 1955.

The Kirwin Irrigation District began operation during the 1957 irrigation season. During that year, 5,530 acre-feet of water were released to the Kirwin Canal to irrigate 1,336 acres of land. The quantity of irrigation water released from Kirwin Reservoir to the canal system increased to 27,679 acre-feet during 1976, with irrigated acreage increasing to 9,266 acres during the same year. Annual quantities of irrigation water released from Kirwin Reservoir during 1970-79 are shown in figure 4, and the area irrigated by surface-water supplies is shown in figure 5. Generally, the irrigation season runs from June through August of each year.

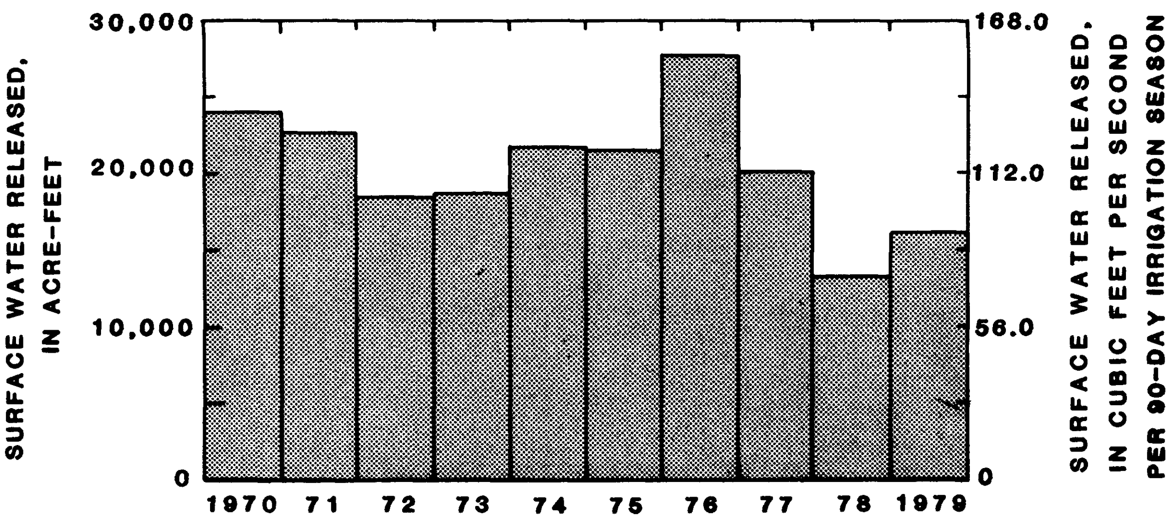


Figure 4.--Annual quantities of surface water released to Kirwin Canal from Kirwin Reservoir, 1970-79 (data from Allacher, 1980).

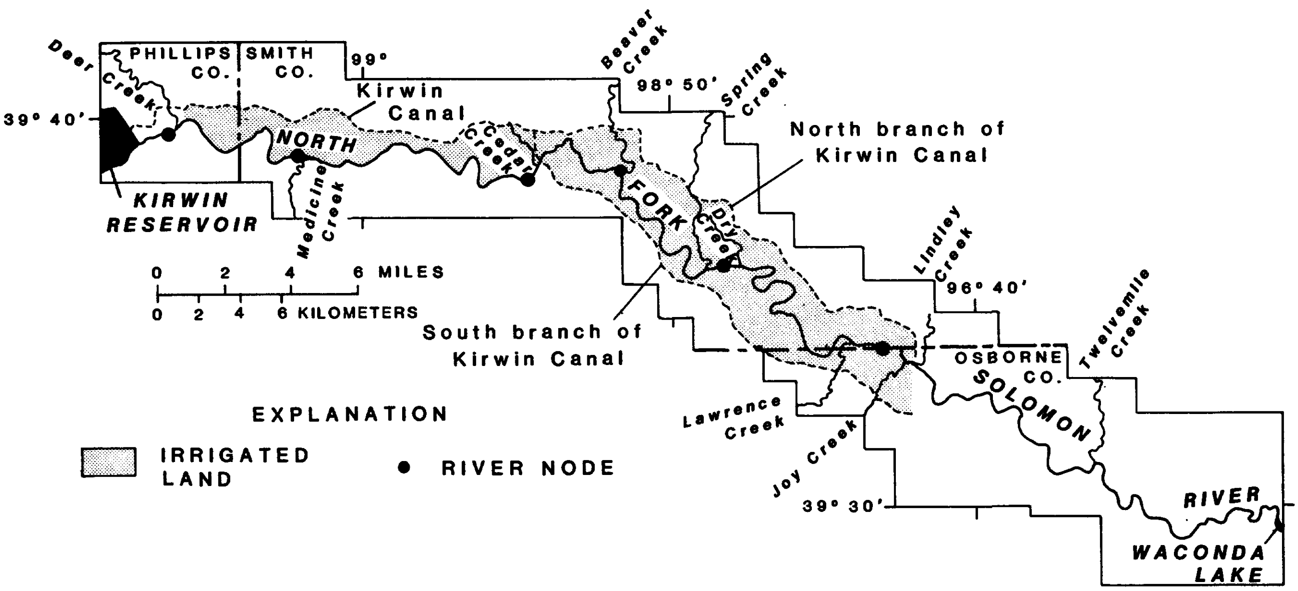


Figure 5.--Irrigated land within Kirwin Irrigation District and tributaries of North Fork Solomon River in the study area.

Ground Water

Nearly all irrigation wells located within the Kirwin Irrigation District are used to supplement surface-water supplies. Ground-water-diversion rates from irrigation wells were determined from water-right records provided by the Division of Water Resources of the Kansas State Board of Agriculture and by reviewing these records with Mr. Leland Stroup, manager of the Kirwin Irrigation District.

Dates on which irrigation wells began operation were obtained from water-rights records. Land area irrigated by wells located within the District was based on the manager's estimates of how much acreage each irrigator was irrigating. An application rate of 0.5 acre-foot per irrigation season was used for wells on lands irrigated by both ground and surface water, and an application rate of 1.0 acre-foot per season was used for wells on lands not irrigated by surface-water applications. These figures are based on estimates made by Mr. Stroup.

The use of ground water for irrigation has increased rapidly since 1970 in the study area. By 1979, 113 irrigation wells were in operation, increasing from approximately 25 during 1970 (plate 1). The cropland irrigated by ground water has increased from about 1,000 acres during 1970 to about 6,000 acres during 1979, and ground-water withdrawal has increased from about 600 acre-feet during 1970 to about 3,300 acre-feet during 1979. Annual estimated ground-water-diversion rates for the 1970-79 irrigation seasons are shown in figure 6.

The use of ground water for municipal purposes has remained constant throughout the 1960's and 70's. An annual total of 427 acre-feet is withdrawn by municipal wells serving population centers of Downs, Gaylord, and Portis, and two rural water districts.

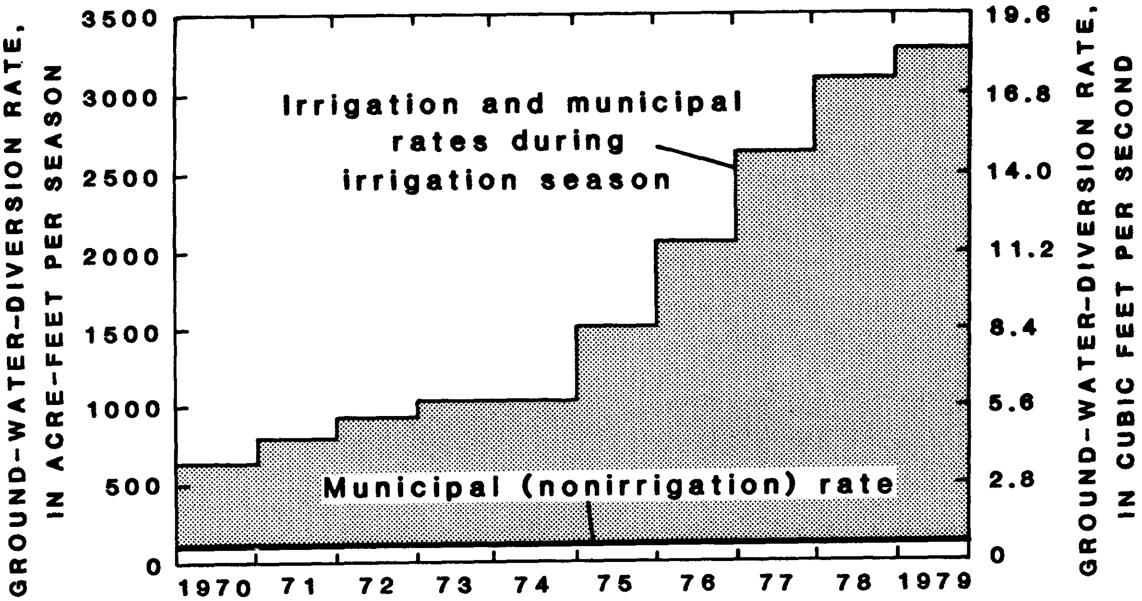


Figure 6.--Estimated irrigation-season ground-water-diversion rates, 1970-79.

Hydrographs of seven observation wells located within the study area are shown in figure 7, and location of these well sites are shown on plate 1. All observation wells are located within the Kirwin Irrigation District and are close to irrigation laterals. Water levels generally increased during 1973 and 1974, reflecting above-average precipitation that fell during 1973. However, from 1974 through 1979, water levels tended to decrease due to continued increases in ground-water-diversion rates for irrigation, decreases in applied surface water for irrigation, and below-average precipitation for this period.

MATHEMATICAL SIMULATION OF STREAM-AQUIFER SYSTEM

Numerical Model

A two-dimensional finite-element model, developed by the U.S. Geological Survey, was used to simulate the stream-aquifer system (Dunlap and others, 1984). The model was formulated to produce an approximate solution to the partial differential ground-water-flow equation as:

$$\frac{\partial}{\partial x}\left(Kb\,\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(Kb\,\frac{\partial h}{\partial y}\right) = S\,\frac{\partial h}{\partial t} + Q, \qquad (1)$$

where

x and y are the coordinate axes [L];
K is the hydraulic conductivity of the aquifer [Lt^{-1}];
b is the saturated thickness [L];
h is the hydraulic head [L];

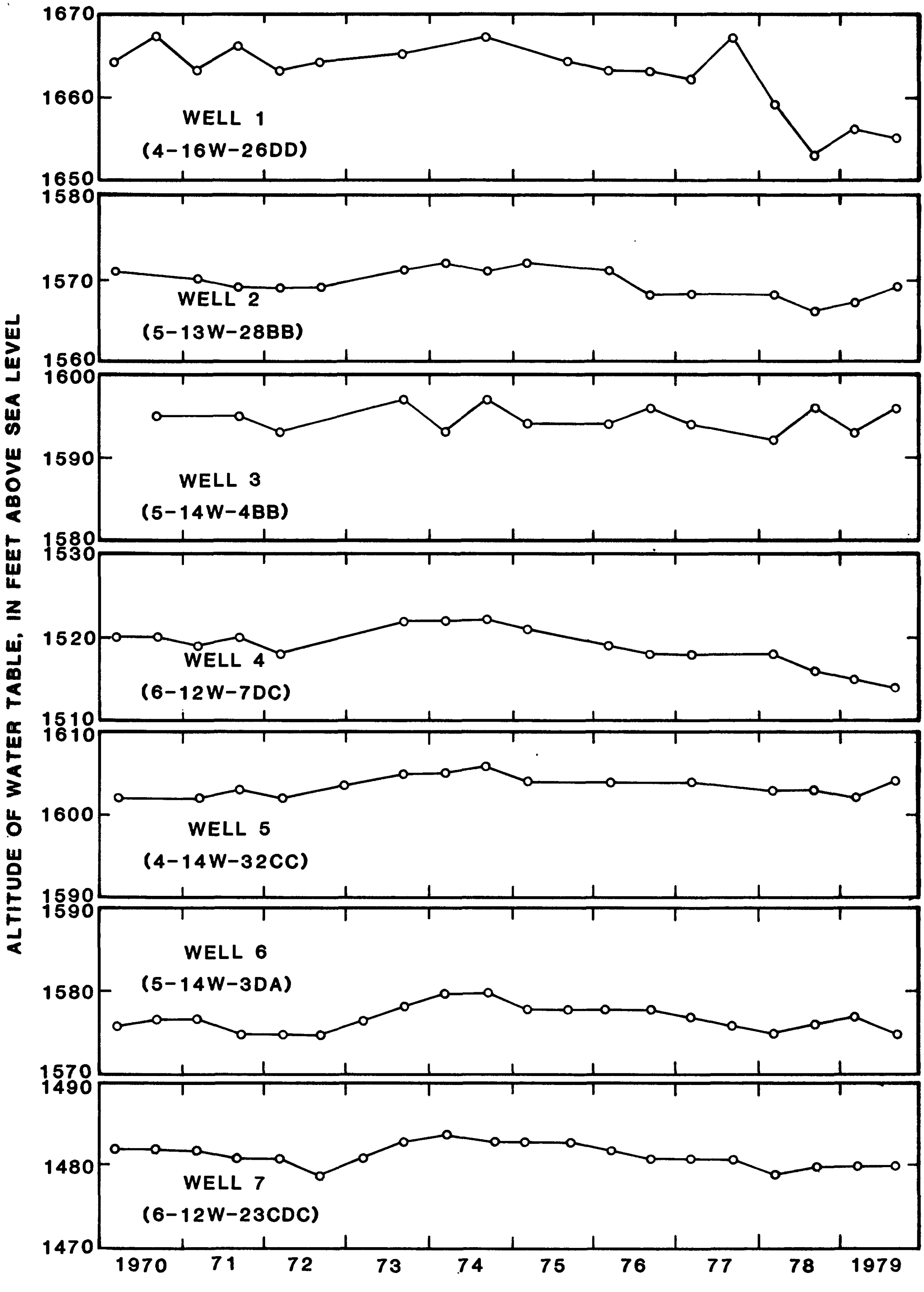


Figure 7.--Hydrographs of selected observation wells, 1970-79.

S is the storage coefficient (dimensionless);
t is the time [t]; and
Q = Q (x, y, t) is the net vertical flux into the aquifer from point or distributed sources (sinks), such as wells, evapotranspiration, ground-water percolation, or river-aquifer interaction [L/t].

Equation 1 is solved using the Galerkin (weighted-residual) method.

The modeled area was subdivided into triangular finite elements illustrated in figure 8, which is an enlargement of section A shown on plate 1. Points of intersections of the sides of the elements represent nodes. Values of hydraulic conductivity, specific yield, and altitudes of the base of the aquifer, land surface, and water surface were specified at each node.

Nodal points were established within the modeled area to produce elements small enough so that errors associated with the finite-element numerical analyses would be minimized. A time step of 5 days was used to produce good numerical solutions to the flow equation. Saturated thicknesses were computed by the model for each time step at each grid node by subtracting altitudes of the aquifer base from hydraulic heads calculated for the previous time step.

Additional interior nodal points were used to simulate stream-aquifer interaction, discharge from irrigation wells, and canal and lateral losses and are illustrated in figure 8. Nodal grid points 20, 25, 30, 35, 16, 21, 26, and 31 represent exterior, no-flow finite-element nodes. Interior nodal points 111, 112, and 155 are recharge wells used to simulate canal and lateral losses during the irrigation seasons. Interior nodal points 3 to 11 are sites used to simulate withdrawal from irrigation wells. All recharge and discharge nodes are treated as point sources, meaning that withdrawal or recharge takes place at a point rather than over a triangular area. Interior nodal points 2 and 3 are used to simulate stream-aquifer interaction. These nodes generally are located where the stream bends. Additional discussion of stream-aquifer interaction occurs in sections on "Stream-Aquifer Flux" and "Model Calibration" in this report.

Boundary Conditions

Boundary conditions were used in the model to control flow at the edge of the aquifer system and are shown on plate 1. The upstream end of the modeled area at Kirwin Dam and the downstream end at the inlet to Waconda Lake were treated as constant-head boundaries. These boundaries were used to simulate subsurface flow into and out of the aquifer along the alluvial basin. Most of the north and south sides of the modeled area were considered as no-flow boundaries. These boundaries were used to represent the effect of lateral termination of the aquifer system against relatively impermeable bedrock boundaries. In areas where tributaries and terrace deposits intercept model boundaries, selected constant-flux nodes were employed to simulate subsurface ground-water flow from these units.

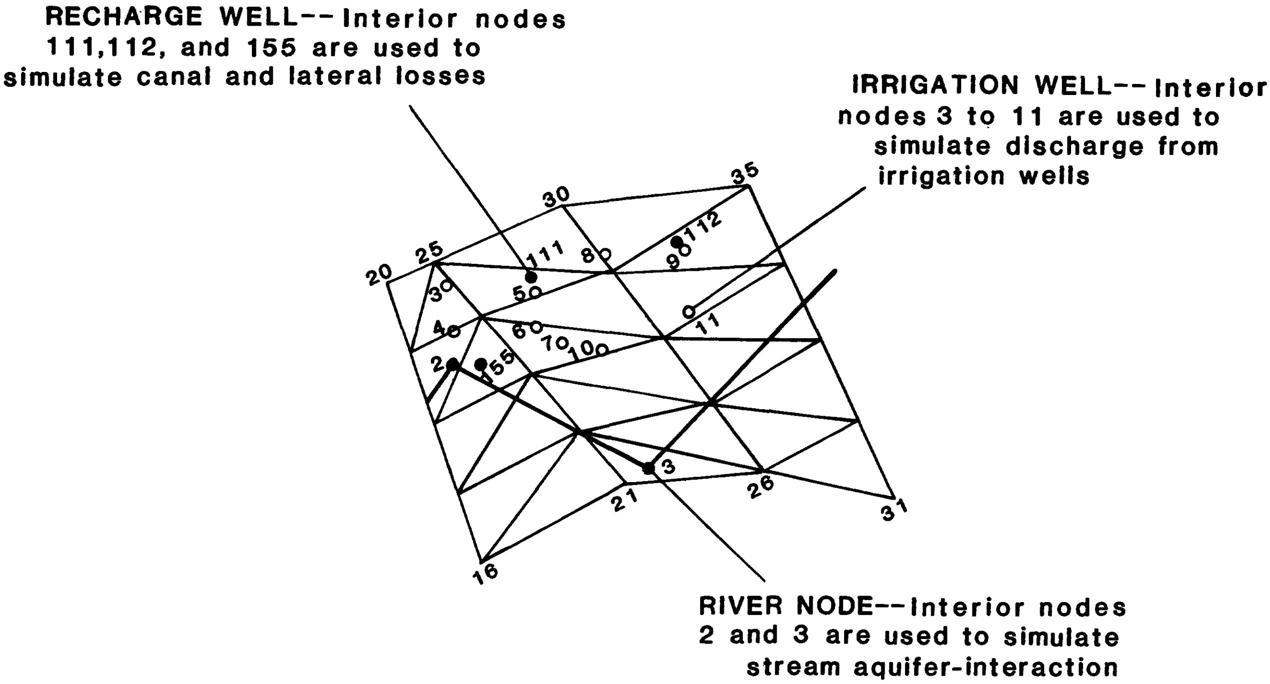


Figure 8.--Example of finite-element grid system used in mathematical simulation of stream-aquifer system (enlargement of section A on plate 1).

Discharging Wells

Discharging nodes were used to simulate the withdrawals by both irrigation and municipal wells. Some discharging nodes in the model included more than one well when wells occurred close to one another. In making transient simulations from 1970 through 1979, each year was divided into two pumping periods, one period representing the nonirrigation season (September through May) and the other period representing the irrigation season (June through August). As previously mentioned, the location of municipal and irrigation sites at which withdrawals occurred are shown on plate 1, and the total quantities of water withdrawn from the wells during the calibration period are shown in figure 6.

Canal and Lateral Flux

Water losses from the Kirwin Canal and laterals during the 1970-79 irrigation seasons are depicted in figure 9. The data used to compile figure 9 were supplied by the U.S. Bureau of Reclamation (Allacher, 1980). Forty-six recharge wells were used to simulate main-canal losses on a seasonal basis. These 46 points were located about 1-mile apart along the main canal and the north-branch and south-branch canal segments. Thirty-three recharge wells were used to simulate the lateral losses on a seasonal basis. These points were located about 1-mile apart along the laterals.

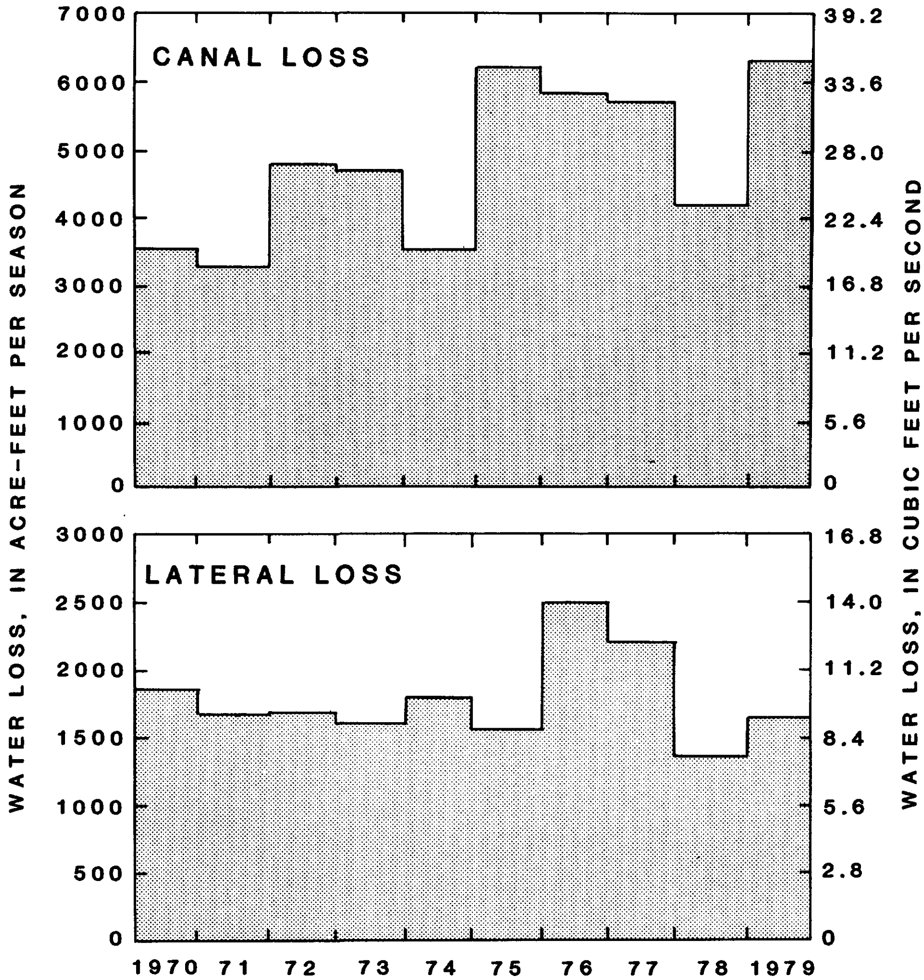


Figure 9.--Water losses from Kirwin Canal and laterals during 1970-79 irrigation seasons (data from Allacher, 1980).

The relationships between surface-water supply and water lost from the main canal and laterals are shown in figures 10 and 11. The graph shown in figure 10 indicates that main-canal losses increase relatively rapidly with the increase in surface-water supply until the main-canal losses reach about 3,000 acre-feet per season, at which point the main-canal losses increase less rapidly. The graph shown in figure 11 indicates that lateral losses are related almost linearly to the surface-water supply for all plotted values of surface-water supply. The relationship between surface-water supply and water delivered to farms is shown in figure 12. The amount of water delivered to farms increases relatively less rapidly with the increase in surface-water supply until the farm delivery reaches about 6,000 acre-feet per season, at which point the farm delivery increases at a relatively more rapid rate.

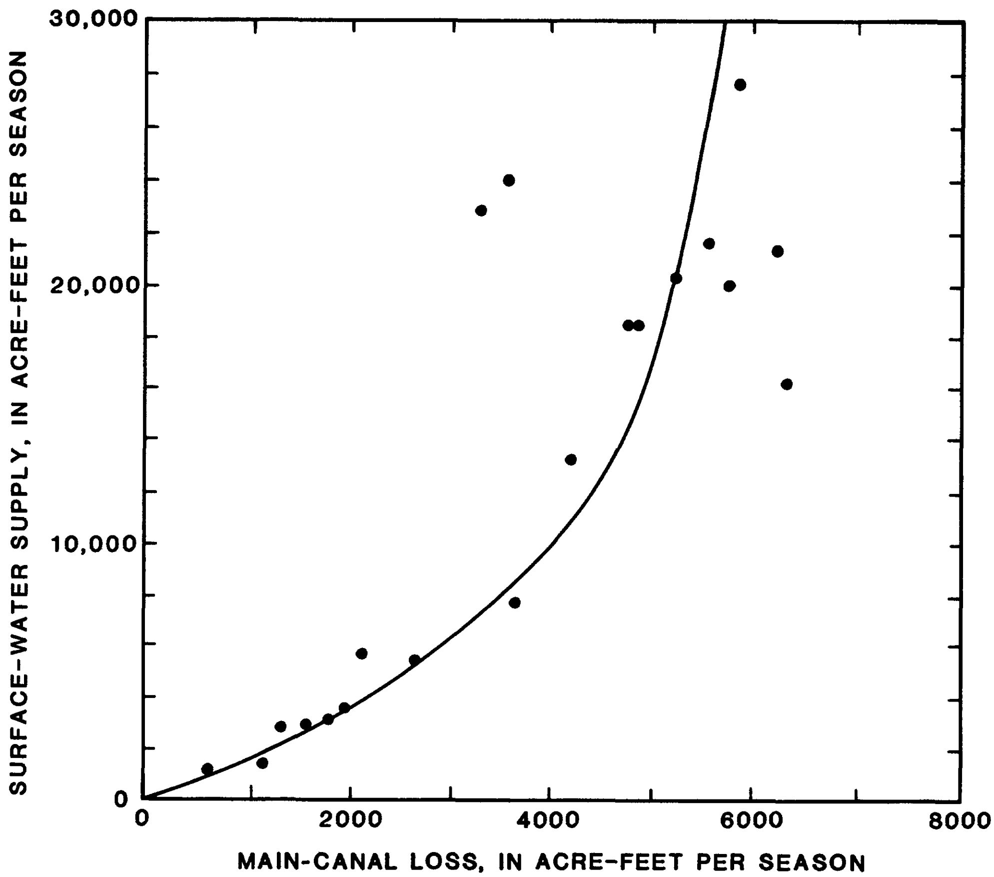


Figure 10.--Surface-water supply versus main-canal loss (data from Allacher, 1980).

Stream-Aquifer Flux

The mathematical model simulates surface-water routing along the North Fork Solomon River through the use of 64 river nodes that simulate stream-aquifer interaction. Six of the 64 nodes, including the first, also simulate surface-water inflow from tributaries along the North Fork Solomon River. Surface-water-discharge values were assigned on a daily basis at the uppermost river node to simulate daily discharge from Kirwin Reservoir and from the Deer Creek tributary into the North Fork Solomon River. Daily surface-water-discharge readings, recorded at the U.S. Geological Survey streamflow-gaging station just below Kirwin Dam, were used throughout the calibration period from 1970 through 1979.

In addition, daily discharge values were assigned to five other river nodes located at the mouths of other large, intermittent tributary streams (fig. 5). Tributaries primarily represented by the six nodes include Deer, Medicine, Cedar, Beaver, Spring, Dry, Lawrence, Joy, and Lindley Creeks.

Because the intermittent stream discharges are ungaged, daily surface-water-discharge values were calculated for the river nodes representing surface inflow from tributaries by averaging daily discharges for four gaged

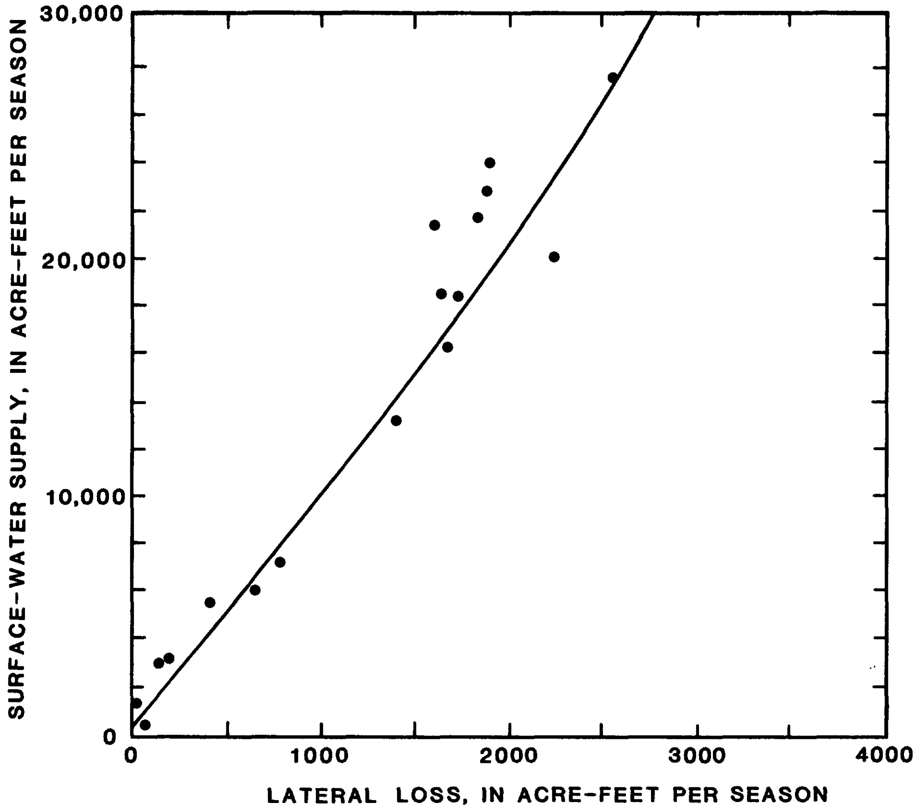


Figure 11.--Surface-water supply versus lateral loss (data from Allacher, 1980).

stream systems outside the study area. The four stream systems were White Rock Creek near Burr Oak; Bow Creek near Stockton; Deer Creek near Phillipsburg; and Kill Creek near Bloomington (U.S. Geological Survey, 1971-80). The average daily discharge values then were divided by the total drainage area included in the four stream systems to give an average daily runoff value per square mile of drainage area. This daily runoff figure was applied to each of the five stream systems draining into the North Fork Solomon River within the modeled area by multiplying the drainage area by the computed daily runoff per square mile. Surface-water-discharge values were computed for each of of the river nodes representing inflow from tributaries for each day of the simulation period from 1970 through 1979.

The computation of stream-aquifer flux for each time step follows. Using a value of discharge at a given river node, a river stage (H_{stage}) is computed from the stage-discharge relationship. The stage-discharge relationship used is shown in figure 13 and is used by the U.S. Geological Survey to determine daily discharge at the streamflow-gaging station located on the North Fork Solomon River near Portis. In addition, a stream-channel area for each river node is computed in the model from assigned length and width dimensions for each river node. Using this information along with assigned values of vertical hydraulic conductivity, streambed thicknesses, and the water level in the aquifer, Darcy's law is applied to compute a stream-aquifer flux. At the next river node downstream, the total surface-water flow computed for the upstream node is used to determine the stream-aquifer flux and the discharge at that

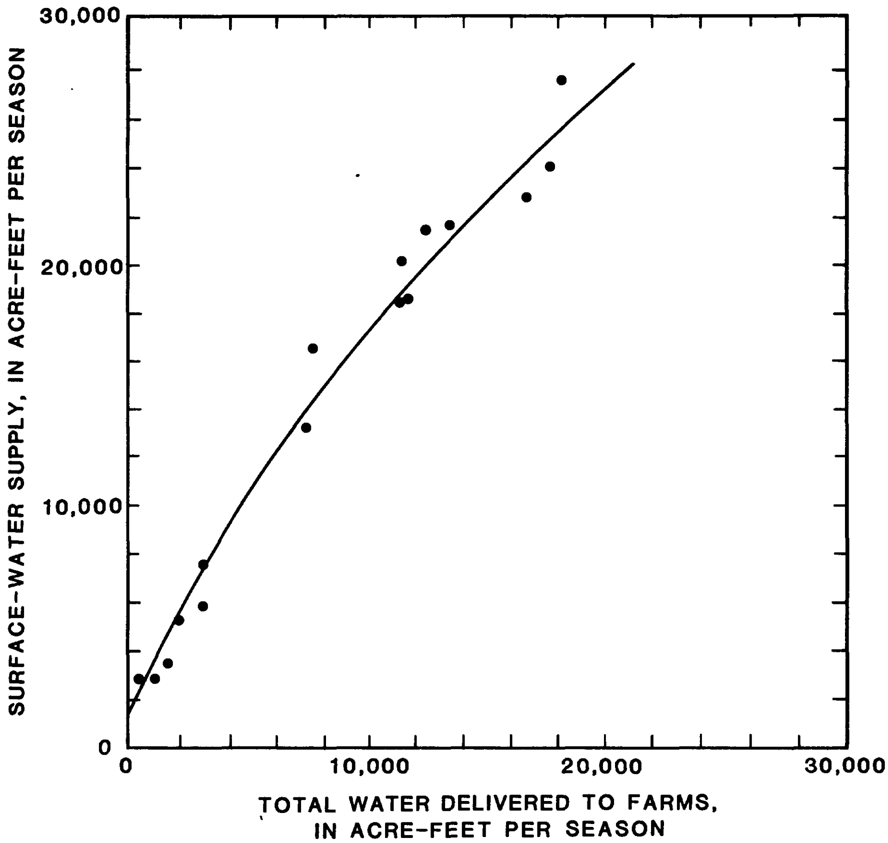


Figure 12.--Surface-water supply versus total water delivered to farms (data from Allacher, 1980).

node, and the computed discharge is carried to the next downstream node, and so forth. The above computational scheme is repeated for all nodes for each time step.

Ground-Water Evapotranspiration Flux

The general flow paths that may occur with regard to evapotranspiration are illustrated in figure 14. Water applied at the land surface is derived from precipitation and irrigation from ground- and surface-water diversions. The effective applied water rate is the computed value of applied water that is available for consumptive use. If the evapotranspiration demand is higher than the effective applied water rate and if the water table is located close to the land surface, then ground-water evapotranspiration will occur. If, on the other hand, the effective applied water rate is greater than the evapotranspiration demand, then recharge to the aquifer will occur.

Mean monthly evapotranspiration-demand values were computed from an equation developed by Eagleman (1976, p. 23):

$$E_T = C(0.035\, e_s) \quad (100 - RH)^{0.5} \quad , \qquad (2)$$

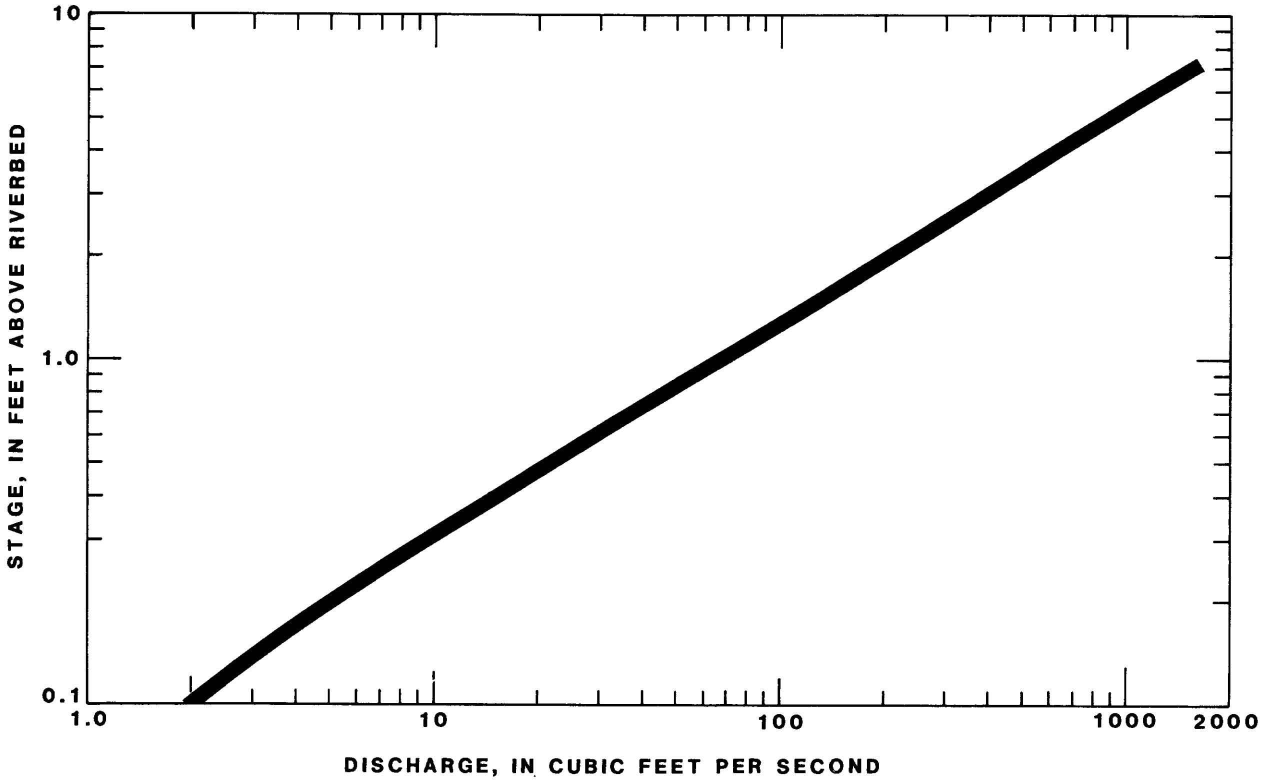


Figure 13.--Stage-discharge relationship, North Fork Solomon River near Portis.

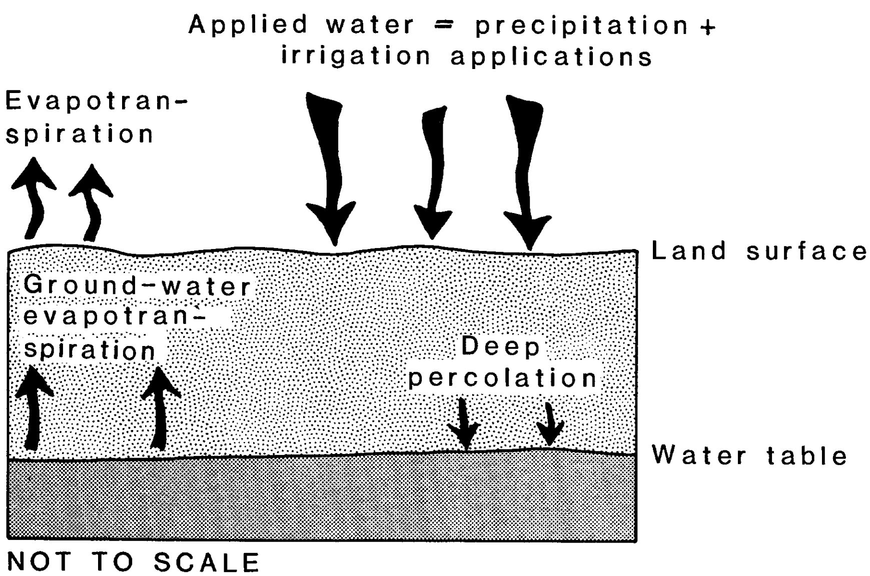


Figure 14.--Evapotranspiration processes.

where E_T is the maximum evapotranspiration demand, in inches per month;

C is the crop factor (C = 0.20 + 0.0133T, where T is mean monthly temperature between 30 and 70 °F; C = 0.6, where T is less than 30 °F; and C = 1.13, where T is greater than 70 °F);

e_s is the saturation vapor pressure, in millibars, corresponding to mean monthly temperature, T, in degrees Fahrenheit; and

RH is the mean monthly relative humidity, in percent.

Equation 2 for computing evapotranspiration demand is basically a function of mean monthly temperature and mean monthly relative humidity since the crop factor (C) and the saturation vapor pressure (e_s) are dependent on mean monthly temperature. The equation is applicable to all areas in the United States. It was shown by Eagleman (1967) that this equation could be used for calculating the evapotranspiration rate with good agreement with measured data. Tests of this method on data from Australia, Africa, and the United States show in all cases that the estimates are closer to lysimeter measurements of water-loss rates than estimates from Thornthwaite's equation, and in some cases the estimates were better than from Penman's equation. The mean monthly values of evapotranspiration demand that were computed from equation 2 are shown in figure 15.

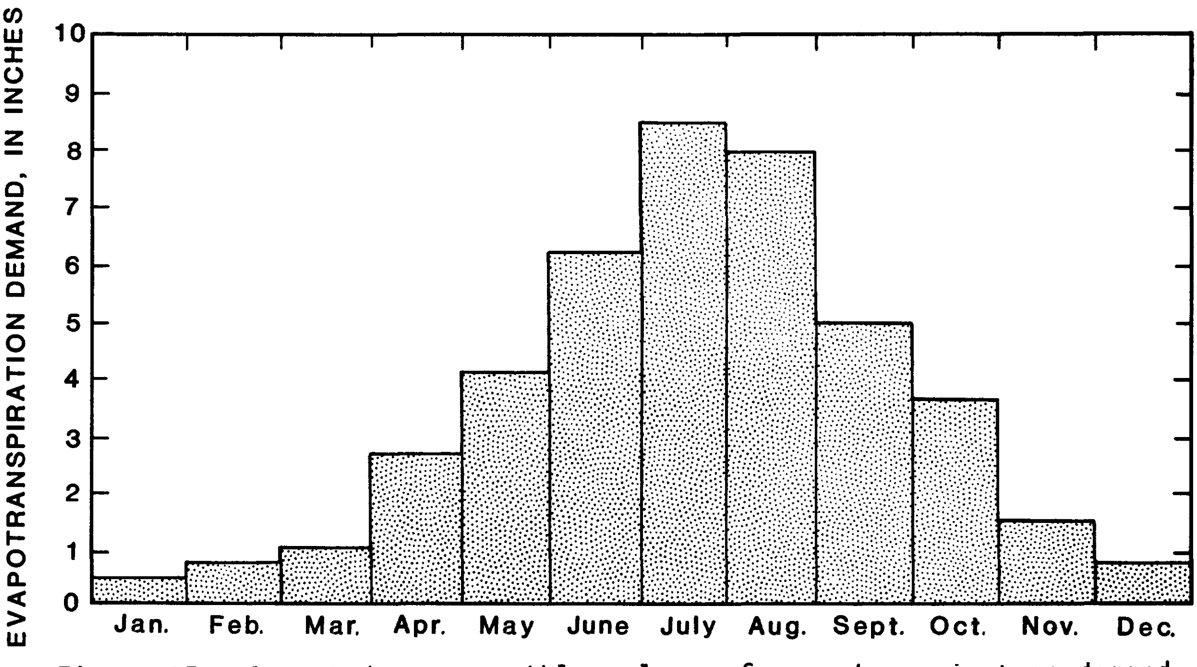


Figure 15.--Computed mean monthly values of evapotranspiration demand.

Monthly values of precipitation as recorded at a weather station at Harlan, located approximately in the center of the modeled area, are shown in figure 16 (U.S. Department of Commerce, 1971-80). These monthly rates were applied uniformly over all finite elements within the modeled area. Applied water rates from ground- and surface-water sources were

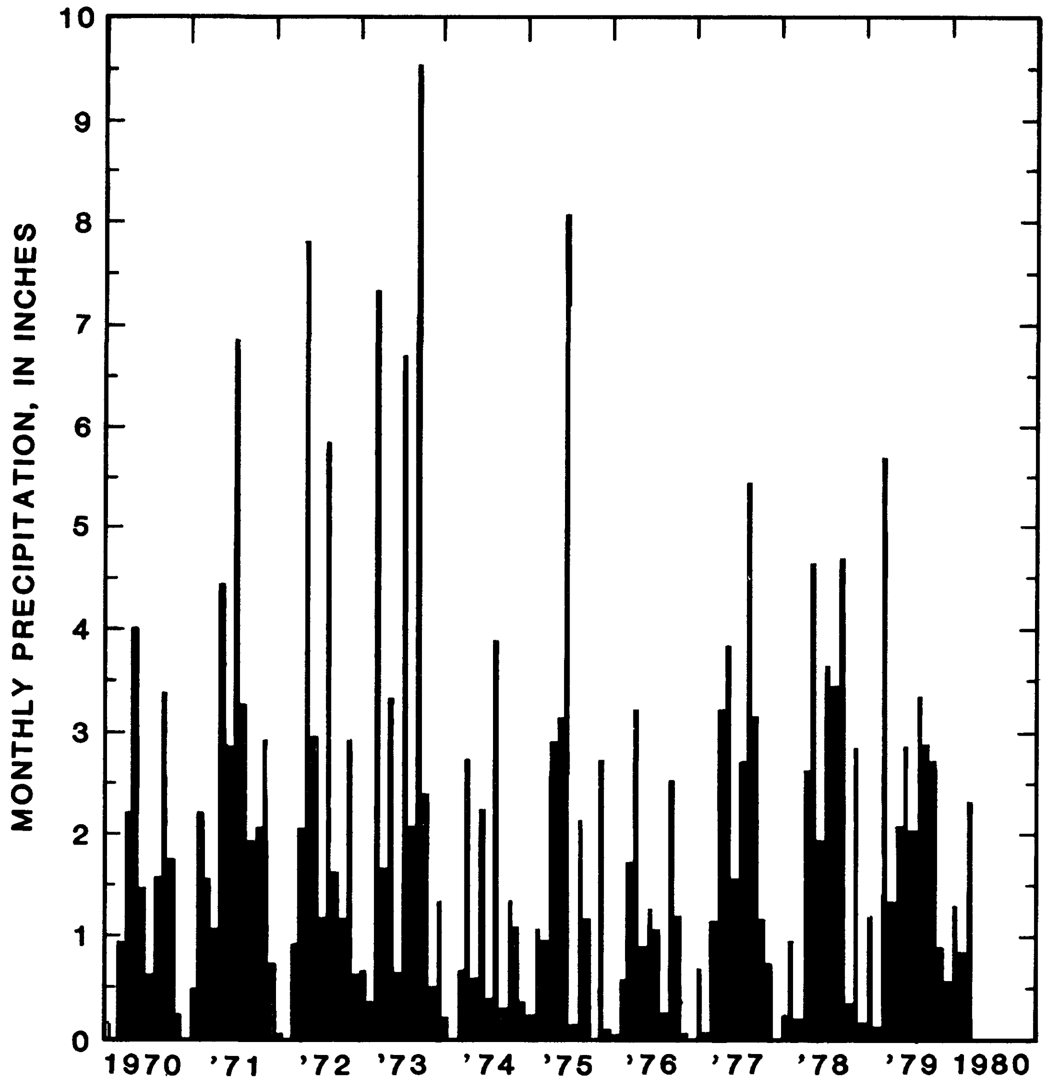


Figure 16.--Mean monthly precipitation at Harlan, 1970-79 (data from U.S. Department of Commerce, 1971-80).

superimposed over many of the elements. Each irrigation ground-water-diversion rate was applied uniformly over the corresponding element area containing an irrigation well. The diversion rates shown in figure 6 equal the total rates (due to pumping all the wells) applied over the land surface irrigated by wells. Rates of surface-water diversions to farm laterals for 1970-79 are shown in figure 17. These farm-delivery rates were supplied by the U.S. Bureau of Reclamation (Allacher, 1980). Annual farm-delivery rates were applied uniformly over all elements located within the Kirwin Irrigation District (fig. 5).

According to a county soil report that includes part of the modeled area, soil-moisture capacities vary between 0 and 13.5 inches (Fleming, 1977). During the process of model calibration, a soil-moisture capacity of 10 inches gave the best results.

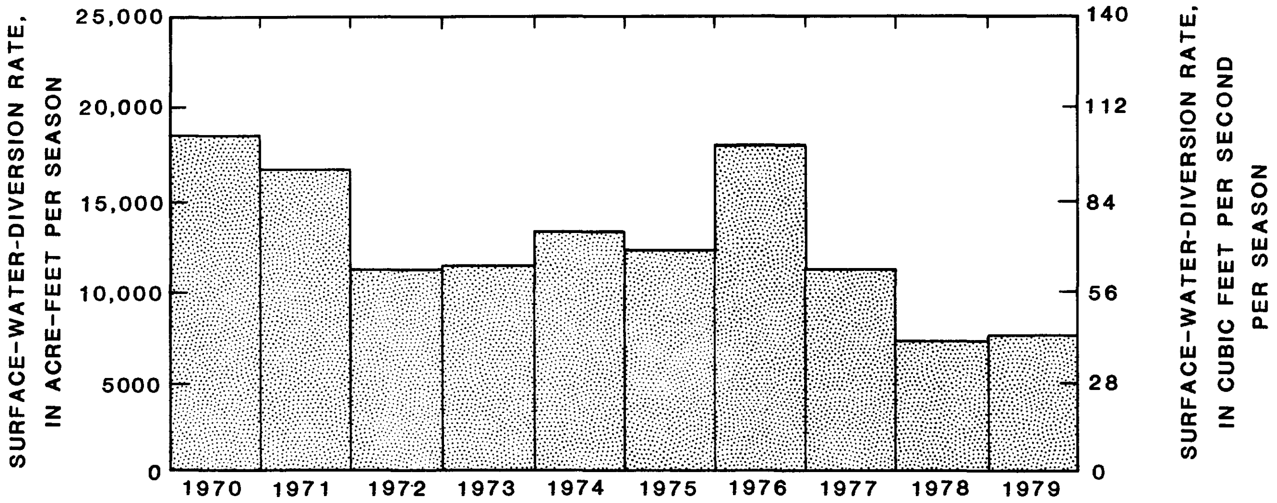


Figure 17.--Rates of surface-water diversions to farm laterals, 1970-79 (data from Allacher, 1980).

Given mean monthly values of evapotranspiration demand, total applied water rate per element (W_t), and soil-moisture capacity (SC), the effective applied water rate (W_e), is computed. The empirical formulation developed by the engineering division of the U.S. Department of Agriculture (1967) and incorporated in the model is:

$$W_e = (0.709\ W_t^{0.824} - 0.115)\ (10^{0.024\ ETD})\ f \quad , \qquad (3)$$

where

$$f = (0.532 + 0.295\ SC - 0.058\ SC^2 + 0.004\ SC^3).$$

Once the effective applied water rate is computed, comparisons are made with the evapotranspiration demand. If the total effective applied water rate is equal to or larger than the evapotranspiration demand, then evapotranspiration equals evapotranspiration demand, and any excess water goes to the saturated zone for recharge. If the total effective applied water rate is less than the evapotranspiration demand, a deficit occurs, and the model considers ground water as a possible source of replacement.

The percentage of the deficit obtained from ground water decreases linearly from 100 percent when the ground-water surface is located at land surface to zero percent when the water surface is located at the extinction depth, the depth at which water can no longer be withdrawn from the saturated zone. During the calibration process, this extinction depth was adjusted between 8 and 10 feet. A final value of 10 feet was used. The mathematical treatment of ground-water evapotranspiration, as discussed above, is illustrated in figure 18.

MODEL CALIBRATION

The stream-aquifer flow model was calibrated by comparing measured and simulated water levels and measured and simulated stream-discharge values for the North Fork Solomon River near Portis. Transient-model simulations

Input

1. Mean monthly applied water, Wt, L
2. Mean monthly evapotranspiration (ET) demand, ETD, L
3. Soil-moisture capacity, SC, L
4. Extinction depth for soil-moisture loss, D, L

↓

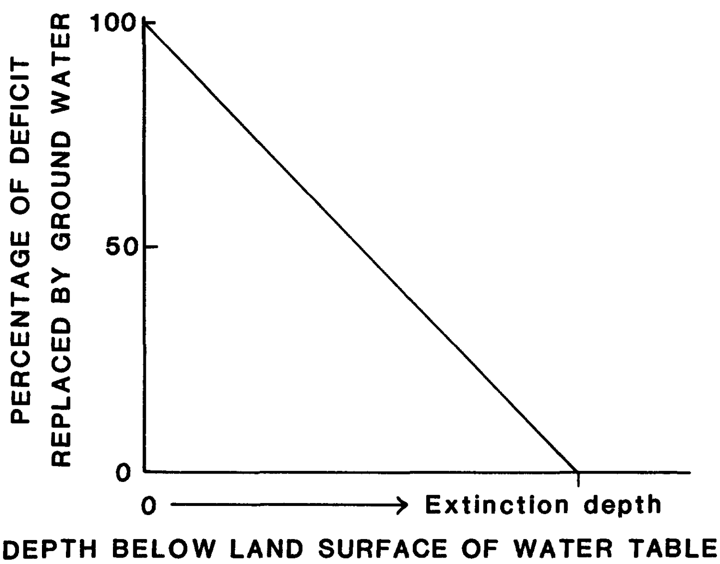
W_E, effective applied water

$$W_E = (0.709Wt^{0.824} - 0.115)(10^{0.024ETD})f$$

where $f = (0.532 + 0.295SC - 0.058SC^2 + 0.004SC^3)$

If $W_E \geq ETD$, then $ET = ETD$ and excess water goes to saturated zone

If $W_E < ETD$, then model considers ground water as a possible source of water.



↓

Output, W_E

Figure 18.--Mathematical treatment of evapotranspiration.

started with conditions during March 1970 and continued through 1979.

The calibration of the transient model began by assigning to the model the hydraulic properties and boundary conditions as described by Jorgensen and Stullken (1981). The reader is referred to that report for a more complete description of the hydrology of the aquifer, such as bedrock contours. The final calibrated value for hydraulic conductivity, 150 ft/d, was the same as that used by Jorgensen and Stullken (1981). Values of hydraulic conductivity were investigated over approximately the same range as those authors, with the final conclusion that a hydraulic conductivity of 150 ft/d gave the best fit to the calibration criteria. A specific yield of 0.2 was used, which was determined to represent the best value for transient simulations in a study of the adjacent South Fork Solomon River valley (Burnett and Reed, 1982). During the calibration procedure, calculated potential-evapotranspiration values were reduced by 16 percent. In addition, vertical hydraulic-conductivity values of the streambed material and the flux at "constant-flux" nodes (used to simulate the subsurface flows moving into the study area along the intermittent tributaries and adjacent terrace deposits) were adjusted to give best results in terms of simulating water levels and stream-discharge values. The total flux into the modeled area from the constant-flux nodes is given in table 1 as tributary and terrace-deposit inflow. Assuming a streambed thickness of 1 foot, values of streambed vertical hydraulic conductivity ranged from 4.5×10^{-2} to 1.4×10^{-1} ft/d.

Contours of water levels simulated for December 31, 1979, were compared to altitudes of water levels measured during January 1980, as shown in figure 19. Some of the disparity between simulated and measured water levels results because the simulated water levels are not computed exactly for the locations of the wells. Other differences could be due to close approximation of observation wells to canals, uniformly modeled hydraulic conductivity, and differential leakage along canals instead of the assumed uniform leakage along the canal.

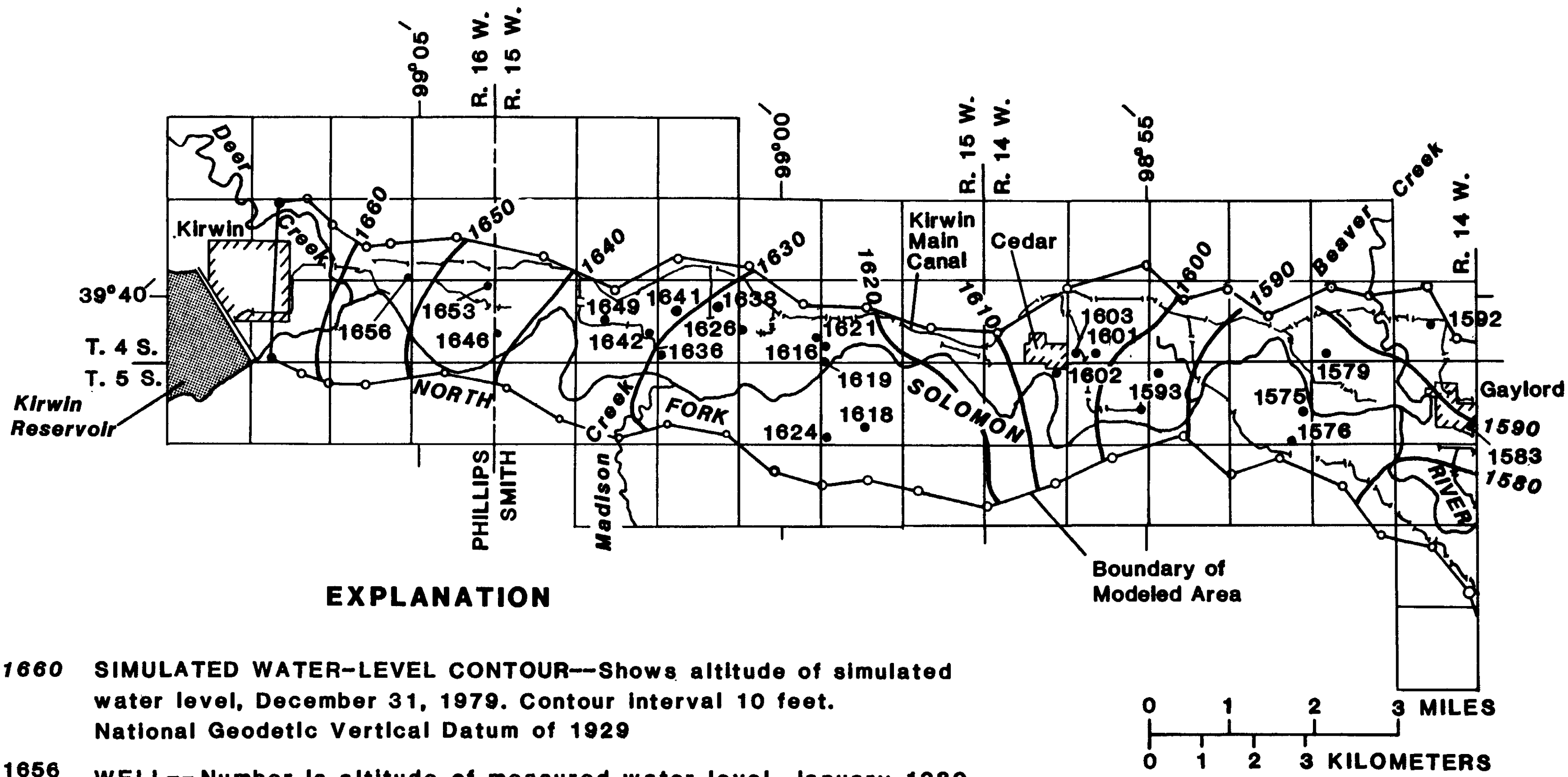
A hydrograph of measured and simulated average monthly discharge values for the North Fork Solomon River near Portis is shown in figure 20. Measured and simulated results agree closely during periods of base-flow conditions, when little or no runoff occurs from the valley sides. However, during periods of storms and significant runoff, measured and simulated results do not agree. These differences are due to errors resulting from the method used for estimating surface-water discharge from the ungaged intermittent tributaries of the North Fork Solomon River. The corresponding difference in river stage resulting from the discrepancies in measured and simulated discharges during periods of peak runoff is, at most, approximately 1 foot. Experimental model simulations showed that during periods in which simulated discharges were either greater or less than measured discharges, the difference in the response of the water table was insignificant, indicating the insensitivity of the aquifer to this modeling error.

Simulated average base flow for the calibration period (1970-79) is in the range determined by several investigators. An analysis by Busby and Armentrout (1965), which involved hydrograph separations to determine base flow along with data collected between 1919 and 1955, indicated a long-term average base flow within the study area of 28.7 ft^3/s; this

Table 1.--Simulated average component flow rates, 1970-79

[Values are given in cubic feet per second]

	Land surface	
	Recharge	Discharge
Precipitation	140.25	0
Irrigation		
Ground water	2.32	0
Surface water	18.33	0
Actual evapotranspiration	0	139.00
TOTALS	160.90	139.00
NET	21.90	0
	Aquifer	
	Recharge	Discharge
Deep percolation	21.90	0
River loss	0	22.92
Subsurface inflow	0.83	0
Subsurface outflow	0	0.45
Tributary and terrace-deposit inflow	4.00	0
Pumpage	0	2.92
Ground-water evapotranspiration	0	10.21
Leakage from surface-water-distribution system	9.63	0
Change in storage	.14	0
TOTALS	36.50	36.50



EXPLANATION

——*1660* SIMULATED WATER-LEVEL CONTOUR--Shows altitude of simulated water level, December 31, 1979. Contour interval 10 feet. National Geodetic Vertical Datum of 1929

1656 ● WELL--Number is altitude of measured water level, January 1980, in feet. National Geodetic Vertical Datum of 1929

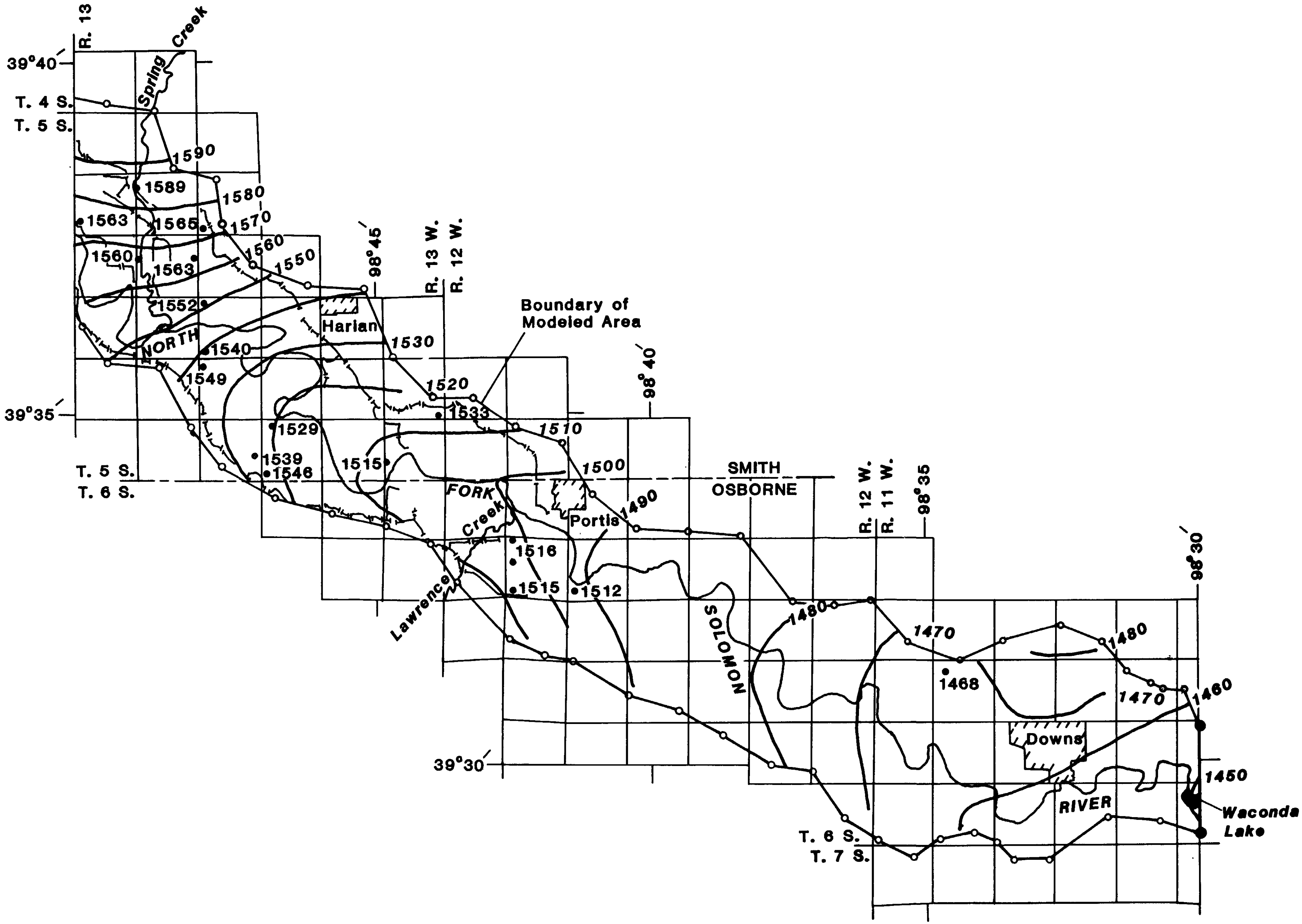


Figure 19.--Comparison of water-level contours simulated for December 31, 1979, and water-level altitudes measured during January 1980, within modeled area.

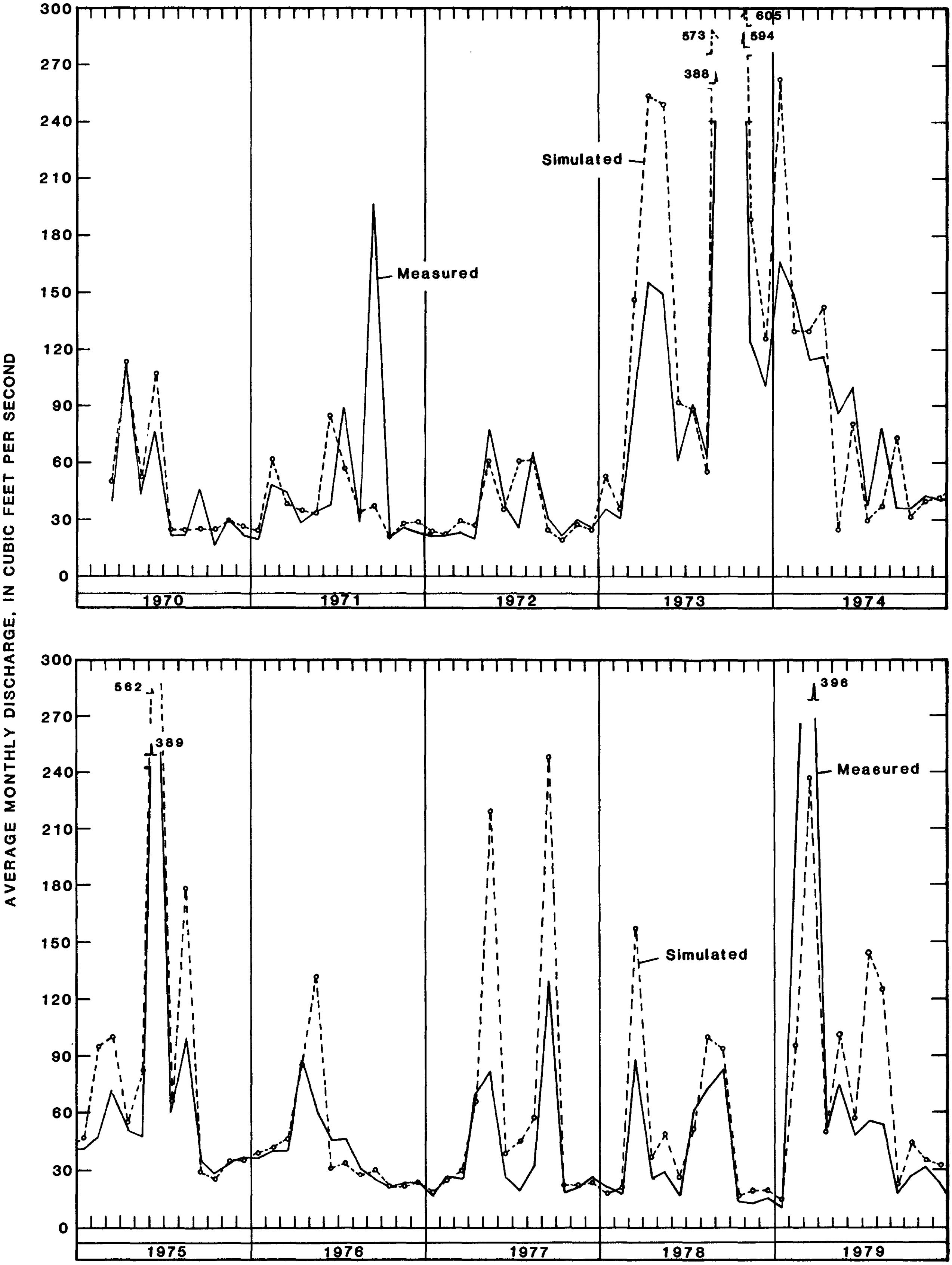


Figure 20.--Measured and simulated average monthly discharge for North Fork Solomon River near Portis, 1970-79.

value includes an adjustment for reduced channel length compared to their analysis. Measurements of base flow made on August 8, 1976, and November 10, 1976 (Jorgensen and Stullken, 1981), indicated a weighted average base flow of 22 ft^3/s in 1976. The model simulated an average base flow of 22.9 ft^3/s for 1970-79 and averaged 22 ft^3/s for 1976. It was assumed that the construction of Kirwin Dam and Reservoir did not significantly change base-flow conditions along the reach of the North Fork Solomon River within the study area.

Average rates of flow into and out of the system, including both the unsaturated and saturated zones, are given in table 1. The component rates of flow represent average conditions for 1970-79. Precipitation provided the greatest rate of inflow to the system at 140.25 ft^3/s. Leakage from the surface-water-distribution system (9.63 ft^3/s) and surface water applied to the land (18.33 ft^3/s) provided the next greatest average component of inflow. Deep percolation occurred at an average rate of 21.90 ft^3/s resulting from precipitation and applied water for irrigation.

Evapotranspiration from the unsaturated zone (139.00 ft^3/s) provided the greatest component of outflow. Other significant outflow components included leakage to the river (22.92 ft^3/s) and ground-water evapotranspiration (10.21 ft^3/s). The storage of water within the aquifer decreased at an average rate of 0.14 ft^3/s during 1970-79.

PREDICTIVE SIMULATIONS OF MANAGEMENT ALTERNATIVES

Eight predictive simulations of alternatives for managing the water resources of the study area were made. The four management schemes involved simulating the resultant hydraulic heads and effects on the stream-aquifer system by:

(1) assuming that the remaining 70 percent of the Kirwin Canal is lined with clay (simulation 1);

(2) reducing the availability of surface water by 25, 75, and 100 percent, while maintaining the 1979 ground-water-diversion rate (simulations 2, 3, and 4);

(3) reducing the availability of surface water by 25, 75, and 100 percent, while at the same time increasing the ground-water-diversion rate to compensate for the decrease in surface water for irrigation (simulation 5, 6, and 7); and

(4) assuming a continuation of 1979 pumping conditions (simulation 8).

Model data for all the predictive simulations mentioned above included an average surface-water supply, based on long-term discharge readings from 1960 through 1979, but modified by assumptions for a given simulation. Mean monthly precipitation used in the evapotranspiration routine were based on recordings made at Harlan and are the normal monthly precipitation values, 1941-70. Mean monthly precipitation and mean monthly evapotranspiration demand used in the simulations are presented in the following tabulation, in inches:

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Mean monthly evapotranspiration demand	0.49	0.63	1.10	2.25	3.49	5.30	7.15	6.74	4.24	3.01	1.17	0.62
Mean monthly precipitation	0.47	0.73	1.41	2.15	3.52	4.62	3.46	2.80	2.84	1.53	0.73	0.59

Surface-water discharges to the North Fork Solomon River from Kirwin Reservoir were based on long-term average daily values for the U.S. Geological Survey streamflow-gaging station located just below Kirwin Dam (North Fork Solomon River at Kirwin). Long-term average daily surface-water discharges to the North Fork from ungaged intermittent tributaries flowing into the modeled area from the valley sides were estimated from average daily discharge readings taken at four surrounding streamflow-gaging stations, as previously discussed. On a daily basis, an average discharge per square mile was computed by averaging discharge per square mile of drainage area at each of the four gaging stations. This average daily runoff per square mile then was applied on a daily basis to all five tributary drainage systems moving into the North Fork Solomon River system by multiplying this average runoff factor by the drainage area of each of the five systems.

For all simulations the reference surface-water land application rate (farm delivery) was based on the resulting application rate (from figure 10) when a surface-water supply of 20,400 acre-feet is assumed to be available. This rate is 69.46 ft^3/s, and from this base value reductions were made as indicated for each projection. The surface-water supply of 20,400 acre-feet was determined from the average water supply from 1960 through 1979. Adjustments to this supply and other values for projections 1 through 8 are shown in table 2.

Simulation 1

Simulation 1 involved simulating hydraulic heads and effects on the stream-aquifer system by assuming that the remaining 70 percent of the Kirwin Canal is clay lined. As reported by personnel of U.S. Bureau of Reclamation, a total savings of 19.0 ft^3/s per season would occur along the canal. Simulation 1 was performed using a value of canal losses of 29.13 ft^3/s as computed from figure 10 by assuming a surface-water supply of 20,400 acre-feet. The resulting leakage after subtracting the savings of 19.0 ft^3/s from 29.13 ft^3/s is 10.13 ft^3/s or 1,808 acre-feet (see table 2). This computed average annual leakage loss of 10.13 ft^3/s was divided evenly among all the recharge-well sites used to simulate canal

Table 2.--Simulated losses in main canal and laterals, farm-delivery rates, and increases in ground-water-pumpage rates based on 25-, 75, and 100-percent reductions in surface-water supply

	Surface-water supply (acre-feet per year)	Computed canal loss (acre-feet per year)	Computed net lateral loss (acre-feet per year)	Farm delivery (acre-feet per year)	Canal waste (acre-feet per year)	Total increase in ground-water pumping (cubic feet per second)	Increase per irrigation well (cubic feet per second)
Simulation 1	20,400	1,808	1,899	12,400	900	--	--
Simulation 2 (25-percent reduction in supply)	15,300	4,900	1,500	8,050	850	--	--
Simulation 3 (75-percent reduction in supply)	5,100	2,600	500	1,900	100	--	--
Simulation 4 (100-percent reduction in supply)	0	0	0	0	0	--	--
Simulation 5 (25-percent reduction in supply)	15,300	4,900	1,500	8,050	850	24.4	0.2389
Simulation 6 (75-percent reduction in supply)	5,100	2,600	500	1,900	100	58.8	.5767
Simulation 7 (100-percent reduction in supply)	0	0	0	0	0	69.5	.6810
Simulation 8 (Continued 1979 pumping conditions)	20,400	5,200	1,900	12,400	900	--	--

leakage. The recharge rate during the irrigation season at each of the 46 recharge wells was 0.22 ft^3/s (10.13/46). Simulation 1 assumes that the 19.0 ft^3/s is retained in the reservoir, and therefore, laterals receive the same amount of water supply as they did before installation of the clay lining and therefore maintain the same leakage as before (10.64 ft^3/s) with this assumed water supply.

The predictive simulations for average drawdown within the modeled area (total storage change divided by model area and storage coefficient) are shown in figure 21, and the effects on net gains or losses of base flows within the study area for 1980-2000 are shown in figure 22. Average drawdown reached a maximum "steady-state" value of about 1.65 feet by 1995 (simulation 1, fig. 21). Base flow increases from about 11.4 ft^3/s during the end of 1979 to a steady-state value of about 13.0 ft^3/s by the year 2000 (simulation 1, fig. 22).

Simulations 2, 3, and 4

Simulations 2, 3, and 4 consisted of simulating hydraulic heads and effects on the stream-aquifer system by reducing the availability of surface water by 25, 75, and 100 percent, while maintaining the 1979 ground-water-diversion rate.

The relationships developed between surface-water supply versus main-canal loss, surface-water supply versus lateral loss, and surface-water supply versus water delivered to farms, as presented in figures 10, 11, and 12, were utilized (see section on "Canal and Lateral Flux"). A 25-percent reduction (simulation 2), a 75-percent reduction (simulation 3), and a 100-percent reduction (simulation 4) in the initial surface-water supply of 20,400 acre-feet were related to losses in the main canal, canal laterals, and to farm delivery. The calculated losses in the canal mains and laterals and in farm-delivery rates based on 25-, 75-, and 100-percent reductions in surface-water supply are shown in table 2.

The results of simulations 2, 3, and 4 due to computed effects on average drawdown and on base flows within the modeled area are shown in figures 21 and 22, respectively. The average drawdown throughout the modeled area reached the maximum "steady-state" values of about 1.25 feet for simulation 2, 2.25 feet for simulation 3, and 3.70 feet for simulation 4 by the year 2000, as indicated in figure 21. Base flow for simulation 2 increased to a "steady-state" value of about 14.3 ft^3/s by the year 2000, as shown in figure 22. Base flow decreased to 8.2 ft^3/s for simulation 3 and to 4.3 ft^3/s for simulation 4 by the year 2000.

Simulations 5, 6, and 7

Simulations 5, 6, and 7 consisted of simulating hydraulic heads and effects on the stream-aquifer system by reducing the net supply of surface water by 25, 75, and 100 percent, while increasing the ground-water-diversion rate simultaneously to compensate for the decrease in surface water available for irrigation. For each of these three simulations, main-canal and lateral losses and farm-delivery rates were based on the relationships developed for simulations 2, 3, and 4 and were equal to the losses computed for those simulations.

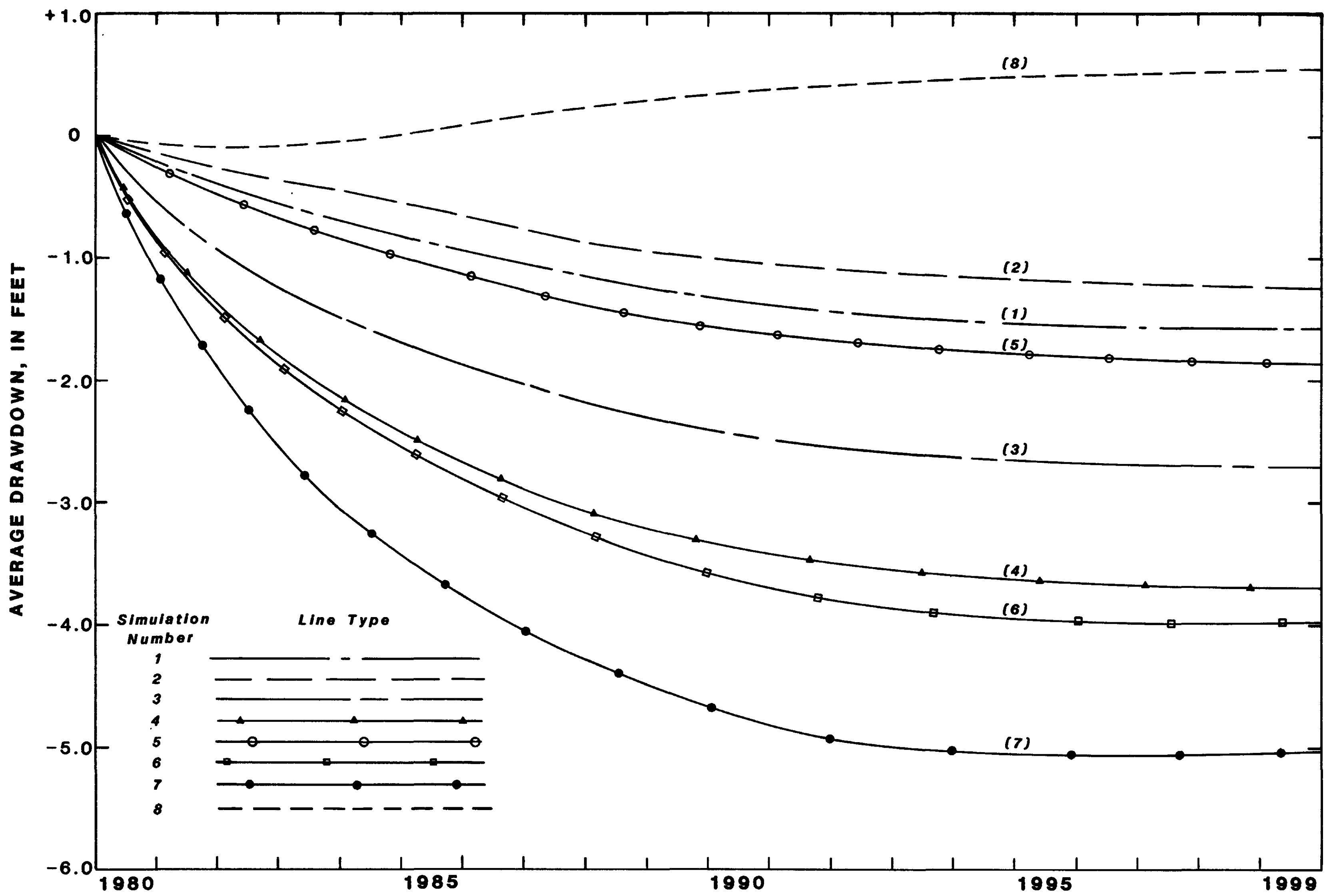


Figure 21.--Average drawdown versus time for all predictive simulations, 1980-2000.

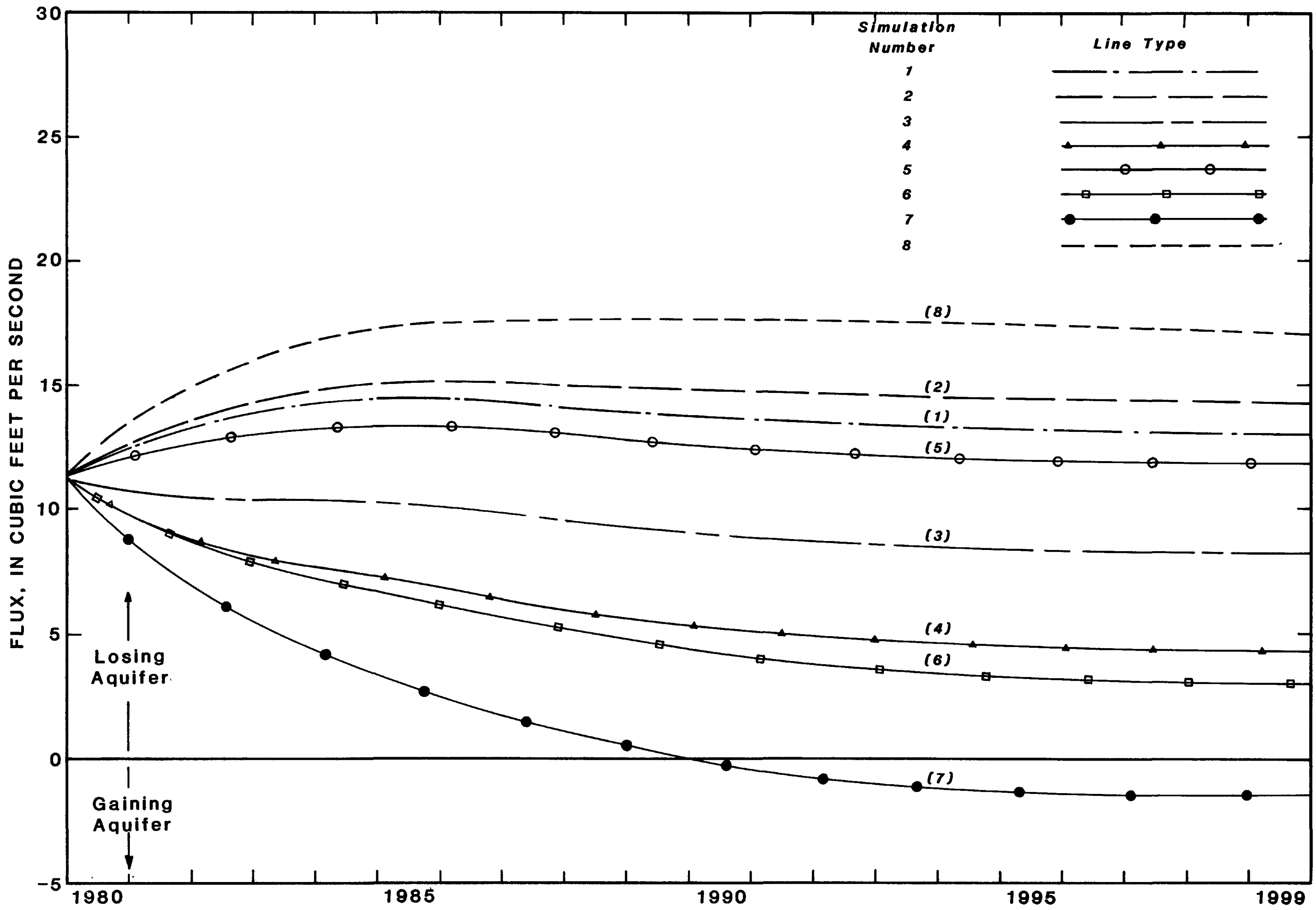


Figure 22.--Stream-aquifer flux versus time for all predictive simulations, 1980-2000.

Increases in ground-water pumping rates were determined for each simulation by subtracting the computed farm-delivery rate for each simulated reduction in surface-water supply from the farm-delivery rate determined for the average water supply of 20,400 acre-feet. This difference represents the net increase in the ground-water pumping rate for all irrigation wells located within the Kirwin Irrigation District. Irrigation-well pumping rates were increased uniformly among all wells located within the irrigation district.

The results of computed losses in the main canal and laterals and in farm-delivery rates and of computed increases in ground-water pumping rates are shown in table 2. Results of simulations 5, 6, and 7 due to computed effects on average drawdown and base flows within the modeled area are shown in figures 21 and 22. The average drawdown throughout the modeled area reached maximum "steady-state" values by 2000 of about 1.85 feet for simulation 5, 3.95 feet for simulation 6, and 5.5 feet for simulation 7, as shown in figure 21. Base flow increased by 2000 to a constant value of about 11.8 ft^3/s for simulation 5 and decreased to a constant value of about 3.0 ft^3/s for simulation 6 and to a value of about -1.4 ft^3/s for simulation 7, as shown in figure 22.

Simulation 8

Simulation 8 consisted of simulating hydraulic heads and effects on the stream-aquifer system by projecting the 1979 irrigation and non-irrigation pumping conditions to 2000. Long-term average values of precipitation, surface-water availability, and river and tributary flow rates were assumed to remain constant.

Results of simulation 8 for average drawdown and effects on net gains or losses of base flows within the study area for 1980-2000 are shown in figures 21 and 22. The average drawdown response throughout the modeled area reached a "steady-state" value of about 0.55 foot less than that of 1979 by the year 2000, as shown in figure 21. Base flow decreased to a constant value of about 17.0 ft^3/s by 2000, as shown in figure 22.

As shown in figure 21, the stream-aquifer flux increased relative to that at the end of 1979 for simulations 1, 2, 5, and 8. Also, as shown in figure 20, the average drawdown is less than that at the end of 1979 for simulation 8. The increases in stream-aquifer flux and the decrease in average drawdown are because of the greater average precipitation and available surface-water supplies assumed for the projection runs relative to the actual amounts available during the later part of the 1970's.

The model data indicate that the aquifer has the capacity to sustain the present (1979) amount of ground-water pumpage without strong adverse effects on streamflow or aquifer storage with the assumed amounts of projected available precipitation and surface-water supplies.

SUMMARY AND CONCLUSIONS

The alluvial aquifer in the North Fork Solomon River valley between Kirwin Dam and Waconda Lake underlies an area of about 100 square miles and has saturated thicknesses ranging from near zero at the valley sides to as much as 55 feet near the center of the valley. The alluvial material has contained sufficient quantities of water to sustain pumpage from municipal and irrigation wells. However, since 1974 water levels in the study area generally have decreased due to increases in ground-water pumpage, decreases in applied surface water for irrigation, and below-average precipitation.

A finite-element model was used to simulate transient conditions between 1970 and 1979. The model was calibrated by comparing measured and simulated water levels and measured and simulated streamflow responses. Model results indicate that precipitation and canal and lateral leakage are the major sources of aquifer recharge. Between 1970 and 1979, recharge to the aquifer per year due to precipitation and application of water for irrigation was found to average about 15,850 acre-feet per year year. Between 1960 and 1979, an annual average of about 20,400 acre-feet of water was released from Kirwin Reservoir into the irrigation system during the irrigation seasons. Of this amount, an average of approximately 6,110 acre-feet leaked from the irrigation canal and laterals to the saturated zone, representing a loss of about 30 percent of the amount released.

The model study showed that major components of aquifer discharge consist of discharge to the river, ground-water evapotranspiration, and ground-water pumping for irrigation. The amount of ground water used for irrigation increased rapidly during the 1970's, from about 600 acre-feet during 1970 to about 3,300 acre-feet during 1979. Model simulations for 1970 through 1979 indicated that the ground-water discharge to the stream averaged about 22.9 ft^3/s or 16,590 acre-feet per year.

Predictive model simulations were made based on management alternatives involving the clay-lining of irrigation ditches, the reduction of surface-water availability with and without an increase in ground-water pumping, and the continuation of 1979 pumping conditions. The predictive simulations were made to the year 2000, based on long-term average monthly precipitation and on long-term average daily streamflow. Results of the predictive simulations were reported in terms of effects on average drawdown within the modeled area and effects on base flow.

The predictive simulations indicated that as much as 5.5 feet of additional average drawdown after 1979 would occur by the year 2000 if the surface-water supply were reduced 100 percent and ground-water pumpage increased (simulation 7). In this case, the stream system would become a losing system, losing as much as 1.4 ft^3/s or about 1,000 acre-feet per year by the year 2000. The predictive simulations also indicated that a rise in average water levels of 0.55 foot after 1979 would occur by the year 2000 and that the base flow to the stream would increase to 17.0 ft^3/s or 12,300 acre-feet per year by the year 2000 if 1979 pumping conditions remained constant, if a surface-water supply equal to the average supply from 1960 to 1979 was available, and if precipitation was equal to the normal precipitation from 1941 to 1970 (simulation 8). The analyses and results of this study indicate that simulation 8 had the least effect on water-level and base-flow declines.

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