DEVELOPMENT AND CALIBRATION OF A TWO-DIMENSIONAL DIGITAL MODEL FOR THE ANALYSIS OF THE GROUND-WATER FLOW SYSTEM IN THE SAN ANTONIO CREEK VALLEY, SANTA BARBARA COUNTY, CALIFORNIA

By Peter Martin

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CONTENTS

ILLUSTRATIONS

TABLES

CONVERSION FACTORS

For this report, the inch-pound system of units was used. For those readers who may prefer metric units rather than inch-pound units, the conversion factors for the terms used in this report are listed below:

Multiply	By	To obtain
acres	4,047	square meters
acre-ft (acre-feet)	1,233	cubic meters
acre-ft/yr (acre-feet per year)	1,233	cubic meters per year
ft (feet)	0.3048	meters
ft/d (feet per day)	0.3048	meters per day
ft^2/d (feet squared per day)	0.0929	meters squared per day
ft^3/s (cubic feet per second)	0.02832	cubic meters per second
ft/mi (feet per mile)	0.1894	meters per kilometer
ft/yr (feet per year)	0.3048	meters per year
gal/min (gallons per minute)	0.003785	cubic meters per minute
inches	25.4	millimeters
mi (miles)	1.609	kilometers
mi^2 (square miles)	2.590	square kilometers

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ABSTRACT

A two-dimensional finite-difference model was developed and used to simulate ground-water flow conditions in San Antonio Creek valley, Santa Barbara County, California. The purpose of the study was to update a water budget compiled in an earlier phase of the study and to develop a tool to help manage the ground-water resources.

The ground-water basin in the valley underlies an area of about 110 square miles and consists of unconsolidated valley-fill deposits that range in thickness from zero feet along the perimeter of the basin to about 3,000 feet in the central part of the basin. At the western edge of the ground-water basin, about 5 miles east of the Pacific Ocean, consolidated rocks form a barrier to the seaward flow of ground water. Upwelling of ground water east of the barrier has created a 660-acre marshland known as Barka Slough.

The development of the mathematical ground-water model of the valley was begun by simulating steady-state conditions as approximated by ground-water conditions in 1943. During steady-state conditions, the basin received about 6,200 acre-feet of water per year from rainfall and stream seepage. The same amount was discharged by pumpage, evapotranspiration, and base flow along the San Antonio Creek channel. The transmissivity of the aquifer and the vertical hydraulic conductivity of the confining bed underlying Barka Slough were calibrated during the steady-state simulation of the model.

The model was then calibrated to simulate transient or time-dependent conditions during 1944-77. Net-flux values (the difference between recharge and net discharge) were adjusted during the transient-state simulation so that computed water-level declines matched measured values. The net-flux values along the San Antonio Creek channel were determined by a least-squares calibration technique developed for this study. Results of the calibration indicated that the net flux out of the system along the stream channel increased by about 4,030 acre-feet per year from 1943 to 1977. Model results indicated that about 70 percent of the increase in net flux was derived from the reduction in ground-water discharge to Barka Slough and about 30 percent from water coming out of storage.

To verify the model and the calibrated input values, the model was used to simulate the period 1978-82 using the annual pumpage determined during this study. During the verification period, model-generated water-level changes duplicated measured changes throughout most of the ground-water basin. The results of this run verified the model and the calibrated input values within acceptable limits of cost and time. The calibrated model can now be used to predict water levels based on different management schemes.

INTRODUCTION

Demand for ground water in the predominantly rural San Antonio Creek valley has increased significantly since 1978 because of the establishment of extensive irrigated vineyards on formerly nonirrigated pastureland and increased pumpage from the valley by Vandenberg Air Force Base (VAFB). Future transfer of pastureland to irrigated vineyards and planned programs at VAFB, such as the space shuttle and MX missile testing, will further increase the demand for water in the valley. Previous studies (Muir, 1964; Hutchinson, 1980) have indicated that pumpage in the valley was already greater than perennial yield prior to 1978. To plan for anticipated growth in the San Antonio Creek valley, there is a need to develop methods to evaluate and predict changing ground-water conditions resulting from current and proposed pumpage in the valley.

Purpose and Scope

In 1976 the U.S. Air Force, VAFB, entered into a cooperative study with the U.S. Geological Survey to evaluate the long-term availability of the ground-water supply in San Antonio Creek valley. The first phase of the program, completed in 1979 (Hutchinson, 1980), involved developing a comprehensive water budget for the valley for 1958-77. The purpose of the second and current phase of the program was to use the water budget to develop a computer model of the valley that could be used as a predictive tool for testing alternative ground-water management plans. The third and final phase of the program will be to use the model to manage the water resources of the valley.

The study involved: (1) Updating, evaluating, tabulating, and computer processing pumpage and other hydrologic data compiled for the water budget previously developed for the valley; (2) developing a steady-state and transient-state computer ground-water flow model; and (3) verifying the model and the calibrated input values with data collected during the study period.

Description of the Study Area

The San Antonio Creek valley consists of a 154-mi^2 area in the west-central part of Santa Barbara County about 55 miles northwest of Santa Barbara (fig. 1). The valley is about 30 miles long and 7 miles wide and lies between the Santa Maria Valley to the north and the Santa Ynez Valley to the south. The Solomon and Casmalia Hills form the northern border of the valley and the Purisima Hills form the southern border. The surrounding hills slope steeply to a narrow valley floor that slopes gently westward from an altitude of about 800 feet in the eastern part to sea level at the Pacific Ocean.

The ground-water basin in the valley underlies an area of about 110 mi^2 and consists of unconsolidated valley-fill deposits. The basin is bounded by relatively impermeable consolidated rocks of the Purisima, Casmalia, and Solomon Hills. At the western end of the ground-water basin, about 5 miles east of the Pacific Ocean, consolidated rocks form a barrier to the seaward flow of ground water. Upwelling of ground water east of the barrier has created a 660-acre marshland known as Barka Slough. This slough, one of the few pristine marshlands in southern California, is known or believed to be inhabited by at least nine threatened or endangered species of wildlife (Descheneaux, 1975). San Antonio Creek runs the length of the valley and is fed by tributaries that are slowly dissecting the surrounding hills. All the streams in the area are intermittent except for San Antonio Creek west of Barka Slough. Ground water forced to the surface by the consolidated rock barrier on the western edge of the slough results in perennial flow in the stream from the barrier to the ocean.

Land in the San Antonio Creek valley is used primarily for agriculture. Historically, the upland parts of the valley have been used for dry farming or pastureland and the flatlands along the streams for irrigated farming. Since 1980, however, large sections of formerly unirrigated pastureland in the upland parts of the valley have been converted to irrigated vineyards.

The western quarter of the valley is owned by VAFB, and the remainder of the valley is privately owned. The town of Los Alamos, population about 900 (Hutchinson, 1980, p. 6), covers about 0.5 mi^2 in the east-central part of the valley.

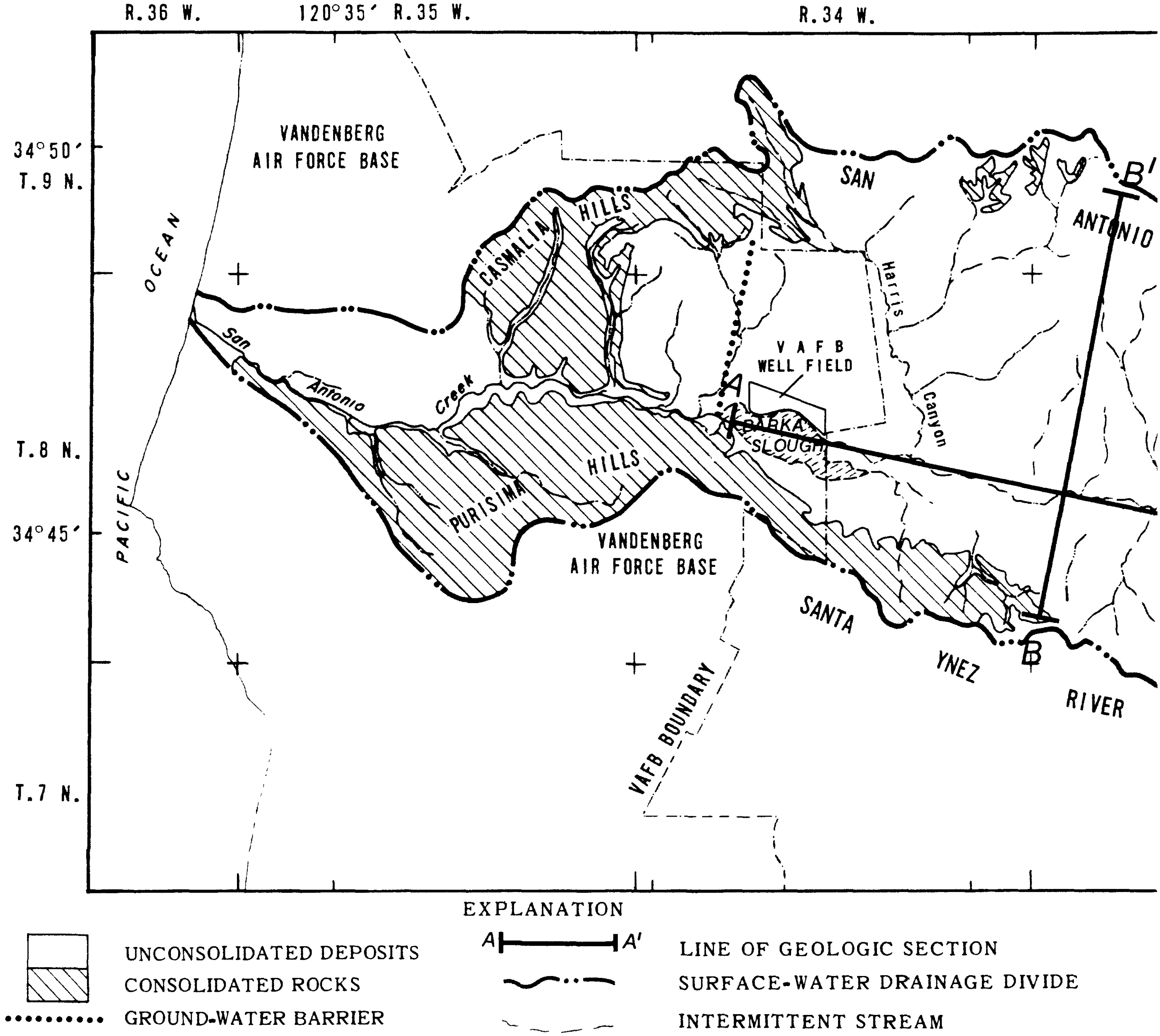


FIGURE 1.--Location of geologic sections and location and general features of San Antonio Creek valley.

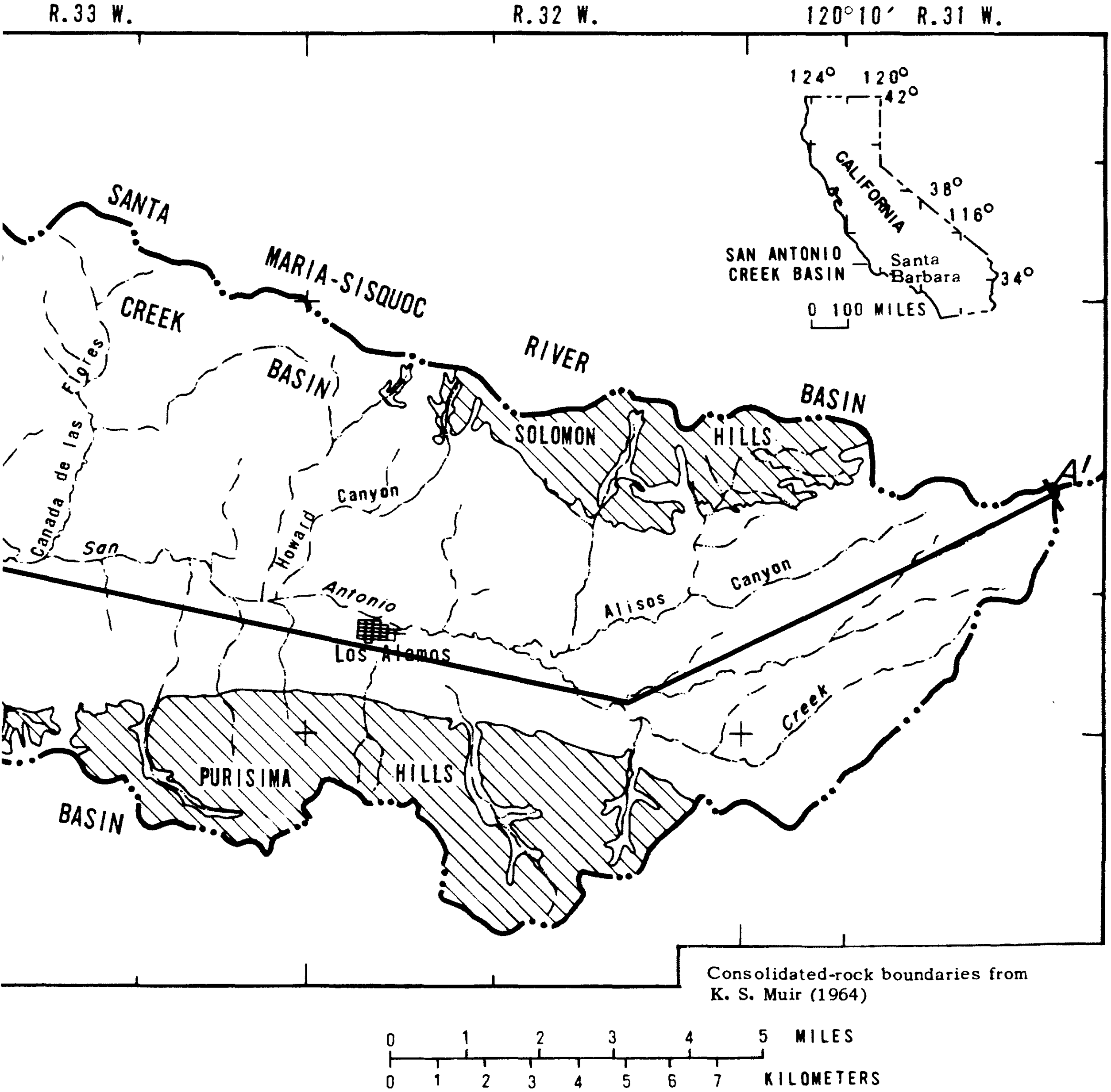


FIGURE 1.--Continued

Well-Numbering System

Wells are numbered according to their location in the rectangular system for the subdivision of public land. The part of the number preceding the slash, as in 8N/34W-14A1, indicates the township (T. 8 N.); the number following the slash indicates the range (R. 34 W.); the number following the hyphen indicates the section (sec. 14); the letter following the section number indicates the 40-acre subdivision according to the lettered diagram below. The final digit is a serial number for wells in each 40-acre subdivision.

D	C	B	A
E	F	G	H
M	L	K	J
N	P	Q	R

GEOHYDROLOGIC SYSTEM

An understanding of the geohydrology of the system is necessary before a numerical model of the San Antonio Creek valley can be developed. This involves defining the aquifer system and the aquifer characteristics, determining the recharge to and discharge from the aquifer system, determining the direction of ground-water movement, and computing a steady-state water budget. The geohydrology of the San Antonio Creek valley is discussed in detail in reports by Hutchinson (1980) and Muir (1964), and only a brief summary of the above-listed components of geohydrology is included here. The reader is referred to the earlier reports for a more complete description.

Definition of the Aquifer System

The San Antonio Creek valley is a northwestward-trending synclinal trough that contains a series of marine and continental sediments, Miocene to Holocene in age, that totals about 10,000 feet in thickness. The lithology of the sediments has been generalized by Muir (1964) into two categories: consolidated non-water-bearing rocks and unconsolidated water-bearing deposits.

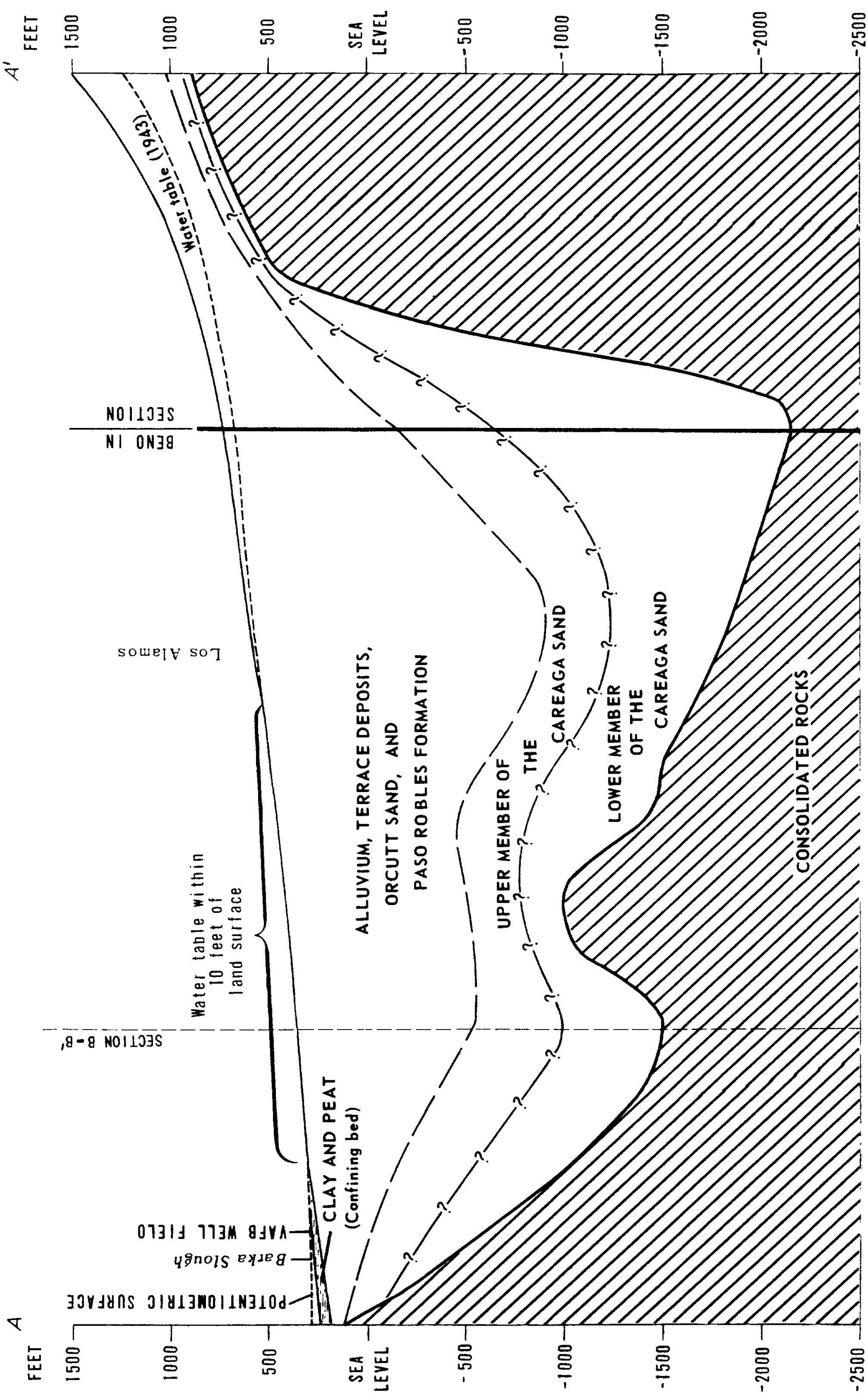
The consolidated rocks are sedimentary rocks, predominantly marine in origin, that are nearly impermeable except for slightly permeable sandstones and for fracture zones. The consolidated rocks form the lower boundary of the aquifer and also form much of the perimeter boundary of the ground-water basin (fig. 1).

Unconsolidated deposits of predominantly sand and gravel blanket the central part of the valley and form the ground-water basin. In upward succession, the unconsolidated deposits include the lower and upper members of the Careaga Sand of Pliocene age, the Paso Robles Formation of Pliocene and Pleistocene age, the Orcutt Sand of Pleistocene age, the terrace deposits of Pleistocene age, and the alluvium of Holocene age. The relative placement of these deposits is shown diagrammatically in figure 2. The unconsolidated deposits range in thickness from zero feet along the perimeter of the basin to about 3,000 feet in the central part of the basin near Los Alamos (Hutchinson, 1980, p. 15).

The lower member of the Careaga Sand is exposed near the perimeter of the basin and occurs at depth within the basin. The formation consists of low-permeability fine-grained sandstone and sand interbedded with siltstone. Wells perforated solely in this formation yield less than 10 gal/min. The lower member of the Careaga Sand is not considered an important part of the aquifer except along the perimeter of the valley where the younger formations are missing or are unsaturated.

The most important water-bearing units in the basin are: (1) The thin prism of alluvium along the San Antonio Creek channel, (2) the underlying thick deposits of the Paso Robles Formation, and (3) the upper member of the Careaga Sand. Hydrologically these formations can be considered as one unit. Vertical and horizontal heterogeneity, within each unit, overshadow any hydrologic differences between the units.

The terrace deposits and Orcutt Sand are highly permeable, but are unsaturated throughout most of the basin. The major hydrologic significance of these units is as a recharge conduit to the underlying formations.



A
A'
FEET
1500
1000
500
SEA LEVEL
-500
-1000
-1500
-2000
-2500
POTENTIOMETRIC SURFACE
Barka Slough
VAFB WELL FIELD
CLAY AND PEAT (Confining bed)
SECTION B-B'
Water table within 10 feet of land surface
Los Alamos
BEND IN SECTION
Water table (1943)
ALLUVIUM, TERRACE DEPOSITS, ORCUTT SAND, AND PASO ROBLES FORMATION
UPPER MEMBER OF THE CAREAGA SAND
LOWER MEMBER OF THE CAREAGA SAND
CONSOLIDATED ROCKS

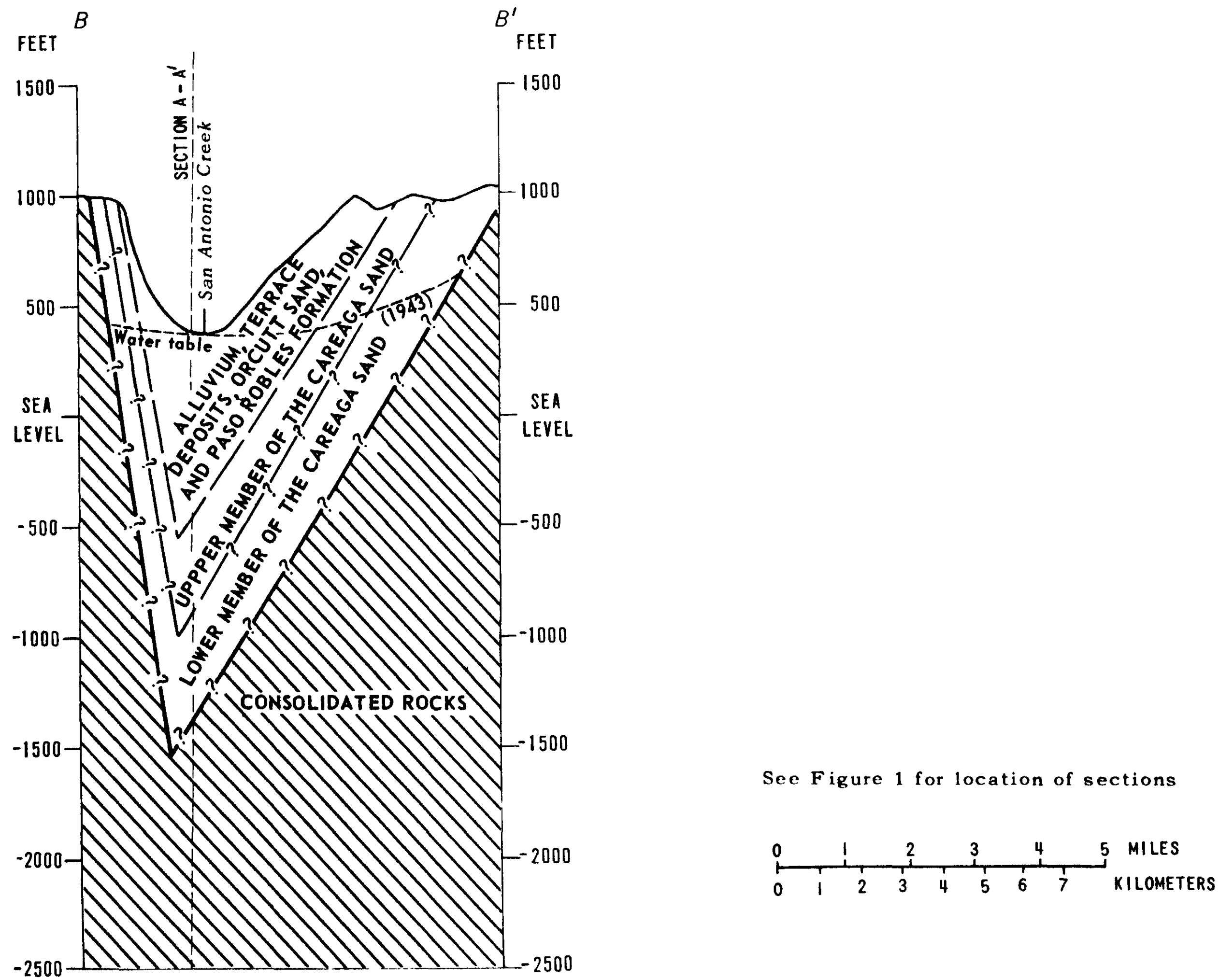


FIGURE 2.--Diagrammatic geologic sections of the San Antonio Creek valley. Modified from Hutchinson (1980).

Aquifer Characteristics

Data on aquifer characteristics are minimal. Aquifer-test results reported by Hutchinson (1980, p. 5) indicate the transmissivity varies widely, ranging from 2,600 to 34,000 ft^2/d. None of the wells used to test the aquifer penetrate much more than half the aquifer thickness. The most reliable test results were obtained at VAFB near the western edge of the basin. Analysis of this test indicated a transmissivity of 14,000 ft^2/d.

The ground-water basin is considered unconfined except in the area of Barka Slough, where the aquifer is confined by a layer of clay and peat. An aquifer test at Barka Slough indicated a storage coefficient of 0.001.

In the nearby Santa Ynez and Cuyama Valleys (off map), the specific yield of the Paso Robles Formation was estimated to be 15 percent (Miller, 1976, p. 37; Singer and Swarzenski, 1970, p. 19). Based on these estimates and on the assumption that the aquifer characteristics of the basins are similar, a storage coefficient of 0.15 was assumed representative of the unconfined areas of the aquifer.

Recharge

Recharge to the ground-water basin is derived entirely from rainfall within the valley. As rain falls on the unconsolidated deposits of the valley it either is held as soil moisture, infiltrates past the root zone to recharge the aquifer, or runs off to stream channels and contributes to recharge by seepage through the permeable streambeds. Because the ground-water divides generally coincide with the topographic divides, there is no surface or ground-water flow into the San Antonio Creek valley (Hutchinson, 1980, p. 27).

Rainfall

The long-term (1909-77) average rainfall in the San Antonio Creek valley is 15.18 inches (Hutchinson, 1980, p. 6). The Santa Barbara County Water Agency has developed a set of functions for determining the infiltration of rainfall over open land (Santa Barbara County Water Agency, 1977, p. C1). Using these rainfall-infiltration functions, Hutchinson (1980, p. 39) computed the average annual infiltration of rainfall in the basin to be 4,700 acre-ft during 1958-77.

Stream Recharge

The amount of recharge contributed by seepage loss from streams is usually estimated from the decrease of streamflow between two gaging stations. In the San Antonio Creek valley, however, it would not be economically feasible to establish gaging stations on enough creeks in the area to accurately measure recharge from seepage losses. Accordingly, no direct estimates have been made of the seepage loss from streams.

Hutchinson (1980, p. 39), from a comprehensive water budget he developed for the basin, estimated the amount of recharge contributed by seepage losses from streamflow to be 5,100 acre-ft/yr during 1958-77. This estimate is probably too high for the predevelopment period in the basin because, prior to 1958, water levels in the western part of the basin beneath San Antonio Creek were near land surface east of Barka Slough and were above land surface at the slough; this would preclude significant seepage loss. A steady-state water budget developed for this study indicates the amount of stream recharge to be 1,500 acre-ft/yr (see section on "Steady-state water budget"). Most of this recharge occurs along the San Antonio Creek channel east of Los Alamos, where water levels were significantly below land surface.

Discharge

Ground water in the San Antonio Creek valley is discharged naturally by evapotranspiration and base flow along San Antonio Creek and artificially by pumpage.

Evapotranspiration

Evapotranspiration is the use of water by growing vegetation, plus water evaporation from adjacent soil. In the San Antonio Creek valley, evapotranspiration takes place in Barka Slough and along the 6-mile reach of San Antonio Creek east of Barka Slough. There is considerable vegetation and a high water table in both areas. Aerial photographs taken in 1943 indicate that dense vegetation covered about 660 acres in Barka Slough and about 130 acres along San Antonio Creek, east of Barka Slough. Prior to significant ground-water development in the valley, native vegetation in Barka Slough consisted predominantly of marsh plants, including cattail, bulrush, saltgrass, and tules. These plants are hydrophytes that require abundant water and live only in shallow ponds or in saturated soil where the depth to water is only a few inches during short, dry periods (Mower and Nace, 1957, p. 10). The native vegetation along the San Antonio Creek channel consisted of cottonwood and willow trees. These plants are phreatophytes that also require abundant water; however, they can extend their roots to the water table and withdraw water like a well.

Muir (1964, p. 33) estimated the average evapotranspiration rate of the vegetation in 1958 to be 4.7 ft/yr in Barka Slough and 3.0 ft/yr in the areas along San Antonio Creek; these evapotranspiration rates were assumed to be representative of predevelopment conditions. By multiplying these average evapotranspiration rates by the area covered by the vegetation, the evapotranspiration prior to significant ground-water development was estimated to be 3,100 acre-ft/yr in Barka Slough and 400 acre-ft/yr along San Antonio Creek. Ground-water development in the valley since 1943 has lowered water levels beneath both areas of dense vegetation and thus has reduced the amount of ground water discharged by evapotranspiration.

Base Flow

The base flow of San Antonio Creek is the component of total runoff sustained by ground-water discharge (Hutchinson, 1980, p. 9). The base flow in San Antonio Creek for 1955-77 was estimated from hydrographs of daily runoff at the gaging station 2 miles west of Barka Slough. During this period, base flow ranged from a high of 1,130 acre-ft in 1958 to a low of 430 acre-ft in 1977 (table 1). The decline in base flow is in response to the increased pumpage in the valley. Estimates of base flow prior to 1955 cannot be computed from runoff measurements because the gaging station was not in operation. Base flow for 1943-54 (table 1) was estimated by extrapolating the trend in the data collected since 1955. The base flow estimates throughout the period of record are probably too high, because runoff from irrigated fields east of Barka Slough was not accounted for. Streamflow measurements east of Barka Slough are needed to determine the contribution of irrigation runoff to the total streamflow.

Pumpage

Ground-water pumpage in the San Antonio Creek valley is principally for municipal, irrigation, and military supplies. The annual distribution of the various pumpages from the ground-water basin for 1943-77 is shown in table 1.

Municipal pumpage is the least significant component of ground-water discharge from the basin. Municipal supplies recorded by the Los Alamos Community Service District increased from about 100 acre-ft/yr in 1943 to 170 acre-ft/yr in 1977.

The most significant component of ground-water discharge is irrigation net pumpage, which is the amount of ground water applied to the crops, minus the amount of return flow that infiltrates past the root zone into the ground-water reservoir. In the San Antonio Creek valley, during 1943-77, irrigation net pumpage steadily increased from about 1,400 acre-ft/yr to about 6,950 acre-ft/yr (table 1). The irrigation net-pumpage estimates were computed by means of the crop-use technique, assuming that 2.1 feet of water is applied per crop-acre and that return flow is 20 percent of the total pumpage (Hutchinson, 1980, p. 11). Irrigated crop acreage was estimated from five crop surveys made in 1943, 1957, 1963, 1968, and 1975 (Muir, 1964, p. 29; Hutchinson, 1980, p. 6). These surveys indicate that irrigated agriculture increased from about 800 acres in 1943 to about 4,750 acres in 1975. The 1975 survey (fig. 3) included, however, about 750 acres in the Harris Canyon area of the valley that was dry farmed or only irrigated occasionally. Therefore, for the purposes of this study, the irrigated agriculture in 1975 was estimated to be 4,000 acres. The irrigated acreage in the undocumented years was estimated from the 5 documented years (see graph inset, fig. 3).

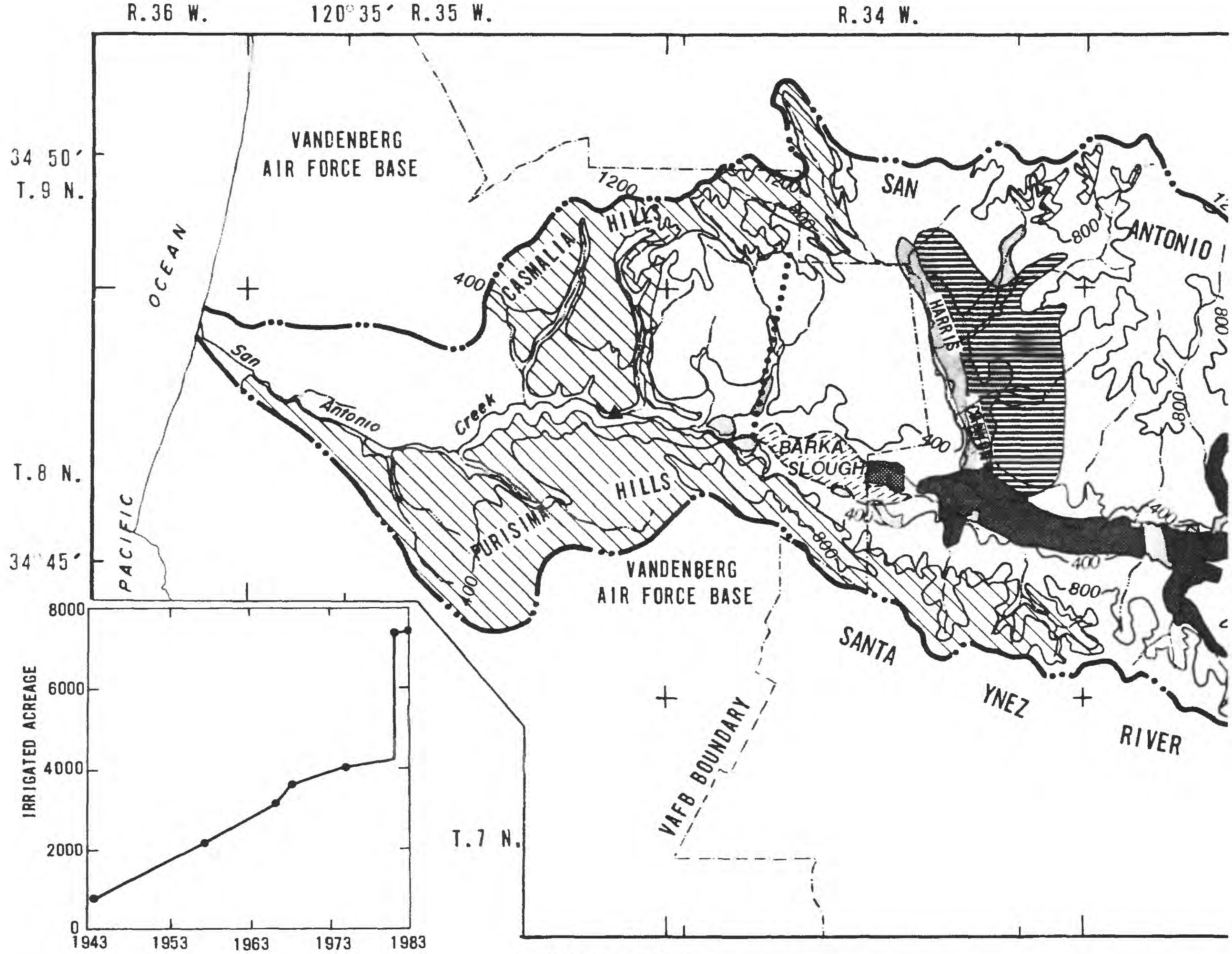
Military pumpage consists of water pumped from the VAFB well field north of Barka Slough. Pumping began in 1963 and increased from an initial rate of 317 acre-ft/yr to 1,829 acre-ft/yr in 1977. None of the water pumped from the well field returns to the aquifer system.

TABLE 1. - Annual base flow and net pumpage from the San Antonio Creek ground-water basin, 1943-77

[All values in acre-feet per year]

Year	Base flow	Irrigation net pumpage	Municipal pumpage	Military pumpage	Total pumpage
1943	[1]1,200	1,400	100	0	1,500
1944	[1]1,180	1,500	100	0	1,600
1945	[1]1,170	1,700	100	0	1,800
1946	[1]1,160	1,800	100	0	1,900
1947	[1]1,140	2,000	100	0	2,100
1948	[1]1,120	2,200	100	0	2,300
1949	[1]1,110	2,300	100	0	2,400
1950	[1]1,100	2,500	100	0	2,600
1951	[1]1,080	2,700	100	0	2,800
1952	[1]1,060	2,900	100	0	3,000
1953	[1]1,050	3,000	100	0	3,100
1954	[1]1,040	3,200	100	0	3,300
1955	1,020	3,400	100	0	3,500
1956	920	3,500	100	0	3,600
1957	830	3,700	100	0	3,800
1958	1,130	3,860	100	0	3,960
1959	990	3,730	100	0	3,830
1960	890	3,860	100	0	3,960
1961	850	4,030	100	0	4,130
1962	1,090	4,200	100	0	4,300
1963	930	4,450	100	320	4,870
1964	650	4,700	100	1,340	6,140
1965	570	4,960	100	1,750	6,810
1966	490	5,210	100	830	6,140
1967	590	5,630	100	1,630	7,360
1968	510	6,050	100	1,350	7,500
1969	790	6,150	100	1,000	7,250
1970	610	6,250	110	1,620	7,980
1971	430	6,350	120	1,310	7,780
1972	460	6,450	140	1,160	7,750
1973	550	6,550	140	1,920	8,610
1974	510	6,650	140	1,330	8,120
1975	540	6,750	150	1,510	8,410
1976	520	6,850	160	1,660	8,670
1977	430	6,950	170	1,830	8,950

[1]Base flow data from 1943 to 1954 were extrapolated from measurements collected since 1955.



EXPLANATION

UNCONSOLIDATED DEPOSITS

CONSOLIDATED ROCKS

800 TOPOGRAPHIC CONTOUR--Shows altitude of land surface. Contour interval 400 feet. National Geodetic Vertical Datum of 1929

GROUND-WATER BARRIER

SURFACE-WATER DRAINAGE DIVIDE

INTERMITTENT STREAM

IRRIGATED LAND, 1975 (Modified from Hutchinson, 1980)

DRY-FARMED LAND, 1975 (Modified from Hutchinson, 1980)

IRRIGATED AGRICULTURE DEVELOPMENT, 1981

SURFACE-WATER GAGE

FIGURE 3.-- Topography and land use in the San Antonio Creek valley.

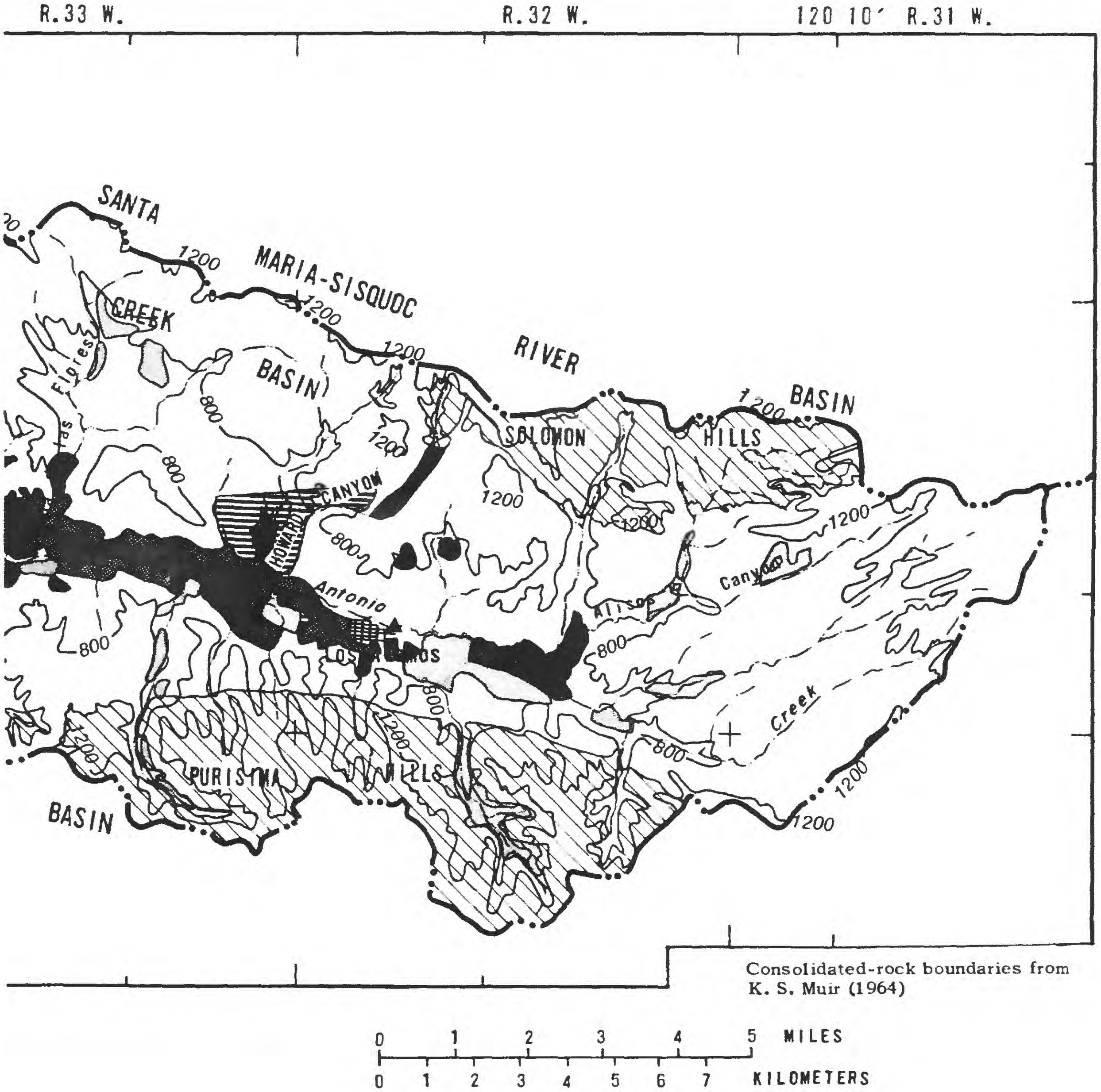


FIGURE 3.--Continued.

Ground-Water Movement

The regional direction of ground-water movement in the San Antonio Creek valley, prior to significant ground-water development, is shown by the 1943 water-level contour map (fig. 4). The 1943 water levels are the highest on record and are considered to be representative of steady-state conditions. Ground-water movement in the valley generally follows the surface drainage pattern. Ground water moves laterally from the divide, and from the contact between the unconsolidated deposits and consolidated rocks near the perimeter of the ground-water basin, toward San Antonio Creek. The water-level surface slopes steeply from an altitude of more than 1,100 feet at the eastern boundary of the basin to 700 feet at the valley floor. Beneath the valley floor, the water-level surface slopes gently from 700 feet to slightly less than 300 feet at Barka Slough, an average gradient of about 30 ft/mi.

An area of ground-water discharge is indicated by the trough in the water-level surface along San Antonio Creek from about Los Alamos to the ground-water barrier. The consolidated rock barrier west of Barka Slough forces ground water toward land surface. A clay and peat layer beneath Barka Slough confines the aquifer; as a result, the potentiometric head is above land surface throughout most of the slough.

Steady-State Water Budget

A steady-state water budget is a quantitative assessment of the inputs and outputs in a ground-water basin. From year to year or season to season, differences between inputs and outputs result in changes in storage, as reflected in fluctuations of ground-water levels, but over the long term, under natural or otherwise steady-state conditions, inputs and outputs tend to balance (Hutchinson, 1980, p. 35). An aquifer system is considered to be under steady-state conditions when water levels remain constant. Prior to 1943, ground-water levels in the San Antonio Creek valley were constant over the long term, although seasonal pumpage and variations in annual precipitation and streamflow caused cyclical variations in water levels. For the purposes of this study, the aquifer was assumed to be under nearly steady-state conditions in 1943.

The steady-state water budget for the San Antonio Creek valley is expressed by the following equation:

$$\text{Inflow} = \text{Outflow}$$

$$I_r + I_{sw} + I_{gw} = O_{bf} + O_{ets} + O_{etv} + O_p, \tag{1}$$

where

I_r is inflow as recharge by infiltration of rainfall over open land; in this case, 4,700 acre-ft. This value was estimated by rainfall-infiltration functions (Hutchinson, 1980, p. 39).

I_{sw} is inflow as surface-water seepage along stream channels; in this case, 1,500 acre-ft. The value of this component was computed as a residual in the water budget.

I_{gw} is inflow as ground-water underflow from adjacent basins; in this case, zero acre-ft. This component of inflow is negligible because the San Antonio Creek basin boundary coincides with the ground-water divides.

O_{bf} is outflow as base flow to San Antonio Creek; in this case, 1,200 acre-ft. This value was estimated by extrapolating base flow data collected since 1955 (table 1).

O_{ets} is outflow as evapotranspiration by hydrophytes at Barka Slough; in this case, 3,100 acre-ft. This value was estimated by multiplying the average evapotranspiration rate of the vegetation in the slough, 4.7 ft/yr (Muir, 1964, p. 32), by the area of the slough, 660 acres.

O_{etv} is outflow as evapotranspiration by phreatophytes along the banks of San Antonio Creek, 6 miles east of Barka Slough; in this case, 400 acre-ft. This value was estimated by multiplying the average evapotranspiration rate of the phreatophytes along the creek, 3.0 ft/yr (Muir, 1964, p. 32), by the area of the phreatophytes, 130 acres.

O_p is outflow by net ground-water pumpage from the basin, including municipal and irrigation pumpages; in this case, 1,500 acre-ft. Military pumpage did not begin until 1963. This component of outflow was computed by Muir (1964, p. 31 and 32).

During the steady-state period, inflow and outflow were equivalent at 6,200 acre-ft/yr. The components of the steady-state water budget are illustrated in figure 5.

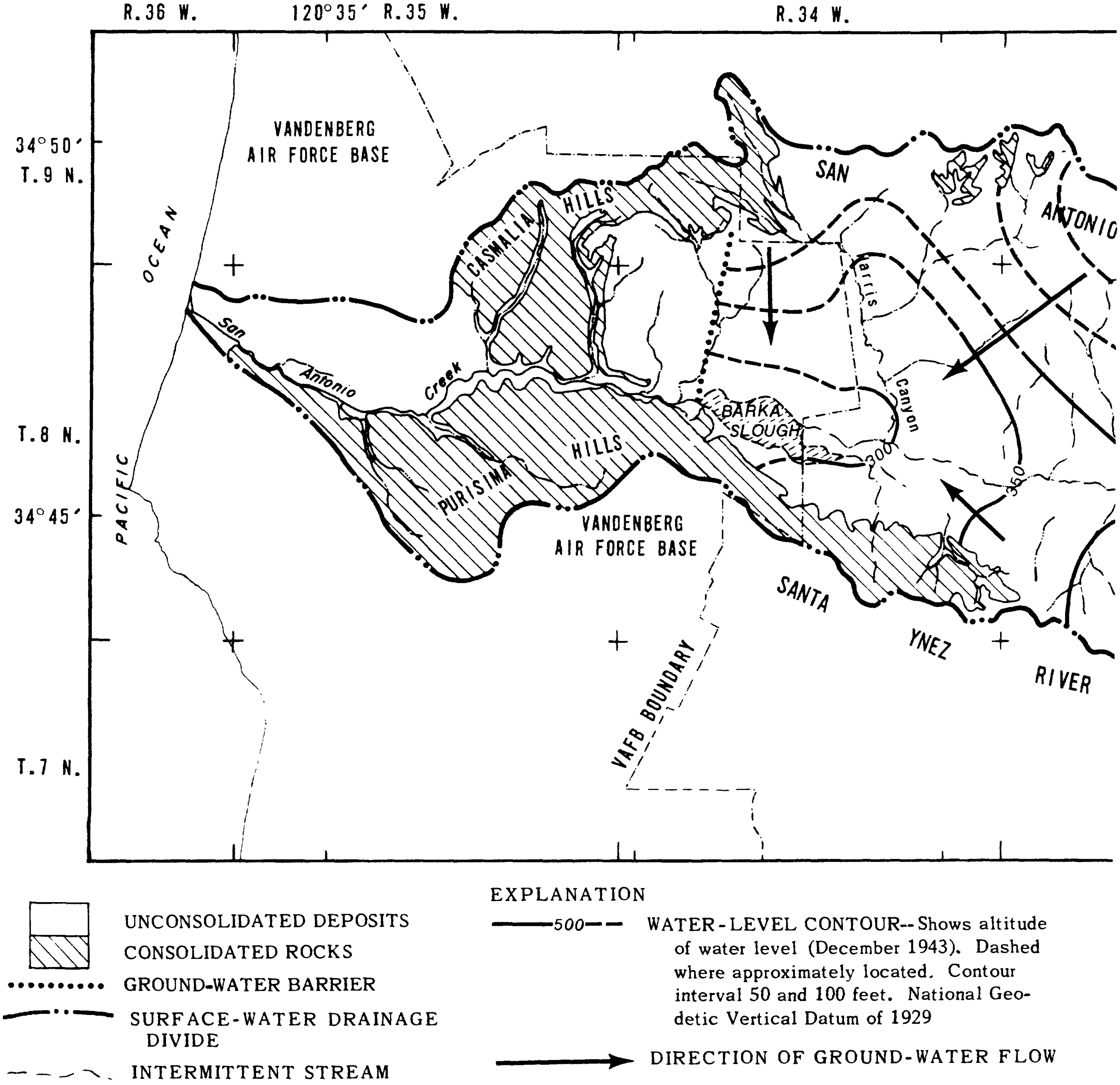


FIGURE 4.--Water-level contours, December 1943.

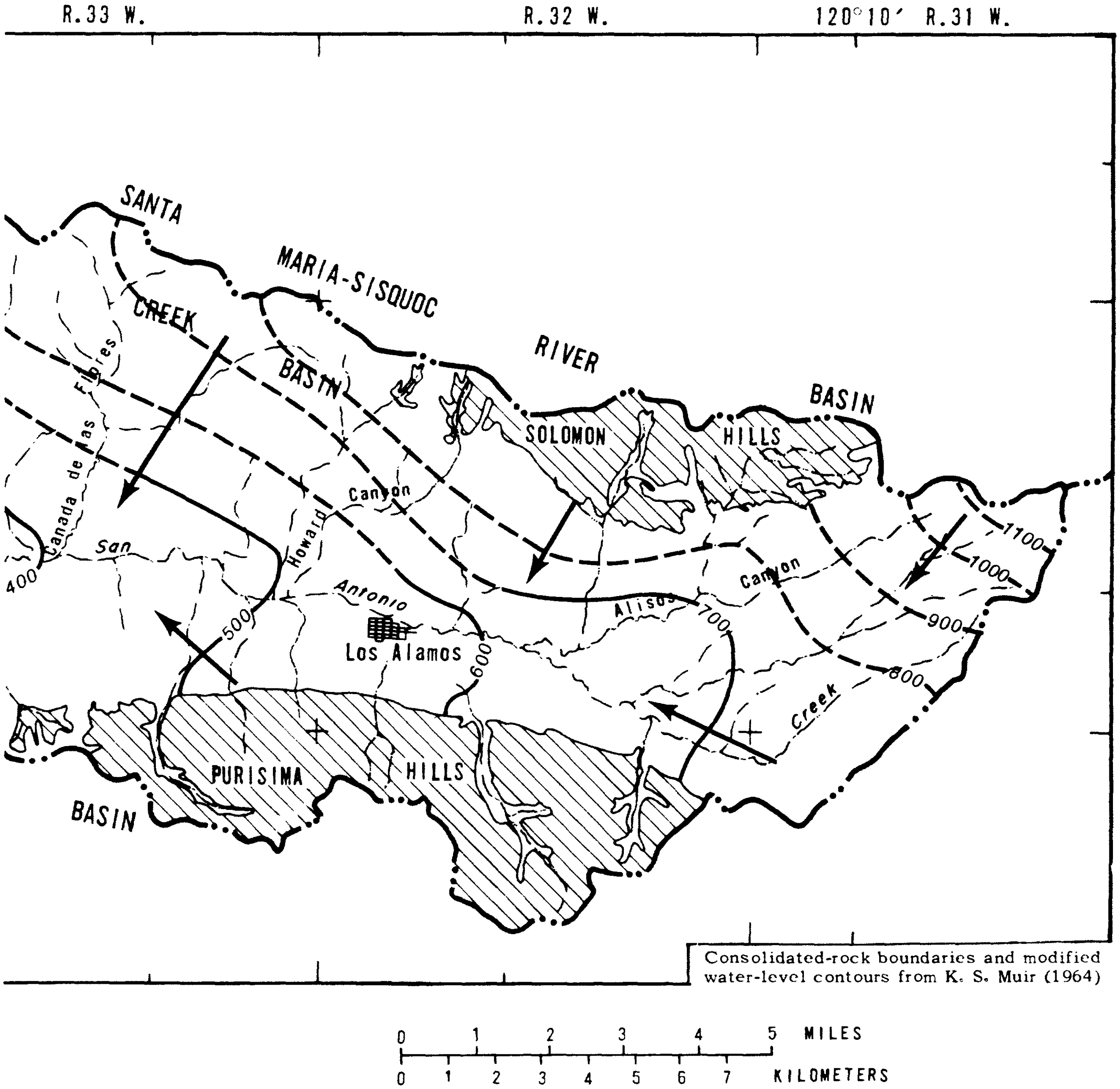


FIGURE 4.--Continued.

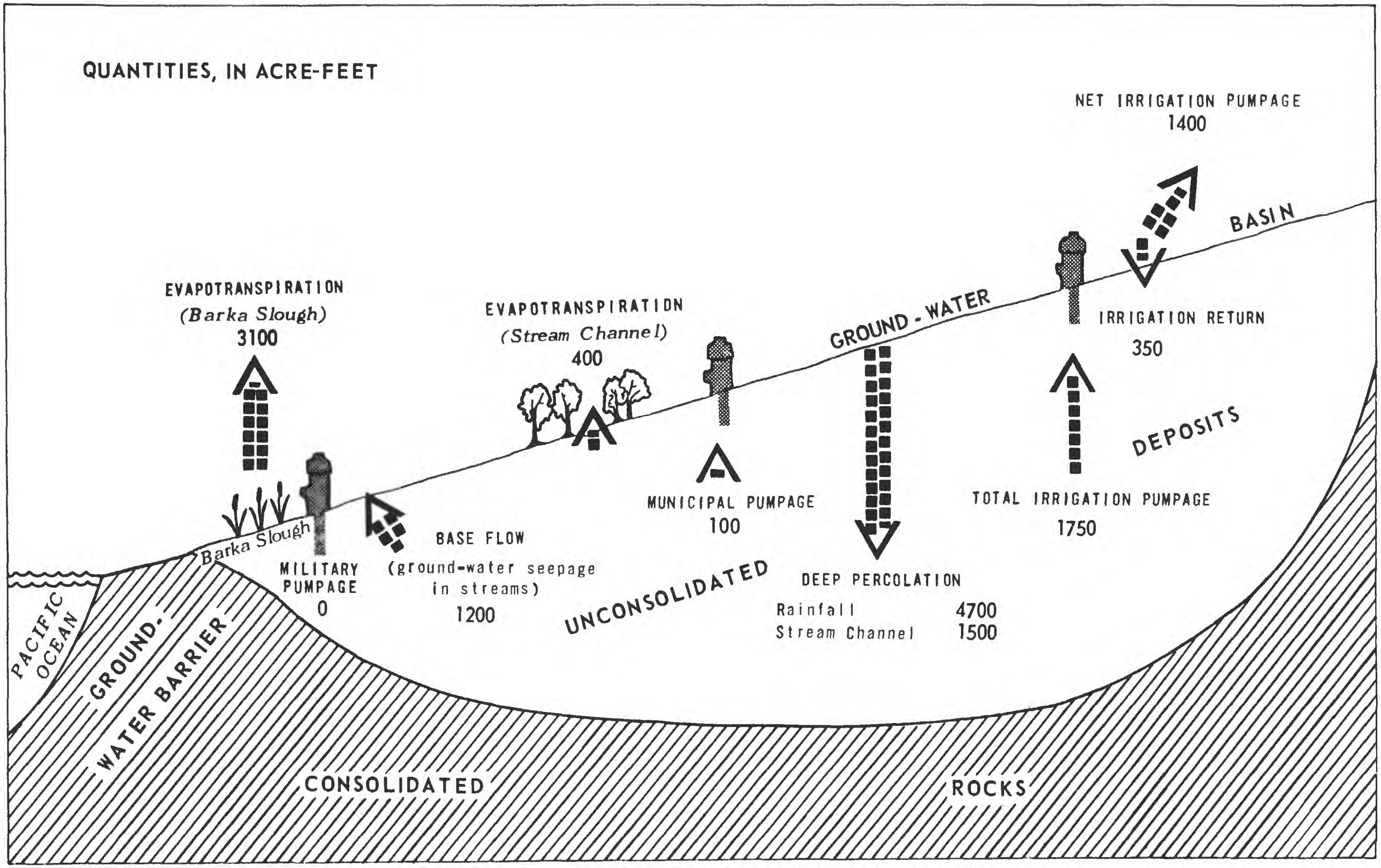


FIGURE 5.--Conceptual model of the steady - state water budget for the San Antonio Creek valley.

MODELING PROCEDURE

The objective in constructing a numerical ground-water flow model of the San Antonio Creek valley was to develop a better understanding of the aquifer system in the study area and to develop a tool that could be used to predict water levels based on projected water-use requirements. The model mathematically calculates water levels based on: (1) The ability of the aquifer to transmit water (transmissivity) and its capacity to store and release water (storage coefficient), (2) the quantity of water entering the aquifer (recharge), and (3) the quantity of water leaving the aquifer (discharge). The mathematical model, however, cannot exactly duplicate the actual system due to the complex geohydrologic relations in the ground-water basin. Model development requires the use of assumptions and approximations that simplify the physical system. It cannot be overemphasized that the model is only as accurate as the assumptions and data used in its development.

Basis of the Model

The numerical model chosen for this study was developed by McDonald and Harbaugh (1984) and utilizes the finite-difference numerical method of solution. A full explanation of the theoretical development, the solution technique used, and the mathematical treatment of each simulated condition is included in McDonald and Harbaugh (1984).

The numerical model solves the following generalized equation of two-dimensional ground-water flow in an isotropic heterogenous aquifer:

$$\frac{\partial}{\partial x} T \frac{\partial h}{\partial x} + \frac{\partial}{\partial y} T \frac{\partial h}{\partial y} = S \frac{\partial h}{\partial t} + W + q, \tag{2}$$

where:

T is transmissivity, the rate of flow of water through a vertical strip of the aquifer of unit width under a unit hydraulic gradient;

h is hydraulic head;

S is storage coefficient, the volume of water an aquifer releases or takes into storage per unit surface area of the aquifer per unit change in head;

t is time;

W is source or sink function, volume flux per unit area (pumpage or recharge);

q is vertical leakage, volume flux per unit area; and

x and y are cartesian coordinates.

The vertical-leakage term q is defined as:

$$q = \frac{k' \ (h_a - h_{cb})}{m}; \qquad q = 0 \text{ when } h_a - h_{cb} \leq 0, \tag{3}$$

where:

k' is vertical hydraulic conductivity of the confining bed of clay and peat underlying Barka Slough;

m is thickness of the confining bed;

h_a is hydraulic head in the aquifer; and

h_{cb} is hydraulic head on the other side of the confining bed.

As indicated in the above equation, leakage is simulated from the aquifer through the confining bed to Barka Slough, but not from the slough to the aquifer. No leakage was simulated between Barka Slough and the aquifer because evapotranspiration by vegetation in the slough was assumed to use all available surface water in the slough.

Grid Network

The finite-difference grid, designed to divide the area into discrete blocks, was oriented with the axis of the valley to minimize the number of blocks outside the principal aquifer system (fig. 6). The grid consisted of 720 rectangular blocks. A variable grid size was used to produce better results where data density was high, or where large variation in aquifer properties or stresses occurred. The lengths of the blocks were 2,640 feet throughout the valley; however, the widths of the blocks decreased from 5,280 feet at the perimeter of the valley to 1,000 feet along the San Antonio Creek channel, which follows the axis of the valley (fig. 6). The smallest block dimensions were situated along the river channel because most of the wells in the valley are located in the fields near the river channel.

Data Input

In order to use the model for approximating the flow system and for predicting water-level changes, it is necessary to determine the boundary conditions for the aquifer, estimate the aquifer properties within the model area, and estimate the rates and distribution of recharge and discharge to the aquifer system. The accuracy of the model is directly related to the accuracy of these input data.

Boundary Conditions

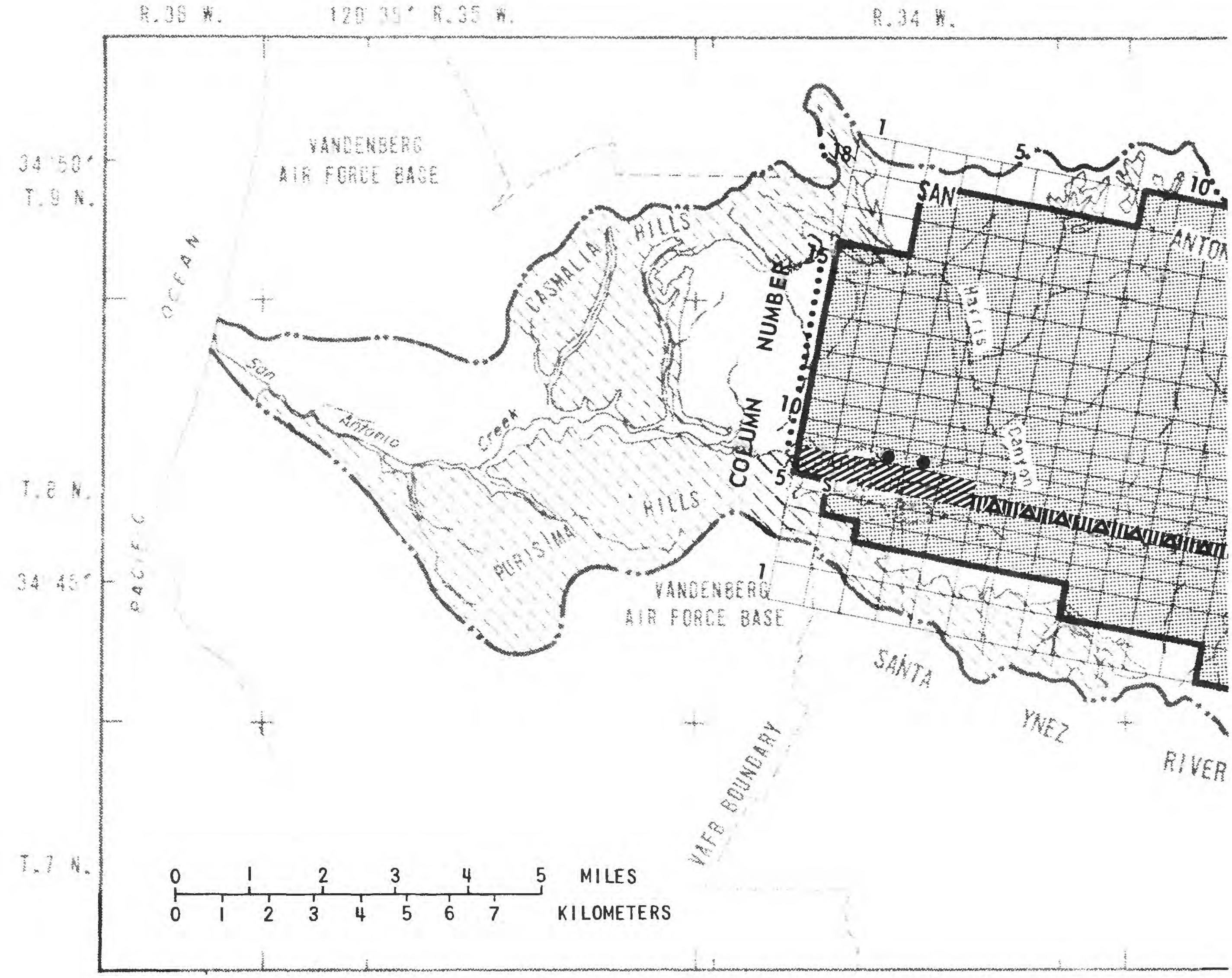
The hydrologic boundaries of the San Antonio Creek valley are simulated as no-flow boundaries. A no-flow boundary indicates that no water enters or leaves through the boundary. The location of the no-flow boundaries corresponds to the contact between unconsolidated deposits and the less permeable consolidated rocks or to the ground-water divide where the consolidated rocks are not exposed (fig. 6). Because the ground-water divides generally coincide with topographic divides, it is evident that there is no significant movement into or out of the San Antonio Creek valley (Hutchinson, 1980, p. 27).

Aquifer Properties

Transmissivity and storage values in the ground-water basin vary considerably due to the nonhomogeneity of the material in the aquifer. To simulate the variability precisely would require a model the size of the real system itself. The system can be approximated, however, by using average conditions. The transmissivity and storage values used in this model should be considered as average values and are not necessarily related to any specific strata or geologic formations.

Transmissivity.--The initial distribution of transmissivity used in this model was derived from aquifer tests, specific-capacity tests, and drillers' logs. These values were then modified during the steady-state simulation of the model until the final distribution of transmissivity was derived (fig. 7).

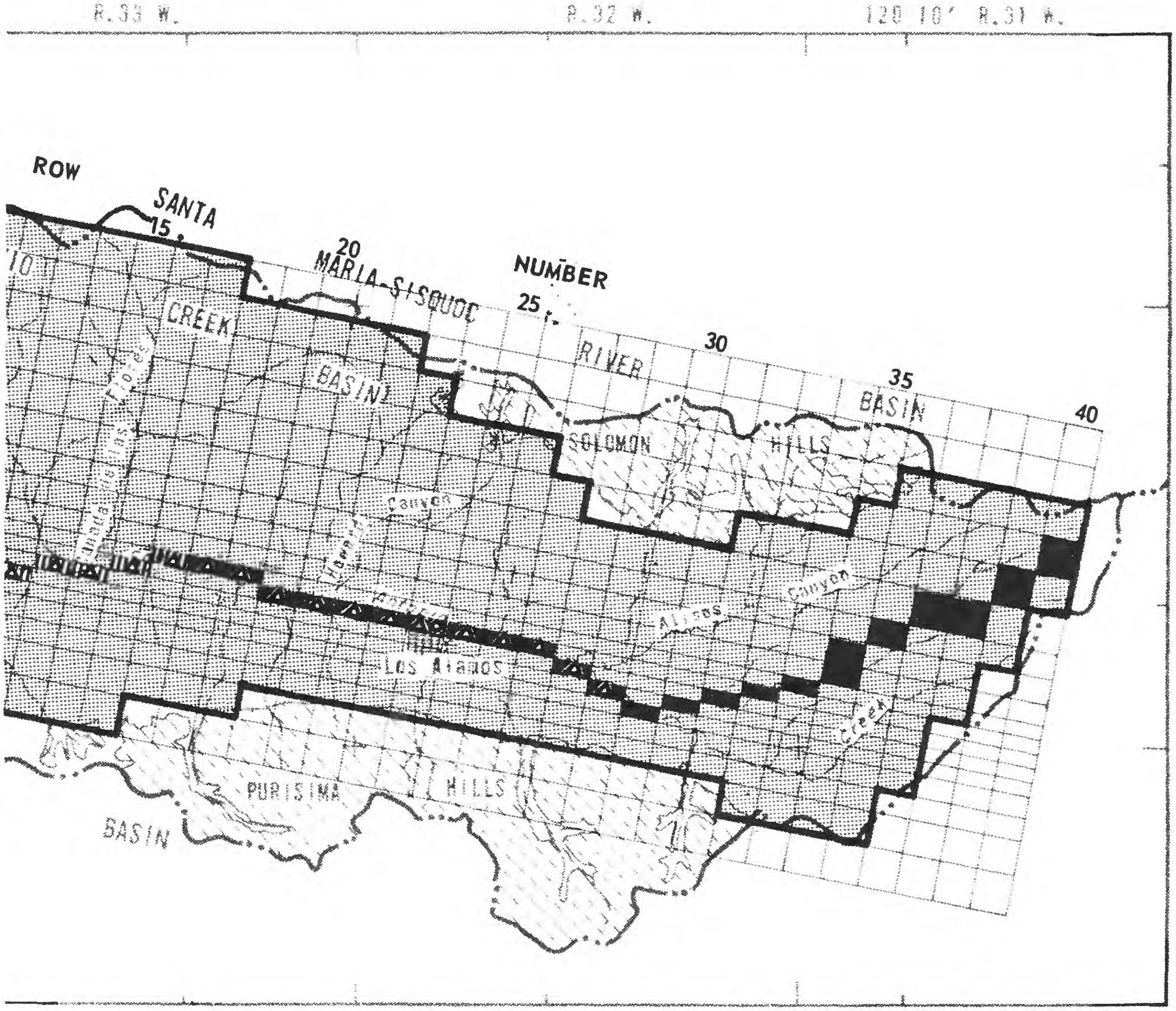
The transmissivity values used in this model do not change with time. Because transmissivity values are computed as a product of the saturated thickness of the aquifer and the average hydraulic conductivity, errors would be introduced in the model if changes in the saturated thickness due to water-level changes were not small compared to the total thickness of the aquifer. In the study area, the change in saturated thickness during the period of study was small (<20 feet) and had little effect on the transmissivity values used in the model.



EXPLANATION

UNCONSOLIDATED DEPOSITS
CONSOLIDATED ROCKS
BOUNDARY OF MODELED AREA
SURFACE-WATER DRAINAGE DIVIDE
INTERMITTENT STREAM
GROUND-WATER BARRIER
MUNICIPAL PUMPAGE
IRRIGATION PUMPAGE
MILITARY PUMPAGE

FIGURE 6.—Finite-difference grid with model blocks used to simulate recharge and discharge.



EXPLANATION

EVAPOTRANSPIRATION ALONG THE STREAM CHANNEL

EVAPOTRANSPIRATION AND BASE FLOW AT BARKA SLOUGH (Leakage)

AREAL RECHARGE

STREAM AND AREAL RECHARGE

FIGURE 6.--Continued.

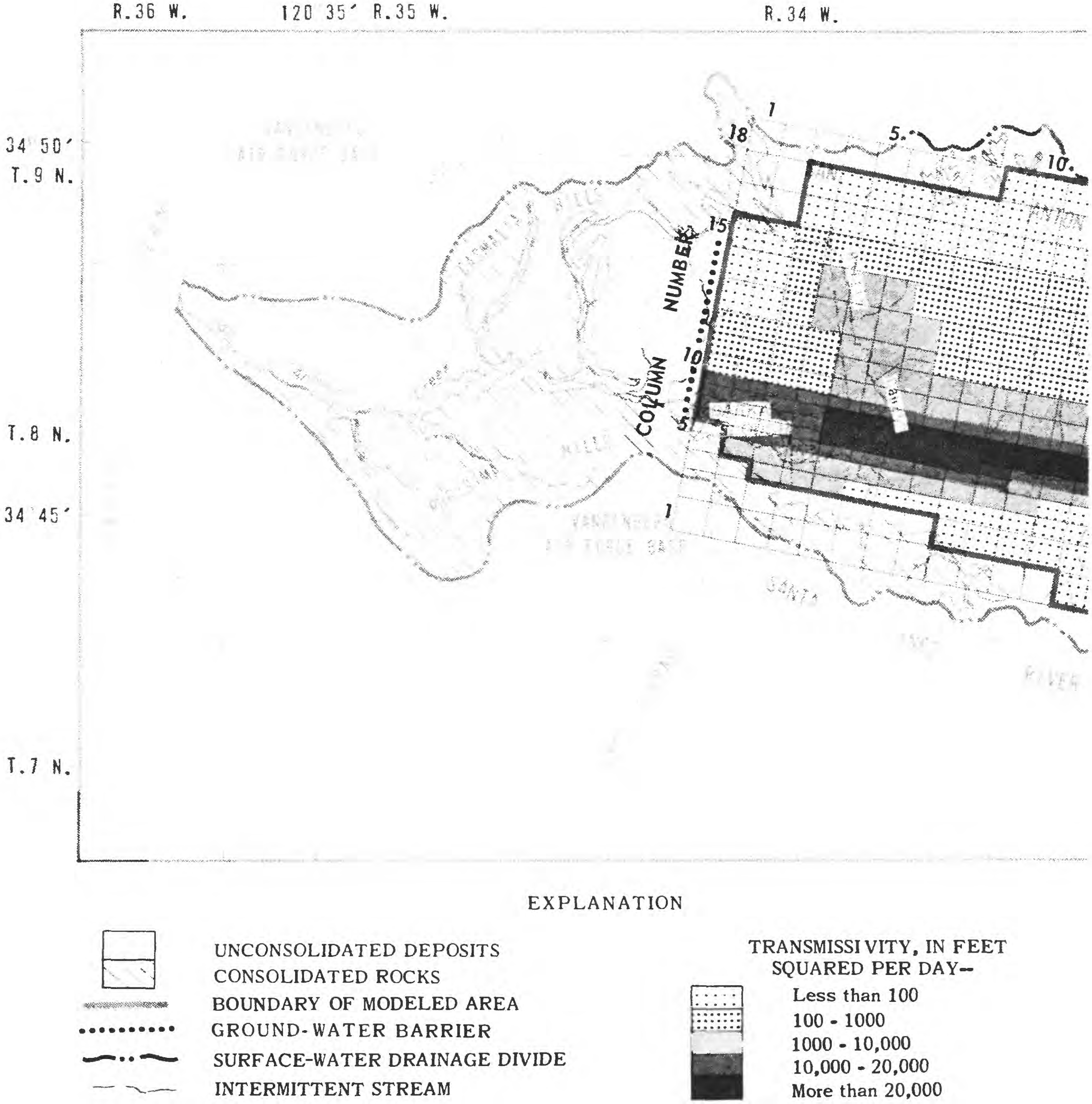


FIGURE 7.--Distribution of transmissivity as calibrated by the model.

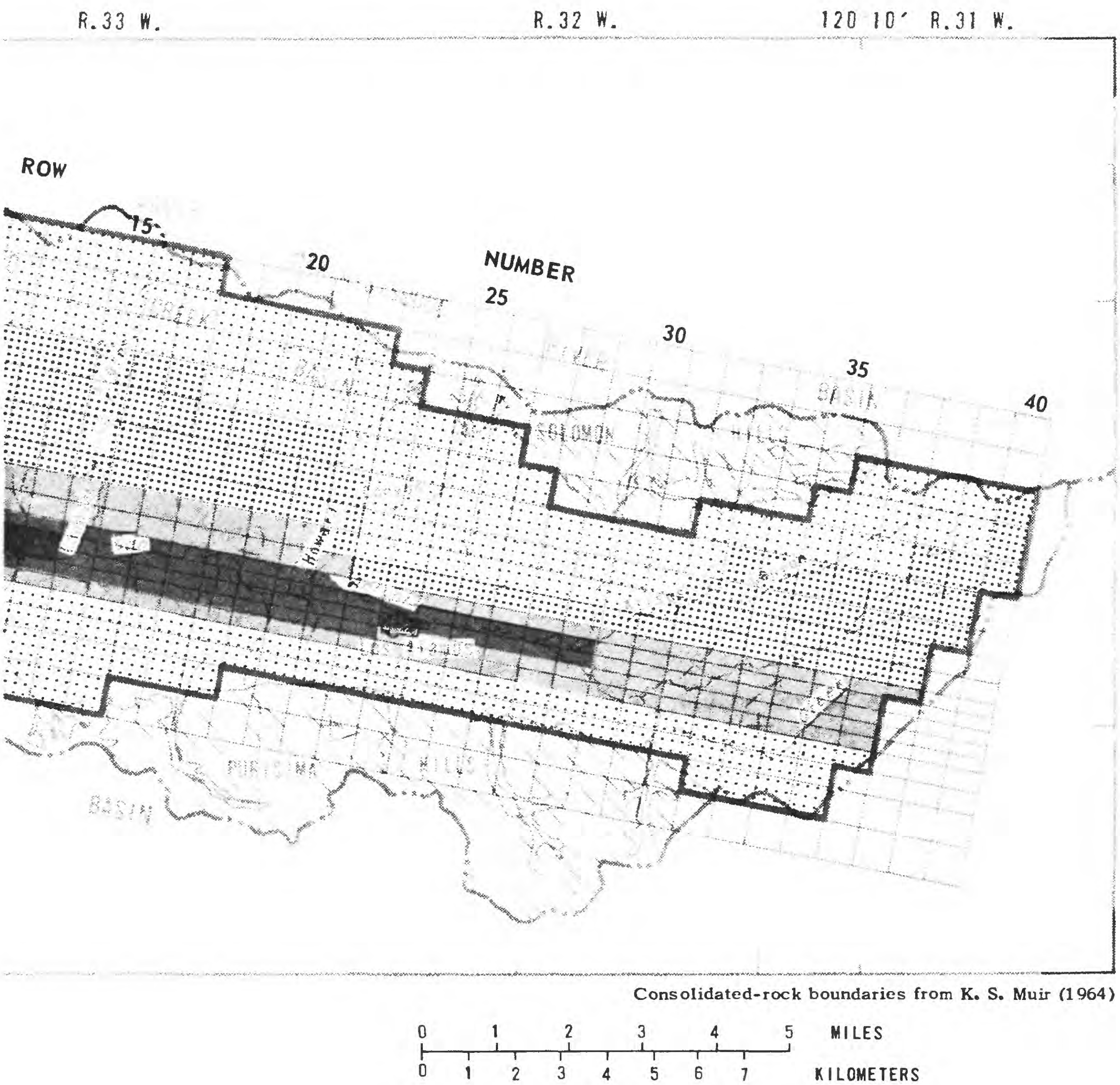


FIGURE 7.--Continued.

Storage coefficient.--The aquifer was simulated as unconfined except in the area of Barka Slough where the aquifer is confined by a layer of clay and peat. An aquifer test at Barka Slough indicated a storage coefficient of 0.001 (Hutchinson, 1980, p. 5), and this value was selected as representative for the confined part of the aquifer. The storage coefficient of the remainder of the aquifer is equivalent to the specific yield. Specific yield was estimated to be 0.15 in the nearby Santa Ynez and Cuyama Valleys (Miller, 1976, p. 37; Singer and Swarzenski, 1970, p. 19). Based on the assumption that the aquifer characteristics of the basins are similar, a storage coefficient of 0.15 was selected as representative of the unconfined part of the aquifer.

The values of storage coefficient used for the model do not change with time except beneath Barka Slough. In this part of the aquifer, the storage coefficient can change from a value representative of confined conditions (0.001) to a value representative of unconfined conditions (0.15) as the water level drops below the bottom of the confining layer of clay and peat underlying Barka Slough.

Recharge and Discharge

Recharge and discharge in the model were simulated using blocks that represent areal recharge, stream recharge, evapotranspiration, leakage, and pumpage (fig. 6). The recharge and discharge simulated by the model occur at constant rates over designated periods of time. The model does not simulate seasonal or other short-term variations in recharge or discharge, but instead simulates long-term average quantities.

Areal recharge.--In this model the areal-recharge term is the average annual infiltration of rainfall in the basin. The areal-recharge value used in the model is 4,700 acre-ft/yr. This value was computed using rainfall-infiltration functions developed by the Santa Barbara County Water Agency (Hutchinson, 1980, p. 39). During the steady-state simulation a uniform areal-recharge rate was simulated in all the model blocks except those where water levels were near or above land surface (fig. 6), which were assigned a value of zero. Areal recharge was assumed to remain constant during all model runs.

Stream recharge.--During steady-state conditions, stream recharge was simulated only in the blocks representing San Antonio Creek in the central to eastern part of the valley (fig. 6). Stream recharge in the western part of the valley was considered insignificant because water levels were near land surface along the 6-mile reach of San Antonio Creek east of Barka Slough and above land surface at Barka Slough. The steady-state stream-recharge value is 1,500 acre-ft/yr. This value was calculated as a residual in a steady-state water budget developed for this study. The stream-recharge rate is uniform in all the blocks that represent the stream during the steady-state simulation; stream recharge is simulated in the model by recharge wells.

During the transient-state simulation, the stream-recharge rate in the blocks representing San Antonio Creek was the same as during the steady-state simulation in the eastern part of the valley. The stream-recharge rate in the western part of the valley was adjusted with evapotranspiration and pumpage along the stream channel during the transient-state calibration of the model (see section on "Transient-state simulation").

Evapotranspiration.--Evapotranspiration by phreatophytes along the San Antonio Creek channel, 6 miles east of Barka Slough, was estimated to be 400 acre-ft/yr prior to significant ground-water development in the valley (Muir, 1964, p. 33). During the steady-state calibration of the model, the uniform evapotranspiration rate was simulated by discharge wells (fig. 6). During the transient-state calibration, the evapotranspiration rate was adjusted with stream recharge and pumpage along the San Antonio Creek channel (see section on "Transient-state simulation").

Leakage.--In this model, base flow and evapotranspiration by vegetation in Barka Slough are simulated together as leakage. This assumes that the vegetation at Barka Slough does not directly discharge ground water from the aquifer, but rather uses water that has leaked through the confining layer of peat and clay underlying Barka Slough. Because the vegetation at Barka Slough throughout most of the transient-simulation period (1944-77) consisted predominantly of hydrophytes that do not have deep root systems, this assumption is probably reasonable.

Leakage from the aquifer to Barka Slough occurs whenever the head in the aquifer is higher than the elevation of the head in the slough. The rate at which this leakage occurs is controlled by the thickness and vertical hydraulic conductivity of the confining layer and the head difference across this layer (see eq. 3). Evapotranspiration by the vegetation in the slough was assumed to use all available surface water in the slough; therefore, no leakage was simulated between Barka Slough and the aquifer when ground-water levels dropped below the land-surface elevation in the slough.

During steady-state simulations, the total leakage rate was estimated to be 4,300 acre-ft/yr, with evapotranspiration accounting for 3,100 acre-ft/yr and base flow 1,200 acre-ft/yr (see section on "Discharge"). The leakage flux can then be computed to be 0.018 ft/d by dividing the total leakage rate (4,300 acre-ft/yr or 11.8 acre-ft/d) by the area of Barka Slough (660 acres). The thickness of the confining layer was estimated from drillers' logs to average 25 feet, ranging from 50 feet on the western edge of the slough to zero feet on the eastern edge. Historical water-level data in Barka Slough are scarce, but trends in water level shown in hydrographs indicate that heads averaged 10 feet above land surface. By substituting these average estimates of the vertical leakage flux, thickness of the confining bed, and head difference in equation 3, the average vertical hydraulic conductivity of the confining layer was computed to be 0.045 ft/d. This estimate of vertical hydraulic conductivity was then adjusted during the steady-state calibration of the model (see section on "Steady-state simulation").

Ground-water pumpage.--Ground water is pumped in the San Antonio Creek valley for municipal, military, and irrigation purposes. The annual distribution of the various pumpages from the ground-water basin is presented in table 1 and shown graphically in figure 8 for 1943-77. The estimated historical pumpage was divided into six pumping periods in the transient-state simulation. The divisions were determined by the uniformity of the annual pumpage within a period of time and by the availability of comparative water-level data.

Municipal and military pumpages are metered supplies and were used in the model without modifications because it was assumed that none of the water returned to the aquifer system. Irrigation pumpage was estimated using the crop-use technique for determining water use by assuming that 2.1 feet of water is applied per crop acre and that return flow is 20 percent of the total pumpage (Muir, 1964, p. 31; California Department of Water Resources, 1964, p. 48). During the transient-state calibration of the model, the estimated irrigation pumpage was adjusted with stream recharge and evapotranspiration along the San Antonio Creek channel (see section on "Transient-state simulation").

Steady-State Simulation

The development of the mathematical ground-water model of the San Antonio Creek valley was begun by simulating steady-state conditions. A steady-state condition exists when natural inflow to the system equals the outflow from the system and storage does not change. In the San Antonio Creek valley, ground-water conditions in 1943 best represent steady-state conditions. Antecedent conditions were neither unusually wet nor dry, and sufficient hydrologic data were available for that year to permit reasonable simulation.

Steady-state water levels are dependent on the quantities of recharge to and discharge from the ground-water basin, the transmissivity of the aquifer, and the vertical hydraulic conductivity of the confining bed of clay and peat underlying Barka Slough. The recharge and discharge quantities used in the steady-state simulation are shown in table 2. These values were estimated in a steady-state water budget developed for the valley (see section on "Steady-state water budget"). The transmissivity of the aquifer and the vertical hydraulic conductivity of the confining bed were calibrated during the steady-state simulation of the model. These parameters were adjusted during numerous calibration runs until model-calculated water levels matched measured water levels.

TABLE 2. - Steady-state water budget

Water-budget components	Quantity (acre-ft/yr)
Inflow	
Areal recharge	4,700
Stream recharge	1,500
Total	6,200
Outflow	
Municipal pumpage	100
Net irrigation pumpage	1,400
Evapotranspiration (east of Barka Slough)	400
Leakage to Barka Slough (evapotranspiration and base flow)	4,300
Total	6,200

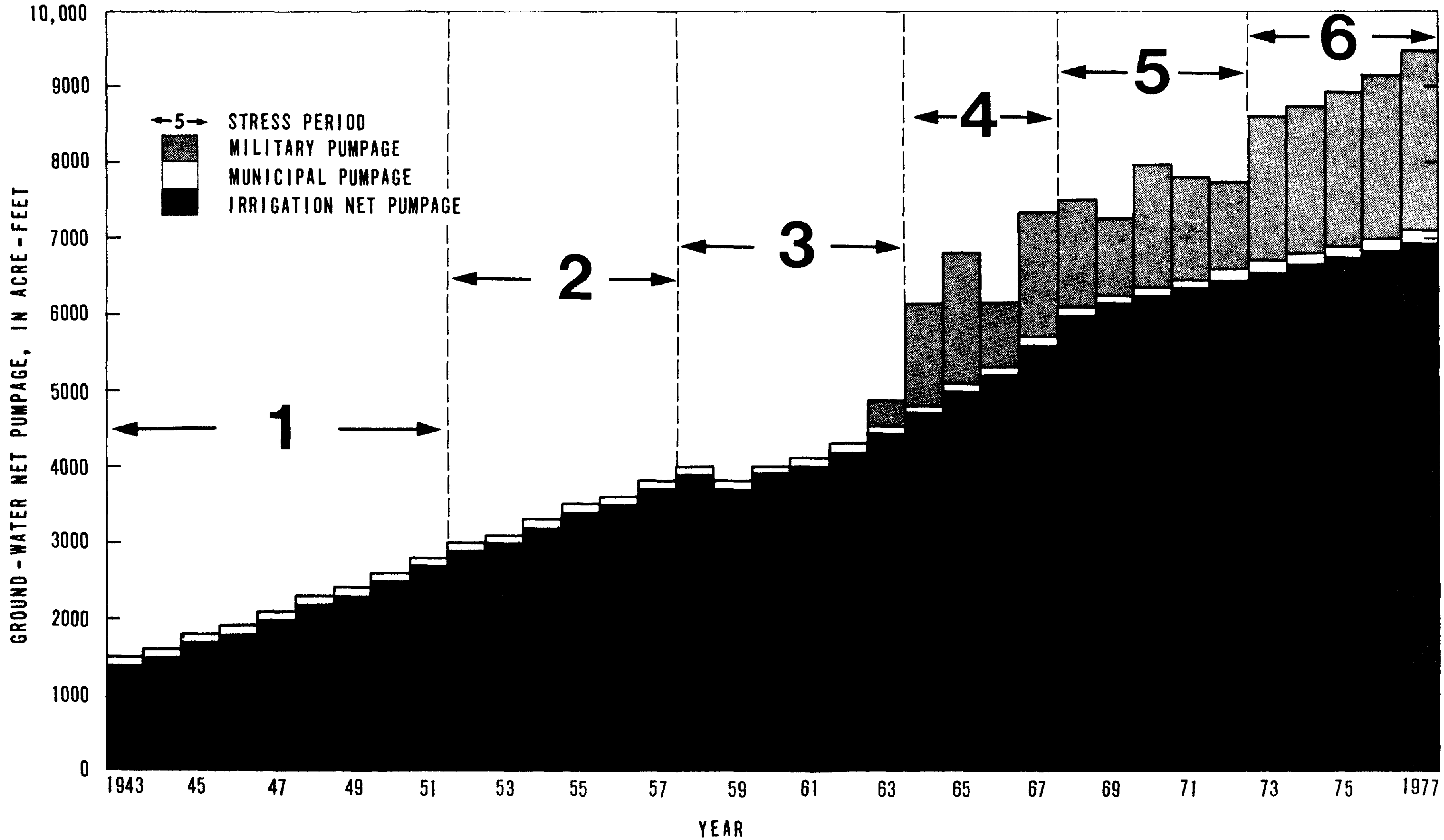


FIGURE 8.--Components of annual net pumpage from the San Antonio Creek ground-water basin, 1943-77.

Water-level contours produced from the steady-state simulation and those based on field data are shown in figure 9. The water-level contours in the valley floor were constructed from field data, whereas the contours along the perimeter of the valley were constructed from trends in water levels shown by sparse historical data. Model calibration was considered acceptable when the difference between model-calculated and measured water levels was within ±5 feet along the San Antonio Creek valley floor, and within ±25 feet along the perimeter of the valley. A greater difference was accepted along the perimeter of the valley because water-level data were sparse and of questionable accuracy.

The steady-state calibrated transmissivity values ranged from less than 100 ft^2/d along the perimeter of the valley to more than 20,000 ft^2/d along the valley floor east of Barka Slough (fig. 7). The model values along the valley floor were in general slightly higher than the values estimated from aquifer tests. No aquifer-test results were available along the valley perimeter; however, the low model transmissivity values are probably representative of the lower member of the Careaga Sand. Beneath the central part of the valley, the lower member of the Careaga Sand has little effect on the average transmissivity of the aquifer, because of its low permeability and the large saturated thickness of the highly permeable younger formations. Along the valley perimeter, however, because the younger formations are unsaturated or have been eroded away, the saturated thickness of the aquifer consists predominantly of the lower member of the Careaga Sand (fig. 2).

The vertical hydraulic conductivity of the confining bed of peat and clay underlying Barka Slough was determined, after several calibration runs, to be 0.054 ft/d. The model-calibrated value is about 35 percent higher than the original estimate.

Transient-State Simulation

Once the transmissivity values of the aquifer and the vertical-hydraulic-conductivity values of the confining layer underlying Barka Slough had been determined by the steady-state simulation, the model was calibrated to simulate transient or time-dependent conditions. Transient conditions are the result of stress on the system imposed by man's use of the water resources, including the pumpage of ground water for municipal, agricultural, and military use. During 1944-77, ground-water pumpage was in excess of the natural recharge, and the water was derived from either aquifer storage or from a reduction in the amount of ground water naturally discharged from the aquifer.

The magnitude of the water-level declines is dependent on the storage coefficient of the aquifer and the net flux of the system (the difference between natural recharge and net discharge), assuming the transmissivity and vertical-hydraulic-conductivity values calibrated during the steady-state simulation remain constant during transient conditions. If the net-flux values are known, the storage-coefficient distribution can be determined during the transient-state simulation by adjusting storage-coefficient estimates until model-generated water-level changes match measured data; if the storage coefficient distribution is known, the net-flux values can be adjusted.

In this model the net-flux values, rather than the storage-coefficient estimates, were adjusted during the transient-state simulation because the recharge and discharge data were considered less reliable. During steady-state conditions, water levels were near or above land surface along the San Antonio Creek channel west of Los Alamos, and ground water was naturally discharged from the aquifer by evapotranspiration and base flow along the stream channel. The high water levels along the stream channel precluded significant recharge from streamflow seepage during steady-state conditions. After 1943, however, ground-water pumpage along the San Antonio Creek channel lowered water levels below land surface beneath most of the channel, increasing the quantity of recharge from streamflow seepage and, conversely, reducing the quantity of ground water naturally discharged from the aquifer.

To accurately determine the net-flux values would require many streamflow gaging stations along San Antonio Creek and its tributaries. Measurements, more accurate than those available for this study, would also be needed of net pumpage and evapotranspiration along the San Antonio Creek channel and in Barka Slough. Due to the unreliability of the historical net-flux estimates and the prohibitively high costs of constructing enough gaging stations to accurately measure seepage losses and base flow, the net-flux values along the San Antonio Creek channel from the eastern edge of Barka Slough to about 2 miles east of Los Alamos were determined by model calibration during the transient-state simulation. The quantity of ground-water leakage to Barka Slough (base flow and evapotranspiration) was computed by the model. The net-flux values everywhere else in the model were assumed to be the same as the steady-state values or were measured fluxes, such as military or municipal pumpage, that could be entered directly into the model.

Net-Flux Calibration

Model parameters, such as net flux, can be calibrated during the transient simulation by a slow and tedious trial-and-error method or by much faster statistical techniques. In this study, the statistical technique of least squares was used to determine the net flux along the San Antonio Creek channel, so that the difference between computed and measured water-level changes was at a minimum. A description of the mathematical development of the least-squares technique is included in an appendix of this report.

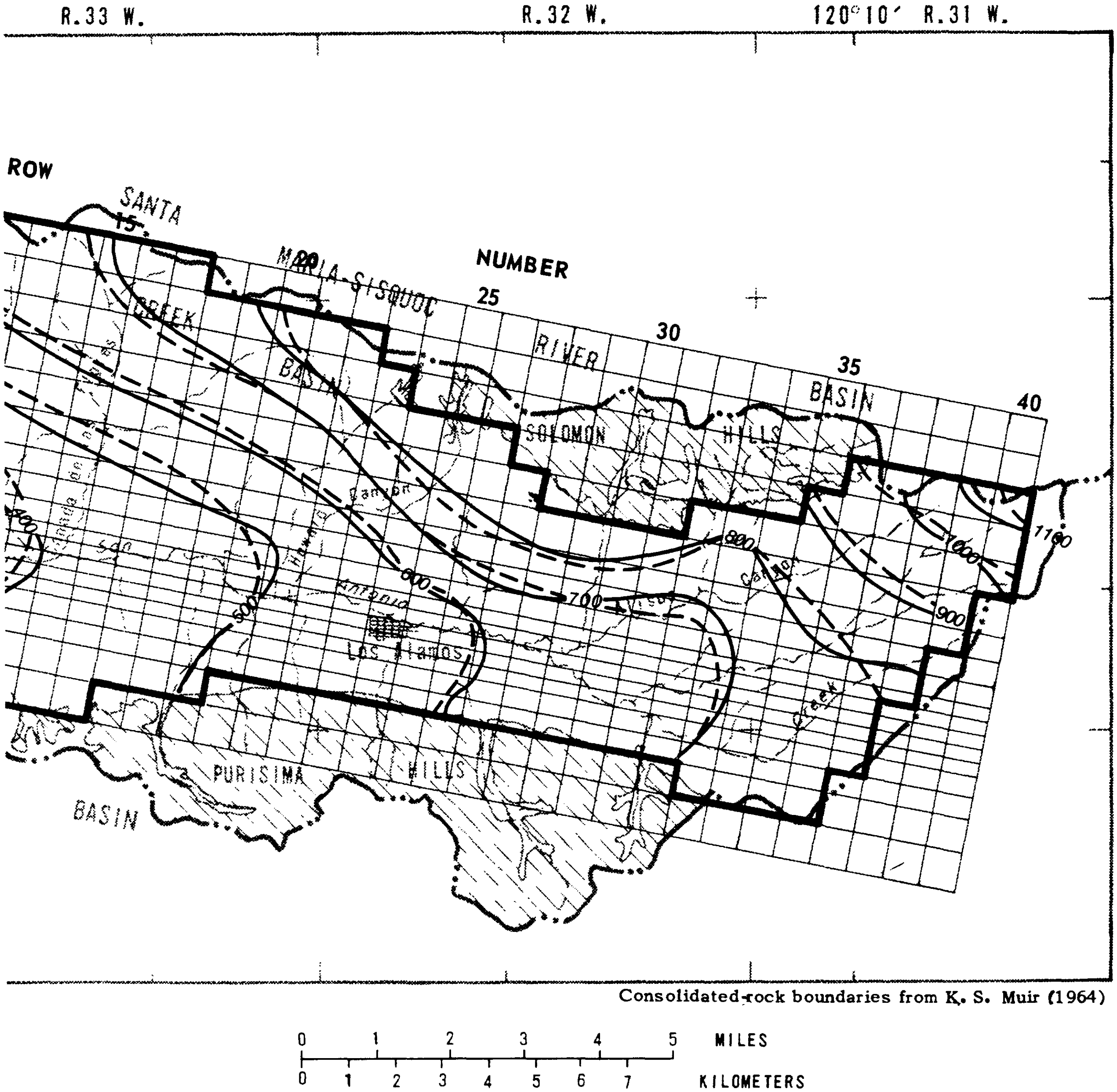
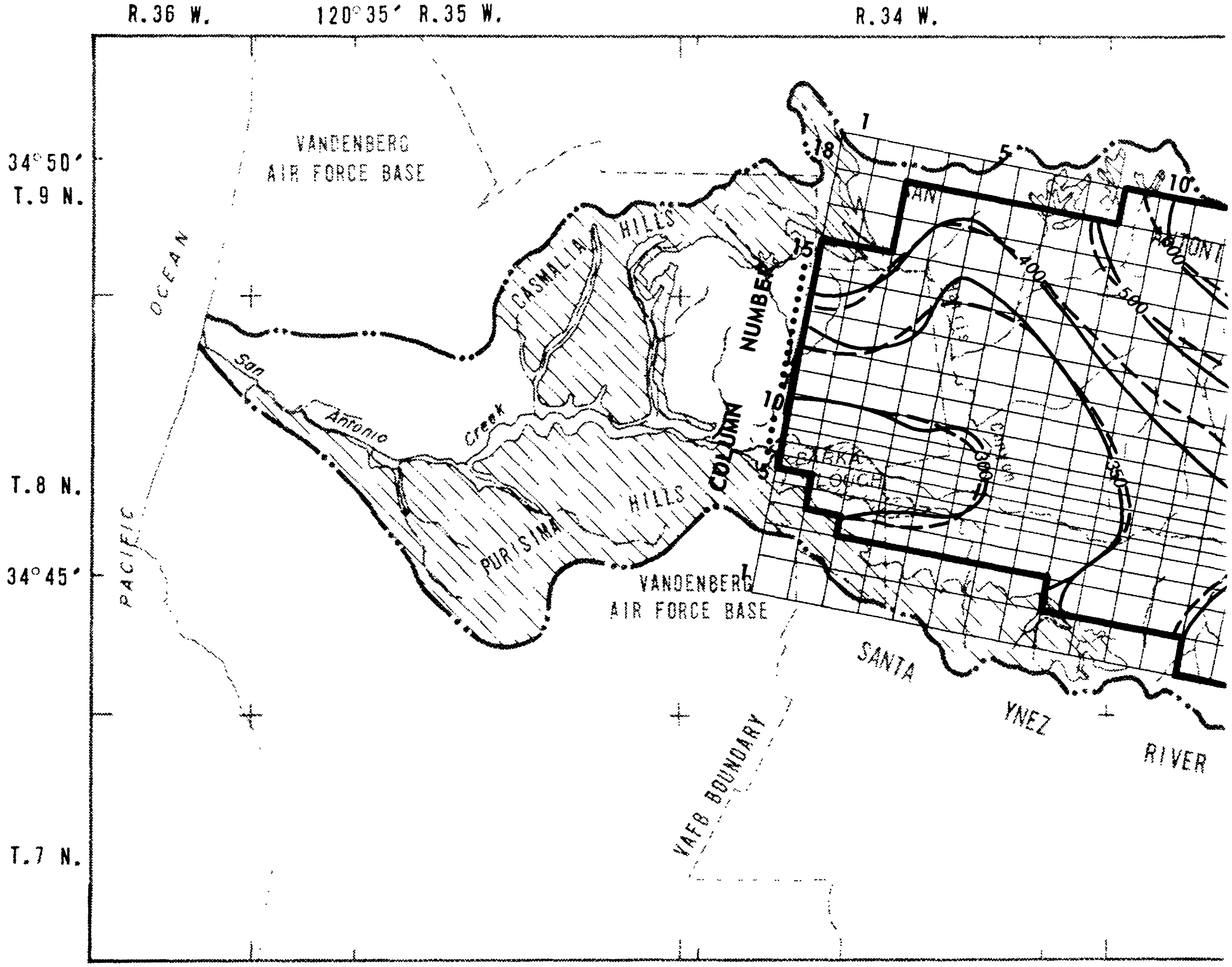


FIGURE 9.—Continued.



EXPLANATION

	UNCONSOLIDATED DEPOSITS
	CONSOLIDATED ROCKS
	BOUNDARY OF MODELED AREA
	GROUND-WATER BARRIER
	SURFACE-WATER DRAINAGE DIVIDE
	INTERMITTENT STREAM
—350—	MEASURED WATER-LEVEL CONTOUR—Shows altitude of water level based on water levels measured in wells, December 1943. Contour interval 50 and 100 feet. National Geodetic Vertical Datum of 1929
— —350— —	MODEL-GENERATED WATER-LEVEL CONTOUR—Shows altitude of water level as generated by the steady-state model. Contour interval 50 and 100 feet. National Geodetic Vertical Datum of 1929

FIGURE 9.—Comparison of measured and model-generated water levels for steady-state conditions (1943).

The least-squares technique was used to determine the net-flux values along the San Antonio Creek channel for 1944-77. The calibration procedure was divided into six different stress periods: (1) 1944-52, (2) 1953-57, (3) 1958-63, (4) 1964-67, (5) 1968-72, and (6) 1973-77. To simplify the calibration procedure, the San Antonio Creek channel from the ground-water barrier west of Barka Slough to about 2 miles east of Los Alamos was divided into seven sections (fig. 10). Each of the sections was one model block wide and from two to five blocks long. The first section includes the model blocks used to simulate military pumpage at the Vandenberg Air Force Base well field near Barka Slough, and the remainder of the sections include model blocks used to simulate seepage losses, evapotranspiration, and irrigation net pumpage along the stream channel.

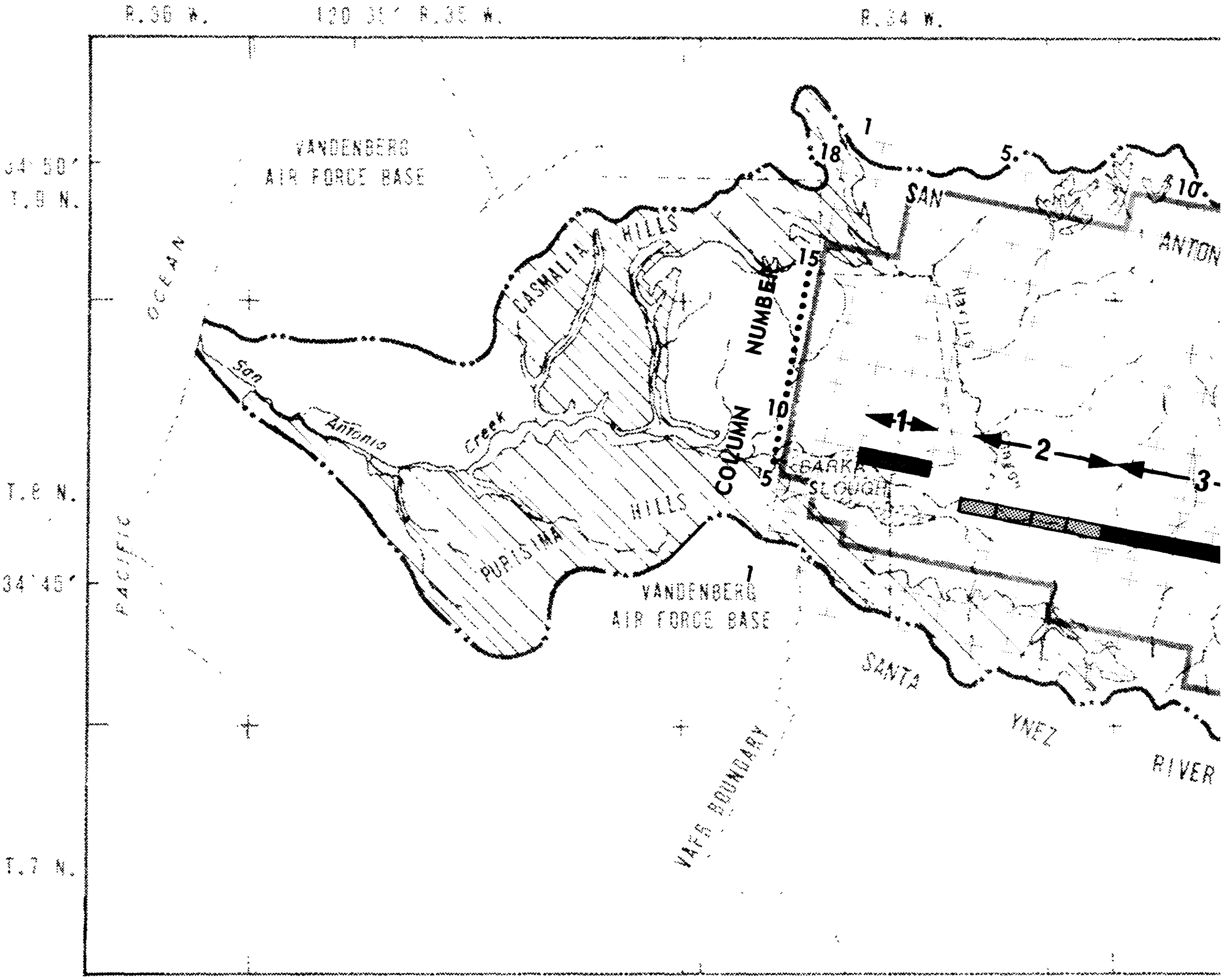
The first step in the calibration procedure involved determining the average water-level changes and unit-response values for each of the sections. The average water-level changes were estimated from composite water-level hydrographs constructed from available data for each of the sections (fig. 11). Because few water-level data were available, portions of the water-level hydrographs prepared for sections 4, 5, and 7 were estimated from hydrographs prepared for adjacent sections. The unit-response values were determined for the different stress periods by superimposing a 1.0-ft^3/s negative flux (pumpage) on the steady-state flux at a designated section of the model for the length of the stress period and recording the water-level change in response to this unit flux change in each of the sections at the end of the stress period. For sections represented by more than one block of the finite-difference grid, the unit net flux change was distributed equally over all the blocks with a cumulative rate of -1 ft^3/s. While the unit response for a particular section was being determined, all other fluxes were kept the same as steady-state conditions. This procedure was repeated for each of the sections in consecutive order. The computed unit-response values can be written in matrix notation as:

$$(\alpha_{ji}) = \begin{pmatrix} \alpha_{11} & \alpha_{12} & \cdots & \alpha_{17} \\ \alpha_{21} & \alpha_{22} & \cdots & \alpha_{27} \\ \vdots & \vdots & & \vdots \\ \alpha_{71} & \alpha_{72} & \cdots & \alpha_{77} \end{pmatrix},$$

where the element α_{ji} in the j^{th} row and the i^{th} column represents the computed water-level change at the end of a stress period in the i^{th} section in response to a unit-net-flux change in the j^{th} section. The computed unit-response values are presented in table 3 for each of the stress periods.

TABLE 3. - Unit-response matrices for the different stress-period lengths

Input section	Response section						
	1	2	3	4	5	6	7
4-year unit-response matrix (stress period 4)							
1	2.58	0.55	0.14	0.04	0.01	0.00	0.00
2	.38	3.08	1.27	.35	.11	.01	.00
3	.12	1.07	3.88	1.69	.59	.08	.01
4	.03	.28	1.59	4.70	2.23	.38	.07
5	.01	.07	.47	1.93	4.74	1.61	.33
6	.00	.01	.10	.47	1.51	6.04	1.82
7	.00	.00	.02	.09	.32	2.61	6.62
5-year unit-response matrix (stress periods 2, 5, and 6)							
1	2.59	0.59	0.18	0.06	0.07	0.00	0.00
2	.42	3.29	1.52	.48	.17	.02	.00
3	.15	1.28	4.33	2.05	.79	.13	.02
4	.04	.38	1.95	5.23	2.68	.54	.11
5	.01	.11	.64	2.34	5.33	2.02	.48
6	.00	.02	.15	.66	1.89	6.76	2.28
7	.00	.00	.03	.14	.46	3.17	7.46
6-year unit-response matrix (stress period 3)							
1	2.61	0.63	0.21	0.07	0.03	0.00	0.00
2	.44	3.46	1.74	.61	.23	.04	.01
3	.17	1.47	4.72	2.39	.99	.19	.04
4	.06	.49	2.28	5.70	3.09	.71	.16
5	.02	.16	.82	2.73	5.85	2.40	.64
6	.00	.04	.22	.85	2.26	7.40	2.73
7	.00	.00	.05	.20	.61	3.69	8.21
9-year unit-response matrix (stress period 1)							
1	2.63	0.65	0.24	0.11	0.06	0.02	0.01
2	.46	3.50	1.86	.85	.45	.15	.06
3	.19	1.58	4.91	2.78	1.46	.50	.20
4	.09	.71	2.68	6.07	3.62	1.24	.49
5	.04	.34	1.28	3.28	6.35	3.07	1.21
6	.02	.15	.59	1.51	3.15	8.42	3.86
7	.01	.06	.22	.56	1.17	4.48	8.97



EXPLANATION

UNCONSOLIDATED DEPOSITS

CONSOLIDATED ROCKS

BOUNDARY OF MODELED AREA

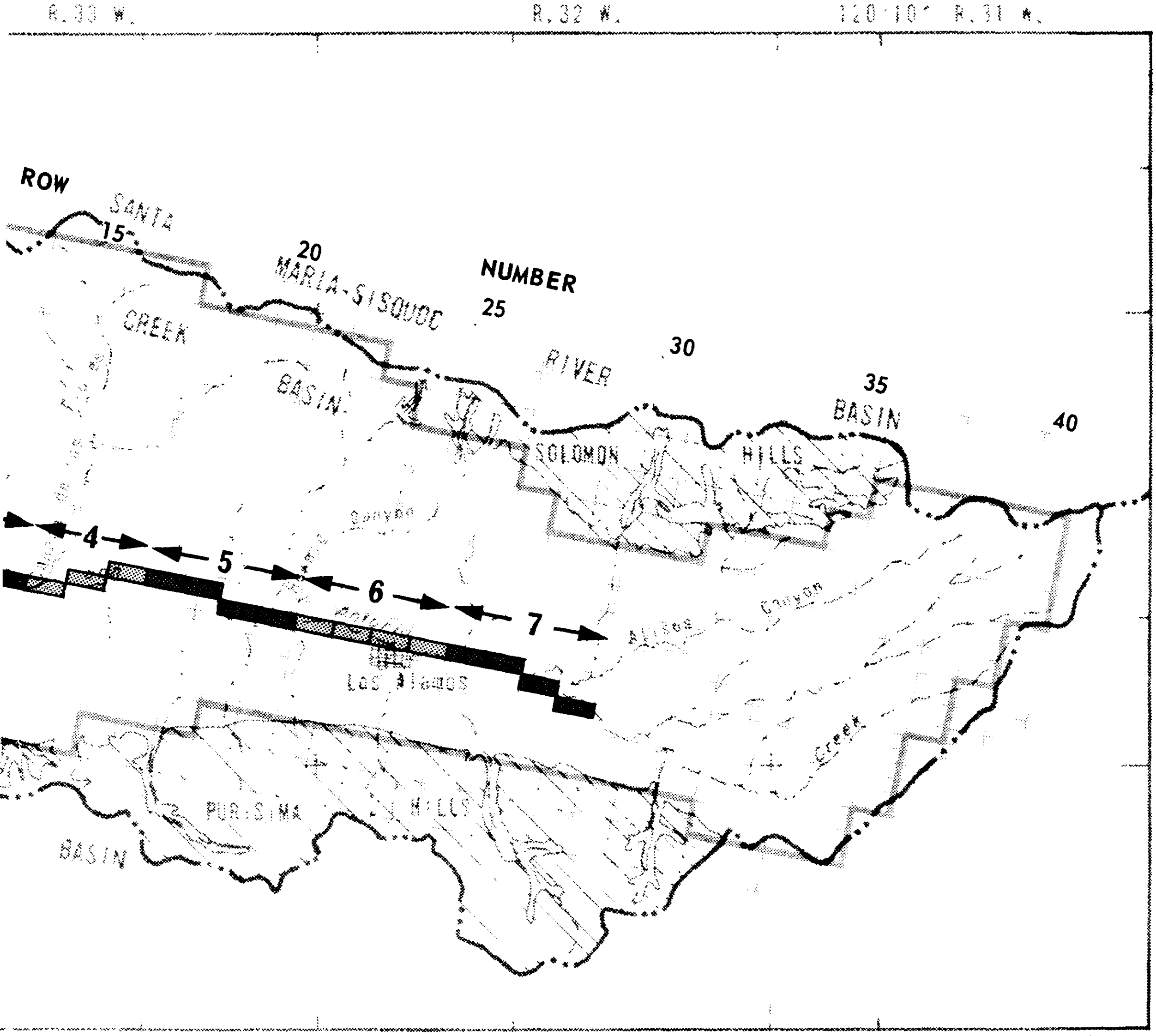
SECTION AND NUMBER--Alternate dark and light patterns are used to distinguish section parameters

SURFACE-WATER DRAINAGE DIVIDE

INTERMITTENT STREAM

GROUND-WATER BARRIER

FIGURE 10.--Location of sections used in the net-flux calibration.



Consolidated-rock boundaries from K. S. Muir (1964)

FIGURE 10.--Continued.

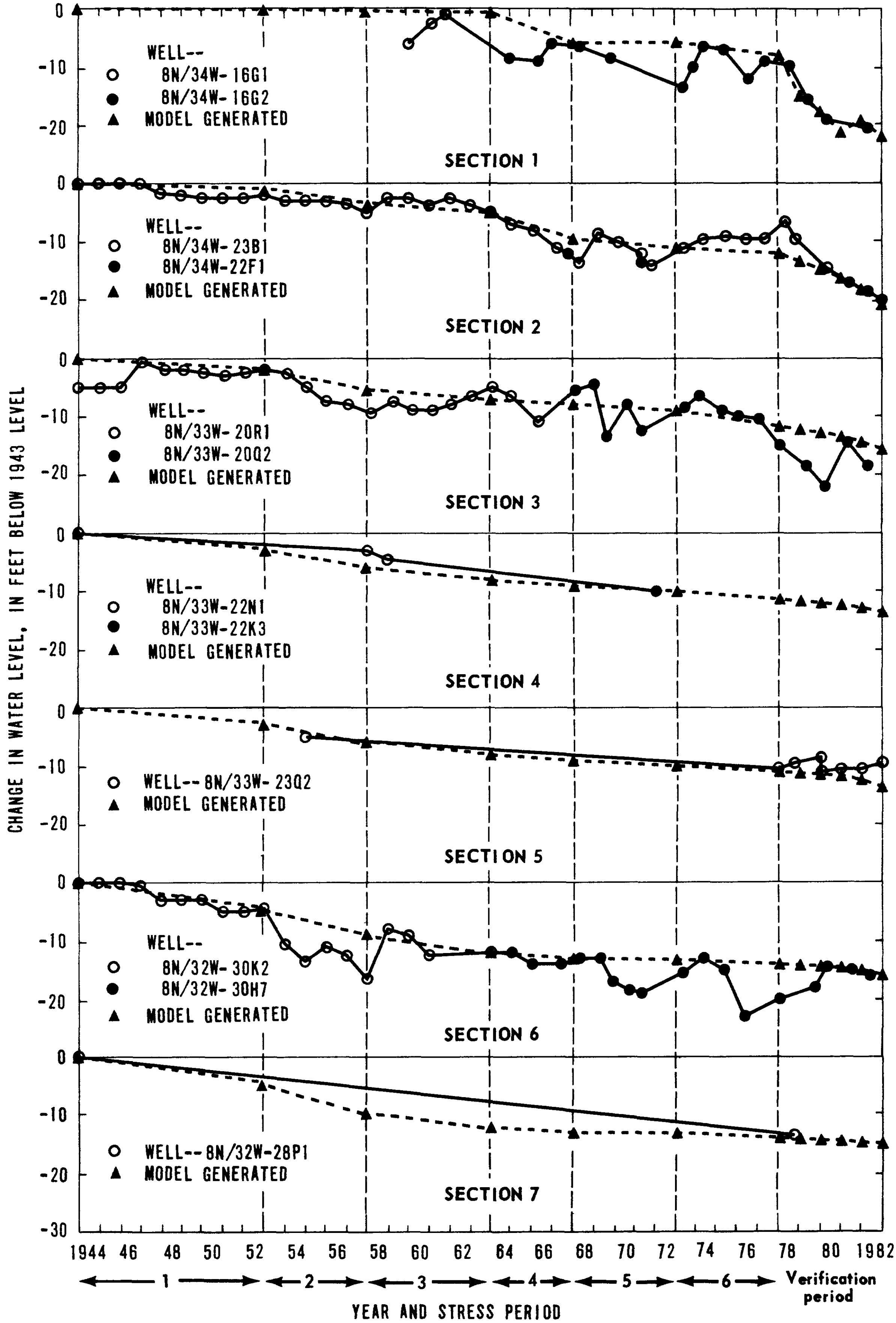


FIGURE 11.--Hydrograph showing comparison of model-generated and historical measured water levels.

The second step of the calibration procedure involved using the least-squares equation (eq. 11, appendix) to compute the change in the net flux (Δq_j) for each stress period. The Δq_j values cause an incremental water-level change (Δh_i) in each stress period; Δh_i can be computed for a particular stress period from the technique of superposition. This technique requires that the natural flow system can be approximated as a linear system, one in which total flow is the addition of the individual flow components resulting from distinct stresses (Reed, 1980, p. 3). For instance, consider a situation (fig. 12) where a well has been pumping for the length of a stress period at a constant rate q_1, and the drawdown trend for that pumping rate has been established. Assume that the pumping rate increases by some amount Δq in the next stress period. Then the incremental water-level change for that increase in the next flux (Δh_i^2 in fig. 12) will be the difference in the water-level change from that occurring due to the pumpage q_1.

The Δh_i values can be computed for a particular stress period as:

$$\Delta h_i^n = \hat{h}_i^n - ph_i^n \qquad \begin{array}{l} i = 1,2,\ldots,7 \\ n = 1,2,\ldots,6, \end{array} \tag{4}$$

where:

Δh_i^n is the incremental water-level change at the i^{th} section in the n^{th} stress period;

$\hat{h}_i^n$ is the measured water-level change at the i^{th} section in the n^{th} stress period; and

ph_i^n is the predicted water-level change at the i^{th} section in the n^{th} stress period occurring due to the net flux in the n-1 stress period.

The relationship between Δh_i, $\hat{h}_i$, and ph_i is shown in figure 12. The ph_i values were computed for a particular stress period by inputting the net-flux values determined for the previous stress period into the current stress period of the model and recording the water-level change. The ph_i values were set equal to zero in stress period 1 because there were steady-state conditions prior to the stress period.

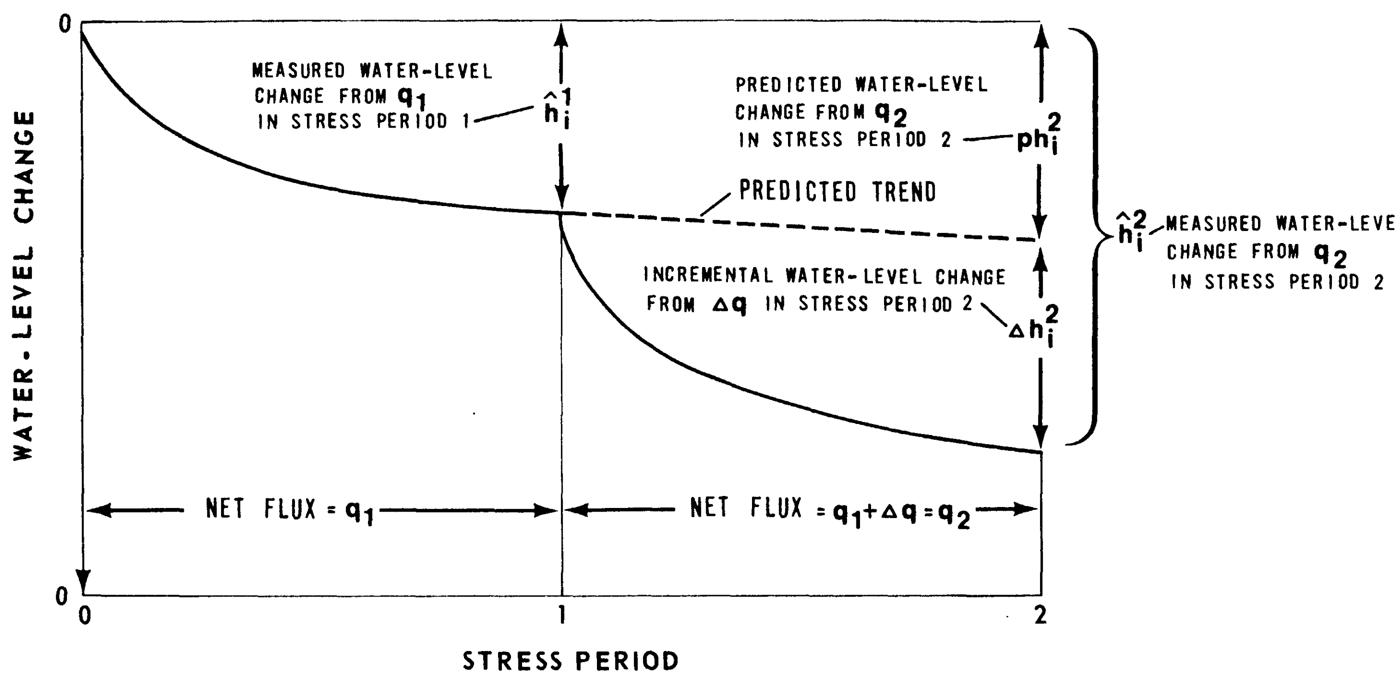


FIGURE 12.--The application of the principal of superposition to the least-squares calibration technique.

The Δq_j values were computed for each stress period by inputting the Δh_i and α_{ji} values determined for each section into the following form of the least-squares equation:

$$(A)\ \{\Delta q_j\} = \{b\}, \tag{5}$$

where:

$$(A) = \begin{pmatrix} a_{11} & a_{12} \cdot & \cdot & \cdot & a_{1i} \\ a_{21} & a_{22} \cdot & \cdot & \cdot & a_{2i} \\ \cdot & \cdot & & & \cdot \\ \cdot & \cdot & & & \cdot \\ \cdot & \cdot & & & \cdot \\ a_{j1} & a_{j2} \cdot & \cdot & \cdot & a_{ji} \end{pmatrix} = (\alpha_{ji})(\alpha_{ji})^T,$$

and

$$b = \begin{Bmatrix} b_1 \\ b_2 \\ \cdot \\ \cdot \\ \cdot \\ b_i \end{Bmatrix} = (\alpha_{ji})\ \{\Delta h_i\}$$

$i = 1,2,\ldots,7$
$j = 1,2,\ldots,7$ (see appendix for derivation).

This matrix equation represents a series of seven linear equations with seven unknown Δq_j values. The value of Δq_1 (the change in net flux in section 1) is known, however, since it simulates the military pumpage, a metered supply, at Barka Slough. Therefore, the value of Δq_1 was set equal to the measured change in military pumpage in each of the stress periods. This was done by setting the a_{11} term in the A matrix equal to 1 and the remaining terms in the first row of the matrix $(a_{12},a_{13},\ldots,a_{17})$ equal to 0, and the b_1 term is the b vector equal to the measured change in the military pumpage. The series of linear equations was then solved for $\Delta q_2,\Delta q_3,\ldots\Delta q_7$ (the change in the net flux in sections 2-7) using a computer program written by the SAS Institute Inc. (1982).

After the Δq_j values were computed for a particular stress period, the computed values were added to the net-flux values of the previous stress period, and the resulting sums were entered into the model as the net flux in the stress period currently being calibrated. The model was then run for the length of the current stress period and the following stress period with the new net-flux values. The model-computed water-level changes at the end of the stress period currently being calibrated were compared to the measured values at each section to check the calibration procedure. If the differences between computed and measured water-level changes were within ±0.5 foot, the net-flux calibration was considered acceptable. If the differences were greater than ±0.5 foot, the input data were checked for errors, and the calibration procedure for that stress period was repeated. If no errors were found, the stress period was subdivided into smaller periods and the calibration procedure was repeated for each new stress period, as discussed in the following paragraph. Next, the computed water-level changes in the following stress period were used to determine the Δh_i values from equation 4, and then to calibrate the net-flux values in that stress period. This stepwise procedure was repeated until the net-flux values were computed for all the stress periods. A flow chart of the calibration procedure is presented in figure 13. Results of the net-flux calibration are presented in table 4 for each stress period.

TABLE 4. - Calibrated net-flux values in sections 1-7

[Values in acre-feet per year. Negative sign indicates water being removed from the aquifer system]

Section	Stress period													
	Steady state 1943	1 1952		2 1957		3 1963		4 1967		5 1972		6 1977	
	Net flux	Net flux		Net flux		Net flux		Net flux		Net flux		Net flux	
	Total	Change	Total	Change	Total	Change	Total	Change	Total	Change	Total	Change	Total
1	0	0	0	0	0	0	0	-1,390	-1,390	+160	-1,230	-600	-1,830
2	-930	-100	-1,030	-460	-1,490	-70	-1,560	-940	-2,500	-60	-2,560	+160	-2,380
3	-770	-100	-870	-340	-1,210	+220	-990	+400	-590	+80	-510	-390	-900
4	-100	-200	-300	-20	-320	-30	-350	-20	-370	+20	-350	+70	-280
5	+250	-30	+220	-100	+120	+140	+260	+40	+300	0	+300	+10	+310
6	+150	-230	-80	-70	-150	-60	-210	+50	-160	+100	-60	-10	-70
7	+250	-290	-40	-280	-320	+140	-180	+70	-110	+110	0	-30	-30
Total	-1,150	-950	-2,100	-1,270	-3,370	+340	-3,030	-1,790	-4,820	+410	-4,410	-770	-5,180

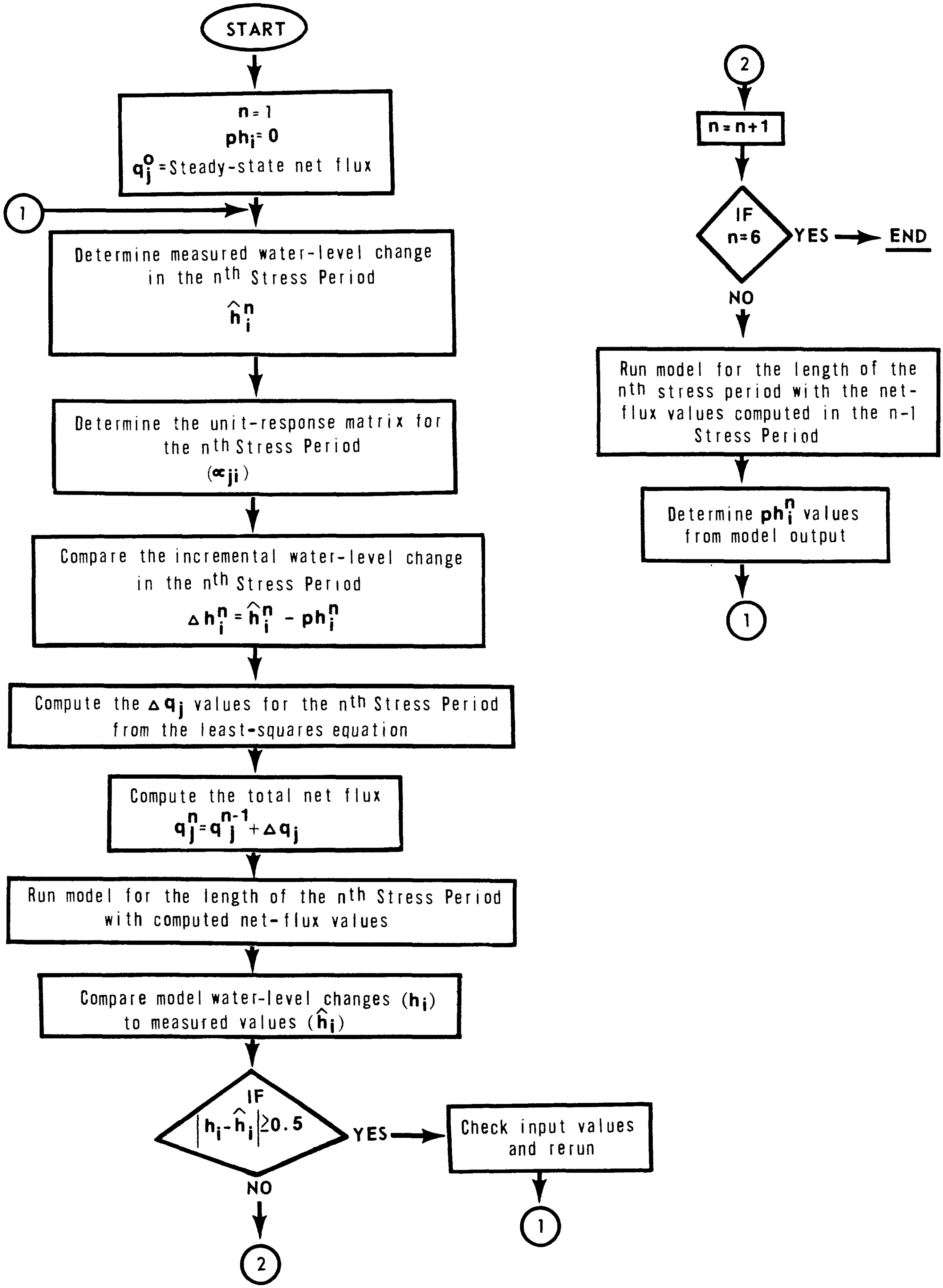


FIGURE 13.--Flow chart of the net-flux calibration.

During the initial calibration of stress period 6, the difference between the computed and measured water-level change in section 1 was more than ±0.5 foot. The large difference was the result of using an average value for military pumpage in section 1. In the final calibration run, stress period 6 was divided into five separate time steps, and annual values of military pumpage were used in the model. The results of the final year of the net-flux calibration of stress period 6 are shown in table 4.

The calibration procedure described above was repeated for each stress period, without specifying the net-flux value in section 1, to see if specifying the net-flux value in section 1 affected the net-flux values computed at the other sections. The computed net-flux values for section 1 are presented in table 5 along with measured military pumpage. As shown in the table, the values compare well. Apparently, specifying the net-flux values in section 1 had little effect on the computed net-flux values.

TABLE 5. - Comparison of computed net-flux values to measured military pumpage in section 1

[All values in acre-feet per year]

Stress period	Computed net-flux value	Measured pumpage (military pumpage)
1 (1944-52)	+10	0
2 (1953-57)	-30	0
3 (1958-63)	-10	0
4 (1964-67)	-1,310	-1,390
5 (1968-72)	-1,220	-1,230
6 (1973-77)	-1,900	-1,830

Transient-State-Model Results

Water levels produced from the calibrated net-flux values were used to construct a water-level contour map for the end of stress period 6, which corresponds to the end of 1977. A water-level contour map constructed from field data collected during January 1978 (Hutchinson, 1980) was used to compare the transient response of the model with the actual system. Comparison of the water-level contours constructed from field data with those constructed from model results shows similar regional patterns of ground-water flow (fig. 14). The similarity of the water-level contours indicates that the calibrated net-flux values closely simulate the hydrologic response of the regional ground-water system, as well as the hydrologic response in the immediate area of the calibrated section of the San Antonio Creek channel.

Results of the transient-state analysis for the 1977 simulation (table 4) indicate that since the steady-state period, the net flux out of the system along San Antonio Creek increased by about 4,030 acre-ft/yr. During this same period, net discharge (irrigation net pumpage, municipal pumpage, military pumpage, and evapotranspiration) along the calibrated sections of the stream channel was estimated to increase by 7,050 acre-ft/yr (table 6). The difference between the calibrated net-flux values and the estimated net-discharge values was assumed to be equal to stream recharge. The total amount of stream recharge along the San Antonio Creek channel was then computed to be equal to this difference plus the 750 acre-ft/yr of stream recharge simulated along the uncalibrated reach of the stream channel in the eastern part of the valley (table 7).

TABLE 6. - Comparison of calibrated net-flux values to estimated net-discharge values in stress periods 1-6

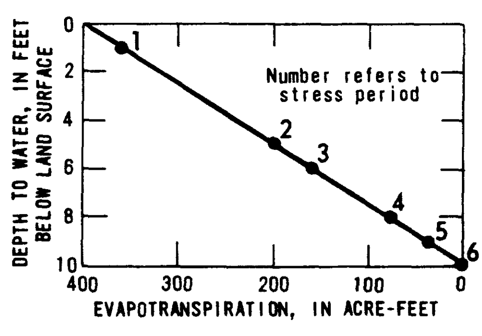
[All values, in acre-feet per year, represent average values for the stress period, except where noted]

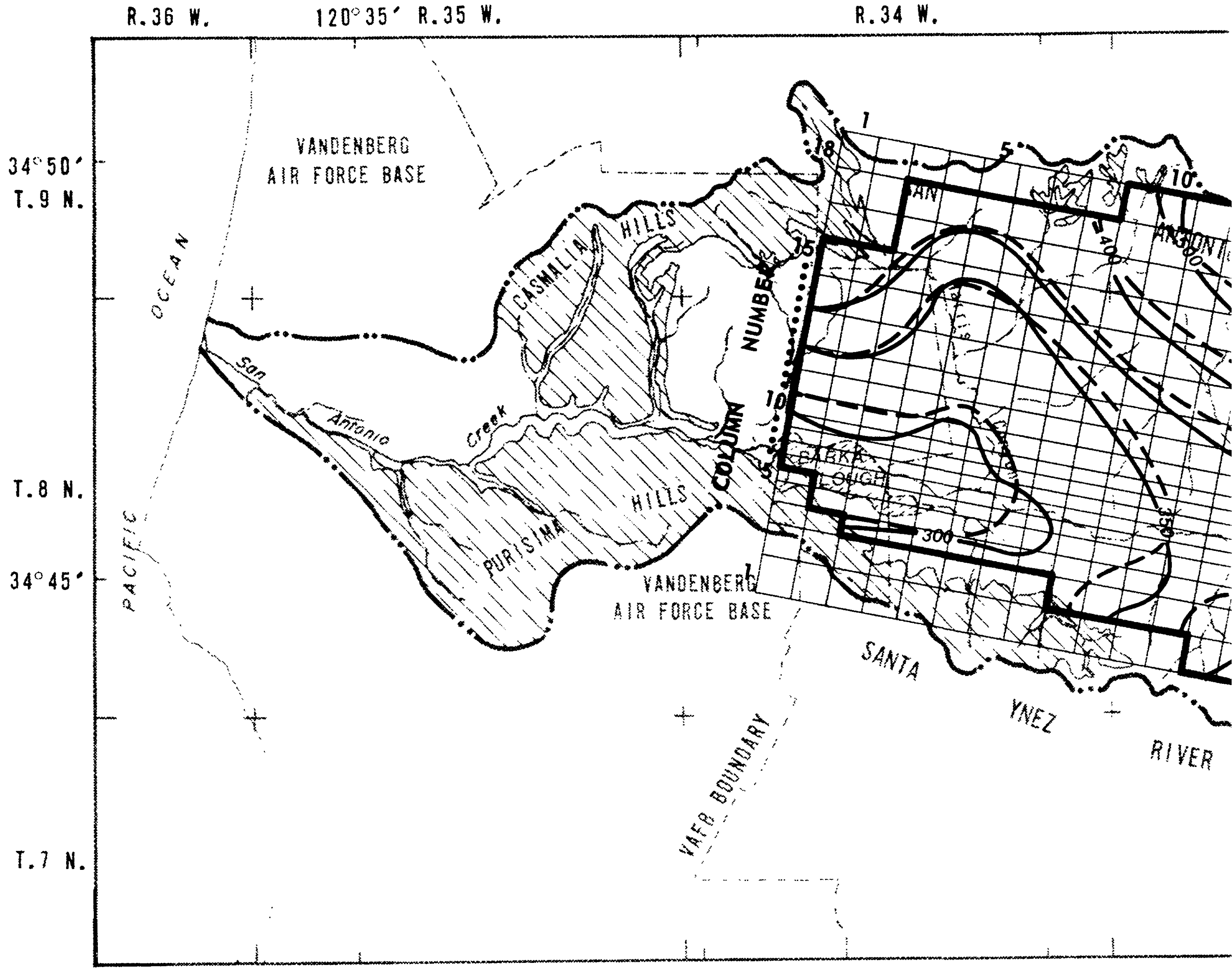
Stress period	Irrigation net pumpage	Municipal pumpage	Military pumpage	Evapotranspiration[1]	Total net discharge	Calibrated net flux	Calibrated net flux minus total net discharge
Steady state	-1,400	-100	0	-400	-1,900	-1,150	+750
1	-2,180	-100	0	-360	-2,640	-2,100	+540
2	[2]-3,700	-100	0	-200	-4,000	-3,370	+630
3	-4,020	-100	0	-160	-4,280	-3,030	+1,250
4	-5,120	-100	-1,390	-80	-6,690	-4,820	+1,870
5	-6,250	-110	-1,230	-40	-7,630	-4,410	+3,220
6[3]	-6,950	-170	-1,830	0	-8,950	-5,180	+3,770

[1]Evapotranspiration was estimated by a linear relation between a maximum evapotranspiration rate at land surface (400 acre-ft/yr) and a depth below land surface (10 feet) at which evapotranspiration ceases (see diagram).

[2]Represents the measured irrigation net pumpage in 1957. During 1957, irrigation pumpage was estimated by both the crop-survey and electrical-energy methods (Muir, 1964, p. 31) and this value was considered more accurate than the average value for the period.

[3]Stress period 6 was divided into five yearly time steps. The table shows only the results of the final year of the stress period, 1977.





EXPLANATION

UNCONSOLIDATED DEPOSITS

CONSOLIDATED ROCKS

BOUNDARY OF MODELED AREA

GROUND-WATER BARRIER

SURFACE-WATER DRAINAGE DIVIDE

INTERMITTENT STREAM

—300— MEASURED WATER-LEVEL CONTOUR—Shows altitude of water level based on water levels measured in wells, January 1978. Contour interval 50 and 100 feet. National Geodetic Vertical Datum of 1929

— —350— — MODEL-GENERATED WATER-LEVEL CONTOUR—Shows altitude of water level generated by the model for the end of Stress Period 6 (1977). Contour interval 50 and 100 feet. National Geodetic Vertical Datum of 1929

FIGURE 14.—Comparison of water-level contours from field data for January 1978 and those from model results for the end of 1977.

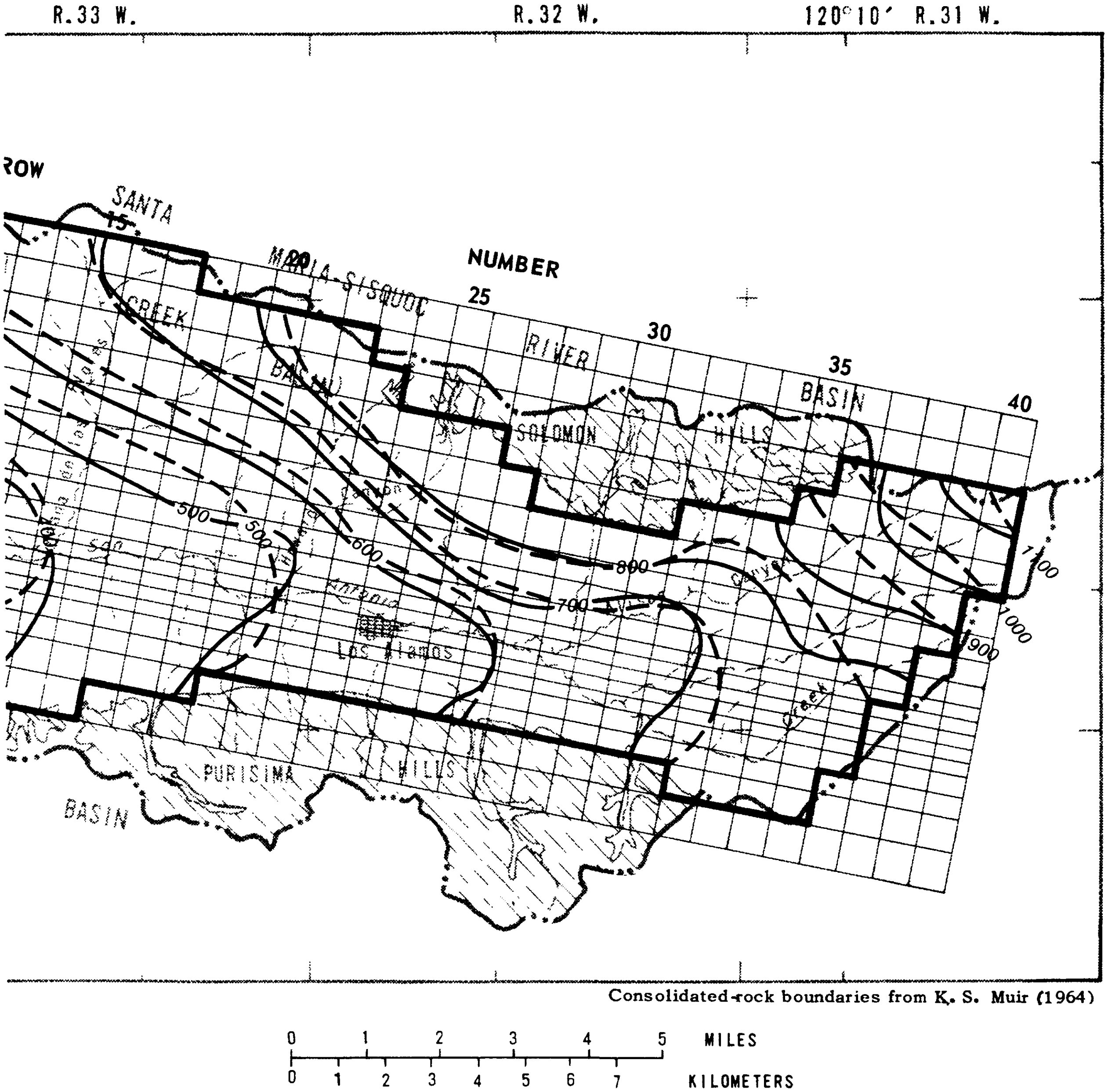


FIGURE 14.—Continued.

TABLE 7. - Estimation of stream recharge from model results in stress periods 1-6

[All values in acre-feet per year]

Stress period	Last year of stress period	Stream recharge			Increase (+) or decrease (-) from steady-state conditions
		Uncalibrated reach	Calibrated reach	Total	
Steady state	1943	750	750	1,500	--
1	1952	750	540	1,290	-210
2	1957	750	630	1,380	-120
3	1963	750	1,250	2,000	+500
4	1967	750	1,870	2,620	+1,120
5	1972	750	3,220	3,970	+2,470
6	1977	750	3,770	4,520	+3,020

The simulated stream-recharge values ranged from a low of 1,290 acre-ft/yr in stress period 1 to a high of 4,520 acre-ft/yr in stress period 6 (table 7). In stress periods 1 and 2, simulated values of stream recharge were 120 to 210 acre-ft/yr lower than steady-state conditions, whereas in the later stress periods the simulated values for stream recharge were 500 to 3,020 acre-ft/yr higher than steady-state conditions. The increase in stream recharge in the later stress periods is the result of increased pumpage along the stream channel. The increased pumpage lowered water levels, which were near or above land surface during steady-state conditions, to below the bottom of the stream channel west of Los Alamos, providing storage space for the seepage of available runoff.

The simulated values of stream recharge (table 7) should only be considered as gross estimates because the irrigation net-pumpage estimates used to help compute the recharge values are probably inaccurate. The pumpage estimates were based on data from five crop surveys made in the study area during 1943, 1957, 1966, 1968, and 1975 (see section on "Discharge"). The irrigation net-pumpage estimates for 1944-57 were based on the 1943 and 1957 crop surveys, and the pumpage estimates for 1958-77 were based on the 1966, 1968, and 1975 crop surveys. Rainfall in 1944-57 averaged 12.98 inches, whereas rainfall in 1943 and 1957 was significantly higher at 18.17 and 14.03 inches. During wet years, fields that are normally irrigated are dry farmed; therefore, crop surveys made during wet years, such as 1943 and 1957, would probably underestimate the irrigation pumpage for the entire period. Rainfall in 1958-77 averaged 14.61 inches, whereas the rainfall in 1966, 1968, and 1975 was significantly lower at 8.08, 11.21, and 10.82 inches, respectively. During dry years, fields that are normally dry farmed require irrigation; therefore, crop surveys made during the dry years of 1966, 1968, and 1975 would probably overestimate the irrigation pumpage for the entire period. To accurately compute the irrigation net pumpage would require more frequent crop surveys or, preferably, metered irrigation pumpage in the valley.

Results of the model simulation indicate about 70 percent (2,810 acre-ft/yr) of the 4,030-acre-ft/yr increase in net flux out of the system along the San Antonio Creek channel as being derived from the reduction of ground-water leakage to Barka Slough, and about 30 percent (1,220 acre-ft/yr) from water coming out of storage (table 8). From 1943 to 1977, ground-water leakage to Barka Slough decreased by about 65 percent, from 4,300 acre-ft/yr to 1,490 acre-ft/yr. The greatest reduction in ground-water leakage occurred in stress period 4 (1964-67) (table 8), when military pumpage was first started in the San Antonio Creek valley. The quantity of ground-water leakage to Barka Slough is sensitive to military pumpage because the military well field is adjacent to the slough and because the aquifer is confined beneath the slough, resulting in rapid water-level fluctuations.

TABLE 8. - Water budget for the steady-state and transient-state conditions as simulated by the model

[Values are in acre-feet per year. Negative sign indicates water being removed from the aquifer system]

Water-budget components	Stress period						
	Steady state 1943	1 1952	2 1957	3 1963	4 1967	5 1972	6 1977
Areal recharge--------	+4,700	+4,700	+4,700	+4,700	+4,700	+4,700	+4,700
Stream recharge along San Antonio Creek east of section 1-----------	+750	+750	+750	+750	+750	+750	+750
Net flux along San Antonio Creek in sections 1-7[1]-------	-1,150	-2,100	-3,370	-3,030	-4,820	-4,410	-5,180
Leakage to Barka Slough--------	-4,300	-4,190	-3,900	-3,740	-2,200	-2,070	-1,490
Storage[2]--------------	0	+840	+1,820	+1,320	+1,570	+1,030	+1,220

[1]The net-flux component includes stream recharge, irrigation net pumpage, municipal pumpage, evapotranspiration along San Antonio Creek east of section 1, and military pumpage. The estimated values of these components are presented in table 6.

[2]A positive value indicates that water is coming out of storage.

The model-simulated reduction in leakage is significantly greater than the measured reduction in base flow (fig. 15), suggesting that there has also been a reduction in the amount of evapotranspiration at Barka Slough. An environmental study of Barka Slough by Dial and Pisapia (1980) concluded that the slough is in transition from an emergent wetland to a more xeric plant community, indicating that evapotranspiration has indeed been reduced over the years. In the northern section of the slough, willows are invading places where marsh plants were abundant, and the eastern end of the slough has been converting to an upland ruderal habitat from a historic inland salt marsh (Dial and Pisapia, 1980, p. 9). Although ground-water leakage to Barka Slough has been reduced over the years, increased pumpage for agriculture in the area necessarily results in increased runoff to San Antonio Creek, thereby compensating for some of the reduction in ground-water leakage.

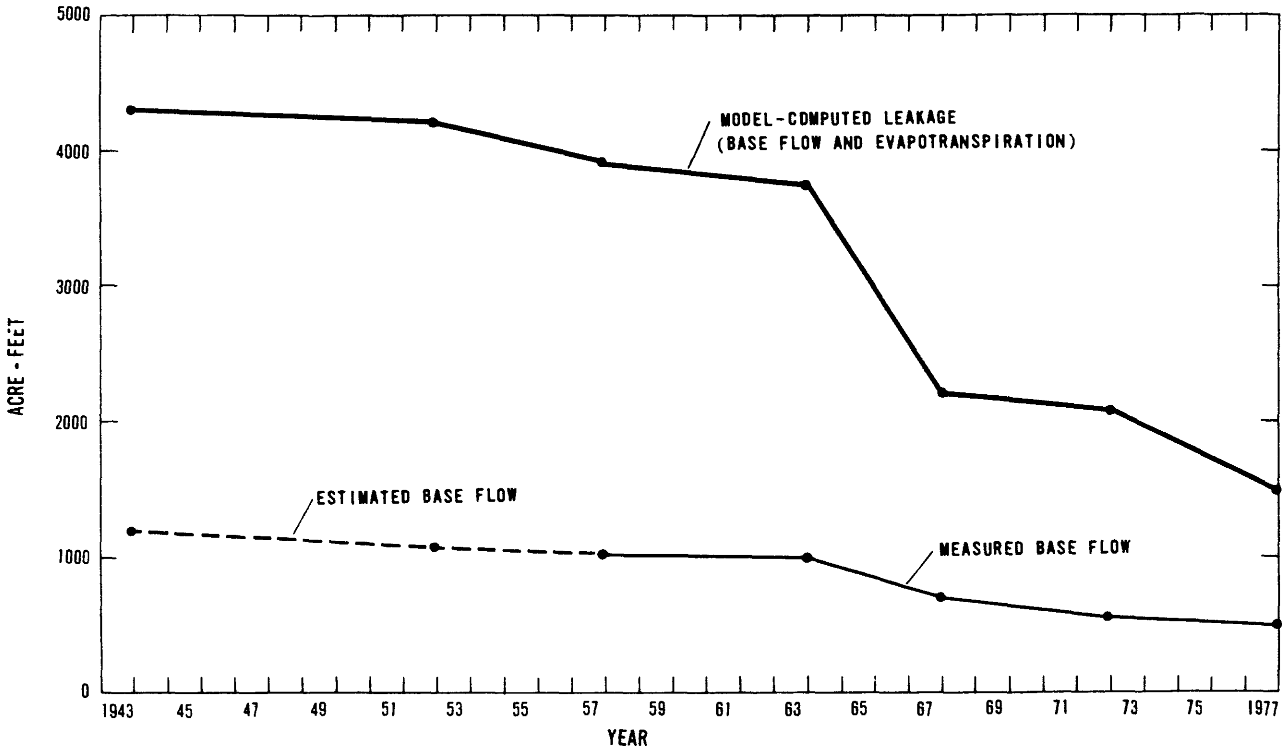


FIGURE 15.--Comparison of measured base flow to model-computed leakage, 1943-77.

Verification

An acceptable fit of the water levels from 1943 to 1977 was generated by adjustments to the transmissivity distribution in the aquifer and the vertical hydraulic conductivity of the confining layer beneath Barka Slough in the steady-state simulation and the net-flux (recharge and discharge) values in the transient-state simulation. The model was then used to simulate the period from 1978 to 1982 using the annual change in net pumpage determined during this study (table 9). The purpose of this run was to verify the model and the calibrated input values.

TABLE 9. - Annual change in net pumpage, 1977-82

[All values in acre-feet per year]

Year	Irrigation net pumpage		Military pumpage	
	Change	Total	Change	Total
1977	--	6,950	--	1,830
1978	0	6,950	1,230	3,060
1979	0	6,950	170	3,230
1980	0	6,950	630	3,860
1981	2,600	9,550	-760	3,100
1982	1,970	11,520	470	3,570
Total change	4,570		1,740	

Changes in Pumpage

During the verification period, extensive irrigated vineyards were established on formerly nonirrigated pastureland. The new agriculture development included about 2,500 acres on the east slope of Harris Canyon and about 900 acres in the lower part of Howard Canyon (fig. 3). The total pumpage for the new irrigated acreage was estimated by the developer to be 3,250 acre-ft in 1981 and 5,700 acre-ft in 1982 (Dana Merrill, San Francisco and Fresno Land Co., written commun., 1982). The net pumpage was estimated to be 2,600 acre-ft in 1981 and 4,570 acre-ft in 1982 by assuming that return flow was 20 percent of the total pumpage. These net-pumpage estimates were simulated in the model blocks in which the irrigation supply wells for the new irrigated acreage are located (fig. 6).

The quantity of military pumpage at Barka Slough also increased during the verification period. From the end of stress period 6 (1977) to 1982, military pumpage increased by about 95 percent or 1,740 acre-ft/yr (table 9).

All other net-flux values were assumed to be the same as calibrated in stress period 6 of the transient-state simulation.

Changes in the Simulation of Barka Slough

In addition to changes in irrigation and military pumpage in the valley, there was also a significant change in the environmental habitat of Barka Slough during the verification period. During the transient-state simulation, the slough consisted predominantly of hydrophytic marshland vegetation. During the later years of the transient-state period and throughout the verification period, however, large areas of the native marshland vegetation were replaced by willow trees (Dial and Pisapia, 1980, p. 37). By 1979, willow trees covered about 350 acres and were the predominant form of vegetation in Barka Slough (Dial and Pisapia, 1980, p. 21). Willow trees are phreatophytes; therefore, unlike the marshland vegetation, they can obtain water from depth by the extension of their roots to the water table.

Evapotranspiration by phreatophytes was simulated in the model during the verification period by using a linear relation between a maximum evapotranspiration rate at land surface and a depth below land surface at which evapotranspiration ceases. The maximum evapotranspiration rate of the willow trees was set equal to 3.0 ft/yr (Muir, 1964, p. 32), and the maximum effective depth of evapotranspiration was assumed to be 10.0 feet. For the purposes of this study, the area of the willow trees was assumed to remain the same throughout the verification period. Therefore, the maximum evapotranspiration value that can be computed by the model when the water table is at or above land surface throughout the slough is 1,050 acre-ft/yr.

Results of the Verification Run

Water-level changes computed by the model during the verification run closely duplicate the measured water levels during the verification period (fig. 11). The areal distribution of the model-generated water-level declines is shown in figure 16. Results of the verification run (table 10) indicate that about 82 percent (5,170 acre-ft/yr) of the 6,310-acre-ft/yr increase in net pumpage from 1977 to 1982 was withdrawn from storage. The remainder of the pumpage increase (1,140 acre-ft/yr) was derived from the reduction of base flow and evapotranspiration at Barka Slough. The model simulated that by the end of 1982 base flow to Barka Slough had ceased, and evapotranspiration by phreatophytes at the slough had been reduced to about 350 acre-ft/yr. As stated before, the increased agricultural pumpage upstream from Barka Slough results in increased runoff to San Antonio Creek, thereby compensating for some of the reduction in ground-water discharge to the slough. A gaging station just upstream from Barka Slough is needed to monitor the contribution of irrigation runoff to streamflow during the dry season.

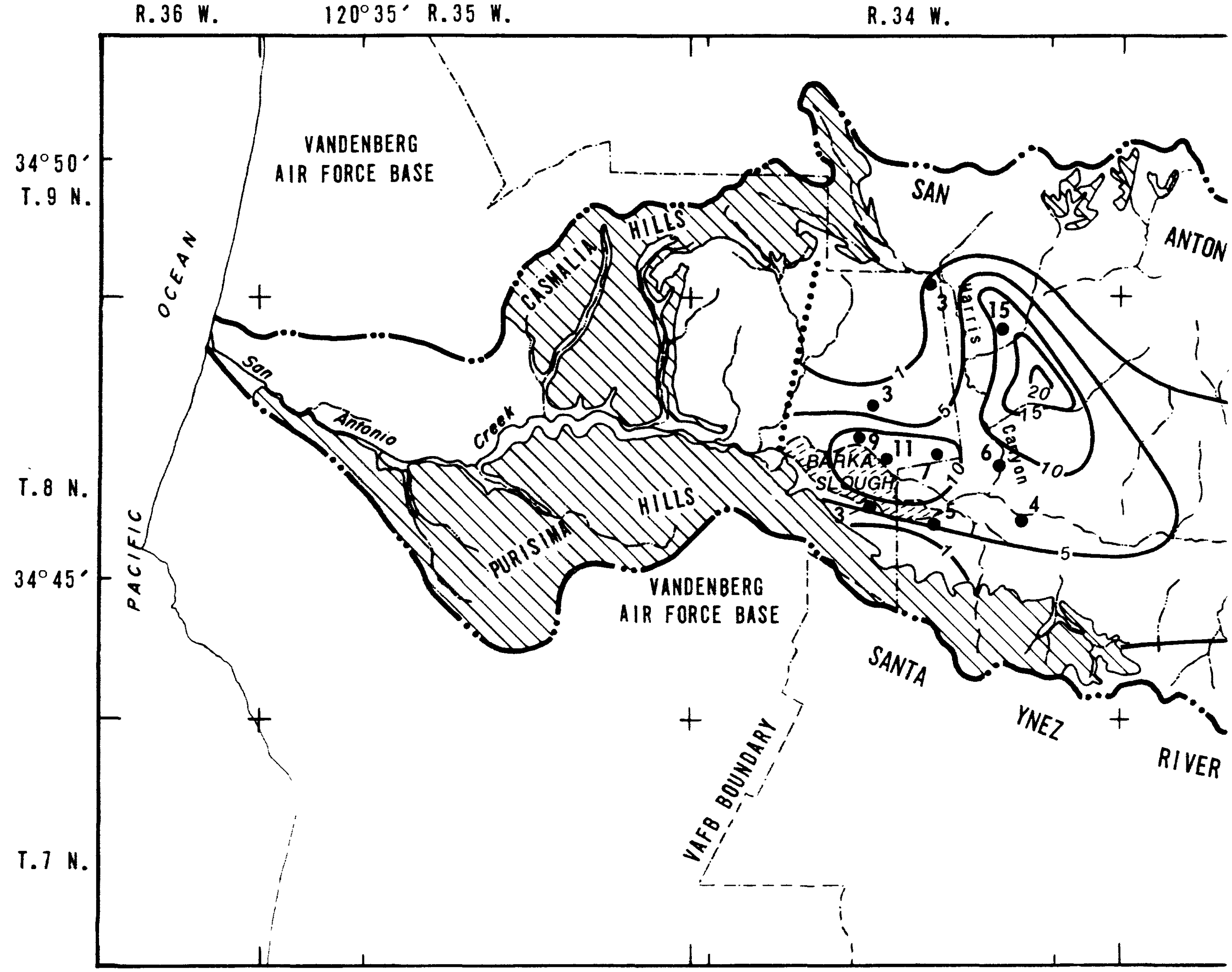
The results of this run substantiate that the model and the calibrated net-flux values were capable of duplicating the measured response of the aquifer within the acceptable limits of cost and time. The model can now be used as a predictive tool to help manage the ground-water basin.

TABLE 10. - Water budget for the verification period, 1978-82

[All values in acre-feet per year. Negative sign indicates water being removed from the aquifer system]

Water-budget components	Verification period				
	1978	1979	1980	1981	1982
Areal recharge[1]-----	+4,700	+4,700	+4,700	+4,700	+4,700
Stream recharge[1]----	+4,520	+4,520	+4,520	+4,520	+4,520
Irrigation net pumpage[1]----------	-6,950	-6,950	-6,950	-9,550	-11,520
Municipal pumpage[1]--	-170	-170	-170	-170	-170
Military pumpage[1]---	-3,060	-3,230	-3,860	-3,100	-3,570
Evapotranspiration at Barka Slough[2]--	-880	-710	-480	-560	-350
Base flow at Barka Slough[2]-----	-190	-70	0	-10	0
Storage[2]------------	+2,030	+1,910	+2,240	+4,170	+6,390

[1]Model-input values.
[2]Model-calculated values.



EXPLANATION

UNCONSOLIDATED DEPOSITS

CONSOLIDATED ROCKS

GROUND-WATER BARRIER

SURFACE-WATER DRAINAGE DIVIDE

INTERMITTENT STREAM

2 LINE OF EQUAL MODEL-GENERATED WATER-LEVEL DECLINE--Interval variable, in feet

5 MEASURED WATER-LEVEL DECLINE, IN FEET

FIGURE 16.--Comparison of measured and model-generated water-level decline, 1977-82.

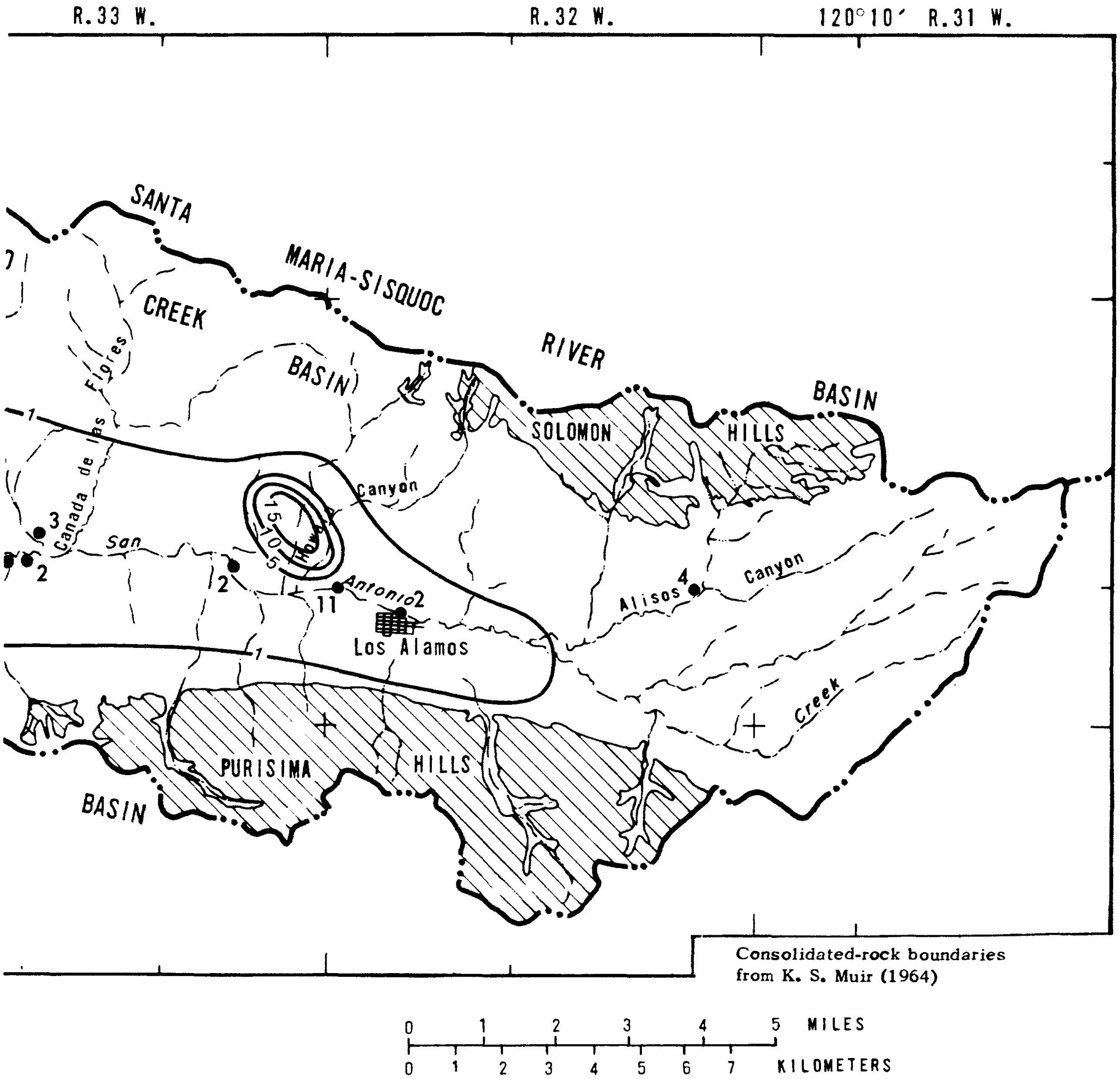


FIGURE 16.--Continued.

SUMMARY

A two-dimensional finite-difference ground-water flow model was developed for the San Antonio Creek valley ground-water basin. The objective of the numerical model was to evaluate present knowledge and concepts of the ground-water system and to develop a tool capable of predicting changing ground-water conditions resulting from current and proposed pumpage in the valley. The data required for the model, and for the analysis of model results, were obtained from detailed reports on the geohydrology of the San Antonio Creek valley by Muir (1964) and Hutchinson (1980).

The ground-water basin in the valley underlies an area of about 110 mi^2 and consists of unconsolidated valley-fill deposits. The unconsolidated deposits of predominantly sand and gravel range in thickness from zero feet along the perimeter of the basin to more than 3,000 feet in the central part of the basin. Consolidated rocks of low permeability form the lower boundary of the aquifer and also form much of the perimeter boundary or the ground-water basin. At the western edge of the ground-water basin, about 5 miles east of the Pacific Ocean, consolidated rocks form a barrier to the seaward flow of ground water. Upwelling of ground water east of the barrier has created a 660-acre marshland known as Barka Slough, which is one of the few pristine marshlands in southern California.

The numerical model was initially calibrated to ground-water conditions in 1943, which best represent steady-state conditions. Steady-state water levels are dependent on the quantity of recharge to and discharge from the ground-water basin, the transmissivity of the aquifer, and the vertical hydraulic conductivity of the confining bed of clay and peat underlying Barka Slough.

The recharge and discharge quantities used in the steady-state simulation, determined in a water budget developed for this study, equaled 6,200 acre-ft/yr. Recharge to the ground-water system consisted of infiltration of rainfall (4,700 acre-ft/yr) and seepage losses along San Antonio Creek (1,500 acre-ft/yr). Discharge from the system consisted of leakage (evapotranspiration and base flow) to Barka Slough (4,300 acre-ft/yr), municipal and irrigation net pumpage (1,500 acre-ft/yr), and evapotranspiration along the San Antonio Creek channel (400 acre-ft/yr).

The transmissivity of the aquifer and the vertical hydraulic conductivity of the confining bed were calibrated during the steady-state simulation. Model calibration was considered acceptable when the difference between model-calculated water levels and measured values was within ±5 feet along the valley floor, and within ±25 feet along the perimeter of the valley. Calibrated transmissivity values ranged from more than 20,000 ft^2/d along the axis of the valley to less than 100 ft^2/d along the perimeter. The vertical hydraulic conductivity of the confining bed was calibrated to be 0.054 ft/d.

Once the transmissivity and vertical-hydraulic-conductivity values had been determined by the steady-state simulation, the model was calibrated to simulate transient or time-dependent conditions during 1944-77. A uniform storage coefficient of 0.15 was assigned to the aquifer, except beneath Barka Slough where a value of 0.001 was assigned. Net-flux values were then adjusted so that computed water-level declines matched measured values. The net-flux values, rather than storage-coefficient estimates, were adjusted because the recharge and discharge data were considered less reliable.

The net-flux values along the San Antonio Creek channel, from the eastern edge of Barka Slough to about 2 miles east of Los Alamos, were determined by a least-squares calibration technique developed for this study. Results of the calibration indicated that the net flux out of the system along the San Antonio Creek channel increased by about 4,030 acre-ft/yr from 1943 to 1977. During this same period, net discharge (irrigation net pumpage, municipal pumpage, military pumpage, and evapotranspiration) was estimated to increase by 7,050 acre-ft/yr. The difference between the calibrated net-flux values and the estimated net-discharge values was assumed to be equal to stream recharge. From 1944 to 1957, computed values of stream recharge were 120 to 210 acre-ft/yr lower than steady-state conditions, whereas, from 1958 to 1977, computed stream-recharge values were 500 to 3,020 acre-ft/yr higher than steady-state conditions. The increase in stream recharge since 1958 was the result of increased pumpage along the stream channel. The increased pumpage lowered water levels, which were near or above land surface during steady-state conditions, to below the bottom of the stream channel, providing storage space for the seepage of available runoff.

The model simulated that about 70 percent of the 4,030-acre-ft/yr increase in net flux out of the system along the San Antonio Creek channel was derived from the reduction in ground-water discharge to Barka Slough and about 30 percent from water coming out of storage. The quantity of ground-water leakage to Barka Slough simulated by the model decreased from 4,300 to 1,490 acre-ft/yr. The model-simulated reduction in ground-water discharge was significantly greater than the measured reduction in base flow, indicating that there has also been a reduction in the amount of evapotranspiration at Barka Slough.

To verify the model and the calibrated net-flux values, the model was used to simulate the period from 1978 to 1982 using the annual change in net-flux values determined during this study. During the verification period, military and irrigation net pumpage increased by 6,310 acre-ft/yr. Results of the verification run indicated that about 5,170 acre-ft/yr of the 6,310-acre-ft/yr increase in net pumpage was withdrawn from storage, and the remainder was derived from the reduction of base flow and evapotranspiration at Barka Slough. The model simulated that, by the end of 1982, base flow to Barka Slough had ceased, and evapotranspiration by the willow trees had been reduced to about 350 acre-ft/yr.

During the verification period, model-computed water-level changes duplicated measured changes throughout most of the ground-water basin. By satisfactorily duplicating the response of the aquifer, the model and the calibrated input values were considered verified, within the acceptable limits of cost and time.

The verified model can now be used to predict water levels from proposed pumpage in the valley. The next phase of the study will be to use the model to predict water levels based on alternative management schemes proposed by Vandenberg Air Force Base.

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APPENDIX--DESCRIPTION OF THE LEAST-SQUARES CALIBRATION TECHNIQUE

A least-squares calibration technique was used to determine unknown net-flux values along the San Antonio Creek channel. The mathematical development of the least-squares calibration technique and a simple example showing the application of the calibration technique are described below:

Mathematical Development of the Least-Squares Calibration Technique

For a given block of the model, there will be a difference between the model-computed water-level change and the measured water-level change. This difference is referred to as the error and may be positive, negative, or zero. A measure of the "goodness of fit" of the model-computed water-level changes to the measured values is provided by the least-squares relationship:

$$S = \sum_{i=1}^{n} (\hat{h}_i - h_i)^2, \tag{1}$$

where

S is the sum of the squared errors,

n is the number of model blocks with a record of measured water-level changes,

$\hat{h}_i$ is the measured water-level change of the i^{th} model block, and

h_i is the model-computed water-level change at the i^{th} model block.

If the sum of the squared errors is small, the fit is good; if it is large, the fit is bad. The model-computed water-level change is a function of the net-flux values input in the model and can be expressed as:

$$h_i \equiv h_i \langle\vec{q}\rangle, \tag{2}$$

where

$\vec{q}$ is the vector representing the net-flux values $q_1, q_2, \ldots q_m$, where m is the number of nodes with a net-flux input.

Substituting this expression for h_i into equation 1 yields

$$S = \sum_{i=1}^{n} (\hat{h}_i - h_i \langle\vec{q}\rangle)^2, \tag{3}$$

which can be differentiated to yield an equation to minimize the error

$$\frac{\partial S}{\partial q_j} = -2 \sum_{i=1}^{n} \left[\hat{h}_i - h_i \langle\vec{q}\rangle\right] \frac{\partial h}{\partial q_j} \langle\vec{q}\rangle \qquad j = 1,2,\ldots,m. \tag{4}$$

By definition, when this equation is at a minimum, $\frac{\partial S}{\partial q_j} = 0$,

therefore

$$0 = \sum_{i=1}^{n} \left[\hat{h}_i - h_i \langle\vec{q}\rangle\right] \frac{\partial h_i}{\partial q_j} \langle\vec{q}\rangle \qquad j = 1,2,\ldots,m; \tag{5}$$

or rewriting the equation

$$0 = \sum_{i=1}^{n} \hat{h}_i \frac{\partial h_i}{\partial q_j} \langle\vec{q}\rangle - \sum_{i=1}^{n} h_i \langle\vec{q}\rangle \frac{\partial h_i}{\partial q_j} \langle\vec{q}\rangle \qquad j = 1,2,\ldots,m. \tag{6}$$

The above equation represents the minimized form of the least-squares equation.

To solve the minimized form of the least-squares equation, the term $h_i \langle\vec{q}\rangle$ was replaced by an equation representing the linear response of the computed water-level changes

$$h_i \langle\vec{q}\rangle = \sum_{j=1}^{m} \alpha_{ji}\, q_j \qquad i = 1,2,\ldots,n, \tag{7}$$

where α_{ji} is the unit-response value. This value is the model-computed water-level change recorded at the i^{th} block in response to a unit net flux at the j^{th} block, and the other variables remain as before. The linear-response equation can be differentiated to yield

$$\frac{\partial h_i}{\partial q_k} \langle\vec{q}\rangle = \frac{\partial}{\partial q_k} \sum_{j=1}^{m} \alpha_{ji}\, q_j = \alpha_{ki} \qquad k = 1,2,\ldots,n. \tag{8}$$

Substituting the results of equations 7 and 8 into equation 6 yields

$$0 = \sum_{i=1}^{n} \hat{h}_i \alpha_{ji} - \sum_{i=1}^{n} \alpha_{ji} \left[\sum_{k=1}^{m} \alpha_{ki} q_k \right] \qquad j = 1,2,\ldots,m. \tag{9}$$

This equation can be arranged and expressed in matrix notation as

$$(\alpha_{ji})(\alpha_{ji})^T \{q_j\} = (\alpha_{ji}) \left\{\hat{h}_i\right\} \qquad \begin{matrix} i = 1,2,\ldots,n \\ j = 1,2,\ldots,m \end{matrix} \tag{10}$$

where

$$(\alpha_{ji}) \text{ is a } (m \times n) \text{ matrix} = \begin{pmatrix} \alpha_{11} & \alpha_{12} & \cdots & \alpha_{1n} \\ \alpha_{21} & \alpha_{22} & \cdots & \alpha_{2n} \\ \vdots & \vdots & & \vdots \\ \alpha_{m1} & \alpha_{m2} & \cdots & \alpha_{mn} \end{pmatrix}$$

$$(\alpha_{ji})^T \text{ is a } (n \times m) \text{ matrix and is the transpose of the } \alpha_{ji} \text{ matrix} = \begin{pmatrix} \alpha_{11} & \alpha_{22} & \cdots & \alpha_{m1} \\ \alpha_{12} & \alpha_{22} & \cdots & \alpha_{m2} \\ \vdots & \vdots & & \vdots \\ \alpha_{1n} & \alpha_{2n} & \cdots & \alpha_{mn} \end{pmatrix}$$

$$\left\{\hat{h}_i\right\} \text{ is a column vector} = \begin{Bmatrix} \hat{h}_1 \\ \hat{h}_2 \\ \vdots \\ \hat{h}_n \end{Bmatrix}$$

$$\{q_j\} \text{ is a column vector } = \begin{Bmatrix} q_1 \\ q_2 \\ \cdot \\ \cdot \\ \cdot \\ q_m \end{Bmatrix}$$

By letting $(A) = (\alpha_{ji})(\alpha_{ji})^T$ and $\{b\} = (\alpha_{ji})\ \{\hat{h}_i\}$, equation 10 can be simplified by substitution to

$$\underset{(m\times n)}{(A)}\ \underset{(m\times 1)}{\{q\}} = \underset{(m\times 1),}{\{b\}} \tag{11}$$

which is a matrix equation for a system of m linear equations with m unknowns, $q_1, q_2, \ldots, q_m$ of the form

$$\begin{array}{ccccccc} a_{11}\ q_1 & + & a_{12}\ q_2 & + \ldots + & a_{1m}\ q_m & = & b_1 \\ a_{21}\ q_1 & + & a_{22}\ q_2 & + \ldots + & a_{2m}\ q_m & = & b_2 \\ \vdots & & \vdots & & \vdots & & \vdots \\ a_{m1}\ q_1 & + & a_{m2}\ q_2 & + \ldots + & a_{mm}\ q_m & = & b_m, \end{array}$$

where the coefficients a_{ji} and b_j are known numbers. The net-flux values $(q_1, q_2, \ldots, q_m)$ can now be easily calculated by solving the above system of linear equations.

Application of the Least-Squares Calibration Technique

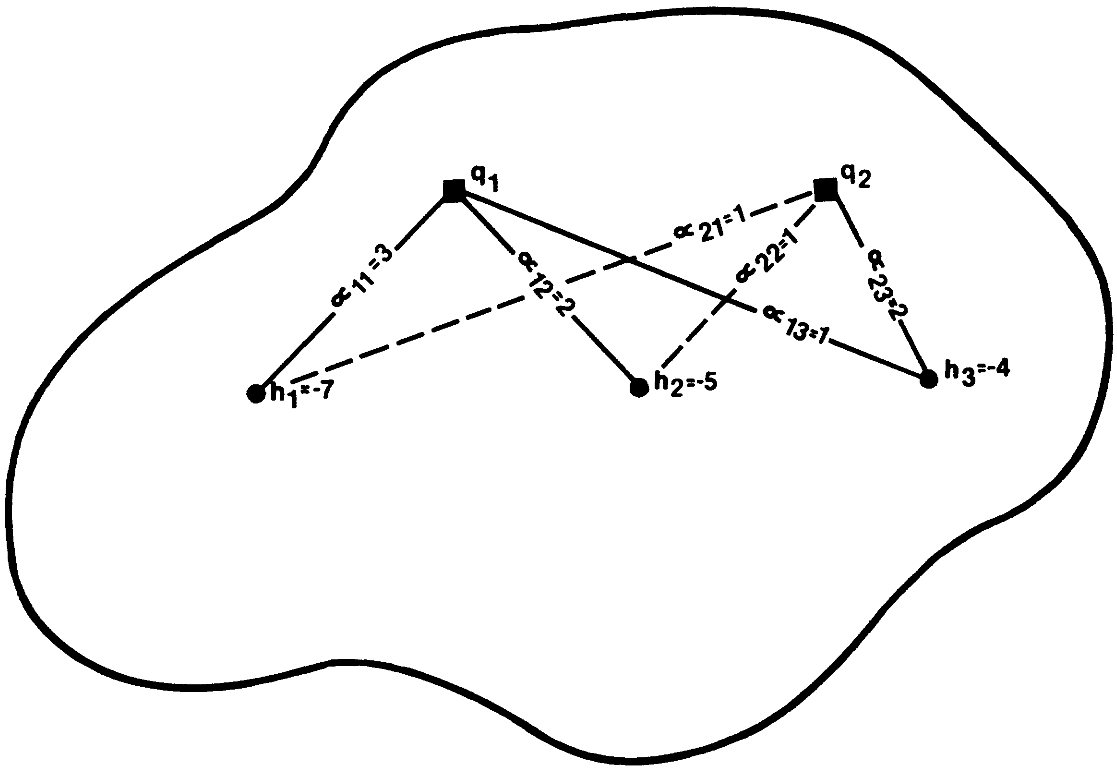
To show how the least-squares technique is used to calculate the net-flux values, a simple example is presented. For the example, assume there are three model blocks with measured water-level changes ($\hat{h}_1, \hat{h}_2, \hat{h}_3,$) caused by an unknown net flux at two blocks (q_1 and q_2). The relationship of the blocks to one another and the unit-response values (α_{ji}) are shown in figure 17. The measured water-level changes and unit-response values can be written in matrix notation as:

$$\{\hat{h}\} = \begin{Bmatrix} \hat{h}_1 \\ \hat{h}_2 \\ \hat{h}_3 \end{Bmatrix} = \begin{Bmatrix} -7 \\ -5 \\ -4 \end{Bmatrix}$$

and

$$(\alpha) = \begin{pmatrix} \alpha_{11} & \alpha_{12} & \alpha_{13} \\ \alpha_{21} & \alpha_{22} & \alpha_{23} \end{pmatrix} = \begin{pmatrix} 3 & 2 & 1 \\ 1 & 1 & 2 \end{pmatrix},$$

where negative values for $\{\hat{h}\}$ indicate water-level declines.



EXPLANATION

———	BOUNDARY OF MODEL
■ q_j	UNKNOWN NET FLUX AT THE j^{th} NET-FLUX BLOCK
● -7	MEASURED WATER-LEVEL CHANGE AT THE i^{th} RESPONSE BLOCK (h_i)
——3——	COMPUTED WATER-LEVEL CHANGE AT THE i^{th} RESPONSE BLOCK IN RESPONSE TO A UNIT NET FLUX AT q_1, IN FEET (α_{1i})
— —2— —	COMPUTED WATER-LEVEL CHANGE AT THE i^{th} RESPONSE BLOCK IN RESPONSE TO A UNIT NET FLUX AT q_2, IN FEET (α_{2i})

FIGURE 17.—Relation of net-flux blocks to response blocks used in the example of the least-squares calibration technique.

Substituting these values into equation 11 where

$$(A) = (\alpha_{ji})(\alpha_{ji})^T = \begin{pmatrix} 3 & 2 & 1 \\ 1 & 1 & 2 \end{pmatrix} \begin{pmatrix} 3 & 1 \\ 2 & 1 \\ 1 & 2 \end{pmatrix} = \begin{pmatrix} 14 & 7 \\ 7 & 6 \end{pmatrix}$$

and

$$\{b\} = (\alpha_{ji})\{\hat{h}_i\} = \begin{pmatrix} 3 & 2 & 1 \\ 1 & 1 & 2 \end{pmatrix} \begin{pmatrix} -7 \\ -5 \\ -4 \end{pmatrix} = \begin{pmatrix} -35 \\ -20 \end{pmatrix},$$

yields the matrix equation

$$\begin{pmatrix} 14 & 7 \\ 7 & 6 \end{pmatrix} \begin{Bmatrix} q_1 \\ q_2 \end{Bmatrix} = \begin{Bmatrix} -35 \\ -20 \end{Bmatrix},$$

which is equivalent to the linear equations

$$14q_1 + 7q_2 = -35$$

$$7q_1 + 6q_2 = -20.$$

These equations can be solved by subtracting a multiple of the first equation from the second to eliminate q_1, yielding the value of q_2

$$\begin{aligned} -7q_1 - 3.5q_2 &= 17.5 \\ 7q_1 + 6q_2 &= -20 \\ \hline 2.5q_2 &= -2.5 \\ q_2 &= -1 \end{aligned}$$

and by backsubstituting, the value for q_1 can be obtained

$$-7q_1 - 3.5\,(-1) = 17.5$$

$$-7q_1 = 14$$

$$q_1 = -2.$$

In this example a negative net-flux value indicates that discharge is greater than recharge.

To check the calculated net-flux values, the computed water-level changes are compared to the measured values. Since $h_i \equiv h_i \langle \vec{q} \rangle$ (eq. 2), equation 5 can be rewritten as

$$h_i = \sum_{j=1}^{m} \alpha_{ji} \, q_i \qquad i = 1,2,\ldots,n, \tag{12}$$

or in matrix notation as

$$\{h\} = (\alpha)^T \{q\}. \tag{13}$$

Substituting the computed values for $(\alpha)^T$ and $\{q\}$ into equation 11 yields the computed water-level changes

$$\{h\} = \begin{pmatrix} 3 & 1 \\ 2 & 1 \\ 1 & 2 \end{pmatrix} \begin{pmatrix} -2 \\ -1 \end{pmatrix} = \begin{pmatrix} -7 \\ -5 \\ -4 \end{pmatrix},$$

which are equivalent to the measured water-level changes.

As shown in the example, it is relatively simple to compute a small number of net-flux values for a given number of water-level changes. However, as the number of unknown net-flux values increases, the number of linear equations increases accordingly, and solution of the net-flux values becomes difficult by hand calculation. The equations can be solved quickly, however, with appropriate computer programs.